Final Thesis Report

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Structural Option

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Orange Regional Medical Center

Middletown, NY

ORMC ORANGE REGIONAL MEDICAL CENTER

Middletown, NY

Ryan Blatz - Structural

Architecture

◊ Pod construction, leaving voids for future additions

◊ Patient rooms comparable to hotel rooms

◊ Brick façade with removable full glass windows

General Information

◊ Owner: Orange Regional Medical Center

◊ Building Type: Hospital

◊ Size: 600,000 SF

◊ Floors: 6 above grade and 1 partially below

◊ Total Cost: $320 million

◊ Design/Construction Team: HBE

Mechanical | Plumbing

◊ Several VAV and Constant Volume systems heat and cool with supply capacities between 34,500 and 80,400 cfm

◊ Emergency exhaust in all surgical departments

◊ Snow melt system for helipad and sidewalk

Lighting | Electrical

◊ Main Switchgear is 13.2 kV (1200A)

◊ 480/277V 3PH - 4W Main Power

◊ Primarily indirect fluorescent lighting with warm incandescent lighted patient rooms

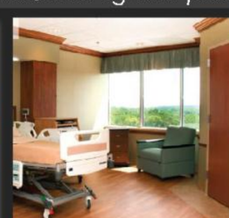
Structure

◊ Composite deck with 3¼" of light weight concrete

◊ Steel frame with composite beams and girders

◊ Lateral load resisted by concrete shear walls on ground floor and by eccentrically braced steel frames first floor and above

◊ Spread footing foundation



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EXECUTIVE SUMMARY

The existing steel structure of Orange Regional Medical Center effectively handles the various loadings it is exposed to; this was made clear from earlier technical reports. However, there were areas that raised interest and brought up the question of whether a more efficient system exists. From the first look at the existing lateral system, one would question whether fifty lateral frames is necessary to control lateral drifts. From this question, it was determined to redesign the structure of ORMC as a concrete flat slab system, using moment frames for lateral support.

From early analysis in this report, it was made clear that the flat slab system would be effective against gravity loading, with all deflection, shear and moment values falling well within their limits. However, once the building was subjected to seismic loading, which was the predominant lateral force, issues arose in moment capacity and story drifts. The 8,394 kip base shear caused moment concentrations at the columns of the second floor where the building geometry changes. As an end result, it was determined that shear walls were necessary around the elevator shafts to control these forces and bring down the story drift values. The moment frames still serve as the primary lateral resistance system, taking over 75% of the lateral load. This was accomplished with an 11 inch slab teamed with column sizes of 30x30's spanning the entire height, 20x20's for the lower section, and 24x24's in the administration wing to control drift values.

A cost and schedule analysis was run for comparison purposes with the existing structure. The results showed that the concrete system would cost roughly $20 million. This is almost twice the cost of the existing structure, and since ORMC had to work with a tight budget, this ultimately labeled the flat slab system as not being a viable alternative. The construction schedule yielded expected results with the concrete system taking about six weeks longer to construct. This also adds cost to this system and gives additional reasons for why the concrete system would not be a better alternative.

Successful architectural layouts were established in a redesign of the medical departments. The goal was to provide efficient flow by placing the Emergency Room, Operating Room, and Intensive Care Unit next to one another. This redesign only impacted the first and third floors, but all departments were able to maintain their existing square footage. The redesign also focused on comfort by relocating the healing garden to the second story roof where it would be more accessible to patients and also provide better window views. This raised concerns with the structural force concentration on the second story. The added weight of the green roof would require either the upsizing of columns or the addition of shear walls. Ultimately, this would be a call made by the owner.

ACKNOWLEDGEMENTS

Orange Regional Medical Center

Special thanks to Jonathan Schiller, Chief Operating Officer and John King, Facilities Engineer for connecting me with my thesis topic and supporting me throughout the year. I know how busy everyone at ORMC was during to move to the new hospital so I can't begin to express my level of appreciation.

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There have been so many good times over these past couple years, and I'm sure there will be many more.

INTRODUCTION

Built with the future in mind, the new Orange Regional Medical Center (ORMC) set to unite ORMC and its branch campus at one new location in Middletown, NY. Not only does this 600,000 square foot hospital provide enough space for the merging of these two locations, but it was also designed to allow for future expansion. From the second story and up, the floor plan was architecturally designed in the shape of a medical cross. This design allows for medical departments to branch off of a central elevator core for easy circulation. In addition, this design will provide seamless building expansion. Currently, the top two floors of this medical cross floor plan are missing two legs of the cross. This provides space for future additions as the community grows over the lifetime of the building. Figure 1 shows the rooftop voids where future additions will be constructed. It is important to note that the analysis and design in this report account for these future additions, and treat the hospital as a 722,000 square foot structure.

Figure 1: *Location of Future Additions*

Figure 2: *Patient Rooms*

Patient and employee comfort was a primary concern in the design of the new hospital. This is not only evident in the finishing touches, but in numerous design features of the building as well. ORMC features patient rooms that rival the rooms of hotels (Figure 2). Carpeting was also installed throughout all hallways of ORMC to provide a quieter atmosphere for recovering patients and diligent employees. This is all based around studies showing that patients are quicker to recover in comfortable, quieter spaces.

Finished in April 2011, Orange Regional Medical Center is comprised of six stories above grade and one partially below, which create a height of 97.5 feet from ground floor. The first floor is significantly larger in square footage than the upper five floors, allowing for major medical departments, such as the Emergency and Operating Rooms, to be readily accessible to incoming patients. The overall design of ORMC is one that has been used before by the design and construction company, HBE. In fact, the one architectural feature setting this hospital apart from HBE's other designs is the cathedral ceilinged lobby, which features gift and coffee shops for the visitors of Orange Regional Medical Center.

EXISTING STRUCUTRAL SYSTEM

Foundations

The foundations are determined from recommendations of the geotechnical report by Melick-Tully and Associates. Square, concrete spread footings are set on virgin soil or engineered, compacted soil with a bearing stress of 4000 psi. Of all 351 columns, 167 carry load down to footings at the first story level, where the rest are carried down to the ground floor level.

Floor System

Out of the Vulcraft catalog, the existing floor system of ORMC consists primarily of 2VLI20 composite deck with 3¼" of light weight concrete, making for a total floor thickness of 5¼". The decking runs three spans, perpendicular to the joists, where typical spans are in the range of 7'4". However, the decking may see longer spans due to the lack of bay size uniformity.

Gravity System

The existing composite steel frame of this structure comes in a variety of sizes. On the first floor alone, there are a total of twelve different wide flange composite beams used, but in general, W16x26's and W16x31's serve as the primary beams throughout the building with an average spacing of about 7 feet and an average span of about 26 feet, as shown in the typical bay in *Figure 3*. W18x35's and W21x44's are the most common choice for girders with spans ranging between 14' 8" and 27' 1". This size dispersion also follows the load path to the columns. A majority of the columns are W12's with a small grouping of W10's and W8's. As mentioned earlier, structural columns for the future additions are also shown on the column schedule. Traveling up the building, the columns continue to carry less of the building load and therefore, reduce in size. Typically, each column has two splices occurring just above the second and fourth floors. However, there are special cases where splices occur on the third and fifth floors instead. The structural notes specify that all splice connections must be slip critical connections. Looking further into the frame connections, steel beam connections are detailed as simple span beams, with the a few exceptions. There are only a handful of moment frames specified throughout the building which must be considered as continuous beams.

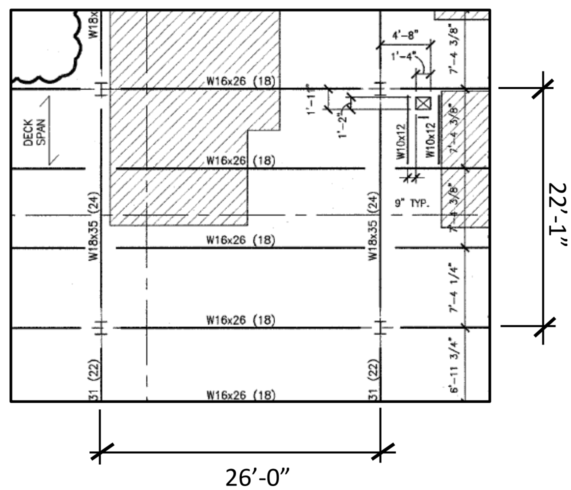


***Figure 3**: Typical Bay*

Lateral System

To resist the lateral forces from wind and seismic activity, the structure utilizes concrete shear walls on the ground level. These shear walls only extend up to the first floor. From the first floor and above, the lateral forces are then resisted by forty-eight eccentrically braced steel frames and two concentrically braced frames. These braced frames are present in varying heights. A majority of the braced frames fall within the exterior walls, so those frames around the perimeter of the first floor typically end at the second story. A number of braced frames continue up to the roof and resist lateral loading in each of the legs of the cross shape floor plan, as shown in *Figures 4 and 5*.

Red: Full Height Braces
Blue: Two Story Braces

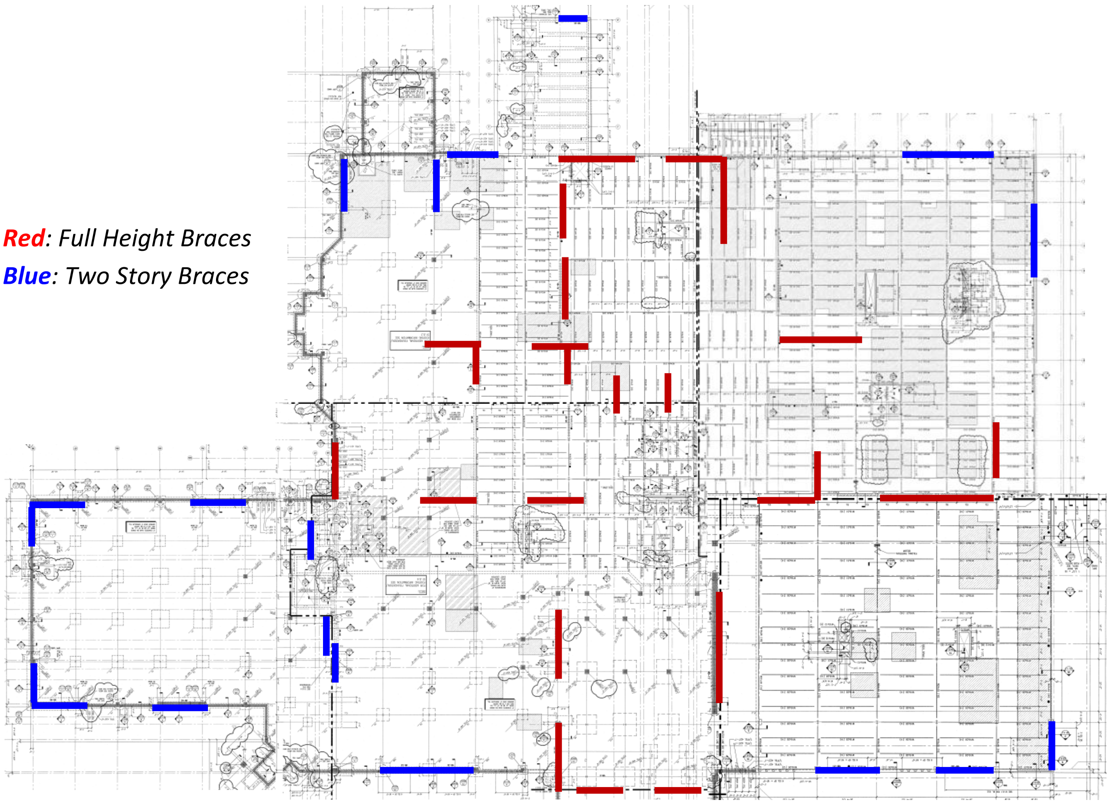


Figure 4: *Braced Frames Location*

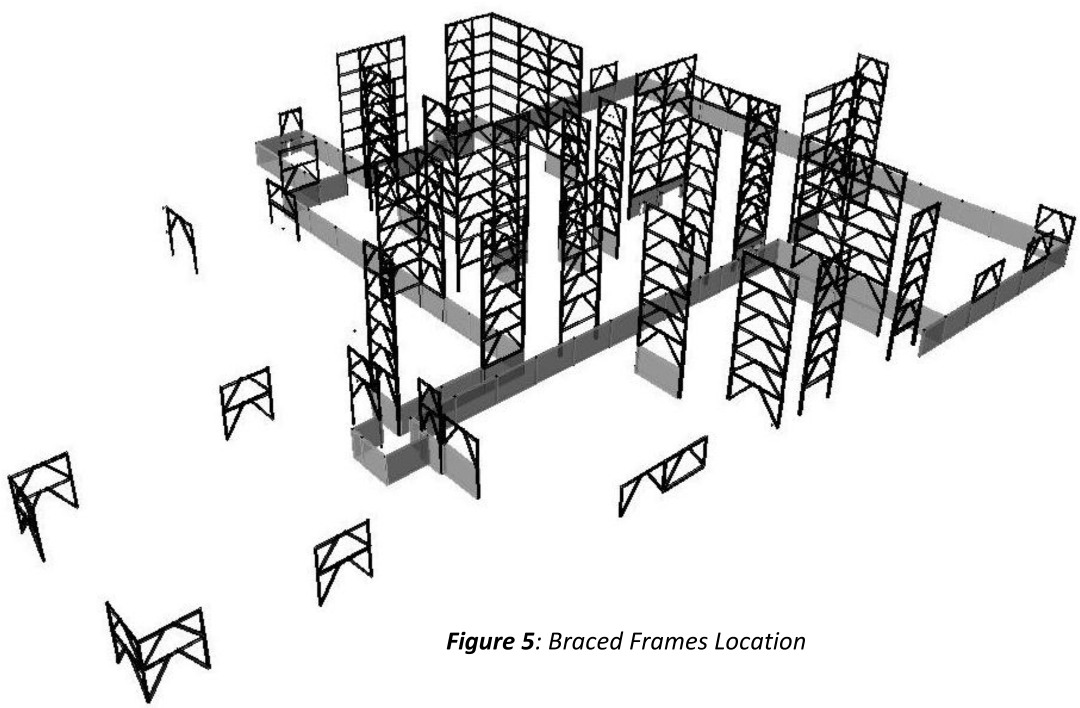


Figure 5: *Braced Frames Location*

GENERAL STRUCTURAL INFORMATION

In the original analysis of the steel structure, the primary codes considered through the calculations were ASCE7-10 and AISC-14th Edition. ASCE was used for determining Live, Snow, and Lateral Loadings, where the Main Wind Force Resisting System (MWFRS) and Equivalent Lateral Force Method (ELF) were used for Wind and Earthquake analysis, respectively. In the redesign, ASCE7-10 is still used for Live, Snow, and Lateral loadings (MWFRS for wind and ELF for seismic). However, AISC is switched for ACI318-08 in the design of the concrete frame.

Materials and Standards

Existing Steel Structure

a.	W's and WT's	ASTM A992
b.	Plates and other shapes	ASTM A36
c.	HSS	ASTM A500, Grade B
d.	Pipe	ASTM A53, Grade B
e.	Bolts	ASTM A325, or F1852 where indicated
f.	Anchor Rods	ASTM F1554, Grade 36
g.	Threaded Rod	ASTM A36
h.	Headed Studs	AWS D1.1, Type B

Redesigned Structure

a.	Concrete	**f'c (psi)**	**Unit Weight (pcf)**
	Columns, Slabs, Drops	4,000	150
	Shear Walls	6,000	150
b.	Reinforcement	ASTM A-615, Grade 60	

THESIS OBJECTIVE

Problem Statement

As noted earlier, the existing system requires fifty lateral braced frames to control the story drifts in each leg of the cross-shape floor plan. Each of these frames requires an increased level of attention to ensure proper lateral performance. For instance, the heavy bracing connections must be properly designed to prevent moment at the connection joint. The link of the eccentric braced frames also requires stiffeners to avoid local buckling. All these details call for site inspections, which increase cost and length of construction. Additionally, many of these braced frames tie into shear walls at the first story. Again, these connections become difficult to ensure the proper transfer of later loads and to ensure that the braced frames will be plumb. With this in mind, Orange Regional Medical Center was also working on a limited budget, and therefore, any alternative system would need to be cost effective. The use of braced frames also brought about architectural concerns. When the building opened in 2011, many occupants disliked the braces running through the windows, and since a majority of these frames run along the perimeter of the building, this was the case for most window openings.

Structural Depth - Problem Solution

Given the spans between columns, a flat slab, concrete system would be an effective method against loading from gravity. In fact, the flat slab system is a common system amongst hospitals for numerous reasons. For one, concrete systems typically perform well under vibration, which is a potential issue with hospital equipment. This would also allow for easier connections than the steel braced frames and the connections to the shear walls since everything is cast integrally. These systems also provide space to run mechanical equipment between drop panels. This is especially beneficial, given that hospitals typically require more mechanical systems to provide infection control.

A concrete flat slab, if designed correctly, would also be effective against lateral loads. Essentially, concrete structures provide moment frames for free. This would eliminate the need for braced frames around the perimeter of the structure, allowing for a very flexible floor plan and façade. These are the reasons that the flat slab system was found to be a viable system in Technical Report 2. In addition, that report estimated that the flat slab system would be less expensive than the existing steel structure. The concrete frames would not require any additional fire proofing, and it would also eliminate the need for steel inspectors on site, but a more detailed estimate would have to be made to confirm whether the new system would fall within ORMC's budget.

Using the existing column locations, the wide flange columns will be replaced with square concrete columns. The goal is to completely design the structure for gravity and lateral loading using solely the concrete moment frames. The columns and slab will have to withstand both shear and moment forces produced by the controlling load combination. If it is found that shear walls are necessary to control lateral behavior, they will be placed in the appropriate locations. This should still produce a flexible floor plan, given that there would be far fewer shear walls than braced frames. Technical Report 2 also found the flat slab system to weigh less than the existing steel system. This will have to be confirmed since concrete structures are typically heavier than those of steel. If the system is found to be heavier, this will increase the seismic lateral forces and the gravity loading on the foundation. These changes would be accounted for in the redesign.

Breadth Topic 1 – Cost and Schedule Analysis

An ideal redesign will be less expensive than the existing system, especially since budget was a crucial part of ORMC's decisions. Although an initial estimate from Technical report 2 found the flat system to be more cost effective, a more detailed cost analysis will have to be carried out using RS Means in order to accurately make comparisons with the existing system. This change to a concrete system will also have impacts on the construction schedule. Typically, concrete construction is longer than what we would expect from steel construction. The purpose of this analysis would be to determine how much the critical path would be affected by changing the structural material.

Breadth Topic 2 – Architectural Redesign

After having some time to work at Orange Regional's new location, the employees have voiced concerns regarding the architectural layout of many of the medical departments. This redesign resolves those concerns by fully rearranging the architectural floor plans in a way that's conducive to the medical flow of patients and staff. This redesign also seeks to better achieve the hospital's initial goal of patient comfort. This includes minor architectural details as well as exploration into the use of green roofs.

BUILDING LOADS

Dead Loads

Typical Floor Loading	
Component	Weight (psf)
Concrete	125.00
MEP & Misc.	20.00
	145.00
Roof Loading	
Component	Weight (psf)
Concrete	125.00
Rigid Insulation	2.00
MEP & Misc.	20
Snow	28
Snow (30% Seismic)	8.4
	155.40

***Table 1**: Floor Dead Loading*

The dead loading on the floors is derived from knowledge of material weights and educated estimates. These loadings include self-weight and MEP loading on the floor level, with the exception of the ground floor, which will not have an MEP load. The roof loading consists of self-weight, insulation, and MEP. This is shown in *Table 1* to the left.

Live Loads

Typical Live Loading	
Component	Weight (psf)
Operating Rms, Labs	60
Patient Rooms	40
Corridors Above 1st	80
Corridor 1st Floor	100
Lobby	100
Dining Area	100
Offices	50
Roof	20

***Table 2**: Floor Live Loading*

Design live loads were taken right from the structural drawings provided by HBE to develop an accurate comparison. Throughout the design process, the live load is taken as 100 psf everywhere. This is due to the fact that corridors run through a majority of the bays, and since this is the controlling load, this determines the necessary slab thickness necessary over the entire floor. The various live loads throughout the building are shown in *Table 2* to the left.

Snow Loads

From ASCE7-10, the ground snow load for the building location is found to be 50 psf. This translates to a 42 psf flat snow load on the roof. ACE7-10 also states that for seismic, the snow load is to be taken as twenty percent of flat snow load, meaning that only 8.4 psf is considered toward calculating building weight. Detailed snow load calculations can be found in Appendix A.

Wind Loads

With the redesign from the steel structure to a concrete structure, the geometry of the building remains the same, and since wind loading is dependent on geometry, the loads also remain the same for both types of construction. Therefore, for both structures, MWFRS is applied to determine the wind pressures at each story (shown in *Tables 3 & 4* and *Figures 7 & 8*). From this method, the basic wind design speed for Middletown, NY is 120 mph with an exposure C category. The shape of the hospital was simplified to the shape in *Figure 6* during analysis to provide general wind pressures.

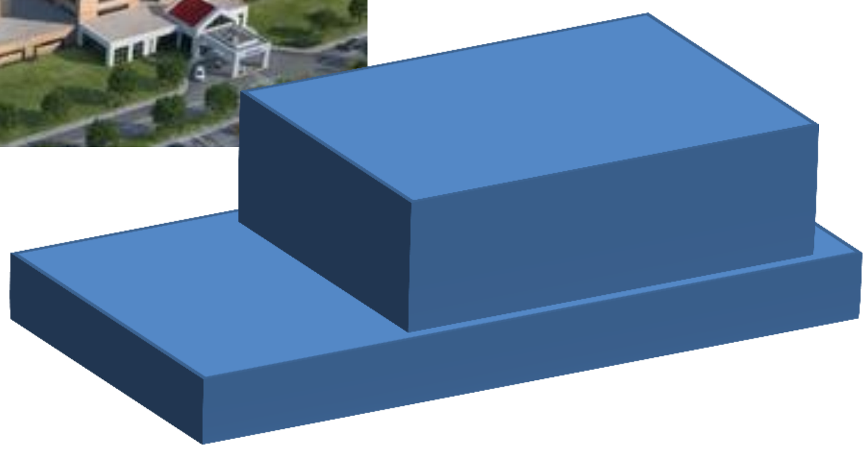


Figure 6: *Simplified Shape for Wind Analysis*

Multiple wind load cases, in *Figure 9*, are applied to the structure to account for directionality and torsional effects of wind. These loads are applied to an ETABS model to determine the story drifts, which follow later in this report. As mentioned earlier, the wind loading has not changed with the redesign, and since seismic was the predominant load in the existing steel structure, wind will continue to be the lesser load as the seismic load increases in the concrete structure. For this reason, load cases with seismic will always control over cases with wind. For detailed wind calculations, refer to Appendix B.

Wind Pressures - North/South										
Floor	z	K_z	q_z	$p_{Windward}$ (psf)	WW (plf)	WW (k)	q_h	$p_{Leeward}$ (psf)	LW (plf)	LW (k)
Ground	0	0.85	26.63	18.6	148.5	72.5	39.32	-16.1	-128.5	-62.7
1	16	0.86	26.95	18.8	300.4	146.6	39.32	-16.1	-257.0	-125.4
2	32	0.99	31.08	21.7	314.1	153.3	39.32	-16.1	-232.9	-113.7
3	45	1.07	33.37	23.3	302.3	108.5	39.32	-16.4	-213.7	-76.7
4	58	1.12	35.16	24.5	318.5	114.3	39.32	-16.4	-213.7	-76.7
5	71	1.17	36.79	25.6	333.2	119.6	39.32	-16.4	-213.7	-76.7
6	84	1.22	38.29	26.7	353.5	126.9	39.32	-16.4	-217.8	-78.2
Roof	97.5	1.26	39.32	27.4	185.0	66.4	39.32	-16.4	-111.0	-39.8

***Table 3**: North/South Wind Pressure*

Wind Pressures - East/West										
Floor	z	K_z	q_z	$p_{Windward}$ (psf)	WW (plf)	WW (k)	q_h	$p_{Leeward}$ (psf)	LW (plf)	LW (k)
Ground	0	0.85	26.63	18.4	147.4	84.3	39.32	-16.5	-132.3	-75.6
1	16	0.86	26.95	18.6	298.4	170.5	39.32	-16.5	-264.6	-151.2
2	32	0.99	31.08	21.5	311.9	178.2	39.32	-16.5	-239.8	-137.0
3	45	1.07	33.37	23.1	300.2	119.0	39.32	-17.0	-221.1	-87.7
4	58	1.12	35.16	24.3	316.3	125.4	39.32	-17.0	-221.1	-87.7
5	71	1.17	36.79	25.5	330.9	131.2	39.32	-17.0	-221.1	-87.7
6	84	1.22	38.29	26.5	351.1	139.2	39.32	-17.0	-225.4	-89.4
Roof	97.5	1.26	39.32	27.2	183.7	72.8	39.32	-17.0	-114.8	-45.5

***Table 4**: East/West Wind Pressure*

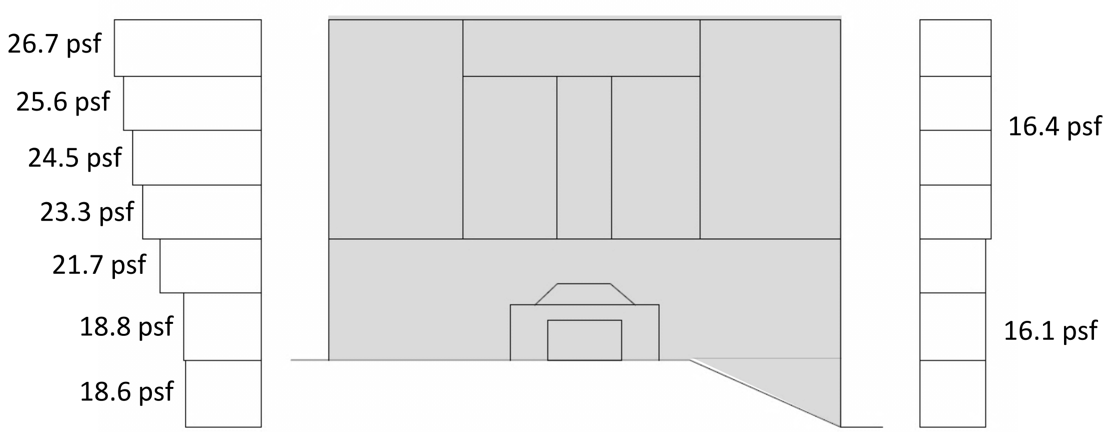


***Figure 7**: North/ South Wind Pressure*

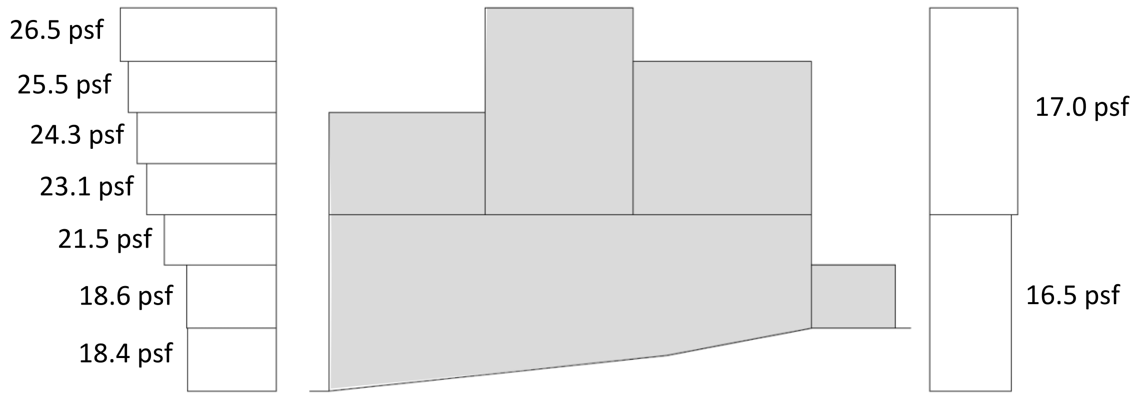


***Figure 8**: East/West Wind Pressure*

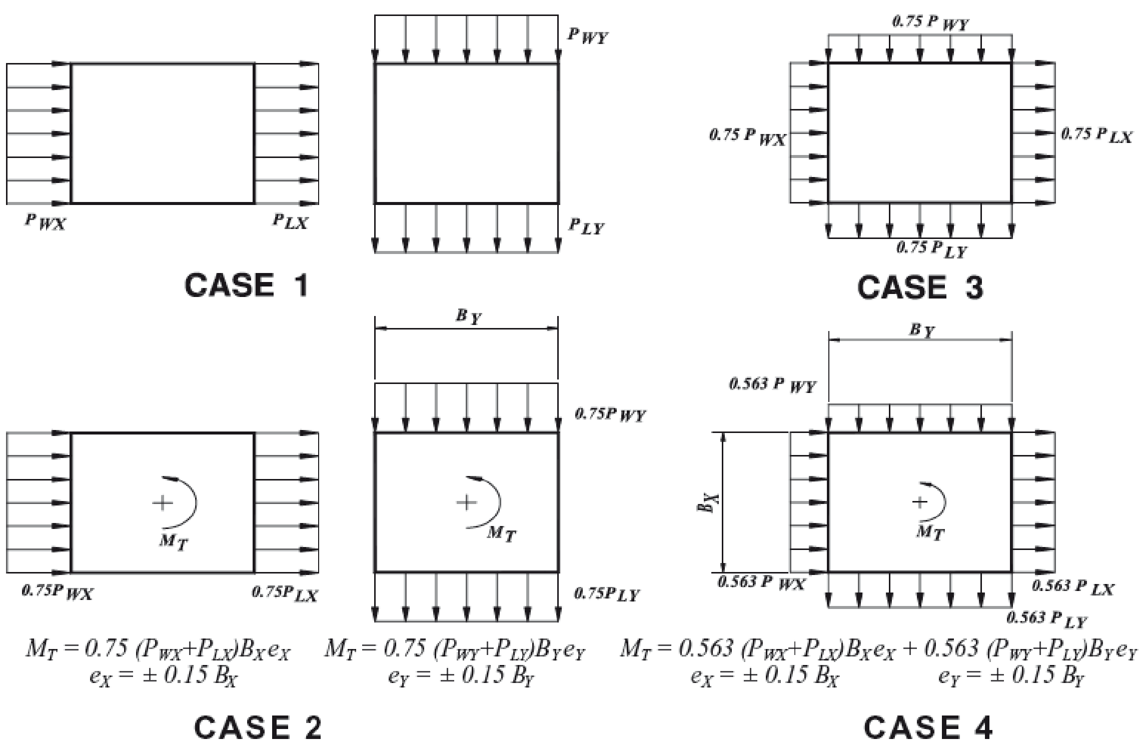


***Figure 9**: ASCE7-10 Wind Load Cases*

Seismic Loads

Equivalent Lateral Force Method from ASCE7-10 is used to determine story shears from seismic loading. Essentially, the only part of the calculations that change when redesigning the structure from steel to concrete is the R factor and the building weight. For the redesigned concrete structure, the R factor remains at 3 for a "dual system with intermediate moment frames capable of resisting at least 25% of prescribed seismic forces." This system utilizes the intermediate moment frames at every column with the addition of ordinarily reinforced concrete shear walls around the elevator shafts. ACI318-08 describes the requirements for creating an intermediate moment frame, which involves the detailing of reinforcement. These include the use of spiral ties and placement of slab reinforcement as seen in *Figure 10*. One thing that does change in the redesign was the building weight. *Tables 5 and 6* show the loading for seismic and the total building weight calculation. From these tables, it is found that the total weight of a concrete structure is 48,000 kips heavier, contrary to the expectedly lighter structure from Technical Report 2. This is a significant increase, considering that it's almost double the weight of the existing steel structure. Story shears become much higher from this increase in weight, especially since the R factor did not increase. The calculations result in a base shear of 8,394 kips (shown in *Table 7*), which is almost twice that of the existing steel structure. Figure 11 shows how this base shear is distributed to each story. For further seismic calculations, see Appendix C.

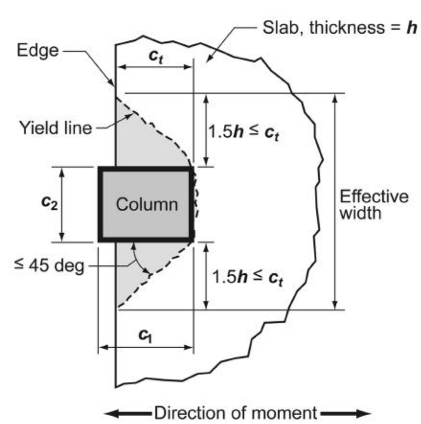
Floor Loading			
Floor	SF	Loading (psf)	Floor Weight (k)
Ground	95676	125.0	11960
1	172144	145.0	24961
2	100167	145.0	14524
3	68865	145.0	9985
4	68865	145.0	9985
5	68865	145.0	9985
6	68865	145.0	9985
Roof	68865	155.4	10702
			102088
Façade Loading			
Floor	Perimeter	Height	Weight on Floor
Ground	1308	8.00	398
1	1681	14.50	926
2	1276	13.00	630
3	1102	13.00	544
4	1102	13.00	544
5	1102	13.00	544
6	1102	13.25	555
Roof	1102	6.75	283
			4424
		Floor Load	102088
		Total Weight	106512

***Table 5**: Building Weight Calculation*

Typical Floor Loading	
Component	Weight (psf)
Concrete	125.00
MEP & Misc.	20.00
	145.00
Roof Loading	
Component	Weight (psf)
Concrete	125.00
Rigid Insulation	2.00
MEP & Misc.	20
Snow (20% Seismic)	8.4
	155.40

***Above – Table 6:** Seismic Floor Loading*

***Below - Figure 10**: Intermediate Frame Slab Detailing (ACI318-08)*



Seismic Loads							
Floor	Weight (k)	Height (ft)	$w_x h_x^k$	C_{vx}	F_x (k)	V_x (k)	M (ft-k)
Roof	6008	97.5	1758311	0.158	1327	1327	129425
6	10529	84	2561495	0.230	1934	3261	162439
5	10529	71	2079445	0.187	1570	4831	111461
4	10529	58	1618221	0.146	1222	6053	70857
3	10529	45	1181328	0.106	892	6945	40133
2	15154	32	1114072	0.100	841	7786	26914
1	25887	16	805732	0.072	608	8394	9733
Ground			11118604		8394		550963

***Table 7**: Story Shear Distribution*

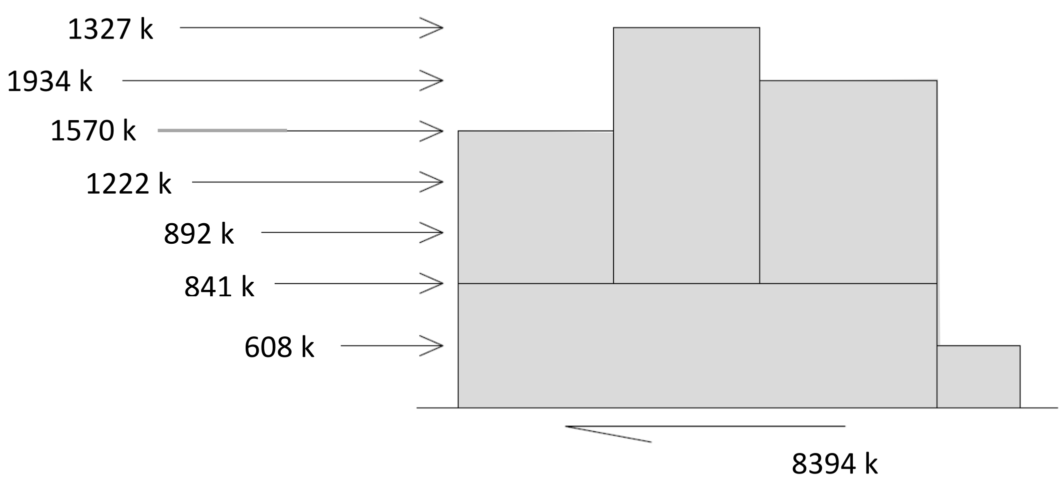


***Figure 11**: Seismic Story Shears*

Overturning Moment and Foundations

Overturning Moments			
Floor	Earthquake	Wind E/W	Wind N/S
Roof	129425	7101	6474
6	162439	11693	10661
5	111461	9316	8494
4	70857	7273	6631
3	40133	5356	4883
2	26914	5704	4904
1	9733	2728	2346
Ground	550963	49173	44393

***Table 8**: Wind & Seismic Overturning Moments*

Table 8 illustrates the overturning moment from wind and seismic. The seismic moment is twice that of steel, but the column to foundation reinforcement is detailed as to create a pin connection at that interface. Therefore, moment is not carried in the foundations. In that case, the foundations only take the axial load of the building. To stay under the 4000 psi max soil bearing stress, this requires minimum 76 in^2 spread footings.

GRAVITY REDESIGN

Slab and Drop Panel Design

By fully understanding the previous load conditions, it becomes evident that the flat slab system could effectively meet the design criteria. To establish preliminary sizes for the slab and drop panels, the CRSI Design Handbook was used for the critical conditions. Using a superimposed load of 120 psf pushes the design load to 200 psf in the handbook. After taking an initial guess of a 10 inch slab, and comparing that with the largest bay spans of 30 feet, the controlling slab system could be determined. These criteria produce 10 foot width by 8.25 inch depth drop panels. The next step was to create a more detailed slab design using actual loads in spSlab. Using this program, a typical column line was designed through the building (shown in *Figure 12*) to determine reinforcement, deflections, shear and moment capacities. All shear and moment fell within the slab's limitations, and the resulting reinforcement for middle and column strips is shown in *Figures 14 and 15* respectively. The reinforcement in these details is considered to control over those provided by CRSI since spSlab also accounts for moment from lateral load. One may also note that the reinforcement in *Figures 14 and 15* have continuous bars running in the top of the slab. This is due to the loading on various span lengths causing other spans to have a reversal of forces and experience tension in the top face of the slab. The largest live load deflection found in the slab is 0.165 inches, which is much less than 0.878 inches for L/360. The same goes for total deflection of 0.284, which was much less than 1.317 (L/240).

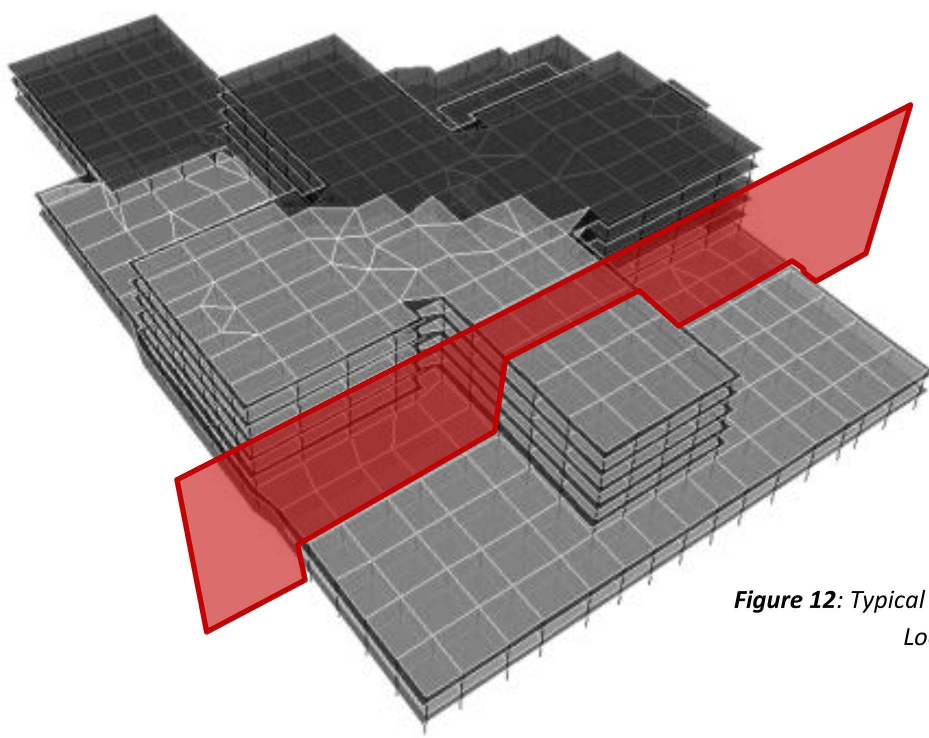


***Figure 12**: Typical Column Line Location*

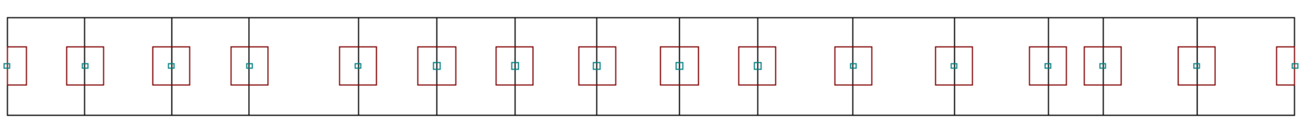


***Figure 13**: Plan View of Typical Column Line*

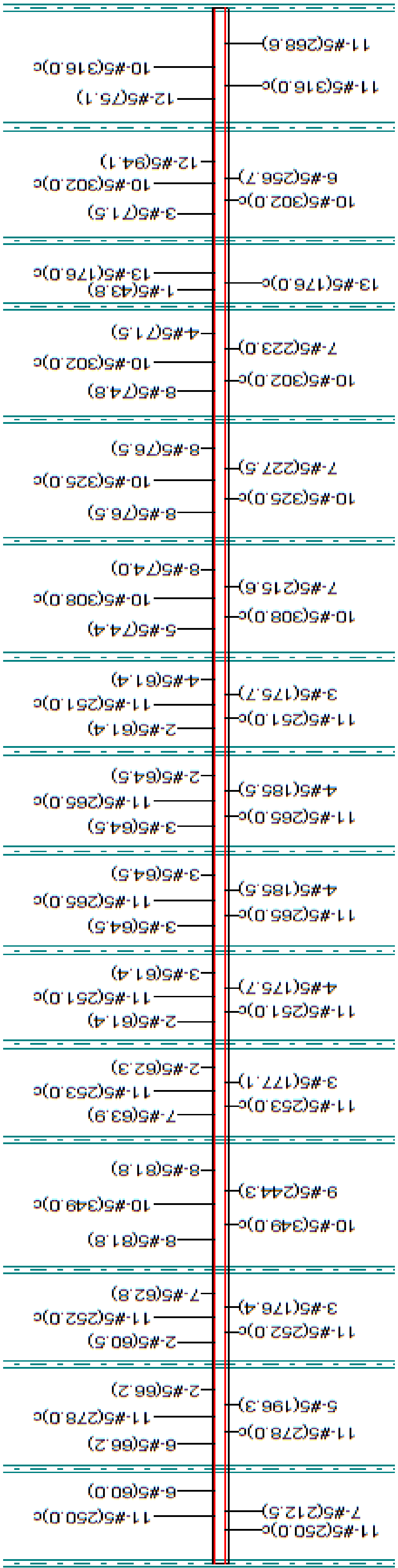


Figure 14: *Middle Strip Flexural Reinforcement*

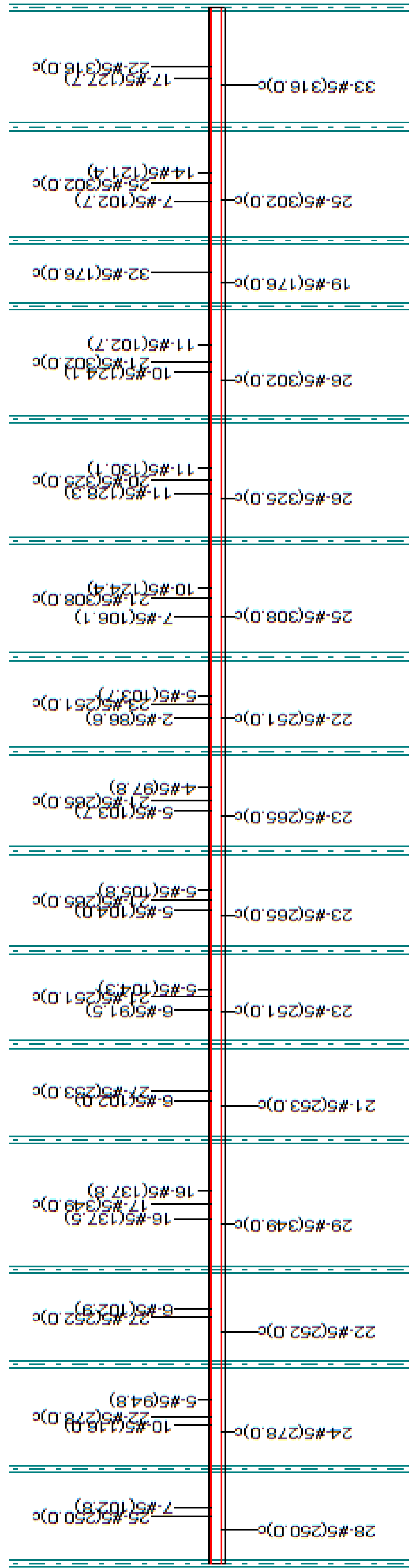


Figure 15: *Column Strip Flexural Reinforcement*

Column Design

The next step in the gravity analysis was to determine the load relationship between the exterior and interior columns. From pure axial calculations it was determined that the façade load makes up for the lower tributary area of an exterior column so that the interior and exterior columns are essentially the same size. In addition, it was also found that CRSI called for larger column sections in each case, so these were still chosen as the controlling design. This process is shown in further detail in Appendix D. From here, the columns were modeled in spColumn and designing became an iterative process between the two programs and spot checks by hand calculation (shown in Appendix H). In general, pure gravity loading did not control the size of the columns. Rather, it was a combination of gravity and lateral loads under the load case 5,

1.2D + 1.0E + L + 0.2S from ASCE7-10 that determined the size of the columns.

LATERAL REDESIGN

Slab and Drop Panel Design

To start the lateral design, portal method was performed by hand through the column line mentioned earlier in *Figure 12* (hand calculations in Appendix G). By applying the new seismic loads mentioned earlier in this report, the moment in the slabs could be determined and then applied in spSlab. In turn, this designed the reinforcement displayed in the gravity section. With the increased load from lateral, the slab thickness was increased to 11 inches to help transfer lateral moment.

Column Design

Starting with the columns designed for pure axial, the preliminary sizes were plugged into spColumn with the moments from the portal method. This became an iterative process of increasing the column cross section until a column size could pass and deliver reinforcement design. The new column sizes were then plugged into ETABS and exposed to seismic loading in both the North/South and East/West directions. At this point, the primary concern was controlling story drifts. Again this became an iterative process between spColumn and ETABS until drifts fell within the acceptable limit, which will be illustrated later in the report.

Although all floors met the drift criteria, the columns were still failing due to concentrated moment at the second story. This is due to the drop in square footage at the second story (illustrated by *Figure 16*) because the story shear from seismic is only carried by 156 columns at the second floor as compared to 351 at the first floor.

***Figure 16**: Floor Plan Reduction at Second Story*

Interestingly enough, simply increasing the column cross section wouldn't fix the problem. Instead, since an increased cross section also means increased stiffness, these columns only continued to take more load to the point of failure. To help take some of the forces at the second story, it was determined that the installation of shear walls would be the most effective method. Once the second story forces were being distributed effectively, drift values were still high at the administration wing of the building, which is the section of purple columns in the figure below. This is due to the fact that torsion is the first mode of the building. Since this "arm" is the only asymmetric piece of the floor plan and also furthest from the center of rigidity, it experiences the highest displacements. To account for this, these bays required increased stiffness, and therefore larger cross section than other comparable columns. The final column layout can be seen in *Figure 17* on the second story floor plan below.

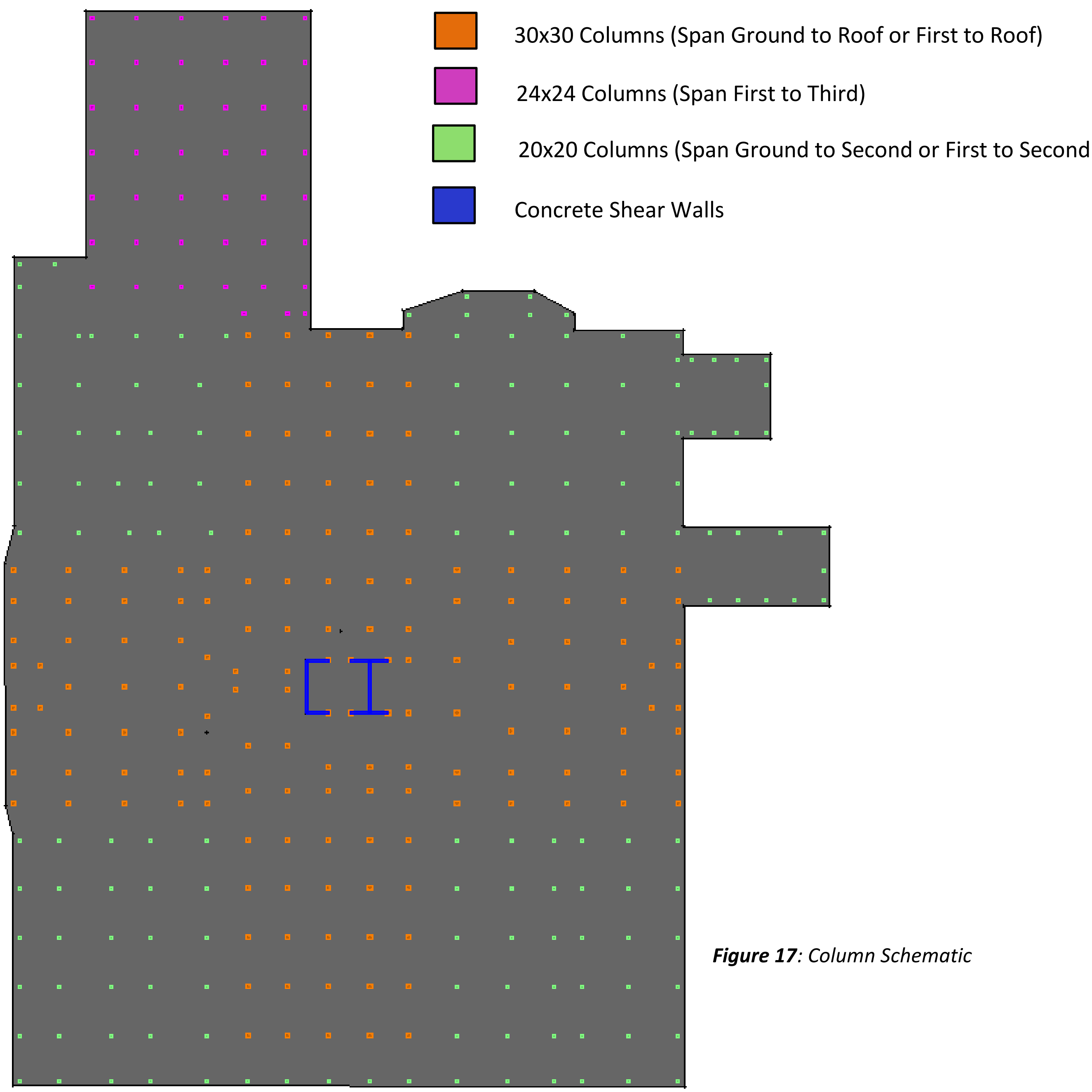


***Figure 17**: Column Schematic*

Shear Wall Design

Given the huge spike in moment at the second story, the most cost effective solution to keep the columns from failing was to add stiffness in the form of shear walls. To actually withstand the concentration of moment at the second story, 156 columns would have had to be made larger than 36x36 columns. Considering the other stories of these columns saw very little moment, it would not be the best design decision to upsize the entire section. Therefore, it was deemed the best solution to construct 18" thick, 6000 psi shear walls around the elevator shafts, which wouldn't interfere with the architectural layout. Refer to *Figure 18* for the shear wall layout. The addition of shear walls then changed the lateral resistance system to a dual system, which also had an R of 3. To meet the criteria of this system, however, the moment frames were still required to carry at least 25% of the lateral force. A relative stiffness check, shown in *Table 9*, was then carried out to verify this criterion. A spot check of the shear walls can be found in Appendix H.

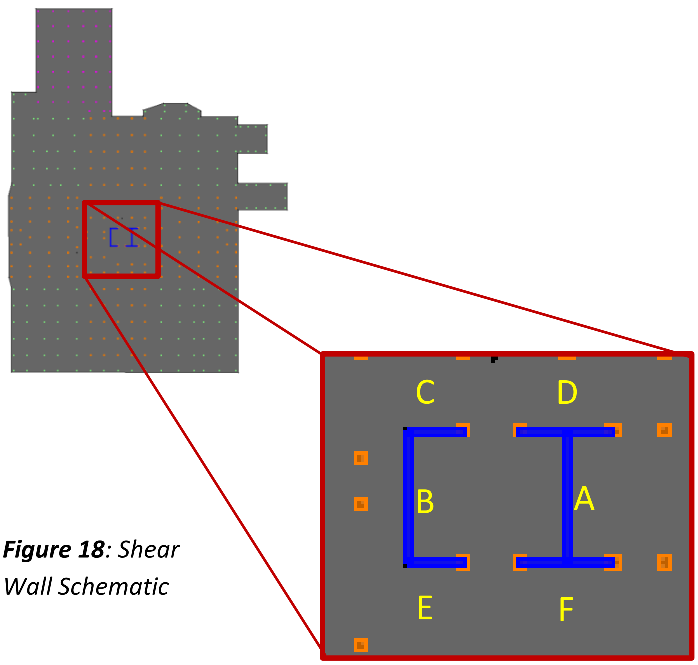


Figure 18*: Shear Wall Schematic*

Relative Stiffness	
North/South Direction	
A	0.144
B	0.155
	29.90%
East/West Direction	
D	0.077
C	0.055
F	0.067
E	0.057
	25.60%

Table 9*: Shear Wall Relative Stiffness*

Story Drifts

Drift analysis is a true test of the lateral effectiveness of a system. Design for drift is crucial to prevent damage to the structure or façade of a building. For wind, drift values should be less that L/400, and for seismic drift, the values for an occupancy category IV should fall underneath $0.01h_{sx}$. *Tables 10 and 11* show the ETABS story drifts compared to the accepted values. All drifts for both wind and seismic fall within the acceptable limits for this structure.

Seismic Story Drifts								
Story	Load	hsx	ETABS Drift X	ETABS Drift Y	Drift X (in)	Drift Y (in)	Allowable	Pass?
Roof	EQx	162	0.004541	0.001415	1.23	0.38	1.62	Yes
Roof	EQy	162	0.001369	0.002426	0.37	0.66	1.62	Yes
6	EQx	156	0.004993	0.001664	1.30	0.43	1.56	Yes
6	EQy	156	0.001628	0.00286	0.42	0.74	1.56	Yes
5	EQx	156	0.005257	0.001855	1.37	0.48	1.56	Yes
5	EQy	156	0.001821	0.003191	0.47	0.83	1.56	Yes
4	EQx	156	0.005039	0.001755	1.31	0.46	1.56	Yes
4	EQy	156	0.00172	0.003134	0.45	0.81	1.56	Yes
3	EQx	156	0.00576	0.00149	1.50	0.39	1.56	Yes
3	EQy	156	0.002393	0.002678	0.62	0.70	1.56	Yes
2	EQx	156	0.005363	0.002279	1.39	0.59	1.56	Yes
2	EQy	156	0.001512	0.002283	0.39	0.59	1.56	Yes

***Table 10**: Seismic Drifts*

WIND STORY DRIFTS					
STORY	ETABS DriftX	ETABS DriftY	DRIFT	ALLOWABLE	PASS?
7	0.000514		0.083	0.405	Yes
7		0.000415	0.067	0.405	Yes
6	0.000624		0.101	0.39	Yes
6		0.0005	0.081	0.39	Yes
5	0.000722		0.117	0.39	Yes
5		0.000579	0.094	0.39	Yes
4	0.000714		0.116	0.39	Yes
4		0.000591	0.096	0.39	Yes
3	0.000715		0.116	0.39	Yes
3		0.000532	0.086	0.39	Yes
2	0.000796		0.129	0.48	Yes
2		0.000582	0.094	0.48	Yes

***Table 11**: Wind Drifts*

ETABS MODEL

ETABS has served as an effective tool for moment frame design. By continuously checking results with hand calculations and other computer programs, one can arrive at very accurate results. There are some assumptions and decisions in modeling worth noting in relation to the analysis and design of the concrete flat slab system. For one, the columns that don't run all the way down to the ground floor are modeled as pinned supports at the first story, which is why there are no drift results at story one. Essentially, the first story is braced against lateral deflection by the 16' of soil between the ground and first floor. All concrete elements also account for cracking with columns using 0.7 Ig, beams using 0.35 Ig, and slabs using 0.25Ig. Thirdly, each individual bay is modeled as a shell element to account for the lateral moments carried into the slab. One final note is the assumption of a 5% accidental eccentricity when applying a seismic load. *Figures 19* illustrates the modeling of the redesigned system.

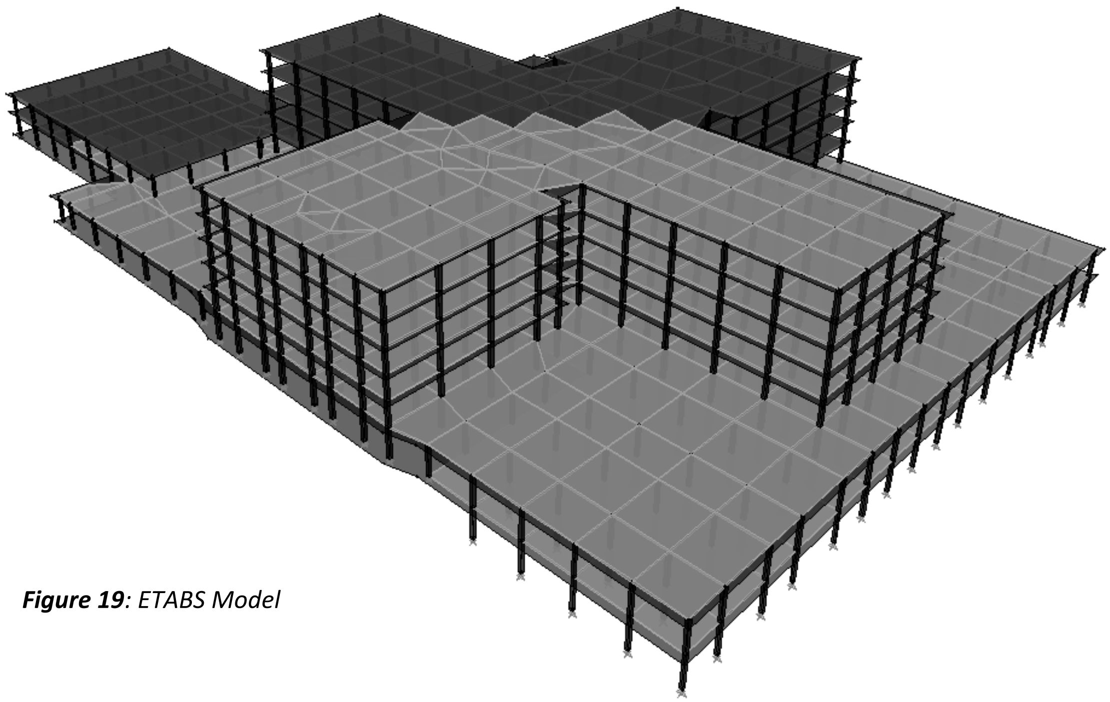


***Figure 19**: ETABS Model*

BREADTH TOPIC 1 – COST AND SCHEDULE

Cost Analysis

A preliminary study in Technical report 2 priced the flat slab system at a lower price than the existing steel structure. If this is the case, the flat slab system could be considered as an viable alternative to the steel construction. Cost is especially crucial in the design by ORMC. The hospital was given a specific budget with little to no variance. The detailed steel cost estimate, using RS Means, included the costs of columns and beams, fireproofing for the steel, floor decking, shear studs, and concrete floor slab. The total estimated cost came to $10,810,000, which is about 5% of the total building cost. Table 12 illustrates an overview of this analysis. For more detailed cost information, refer to Appendix E.

Steel System Costs			
Item	Quantity	Unit	Total
Columns & Beams	122771	L.F.	7032700
Metal Decking	823310	S.F.	1893613
Concrete	8259	C.Y.	187892
Shear Studs	130361	Ea.	243775
Fireproofing	946081.09	S.F.	1455160
			10813141

***Table 12**: Steel Construction Costs*

The redesigned flat slab system includes costs of columns, shear walls, flat slab and drop panels. With the cost of the concrete, reinforcement, placing, formwork, and finishing is included in the RS Means price. The total price of the system is estimated at $20,120,000 which is about 8% of the total building cost and almost double the cost of the steel structure. Of course there are still miscellaneous items that have not been accounted for in both systems such as steel connections for the existing system and the cost of freeze add mixtures in the concrete system, since construction spans over the winter months. Even without these additions however, this cost analysis gives a pretty good idea that the concrete flat slab system would be significantly more expensive, contrary to the estimate from the Technical Report 2. This alone would probably be enough to turn ORMC away from this system. An overview of these costs are outlined in Table 13, but for further detailed costs, refer to Appendix F.

Concrete System Costs			
Item	Quantity	Unit	Total
Columns	12682	C.Y.	5398917
Slab & Drops	31840.3	C.Y.	14504849
Shear Wall	-	-	219227
			20122993

***Table 13**: Concrete Construction*

Construction Schedule

A second important piece in choosing a structural system is the length of time required for construction. As the saying goes, "time is money," and most owners entering the construction phase, seek to complete the building as quick as possible, as cheap as possible while also maintaining a standard of quality. From the existing construction schedule provided by Orange Regional Medical Center, it is shown that construction for the entire steel structure spanned over 15 months and 9 days. This schedule can be found in Appendix F. By using the labor daily output values from RSMeans, a construction schedule was also derived for the concrete flat slab system. On a side note, for the comparison of schedules, this analysis only considers the scope of the structure timeline. Realistically, other trades would also be carried out during this time, but that will be outside of the purposes of this report.

An accurate construction schedule will have tasks overlap so that multiple projects are being carried out at the same time. This piece of the schedule required some assumptions as to how soon a slab could be started after the columns are finished. In most cases, the slab was started a couple weeks before the columns would reach their 28-day strength. This allowed time for formwork and rebar to be set before placing concrete for the slabs and loading the columns below. It was also determined that the slab for the first floor could be started on the same date as the columns on the ground floor since parts of the first story does not have the columns supporting. This is due to the ground level being partially below grade, and a smaller square footage than the first story. This same concept carries over to the drop in square footage at the second story; the second story columns can be started early into the second floor slab construction, given that much of the second story slab is roof. At the completion of the analysis in Microsoft Project, it was found that the concrete system will take 6 weeks longer to construct than the existing steel system. This is expected of a concrete structure, but ultimately this would the decision of the owner as to how critical the 6 weeks is. *Figure 20* shows the schedule of the various flat slab tasks.

	Task Name	Duration	Start	Finish
1	F/R/P Columns - Ground	41 days	Mon 7/28/08	Mon 9/22/08
2	F/R/P Slab and Drops - L1	124 days	Mon 7/28/08	Thu 1/15/09
3	F/R/P Columns - L1	76 days	Sun 9/7/08	Fri 12/19/08
4	F/R/P Slab and Drops - L2	134 days	Sun 10/12/08	Wed 4/15/09
5	F/R/P Columns - L2	46 days	Sun 11/16/08	Fri 1/16/09
6	F/R/P Slab and Drops - L3	68 days	Sun 12/21/08	Tue 3/24/09
7	F/R/P Columns - L3	35.6 days	Sun 2/8/09	Fri 3/27/09
8	F/R/P and Drops - L4	59 days	Sun 3/8/09	Wed 5/27/09
9	F/R/P Columns - L4	36 days	Sun 4/12/09	Fri 5/29/09
10	F/R/P Slab and Drops - L5	59 days	Sun 5/10/09	Wed 7/29/09
11	F/R/P Columns - L5	36 days	Sun 6/14/09	Fri 7/31/09
12	F/R/P Slab and Drops - L6	59 days	Sun 7/12/09	Wed 9/30/09
13	F/R/P Columns - L6	37 days	Sun 8/16/09	Mon 10/5/09
14	F/R/P Slab and Drops - Roof	59 days	Sun 9/13/09	Wed 12/2/09
15	F/R/P Shear Walls	50 days	Sun 10/11/09	Thu 12/17/09

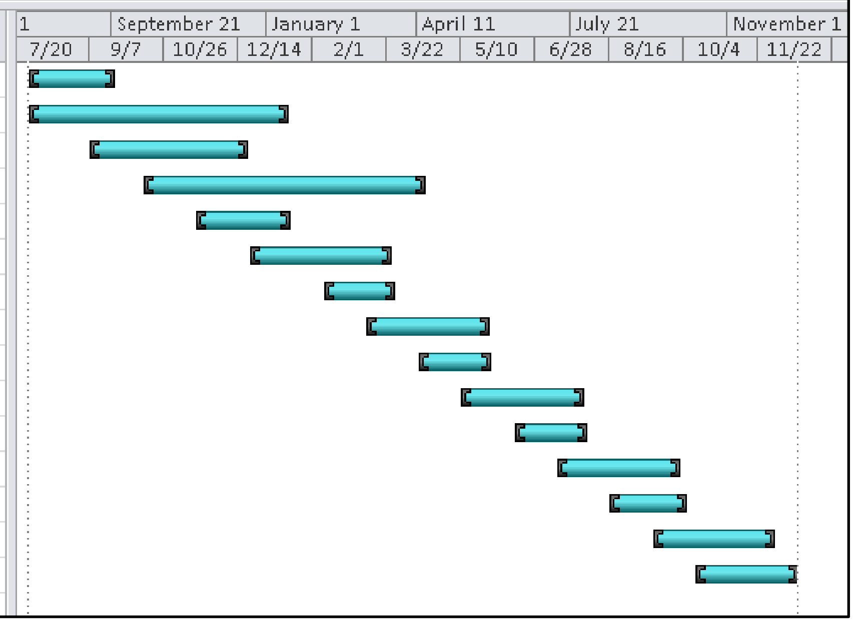


***Figure 20**: Flat Slab Construction Schedule*

BREADTH TOPIC 2 – ARCHITECTURAL REDESIGN

Medical Department Layout

When designing an architectural floor plan, there are so many criteria to keep in mind that some other areas may be overlooked. Near the top of that list however, is functionality for the building occupants. HBE has achieved a very functional layout for patients and employees, but even still there are areas that could benefit from a second look. The original design had to be rotated 90 degrees from its intended position due to site issues. This is something the architects could not have anticipated, and as a result, the emergency room entrance is located on the opposite corner of the site from the main entrance (shown *in Figure 21*). The current design causes confusion in what is often a frantic situation. For this reason, the redesign will focus on moving the emergency room to a location closer to the main entrance.

***Figure 21**: E.R. Entrance vs. Main Highway Entrance*

Now looking inside the building, it is always important to think about the flow of patients and employees. Three particular departments that should always be close to one another are the Emergency Room, Operating Room, and the Intensive Care Unit. If a patient needs to be rushed to either of these departments, time shouldn't be wasted having to wait for the elevator. As a result, the redesign moves the Intensive Care Unit (ICU) down to the first story, where it neighbors the Emergency Room (ER) and Operating Room (OR). To achieve this relocation, departments on the first floor that don't require patient or employee urgency are moved up to the third floor where ICU was originally located. Therefore, the redesign is able to achieve a more logical flow by only changing the first and third floor layouts. During the redesign, all departments maintained roughly the same square footage while attempting to work around the existing circulatory space. The administrative departments were left in the same location since this section of the building follows separate fire codes. *Figures 23 through 25* on the next pages show the original and redesigned floor plans of the first and third floors.

- Emergency Room
- Operating Room
- Intensive Care Unit
- Same Day Surgery
- P.A.C.U.
- Financial Advocates
- Outpatient Infusion Center
- Administration
- Diagnostic Imaging
- Invasive Radiation
- Gift Shops
- Employee Space
- Cardiac Rehabilitation
- Endoscopy
- Non-invasive Cardiology
- Lobb

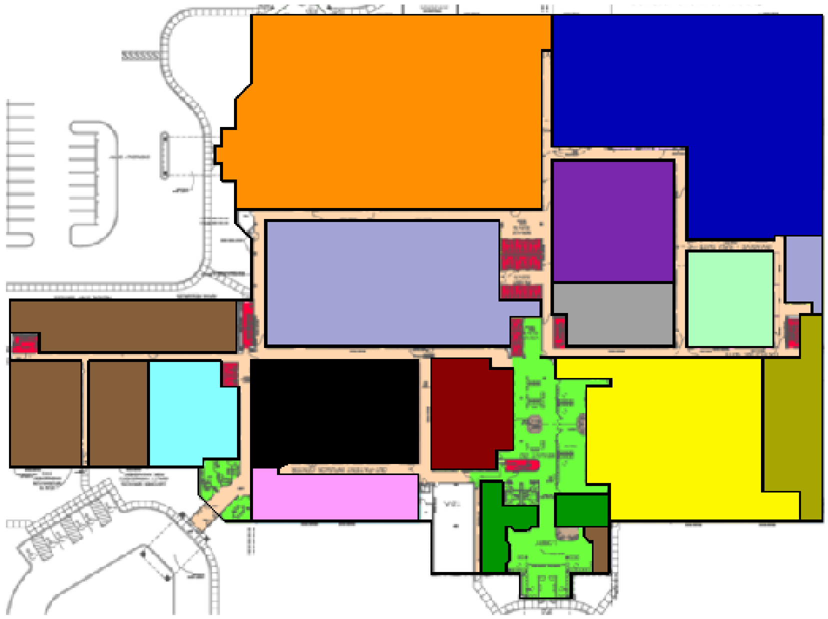


Figure 22: *Original Department Layout – Story 1*

Figure 23: *Redesigned Department Layout – Story 1*

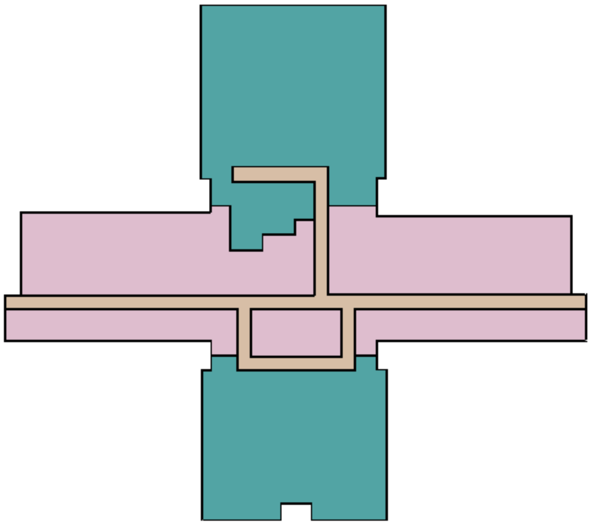


Figure 24: Original Department Layout – Story 3

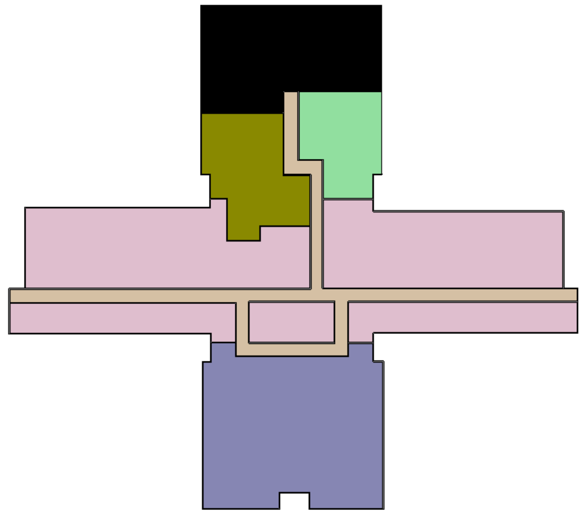


Figure 25: Redesigned Department Layout – Story 3

Intensive Care Unit **Progressive Care Unit** **Diagnostic Imaging**

Invasive Radiation **Non-invasive Cardiology** **Endoscopy**

Green Roof Addition

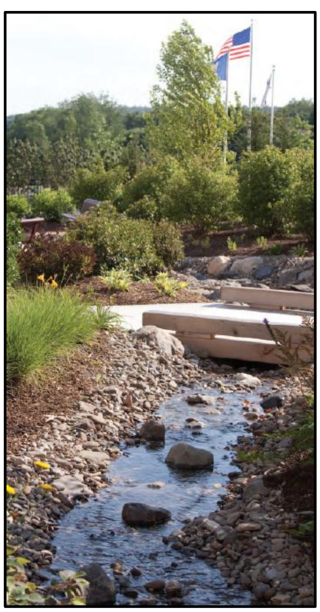


Figure 26: Healing Garden

As mentioned earlier in the report, ORMC strived for patient comfort and quick recovery. Along with the hotel-like patient rooms and carpeted hallways mentioned earlier, ORMC also planted a healing garden, shown in *Figure 26,* next to the lobby entrance. Although this is a great amenity, it doesn't get used as often as it would if it was located where the patients are actually healing. Since the patient rooms are all on the second story and above, why not move the healing garden to the second story roof and make it patient accessible. Currently, the patient rooms look out onto a gravel roof with mechanical rooms and pipes jutting out of the roofing. This seems like a waste, considering that ORMC made it a point to design higher quality rooms, but yet, the rooms have an eyesore right outside the window. Placing a green roof on the second story will allow all patient rooms to look down on a beautiful garden, therefore providing further comfort. In addition, there is the added bonus of thermal properties that go along with a green roof as well as extended life of the roof.

The proposed location can be seen in Figure 27, highlighted in green, with the existing location highlighted in yellow. This would allow patients from Oncology, Respiratory Therapy, Progressive Care Unit, Intensive Care Unit, Physical and Occupational Therapy, Medical/Surgical Units, and the Maternity to have views of the garden from their rooms. The relocation also has its disadvantages, however. Looking at the green roof from a structural prospective raises some concerns. As mentioned in the structural depth study, all of the full height columns had to be upsized to account for the concentration of moment forces at the second story. Since green roofs are typically heavy systems, this would add further shear at the second story level due to seismic forces. Essentially, the addition of a green roof would be a nice feature, especially since it would be more accessible to the patients, but with the added costs of the green roof alone and any structural retrofitting, it would ultimately be a decision made by the owner. The owner would determine the value of such a garden in light of the structural consequences.

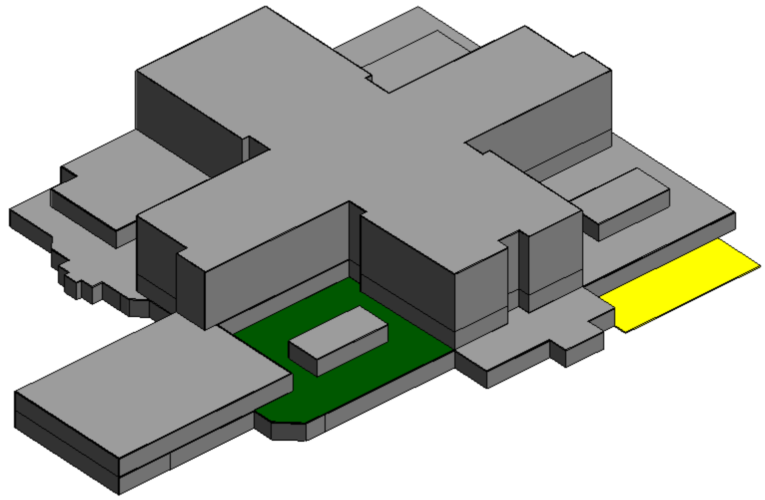


Figure 27*: New Garden Location*

CONCLUSIONS

Given the various loads that act on this hospital, it was found that a concrete flat slab system would be capable of withstanding the predominant seismic load. However, geometry of the structure created force concentration at the second story which would require the addition of shear walls around the elevator shafts. This addition allows the structure to fall within all acceptable limits. The structure cannot be based solely on structural criteria, however. After analyzing cost and construction schedule of the redesigned system, it was found that a flat slab may not be the best method for Orange Regional Medical Center. The cost of the new structure came in at almost twice the original cost which is simply something that ORMC cannot consider on a limited budget. There was also an urgency to move into the new hospital, so a longer schedule would also be a downside especially when anticipating delays from a constructing a concrete system during the winter months. On the other hand, there are architectural changes that could have been done in the original structure that carry many benefits with them. Patient and employee flow and comfort are always of the utmost importance, and the new layout of departments and green roofs would achieve this. But overall, analysis shows that the existing composite steel structure with braced frames is the better solution for the needs of Orange Regional Medical Center.

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APPENDIX A: SNOW CALCULATIONS

APPENDIX A | SNOW CALCULATIONS 1 | RYAN BLATZ

DESIGN CRITERIA - ASCE 7-10

$C_e = 1.0$ (TABLE 7-2) LOWER SECTION - PARTIALLY EXPOSED
$C_e = 1.0$ (TABLE 7-2) UPPER SECTION - PARTIALLY EXPOSED

$C_t = 1.0$ (TABLE 7-3)

$I_s = 1.20$ (TABLE 1.5-2)

$p_g = CS$ (FIGURE 7-1) ⟶ $p_g = 50$ psf (FROM DRAWINGS)

$p_f = 0.7(1.0)(1.0)(1.20)(50) = 42$ psf

SNOW DRIFTS

$\gamma = 0.13(50) + 14 \leq 30$
$\gamma = 20.5$ pcf ≤ 30 ✓

• DRIFT ONTO FIFTH FLOOR ROOF

$l_u = 117'$
$h_c = 13.5'$

$h_d = 0.43\sqrt[3]{117}\sqrt[4]{50+10} - 1.5 = 4.35'$
$w = 4h_d = 4(4.35) = 17.4'$

$P_d = (4.35)(20.5) = 89.18$ psf

• DRIFT ONTO SECOND FLOOR ROOF - NORTH/SOUTH

$l_u = 124.4'$

$h_d = 0.43\sqrt[3]{124.4}\sqrt[4]{50+10} - 1.5 = 4.47'$
$w = 4(4.47) = 17.9'$

$P_d = (4.47)(20.5) = 91.6$ psf

• DRIFT ONTO SECOND FLOOR ROOF - EAST/WEST

$l_u = 126.3'$

$h_d = 0.43\sqrt[3]{126.3}\sqrt[4]{50+10} - 1.5 = 4.5'$
$w = 4(4.5) = 18'$

$P_d = (4.5)(20.5) = 92.3$ psf

APPENDIX B: WIND CALCULATIONS

APPENDIX B | WIND CALCULATIONS 1 | RYAN BLATZ

DESIGN CRITERIA – ASCE 7-10

BASIC WIND SPEED (FIGURE 26.5-1B): $V = 120$ mph

RISK FACTOR (TABLE 1.5.1): IV ESSENTIAL FACILITY

WIND DIRECTIONALITY FACTOR (TABLE 26.6-1): $K_d = 0.85$

EXPOSURE CATEGORY (SECTION 26.7.3): EXPOSURE C

TOPOGRAPHIC FACTOR (SECTION 26.8): DOES NOT APPLY, $K_{zt} = 1.0$

GUST FACTOR: SEE ATTACHED CALCULATIONS

• RIGIDITY CALCULATION

$$L_{eff} = \frac{16(488') + 32(359') + 45(359') + 58(359') + 71(249') + 84(145') + 97.5(145')}{16 + 32 + 45 + 58 + 71 + 84 + 97.5}$$

$L_{eff} = 248.5'(4) = 994' >> 97.5' \longrightarrow$ CALCULATE n USING SECTION 26.9.3

$n_a = 75/h = 75/97.5 = 0.769$ Hz < 1.0 Hz $\therefore$ STRUCTURE NOT CONSIDERED RIGID

$g_Q = 3.4 \qquad g_v = 3.4 \qquad g_R = \sqrt{2\ln(3600(.769))} + \dfrac{0.577}{\sqrt{2\ln(3600(.769))}}$

$g_R = 4.13$

1) GUST CALCULATION – EAST/WEST BOTTOM SECTION

$\bar{b} = 0.65 \qquad \bar{\alpha} = 1/6.5 = 0.154$

$\bar{Z} = 0.6h = 0.6(97.5) = 58.5 > 15$ ✓

$\bar{V}_{\bar{z}} = 0.65\left(\dfrac{58.5}{33}\right)^{1/6.5}\left(\dfrac{88}{60}\right)(120) = 124.93$

$\ell = 500$ ft $\qquad \bar{\epsilon} = 1/5.0$

$L_{\bar{z}} = 500\left(\dfrac{58.5}{33}\right)^{1/5} = 560.66$

$R_n = \dfrac{7.47(3.45)}{(1+10.3(3.45))^{5/3}} = 0.064$

$N_1 = \dfrac{0.769(560.66)}{124.93} = 3.45$

$R_h = \dfrac{1}{2.76} - \dfrac{1}{2(2.76)^2}\left(1-e^{-2(2.76)}\right) = 0.297$

$\eta_h = \dfrac{4.6(.769)(97.5)}{124.93} = 2.76$

$R_B = \dfrac{1}{16.18} - \dfrac{1}{2(16.18)^2}\left(1-e^{-2(16.18)}\right) = 0.060$

$\eta_B = \dfrac{4.6(.769)(571.5)}{124.93} = 16.18$

$R_L = \dfrac{1}{46.26} - \dfrac{1}{2(46.26)^2}\left(1-e^{-2(46.26)}\right) = 0.021$

$\eta_L = \dfrac{15.4(.769)(488)}{124.93} = 46.26$

APPENDIX B: WIND CALCULATIONS

APPENDIX B | WIND CALCULATIONS 2 | RYAN BLATZ

$\beta = 1.0\%$ AS RECOMMENDED IN ASCE7-10, pg. 521

$$R = \sqrt{(1/.01)(.064)(.297)(.06)(.53+.47(.021))} = 0.248$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{571.5+97.5}{560.66}\right)^{0.63}}} = 0.766$$

$$I_{\bar{z}} = 0.2\left(\frac{33}{58.5}\right)^{1/6} = 0.182 \qquad c = 0.20 \quad \text{TABLE 26.9-1}$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.766)^2+(4.13)^2(.248)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.841}$$

2) <u>GUST CALCULATION</u> - EAST/WEST TOP SECTION

- ALL CALCULATIONS NOT SHOWN ARE THE SAME AS PREVIOUS SECTION

$$R_B = \frac{1}{11.23} - \frac{1}{2(11.23)^2}\left(1-e^{-2(11.23)}\right) = 0.085 \qquad \eta_B = \frac{4.6(.769)(396.5)}{124.93} = 11.23$$

$$R_L = \frac{1}{34.03} - \frac{1}{2(34.03)^2}\left(1-e^{-2(34.03)}\right) = 0.029 \qquad \eta_L = \frac{15.4(.769)(359)}{124.93} = 34.03$$

$$R = \sqrt{(1/.01)(.064)(.297)(.085)(.53+.47(.029))} = 0.296$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{396.5+97.5}{560.66}\right)^{0.63}}} = 0.775$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.775)^2+(4.13)^2(.296)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.865} \quad \text{CONTROLS}$$

3) <u>GUST CALCULATION</u> - NORTH/SOUTH BOTTOM SECTION

$$R_B = \frac{1}{13.82} - \frac{1}{2(13.82)^2}\left(1-e^{-2(13.82)}\right) = 0.070 \qquad \eta_B = \frac{4.6(.769)(488)}{124.93} = 13.82$$

$$R_L = \frac{1}{54.17} - \frac{1}{2(54.17)^2}\left(1-e^{-2(54.17)}\right) = 0.018 \qquad \eta_L = \frac{15.4(.769)(571.5)}{124.93} = 54.17$$

$$R = \sqrt{(1/.01)(.064)(.297)(.07)(.53+.47(.018))} = 0.268$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{488+97.5}{560.66}\right)^{0.63}}} = 0.779$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.779)^2+(4.13)^2(.268)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.851}$$

APPENDIX B: WIND CALCULATIONS

APPENDIX B | WIND CALCULATIONS 3 | RYAN BLATZ

4) GUST CALCULATION - NORTH/SOUTH TOP SECTION

$$R_B = \frac{1}{10.17} - \frac{1}{2(10.17)^2}\left(1-e^{-2(10.17)}\right) = 0.093 \qquad \eta_B = \frac{4.6(.769)(357)}{124.93} = 10.17$$

$$R_L = \frac{1}{37.59} - \frac{1}{2(37.59)^2}\left(1-e^{-2(37.59)}\right) = 0.026 \qquad \eta_L = \frac{15.4(396.5)(.769)}{124.93} = 37.59$$

$$R = \sqrt{(1/.01)(.064)(.277)(.093)(.53+.47(.026))} = 0.310$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{359+97.5}{560.66}\right)^{0.63}}} = 0.802$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.802)^2+(4.13)^2(.31)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.871}$$ CONTROLS

MAIN WIND FORCE RESISTING SYSTEM (MWFRS) - DIRECTIONAL PROCEDURE

ENCLOSURE CLASSIFICATION: ENCLOSED , $GC_{pi} = \pm 0.18$ • DO NOT NEED

WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.5$ EAST/WEST — NORTH/SOUTH: $C_p = -0.47$ BOTTOM, $C_p = -0.48$ TOP

SIDE WALL: $C_p = -0.7$

• THE REMAINDER IS CALCULATED USING AN EXCEL SPREADSHEET ; SEE ATTACHED

CASE I (P_{WX} + P_{LX})			
1)	*E/W-DIRECTION*	F (k)	
	ROOF	116.56	
	STORY 6	225.03	
	STORY 5	215.44	
	STORY 4	209.64	
	STORY 3	285.48	
	STORY 2	313.55	
2)	*N/S-DIRECTION*		
	ROOF	106.24	
	STORY 6	203.65	
	STORY 5	196.29	
	STORY 4	191.02	
	STORY 3	185.20	
	STORY 2	269.57	
DEFLECTIONS			
Story	Load	UX	UY
ROOF	XCASE1	0.4117	-0.0032
ROOF	YCASE1	0	0.5179
STORY6	XCASE1	0.3724	-0.0019
STORY6	YCASE1	0.0012	0.4628
STORY5	XCASE1	0.3189	-0.0007
STORY5	YCASE1	0.0023	0.3899
STORY4	XCASE1	0.2514	0.0004
STORY4	YCASE1	0.0034	0.3
STORY3	XCASE1	0.1664	0.0009
STORY3	YCASE1	-0.0025	0.1977
STORY2	XCASE1	0.0936	0.0001
STORY2	YCASE1	0.0007	0.1046

CASE II (.75(P_{WX}+P_{LX})B_X(±.15B_X))				
3)	*E/W-DIRECTION*	F (k)	B_x (in)	M_T (k-in)
	ROOF	87.4	4759.25	-62409
	STORY 6	168.8	4759.25	120486
	STORY 5	161.6	4759.25	115347
	STORY 4	157.2	4759.25	112247
	STORY 3	214.1	6684.375	-214677
	STORY 2	235.2	6857.375	241892
4)	*N/S-DIRECTION*	F (k)	B_x (in)	M_T (k-in)
	ROOF	79.7	4307	51477
	STORY 6	152.7	4307	98676
	STORY 5	147.2	4307	95111
	STORY 4	143.3	4307	92554
	STORY 3	138.9	4307	89737
	STORY 2	202.2	5855	177564
DEFLECTIONS				
Story		Load	UX	UY
ROOF		YCASE2	0.0059	0.3955
ROOF		XCASE2	0.3103	0.0005
STORY6		YCASE2	0.0078	0.3532
STORY6		XCASE2	0.2812	0.0012
STORY5		YCASE2	0.0096	0.2976
STORY5		XCASE2	0.2415	0.0017
STORY4		YCASE2	0.0112	0.2292
STORY4		XCASE2	0.1914	0.002
STORY3		YCASE2	-0.0063	0.1507
STORY3		XCASE2	0.1231	0.0015
STORY2		YCASE2	0.0026	0.0786
STORY2		XCASE2	0.0713	0.0001

CASE III (.75PWX + .75PLX)

5)	E/W-DIRECTION	N/S-DIRECTION
ROOF	87.4	79.7
STORY 6	168.8	152.7
STORY 5	161.6	147.2
STORY 4	157.2	143.3
STORY 3	214.1	138.9
STORY 2	235.2	202.2

DEFLECTIONS

Story	Load	UX	UY
ROOF	CASE3	0.3088	0.3861
ROOF	CASE3	0.2802	0.3457
STORY5	CASE3	0.2409	0.2919
STORY4	CASE3	0.191	0.2253
STORY3	CASE3	0.1229	0.149
STORY2	CASE3	0.0708	0.0785

CASE IV

6) E/W-DIRECTION	F (k)	B_X (in)	N/S-DIRECTION	F (k)	B_Y (in)	M_T (k-in)
ROOF	65.6	4759.25	ROOF	-59.8	4307	-85490.3
STORY 6	126.7	4759.25	STORY 6	114.7	4307	16372.5
STORY 5	121.3	4759.25	STORY 5	110.5	4307	157983.9
STORY 4	118.0	4759.25	STORY 4	107.5	4307	153737
STORY 3	160.7	6684.375	STORY 3	-104.3	4307	-93788.6
STORY 2	176.5	6857.375	STORY 2	151.8	5855	314871.9

DEFLECTIONS

Story	Load	UX	UY
ROOF	CASE4	0.2377	0.1302
STORY6	CASE4	0.2167	0.1295
STORY5	CASE4	0.1879	0.1152
STORY4	CASE4	0.1511	0.0881
STORY3	CASE4	0.0884	0.0514
STORY2	CASE4	0.0553	0.0288

EARTHQUAKE

7) E/W-DIRECTION	F (k)
ROOF	744.3
STORY 6	895.3
STORY 5	741.4
STORY 4	781.6
STORY 3	573.5
STORY 2	541.1

8) N/S-DIRECTION	F (k)
ROOF	744.3
STORY 6	895.3
STORY 5	741.4
STORY 4	781.6
STORY 3	573.5
STORY 2	541.1

DEFLECTIONS

Story	Load	UX	UY
ROOF	EWXQUAKE	1.6427	-0.0259
ROOF	NSYQUAKE	0.0001	2.3692
STORY6	EWXQUAKE	1.4456	-0.018
STORY6	NSYQUAKE	0.0082	2.0666
STORY5	EWXQUAKE	1.1968	-0.0105
STORY5	NSYQUAKE	0.0159	1.6939
STORY4	EWXQUAKE	0.9043	-0.0035
STORY4	NSYQUAKE	0.0226	1.2654
STORY3	EWXQUAKE	0.5614	0.0014
STORY3	NSYQUAKE	-0.0158	0.7939
STORY2	EWXQUAKE	0.2962	0.0004
STORY2	NSYQUAKE	0.0043	0.3928

APPENDIX C: SEISMIC CALCULATIONS

DESIGN CRITERIA: ASCE 7-10

SITE CLASS C (FROM GEOTECHNICAL REPORT)

RISK CATEGORY (TABLE 1.5.1): IV ESSENTIAL FACILITY

IMPORTANCE FACTOR (TABLE 1.5-2): $I_e = 1.5$

$S_s = 0.20$ (FIGURE 22-1) $\quad S_1 = 0.06$ (FIGURE 22-2)

$F_a = 1.2$ (TABLE 11.4-1) $\quad F_v = 1.7$ (TABLE 11.4-2)

$S_{MS} = (1.2)(0.2) = 0.24$ $\quad S_{M1} = (1.7)(0.06) = 0.102$

$S_{DS} = \frac{2}{3} S_{MS} = (\frac{2}{3})(0.24) = 0.16$ $\quad S_{D1} = \frac{2}{3} S_{M1} = (\frac{2}{3})(0.102) = 0.068$

SEISMIC DESIGN CATEGORY: A (TABLE 11.6-1), C (TABLE 11.6-2) } USE HIGHER CLASS C

RESPONSE MODIFICATION COEFFICIENT (TABLE 12.2-1): $R = 3$ • ORDINARY REINFORCED CONCRETE MOMENT FRAMES

EQUIVALENT LATERAL FORCE METHOD

$T_a = C_t h_n^{x} = (0.016)(97.5)^{0.9} = 0.987$ $\quad C_t = 0.016$ (TABLE 12.8-2), $x = 0.9$ (TABLE 12.8-2)

$C_s = \dfrac{S_{DS}}{(R/I_e)} = \dfrac{0.16}{(3/1.5)} = 0.08$

$V = C_s W = (0.08)(104{,}923) = 8{,}394$ KIPS

$F_x = C_{vx} V$ $\quad k = 1.24$ (SECTION 12.8.3)

$C_{vx} = \dfrac{w_x h_x^{k}}{\sum w_i h_i^{k}}$

APPENDIX D: COLUMN DESIGN

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL — With Drop Panels — No Beams

SQUARE INTERIOR PANEL
With Drop Panels(2) — No Beams

SPAN c-c $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	(3) Square Column Size (in.)	Square Column γ_f	Reinforcing Bars (E. W.) Column Strip (1) Top Ext.	+	Column Strip (1) Bottom	Column Strip (1) Top Int.	Middle Strip Bottom	Middle Strip Top Int.	Total Steel (psf)	Moments Edge (−) (ft-k)	Moments Bot. (+) (ft-k)	Moments Int. (−) (ft-k)	Interior Panel Factored Superimposed Load (psf)	(3) Square Column Size (in.)	Reinforcing Bars (E. W.) Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete (cu. ft / sq. ft)
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS																h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS							
25	100	4.25	8.33	12	0.802	12-#5	2	9-#6	14-#5	9-#5	9-#5	2.34	117.9	235.9	317.5	100	12	13-#5	9-#5	9-#5	9-#5	2.19	0.873
25	200	4.25	8.33	16	0.813	12-#5	4	9-#7	14-#6	11-#5	10-#5	2.85	158.4	316.8	426.5	200	19	12-#6	11-#5	9-#5	9-#5	2.53	0.873
25	300	6.25	8.33	18	0.675	12-#5	1	15-#6	20-#5	10-#6	12-#5	3.35	200.2	400.4	539.0	300	22	13-#6	14-#5	11-#5	10-#5	2.95	0.891
25	400	8.25	8.33	20	0.631	12-#5	1	11-#8	12-#7	9-#7	10-#6	4.07	241.3	482.5	649.5	400	24	14-#6	9-#7	13-#5	11-#5	3.47	0.910
25	500	8.25	10.00	22	0.646	13-#5	1	17-#7	18-#6	11-#7	9-#7	4.77	283.0	592.9	761.9	500	25	12-#7	11-#7	11-#6	10-#6	4.14	0.943
25	600	10.25	10.00	24	0.629	13-#5	1	17-#8	26-#5	9-#9	19-#5	5.93	324.4	728.2	873.3	600	25	13-#7	11-#8	10-#7	9-#7	5.10	0.970
26	100	4.25	8.67	12	0.833	12-#5	3	10-#6	16-#5	10-#5	10-#5	2.48	133.1	266.2	358.3	100	12	15-#5	10-#5	10-#5	10-#5	2.34	0.873
26	200	6.25	8.67	16	0.722	12-#5	3	19-#5	18-#5	9-#6	11-#5	2.94	179.5	359.0	483.3	200	19	12-#6	9-#6	10-#5	10-#5	2.66	0.891
26	300	6.25	8.67	19	0.704	12-#5	2	17-#6	12-#7	16-#5	13-#5	3.61	225.7	451.3	607.5	300	22	15-#6	16-#5	12-#5	11-#5	3.20	0.891
26	400	8.25	8.67	20	0.640	12-#5	2	12-#8	13-#7	19-#5	16-#5	4.28	273.1	546.1	735.1	400	24	12-#7	19-#5	15-#5	9-#6	3.72	0.910
26	500	8.25	10.40	23	0.753	14-#5	5	12-#9	12-#8	12-#7	10-#7	5.12	318.2	636.4	856.7	500	26	26-#5	12-#7	17-#5	15-#5	4.31	0.943
26	600	10.25	10.40	24	0.629	15-#5	0	18-#8	12-#8	12-#8	9-#8	6.07	366.1	789.3	985.7	600	26	14-#7	15-#7	20-#5	10-#7	5.15	0.970
27	100	6.25	9.00	12	0.749	12-#5	2	16-#5	15-#5	11-#5	10-#5	2.47	150.1	300.1	404.0	100	12	14-#5	11-#5	10-#5	10-#5	2.26	0.891
27	200	6.25	9.00	16	0.766	12-#5	4	9-#8	15-#6	10-#6	12-#5	3.19	201.9	403.8	543.6	200	19	14-#6	14-#5	11-#5	10-#5	2.76	0.891
27	300	8.25	9.00	19	0.632	12-#5	1	9-#9	12-#7	10-#7	15-#5	3.94	254.5	509.0	685.2	300	22	15-#6	18-#5	10-#6	12-#5	3.34	0.910
27	400	8.25	9.00	21	0.735	14-#5	4	11-#9	27-#5	9-#8	10-#7	4.83	305.5	610.9	822.4	400	25	13-#7	9-#8	9-#7	10-#6	4.20	0.910
27	500	10.25	10.80	23	0.630	14-#5	1	13-#9	12-#8	18-#6	15-#6	5.36	358.2	716.4	964.4	500	27	26-#5	18-#6	14-#6	9-#7	4.60	0.970
28	100	6.25	9.33	12	0.785	13-#5	2	18-#5	17-#5	12-#5	10-#5	2.57	167.8	335.6	451.8	100	12	16-#5	12-#5	10-#5	10-#5	2.31	0.891
28	200	6.25	9.33	17	0.778	13-#5	4	17-#6	16-#6	16-#5	13-#5	3.34	224.7	449.4	604.9	200	20	15-#6	16-#5	12-#5	11-#5	2.95	0.891
28	300	8.25	9.33	19	0.718	13-#5	4	22-#6	18-#6	20-#5	12-#6	4.11	285.3	570.5	768.0	300	22	13-#7	20-#5	11-#6	13-#5	3.61	0.910
28	400	10.25	9.33	21	0.637	14-#5	3	20-#7	27-#5	10-#8	20-#5	4.98	343.1	686.2	923.7	400	25	13-#7	10-#8	10-#7	16-#5	4.36	0.928
28	500	10.25	11.20	23	0.724	16-#5	4	18-#8	13-#8	12-#8	10-#8	5.88	401.9	803.9	1082.2	500	27	16-#7	12-#8	12-#7	10-#7	5.15	0.970
29	100	6.25	9.67	12	0.816	13-#5	3	14-#6	14-#6	13-#5	11-#5	2.75	186.9	373.9	503.3	100	12	18-#5	13-#5	11-#5	11-#5	2.47	0.891
29	200	8.25	9.67	17	0.686	13-#5	3	11-#8	16-#6	10-#7	15-#5	3.61	251.3	502.6	676.5	200	20	15-#6	18-#5	10-#6	12-#5	3.08	0.910
29	300	10.25	9.67	19	0.634	13-#5	3	14-#8	18-#6	12-#7	10-#7	4.34	318.0	635.9	856.1	300	23	23-#5	22-#5	17-#5	15-#5	3.65	0.928
29	400	10.25	11.60	22	0.704	15-#5	4	14-#9	16-#7	12-#8	12-#7	5.43	383.2	766.5	1031.8	400	26	15-#7	11-#8	11-#7	18-#5	4.59	0.970
29	500	12.25	11.60	23	0.632	16-#5	2	17-#9	13-#8	17-#7	11-#8	6.26	450.1	900.3	1211.9	500	27	16-#7	17-#7	10-#8	15-#6	5.32	0.997
30	100	8.25	10.00	12	0.732	14-#5	1	22-#5	14-#6	15-#5	12-#5	2.84	208.3	416.5	560.7	100	12	17-#5	15-#5	12-#5	11-#5	2.49	0.910
30	200	8.25	10.00	17	0.732	14-#5	3	16-#7	18-#6	14-#6	12-#6	3.78	279.2	558.5	751.8	200	20	16-#6	14-#6	15-#5	13-#5	3.24	0.910
30	300	10.25	10.00	19	0.693	14-#5	4	13-#9	15-#7	18-#6	11-#7	4.82	353.6	707.1	951.9	300	23	26-#5	18-#6	10-#7	12-#6	4.07	0.928
30	400	12.25	12.00	22	0.633	15-#5	3	16-#9	16-#7	10-#9	18-#6	5.71	427.6	855.2	1151.2	400	26	15-#7	10-#9	23-#5	20-#5	4.83	0.997

NOTES: (1) 50 percent of these bars may be placed in the middle third of column strip. (2) Drop panels same size as for edge panels. (3) Same column size above and below slab.

APPENDIX D: COLUMN DESIGN

AMPAD

SLAB THICKNESS FOR GRAVITY LOADING

MAX SPANS:

FIRST FLOOR - 30'-0"
SECOND FLOOR - 30'-0" BUSINESS - 24'-0"
THIRD FLOOR - 30'-0" BUSINESS - 24'-0"
FOURTH FLOOR - 30'-0"
FIFTH FLOOR - 30'-0"

SHORT INTERIOR COLUMN LOADING - PURE AXIAL

$150_{pcf}\left(\frac{10}{12}\right)(26')(21.0')(2\text{ FLOORS}) = 136.5$ KIPS SELF WT

$20_{psf}(26)(21)(2\text{ FLOORS}) = 21.8$ KIPS MEP AND MISC.

$100_{psf}(26')(21')(1\text{ FLOOR}) = 54.6$ KIPS LIVE LOAD

24.7 KIPS EARTHQUAKE LOAD

$30_{psf}(26)(21) = 16.4$ KIPS SNOW

$20_{psf}(26)(21) = 10.9$ KIPS ROOF LIVE

$1.4(158.3) = 221.6$ KIPS
$1.2(158.3) + 1.6(54.6) + 0.5(16.4) = 285.5$ KIPS
$1.2(158.3) + 1.6(16.4) + 54.6 = 270.8$ KIPS
$1.2(158.3) + 1.0(24.7) + 54.6 + 0.2(16.4) = 272.5$ KIPS

$P_o = 0.85 f'_c A_c$
$285.5 = (0.85)(4) A_c$
$A_c = 84$ ⟶ NEED A 10×10 COLUMN

CRSI CALLS FOR A 20×20 COLUMN SO ITS OKAY ✓

TALL INTERIOR COLUMN LOADING - PURE AXIAL

$150\left(\frac{10}{12}\right)(26')(21')(7\text{ FLOORS}) = 478$ KIPS SELF WT.
$20(26')(21')(7\text{ FLOORS}) = 76.4$ KIPS MEP AND MISC.
$100(26)(21)(6\text{ FLOORS}) = 327$ KIPS LIVE LOAD
$20(26)(21) = 11$ KIPS ROOF LIVE LOAD
$30(26)(21) = 16.4$ KIPS SNOW LOAD
5.7 KIPS EARTHQUAKE LOAD
$1.4(554.4) = 776.2$ KIPS
$1.2(554.4) + 1.6(327) + 0.5(16.4) = 1196.7$ KIPS
$1.2(554.4) + 1.0(5.7) + 327 + 0.2(16.4) = 1001.3$ KIPS

$P_o = 0.85 f'_c A_c$
$1196.7 = 0.85(4) A_c$
$A_c = 352$
NEED A 19×19 COLUMN ⟵
CRSI CALLS FOR 20×20 ✓

APPENDIX D: COLUMN DESIGN

APPENDIX | COLUMN SIZING

SHORT EXTERIOR COLUMN LOADING - PURE AXIAL

150 pcf $(10/12)(26')(10.5')(2 \text{ FLOORS}) = 68.3$ KIPS SELF WT
20 psf $(26)(10.5)(2 \text{ FLOORS}) = 10.9$ KIPS MEP AND MISC.
38 psf $(26)(27.5') = 27.2$ KIPS FAÇADE LOAD
100 psf $(26)(10.5)(1 \text{ FLOOR}) = 27.3$ KIPS LIVE LOAD
30 psf $(26)(10.5) = 8.2$ KIPS SNOW
20 psf $(26)(10.5) = 5.5$ KIPS ROOF LIVE
41.5 KIPS EARTHQUAKE

$1.4(106.4) = 149$ KIPS
$1.2(106.4) + 1.6(27.3) + 0.5(8.2) = 175.5$ KIPS
$1.2(106.4) + 1.0(41.5) + 27.3 + 0.2(8.2) = 198.1$ KIPS

$P_o = 0.85 f'_c A_c$
$198.1 = 0.85(4) A_c$
$A_c = 58.3$ ⟶ NEED AN 8×8 COLUMN   CRSI CALLS FOR 20×20 COLUMN ∴ OKAY ✓

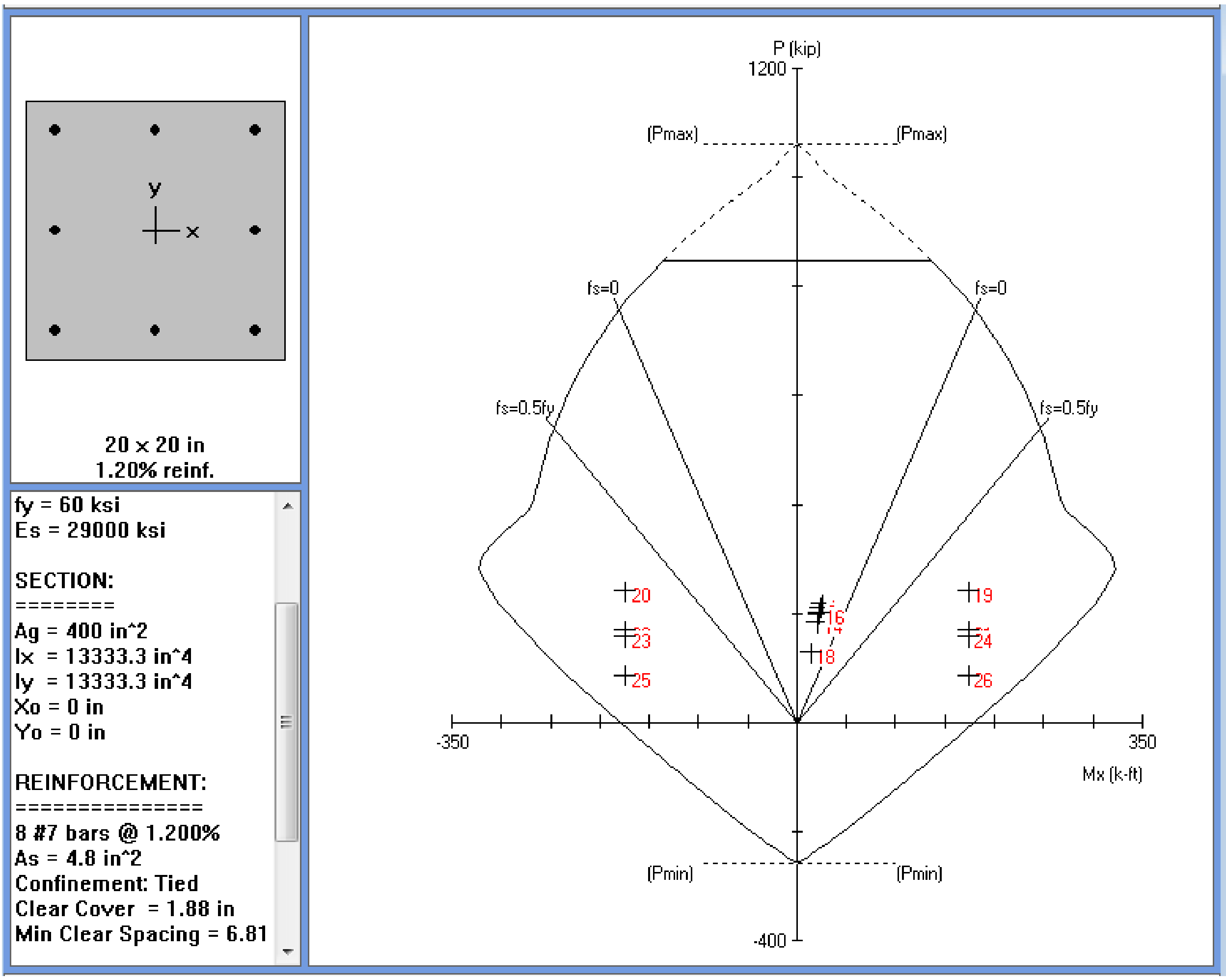
TALL EXTERIOR COLUMN LOADING - PURE AXIAL

150 pcf $(10/12)(26')[(2)(21) + (5)(10.5)] = 307$ KIPS SELF WT
20 psf $(26)[(2)(21) + (5)(10.5)] = 49.1$ KIPS MEP AND MISC.
38 psf $(26)(59') = 58.3$ KIPS FAÇADE LOAD
100 psf $(26)[(2)(21) + (5)(10.5)] = 245.7$ KIPS LIVE LOAD
30 psf $(26)(10.5) = 8.2$ KIPS SNOW
20 psf $(26)(10.5) = 5.5$ KIPS ROOF LIVE
275 KIPS EARTHQUAKE

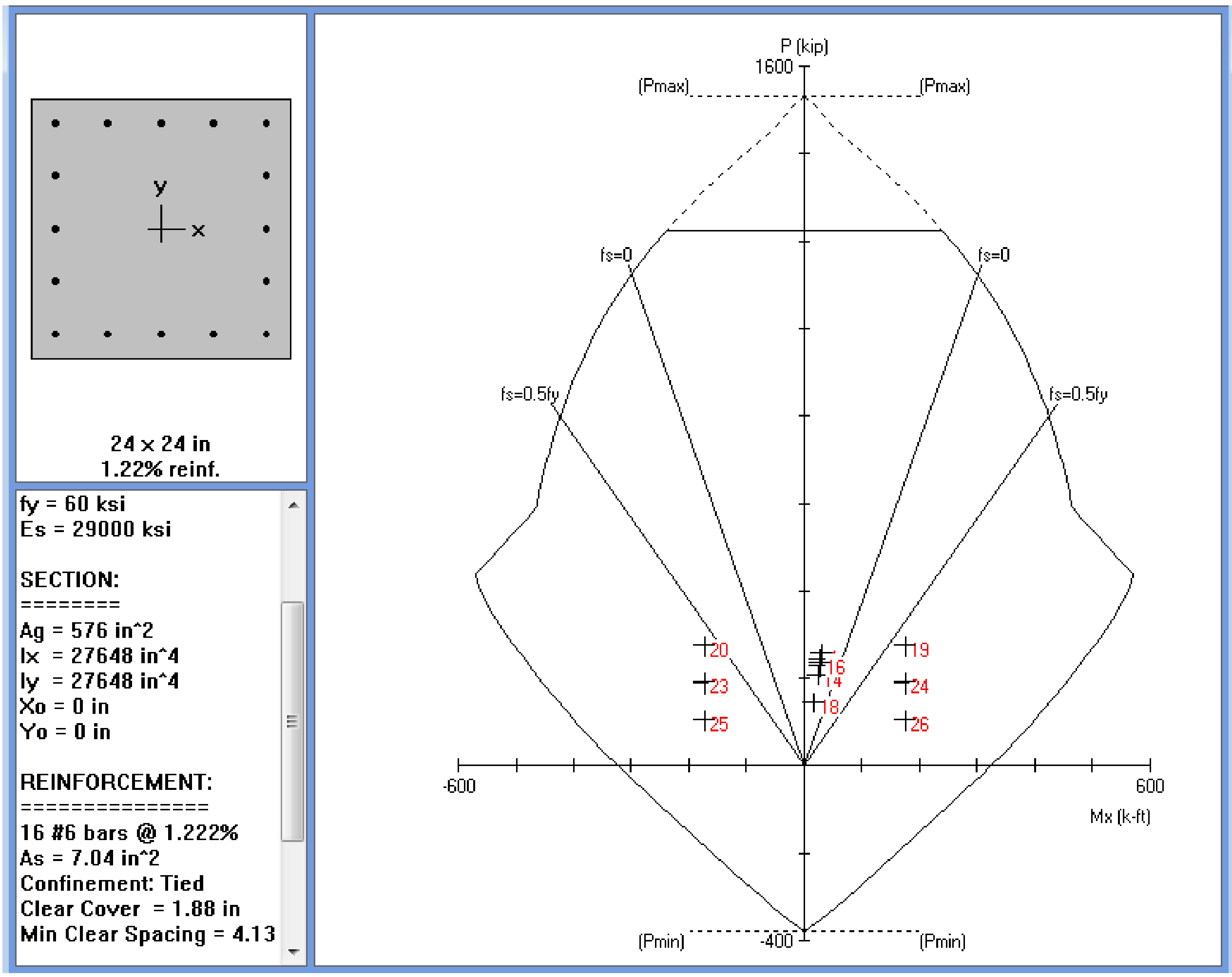
$1.4(414.4) = 580.2$ KIPS
$1.2(414.4) + 1.6(245.7) + 0.5(8.2) = 894.5$ KIPS
$1.2(414.4) + 1.0(275) + 245.7 + 0.2(8.2) = 1019.6$ KIPS

$P_o = 0.85 f'_c A_c$
$1019.6 = 0.85(4) A_c$
$A_c = 300$ ⟶ NEED A 17×17 COLUMN   CRSI CALLS FOR 20×20 COLUMN ∴ OKAY ✓

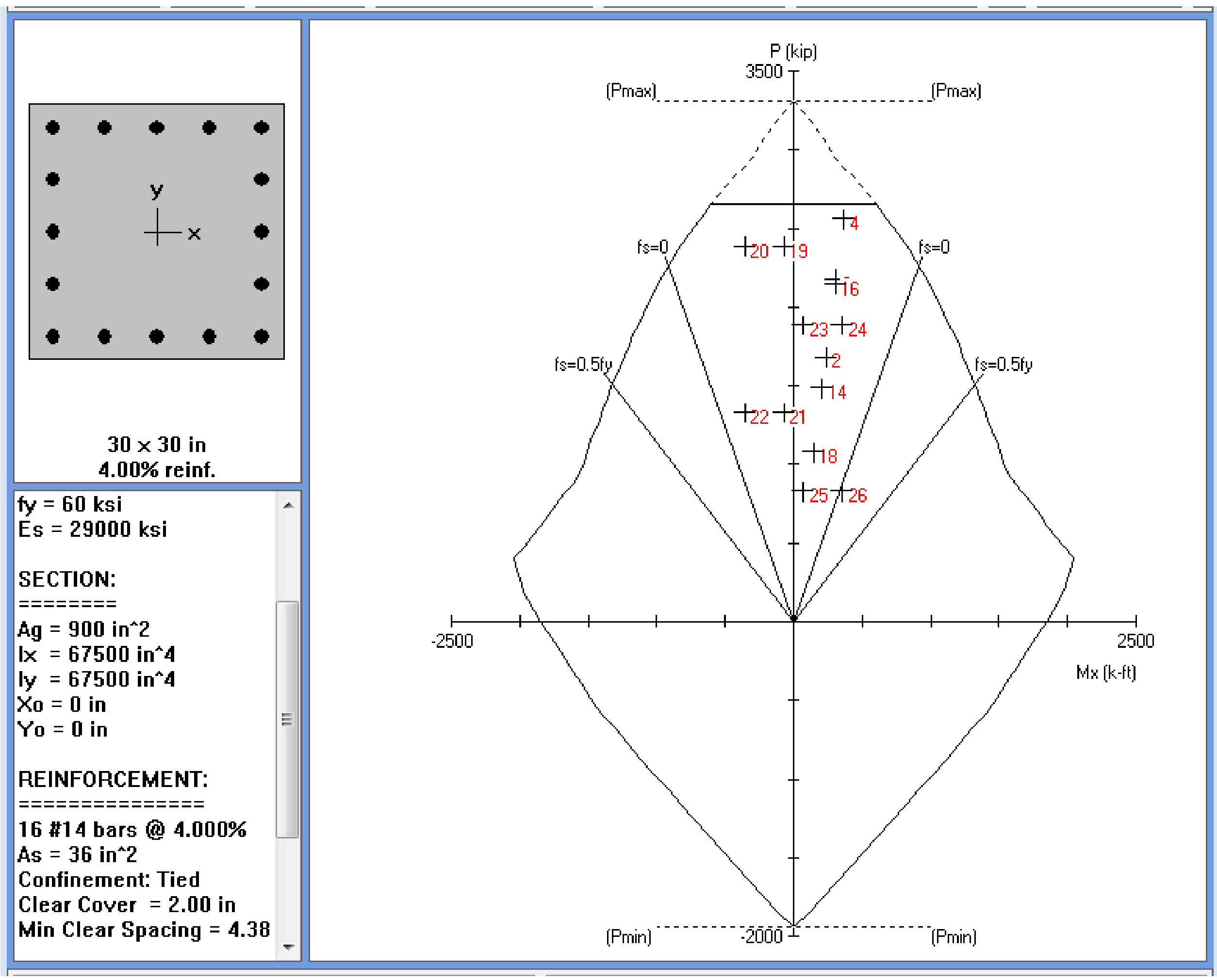
APPENDIX D: COLUMN DESIGN – 20x20's



APPENDIX D: COLUMN DESIGN – 24x24's



APPENDIX D: COLUMN DESIGN – 30x30's



APPENDIX E: COST DATA – STEEL SYSTEM

Total Metal Decking Costs (2" deep, 20 gauge)							
Level	Area (ft^2)	Unit	Crew	Daily Output	Labor Hours	Bare Costs	Total
Roof	70888	S.F.	E4	3600	0.009	2.3	163042
6	70888	S.F.	E4	3600	0.009	2.3	163042
5	70888	S.F.	E4	3600	0.009	2.3	163042
4	70888	S.F.	E4	3600	0.009	2.3	163042
3	99794	S.F.	E4	3600	0.009	2.3	229526
2	172144	S.F.	E4	3600	0.009	2.3	395931
1	172144	S.F.	E4	3600	0.009	2.3	395931
Ground	95676	S.F.	E4	3600	0.009	2.3	220055
							1893613
3/4" Shear Studs Total							
	Quantity	Crew	Unit	Daily Output	Labor hours	Bare Cost	Total
	130361	E-10	Ea.	935	0.017	1.87	243775

Total Steel Beam and Column Costs								
Size	Quantity	Length (ft)	Unit	Crew	Daily Output	Labor Hours	Bare Cost	Total
W8x31	29	841	L.F.	E2	1080	0.052	46.39	39013.99
W8x40	15	435	L.F.	E2	550	0.102	73.63	32029.05
W8x58	4	142	L.F.	E2	550	0.102	73.63	10455.46
W8x67	4	142	L.F.	E2	984	0.057	96.27	13670.34
W10x12	577	7118.65	L.F.	E2	600	0.093	23.49	167217.1
W10x33	39	624	L.F.	E2	550	0.102	53.13	33153.12
W10x39	31	614	L.F.	E2	550	0.102	75.13	46129.82
W10x45	9	144	L.F.	E2	1032	0.054	66.07	9514.08
W10x49	71	1900	L.F.	E2	550	0.102	75.13	142747
W10x60	10	425	L.F.	E2	550	0.102	75.13	31930.25
W10x88	6	194	L.F.	E2	640	0.088	126.56	24552.64
W12x14	115	1713	L.F.	E2	880	0.064	26.77	45857.01
W12x16	20	295	L.F.	E2	880	0.064	26.77	7897.15
W12x19	234	4946.75	L.F.	E2	880	0.064	35.27	174471.9
W12x36	2	72	L.F.	E2	810	0.069	53.19	3829.68
W12x40	36	1320	L.F.	E2	750	0.075	74.6	98472
W12x45	39	1253.5	L.F.	E2	750	0.075	74.6	93511.1
W12x50	4	112	L.F.	E2	1032	0.054	73.07	8183.84
W12x53	67	711.75	L.F.	E2	750	0.075	74.6	53096.55
W12x58	60	2122	L.F.	E2	750	0.075	85.6	181643.2
W12x65	55	1857.5	L.F.	E2	640	0.088	105.56	196077.7
W12x72	27	825	L.F.	E2	640	0.088	105.56	87087
W12x79	13	569	L.F.	E2	640	0.088	105.56	60063.64
W12x87	26	777	L.F.	E2	984	0.057	124.27	96557.79
W12x96	61	2180.5	L.F.	E2	640	0.088	126.56	275964.1
W12x106	9	341	L.F.	E2	900	0.089	152.14	51879.74
W12x135	1	49	L.F.	E2	1050	0.076	206.26	10106.74

APPENDIX E: COST DATA – STEEL SYSTEM

W12x136	15	300	L.F.	E2	1050	0.076	206.26	61878
W12x152	28	896	L.F.	E2	1050	0.076	206.26	184809
W14x22	156	3056	L.F.	E2	990	0.057	40.24	122973.4
W14x30	118	1627.5	L.F.	E2	900	0.062	46.16	75125.4
W14x38	9	285.99	L.F.	E2	810	0.069	64.19	18357.7
W14x48	115	2737.5	L.F.	E2	800	0.07	78.25	214209.4
W14x68	26	765.65	L.F.	E2	760	0.074	107.53	82330.34
W16x26	1255	30825.83	L.F.	E2	1000	0.056	40.2	1239198
W16x31	738	19826.37	L.F.	E2	900	0.062	47.16	935011.6
W16x36	28	682	L.F.	E2	800	0.07	60.25	41090.5
W18x35	349	8134	L.F.	E5	960	0.083	53.76	437283.8
W18x40	187	5129.5	L.F.	E5	960	0.083	60.76	311668.4
W21x16	2	40.5	L.F.	E5	880	0.064	26.77	1084.185
W21x19	32	624	L.F.	E5	880	0.064	35.27	22008.48
W21x20	2	84	L.F.	E5	880	0.064	35.27	2962.68
W21x44	300	7490.91	L.F.	E5	1064	0.075	65.69	492077.9
W21x50	87	2164.41	L.F.	E5	1064	0.075	74.19	160577.6
W21x57	7	167.32	L.F.	E5	1036	0.077	98.83	16536.24
W21x152	7	252	L.F.	E5	1050	0.076	206.26	51977.52
W24x55	122	3092.33	L.F.	E5	1110	0.072	80.48	248870.7
W24x62	30	818	L.F.	E5	1110	0.072	90.48	74012.64
W24x68	11	312.82	L.F.	E5	1110	0.072	98.48	30806.51
W24x76	27	718.98	L.F.	E5	1110	0.072	109.98	79073.42
W27x84	26	715.83	L.F.	E5	1190	0.067	120.64	86357.73
W27x94	1	26	L.F.	E5	1190	0.067	133.64	3474.64
W30x99	5	155	L.F.	E5	1200	0.067	140.6	21793
W33x130	4	118	L.F.	E5	1160	0.069	186.77	22038.86
								7032700

Concrete Costs									
Level	Area (ft^2)	Thickness	Quantity	Unit	Crew	Daily Output	Labor Hours	Bare Costs	Total
Roof	70888	3.25"	711	C.Y.	C-20	140	0.457	22.75	16175
6	70888	3.25"	711	C.Y.	C-20	140	0.457	22.75	16175
5	70888	3.25"	711	C.Y.	C-20	140	0.457	22.75	16175
4	70888	3.25"	711	C.Y.	C-20	140	0.457	22.75	16175
3	99794	3.25"	1001	C.Y.	C-20	140	0.457	22.75	22773
2	172144	3.25"	1727	C.Y.	C-20	140	0.457	22.75	39289
1	172144	3.25"	1727	C.Y.	C-20	140	0.457	22.75	39289
Ground	95676	3.25"	960	C.Y.	C-20	140	0.457	22.75	21840
									187892

Cementitious Fireproofing							
Item	Quantity	Unit	Crew	Daily Output	Labor Hours	Bare Costs	Total
Beams & Columns	122771.1	S.F.	G-2	1500	0.016	1.19	146098
Decking	823310	S.F.	G-2	1250	0.019	1.59	1309063
							1455160

APPENDIX E: COST DATA – CONCRETE SYSTEM

Concrete Column Costs (Icluding 4 use forms, cocrete, reinforcment, placement, and finishing)								
Size	Quantity	Volume per Column	Unit	Crew	Daily Output	Labor Hours	Bare Cost	Total
30x30	156	22.57	C.Y.	C-14A	13.18	15.48	1315	4630010
20x20	4	4.32	C.Y.	C-14A	11.23	18.17	1261	21790
24x24	57	4.3	C.Y.	C-14A	11.23	18.17	1261	309071
20x20	77	3.29	C.Y.	C-14A	11.23	18.17	1261	319449
20x20	57	1.65	C.Y.	C-14A	11.23	18.17	1261	118597
								5398917

Flat Slab Costs (Icluding 4 use forms, cocrete, reinforcment, placement, and finishing)							
Level	Slab Volume	Unit	Crew	Daily Output	Labor Hours	Bare Cost	Total
Roof	2991	C.Y.	C-14B	50.99	4.079	455.55	1362550
6	2991	C.Y.	C-14B	50.99	4.079	455.55	1362550
5	2991	C.Y.	C-14B	50.99	4.079	455.55	1362550
4	2991	C.Y.	C-14B	50.99	4.079	455.55	1362550
3	3460	C.Y.	C-14B	50.99	4.079	455.55	1576203
2	6852	C.Y.	C-14B	50.99	4.079	455.55	3121429
1	6316	C.Y.	C-14B	50.99	4.079	455.55	2877254
Ground	3248.3	C.Y.	C-14B	50.99	4.079	455.55	1479763
							14504849

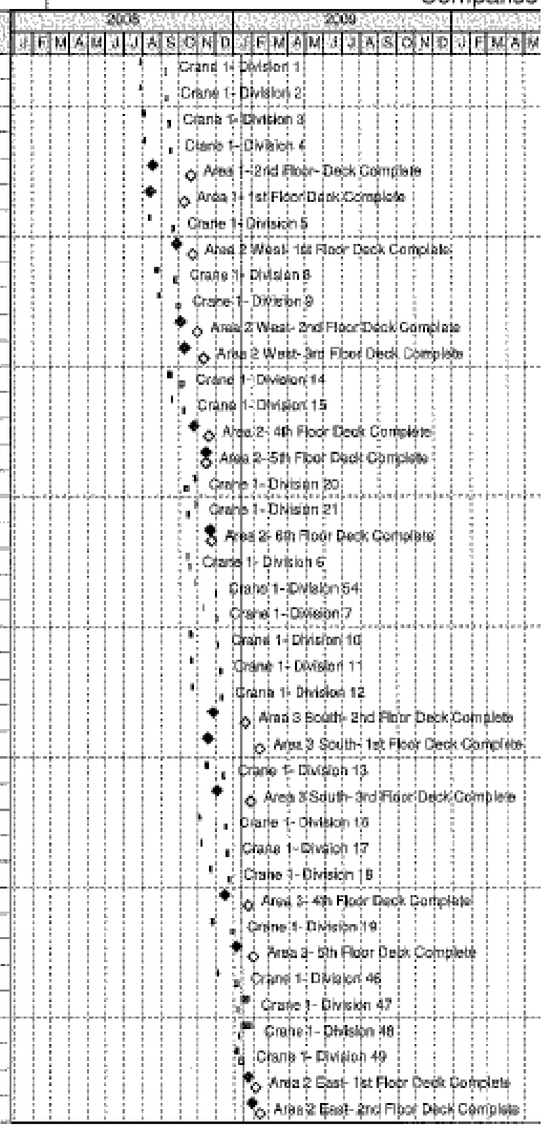
Shear Wall							
Item	Quantity	Crew	Unit	Daily Output	Labor hours	Bare Cost	Total
Reinf. #3-#7	4.1	4 Rodm	Ton	2.3	13.913	1455	5966
Reinf. #8 - #18	3.64	4 Rodm	Ton	3	10.667	1325	4823
Forms - 2 use	13503	C-2	SFCA	345	0.139	6.93	93576
6ksi Concrete	750	-	C.Y.	-	-	127	95250
Placing	750	C-20	C.Y.	120	0.533	26.15	19612.5
							219226.8

APPENDIX F: STEEL CONSTRUCTION SCHEDULE

Medical Center 1-14-09 Baseline Schedule Update 1

Compariso

Activity Name	Original Duration	Start	Finish	BL1 Early Start	BL1 Early Finish
Crane 1- Division 1	2	28-Jul-08 A	29-Jul-08 A		
Crane 1- Division 2	1	28-Jul-08 A	29-Jul-08 A		
Crane 1- Division 3	3	01-Aug-08 A	05-Aug-08 A		
Crane 1- Division 4	2	04-Aug-08 A	05-Aug-08 A		
Area 1- 2nd Floor- Deck Complete	0		19-Aug-08 A		
Area 1- 1st Floor Deck Complete	0		13-Aug-08 A		
Crane 1- Division 5	3	11-Aug-08 A	13-Aug-08 A	14-Aug-08	10-Sep-08
Area 2 West- 1st Floor Deck Complete	0		26-Sep-08 A		
Crane 1- Division 8	3	22-Aug-08 A	26-Aug-08 A		
Crane 1- Division 9	3	27-Aug-08 A	29-Aug-08 A		
Area 2 West- 2nd Floor Deck Complete	0		03-Oct-08 A		
Area 2 West- 3rd Floor Deck Complete	0		10-Oct-08 A		
Crane 1- Division 14	5	11-Sep-08 A	17-Sep-08 A		
Crane 1- Division 15	2	17-Sep-08 A	18-Sep-08 A		
Area 2- 4th Floor Deck Complete	0		24-Oct-08 A		
Area 2- 5th Floor Deck Complete	0		14-Nov-08 A		
Crane 1- Division 20	2	25-Oct-08 A	28-Oct-08 A		
Crane 1- Division 21	2	27-Oct-08 A	28-Oct-08 A		
Area 2- 6th Floor Deck Complete	0		21-Nov-08 A		
Crane 1- Division 6	2	13-Oct-08 A	14-Oct-08 A		
Crane 1- Division 54	1	24-Oct-08 A	24-Oct-08 A		
Crane 1- Division 7	2	10-Nov-08 A	11-Nov-08 A		
Crane 1- Division 10	2	17-Oct-08 A	20-Oct-08 A	08-Sep-08	
Crane 1- Division 11	2	18-Oct-08 A	21-Oct-08 A		
Crane 1- Division 12	2	21-Oct-08 A	22-Oct-08 A		
Area 3 South- 2nd Floor Deck Complete	0		25-Nov-08 A		
Area 3 South- 1st Floor Deck Complete	0		18-Nov-08 A		
Crane 1- Division 13	2	14-Nov-08 A	17-Nov-08 A		
Area 3 South- 3rd Floor Deck Complete	0		02-Dec-08 A		
Crane 1- Division 16	3	31-Oct-08 A	04-Nov-08 A		
Crane 1- Division 17	2	06-Nov-08 A	07-Nov-08 A		
Crane 1- Division 18	2	20-Nov-08 A	21-Nov-08 A		
Area 3- 4th Floor Deck Complete	0		16-Dec-08 A		
Crane 1- Division 19	4	21-Nov-08 A	26-Nov-08 A		
Area 3- 5th Floor Deck Complete	0		05-Jan-09 A		
Crane 1- Division 46	3	03-Dec-08 A	05-Dec-08 A		
Crane 1- Division 47	1	13-Jan-09 A	23-Jan-09		
Crane 1- Division 48	1	13-Jan-09 A	28-Jan-09		
Crane 1- Division 49	4	02-Jan-09 A	07-Jan-09 A		
Area 2 East- 1st Floor Deck Complete	0		23-Jan-09*		
Area 2 East- 2nd Floor Deck Complete	0		30-Jan-09*		



vel of Effort — Actual Work — Critical Remaining W...
l Effort — Remaining Work ◆ ◆ Milestone

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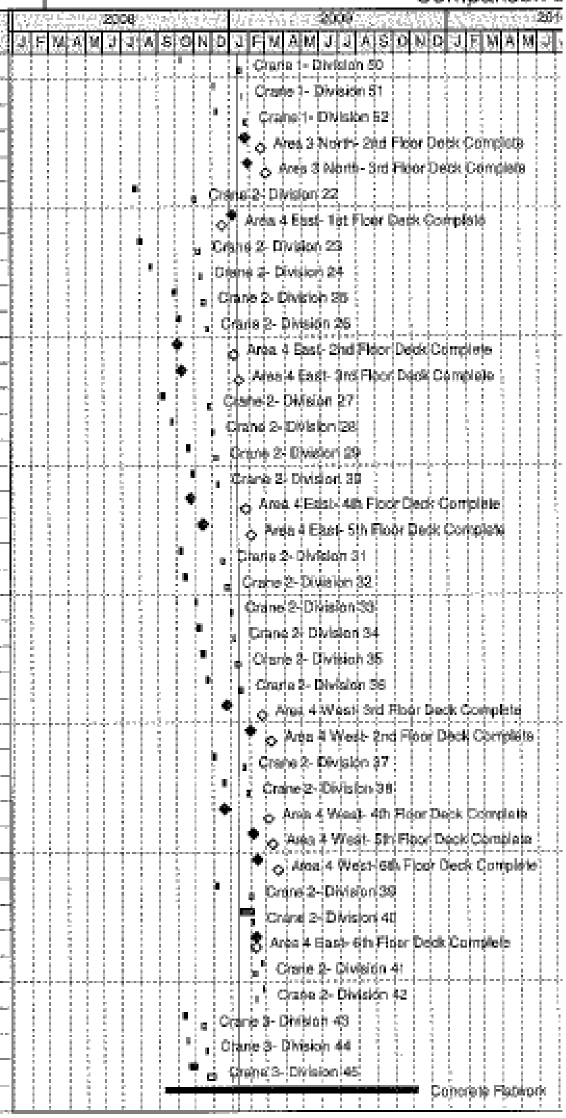
TASK filter: All Activities

APPENDIX F: STEEL CONSTRUCTION SCHEDULE

Medical Center 1-14-09 Baseline Schedule Update 1

Comparison b

Activity Name	Original Duration	Start	Finish	BL1 Early Start	BL1 Early Finish
Crane 1- Division 50	4	07-Oct-08 A	10-Oct-08 A		
Crane 1- Division 51	2	04-Dec-08 A	05-Dec-08 A		
Crane 1- Division 52	3	08-Dec-08 A	10-Dec-08 A		
Area 3 North- 2nd Floor Deck Complete	0		28-Jan-09		
Area 3 North- 3rd Floor Deck Complete	0		30-Jan-09*		
Crane 2- Division 22	5	22-Jul-08 A	28-Jul-08 A		
Area 4 East- 1st Floor Deck Complete	0		05-Jan-09 A		
Crane 2- Division 23	4	30-Jul-08 A	04-Aug-08 A		
Crane 2- Division 24	3	19-Aug-08 A	21-Aug-08 A		
Crane 2- Division 25	4	25-Sep-08 A	30-Sep-08 A		
Crane 2- Division 26	3	02-Oct-08 A	06-Oct-08 A		
Area 4 East- 2nd Floor Deck Complete	0		09-Oct-08 A		
Area 4 East- 3rd Floor Deck Complete	0		10-Oct-08 A		
Crane 2- Division 27	4	05-Sep-08 A	10-Sep-08 A	11-Sep-08	01-Oct-08
Crane 2- Division 28	3	22-Sep-08 A	24-Sep-08 A		
Crane 2- Division 29	3	18-Oct-08 A	22-Oct-08 A		
Crane 2- Division 30	4	25-Oct-08 A	30-Oct-08 A		
Area 4 East- 4th Floor Deck Complete	0		24-Oct-08 A		
Area 4 East- 5th Floor Deck Complete	0		14-Nov-08 A		
Crane 2- Division 31	5	06-Oct-08 A	10-Oct-08 A		
Crane 2- Division 32	5	11-Oct-08 A	17-Oct-08 A		
Crane 2- Division 33	3	31-Oct-08 A	04-Nov-08 A		
Crane 2- Division 34	4	05-Nov-08 A	10-Nov-08 A		
Crane 2- Division 35	4	12-Nov-08 A	17-Nov-08 A		
Crane 2- Division 36	3	20-Nov-08 A	24-Nov-08 A		
Area 4 West- 3rd Floor Deck Complete	0		24-Dec-08 A		
Area 4 West- 2nd Floor Deck Complete	0		02-Feb-09*		
Crane 2- Division 37	4	01-Dec-08 A	04-Dec-08 A		
Crane 2- Division 38	4	18-Dec-08 A	23-Dec-08 A		
Area 4 West- 4th Floor Deck Complete	0		19-Dec-08 A		
Area 4 West- 5th Floor Deck Complete	0		05-Feb-09*		
Area 4 West- 6th Floor Deck Complete	0		13-Feb-09*		
Crane 2- Division 39	3	04-Dec-08 A	08-Dec-08 A		
Crane 2- Division 40	2	14-Jan-09 A	05-Feb-09		
Area 4 East- 6th Floor Deck Complete	0		11-Feb-09*		
Crane 2- Division 41	2	19-Feb-09	20-Feb-09*		
Crane 2- Division 42	2	23-Feb-09	24-Feb-09*		
Crane 3- Division 43	5	10-Oct-08 A	16-Oct-08 A		
Crane 3- Division 44	2	16-Oct-08 A	17-Oct-08 A		
Crane 3- Division 45	10	21-Oct-08 A	03-Nov-08 A		
twork	303	08-Sep-08 A	06-Nov-09	11-Sep-08	07-Jul-09



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of Effort — Remaining Work ◆ ◆ Milestone

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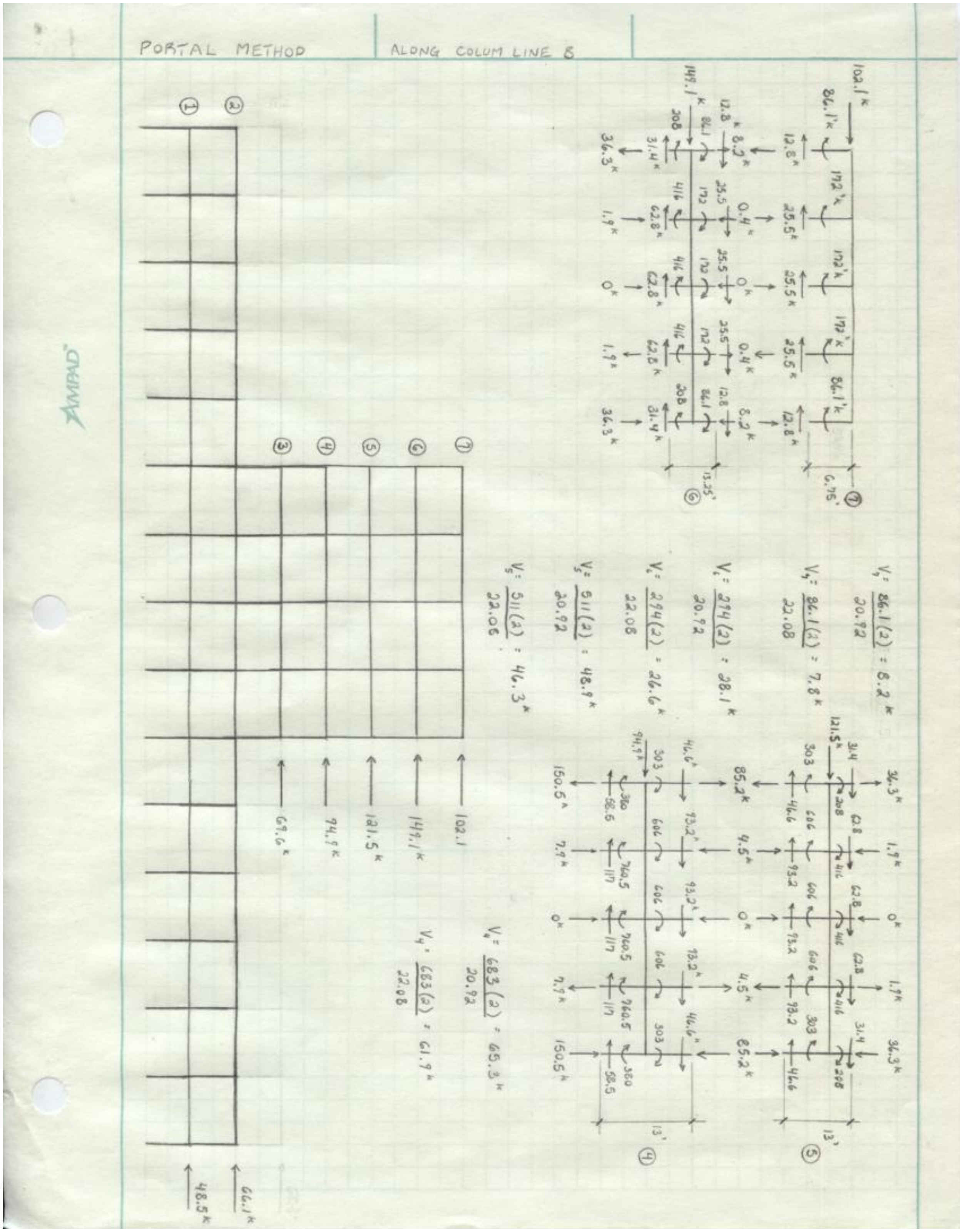
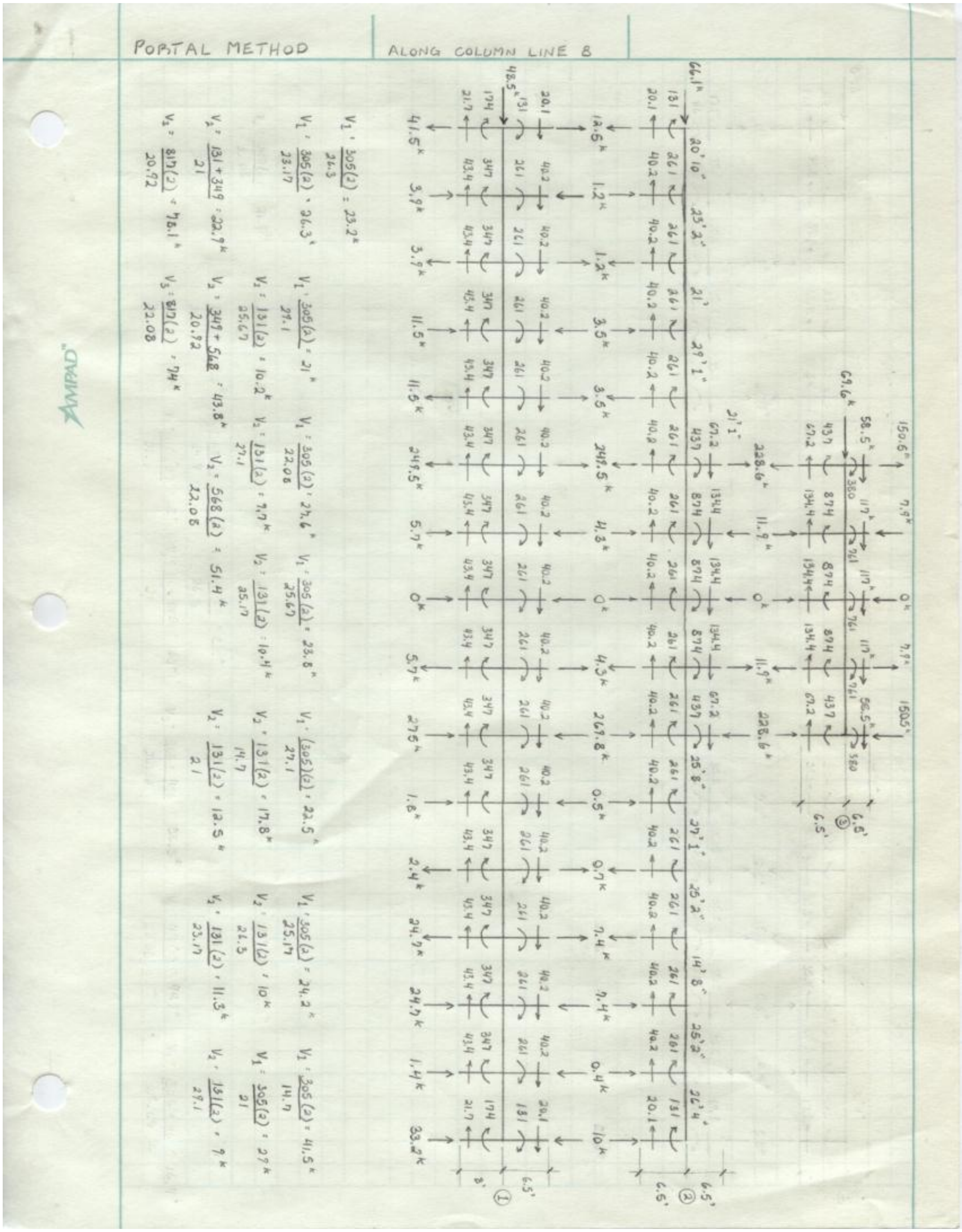
TASK filter: All Activities

APPENDIX G: PORTAL METHOD



APPENDIX G: PORTAL METHOD



APPENDIX H: SPOT CHECKS – COLUMNS

APPENDIX | COLUMN SPOT CHECKS

30 × 30 COLUMN AT CRITICAL STORY – 1^{st} STORY COLUMN AT GRID AG36

DL: $(30/12)^2(150)(81.5) + (10/12)(150)[(46.6)(43) + (46.6)(21)(5)] = 938.5^k$

LL: $(100)(46.6)(43)(1\text{ FLOOR}) + 100(46.6)(21)(5\text{ FLOORS}) = 689.7^k$

E: 249.5^k (FROM PORTAL METHOD)

SDL: $20\text{psf}(46.6)(43) + 20(46.6)(21)(5) = 137.9^k$

FACADE: $38(46.6)(65.5) = 116^k$

S: $30\text{psf}(46.6)(21) = 29.4^k$

$P_u = 1.2D + 1.0E + L + 0.2S$

$P_u = 1.2(938.5 + 137.9 + 116) + 1.0(249.5) + (689.7) + 0.2(29.4)$

$P_u = 2376$ KIPS

$\Sigma P_u = 192{,}239$ KIPS (COMPUTED IN SPREADSHEET ATTACHED)

$\Delta_o = .00536$ (FROM ETABS)

$V_{us} = 841$ KIPS (FROM SEISMIC APPENDIX)

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us}\, l_c} = \frac{(192239)(.00536)}{(841)(13 \times 12)} = 0.008 \leq 0.05 \quad \therefore \text{ NONSWAY FRAME}$$

SLENDERNESS:

$$\frac{kl}{r} = \frac{(1.0)(13 \times 12)}{0.3(30)} = 17.3 < 22 \quad \therefore \text{ COLUMN IS NOT SLENDER}$$

MOMENT MAGNIFICATION (NONSWAY): ACI 10.10.6.1

$$C_m = 0.6 + 0.4\left(M_1/M_2\right) = 0.6 + 0.4\left(-76.68/359.38\right) = 0.51$$

$$EI = \frac{0.4 E_c I_g}{1 + \beta_{dns}} = \frac{0.4(3600)(30^4/12)}{1+1} = 4.86 \times 10^7$$

$$P_c = \frac{\pi^2 EI}{(kl)^2} = \frac{\pi^2(4.86E7)}{(13 \times 12)^2} = 19{,}710^k$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} = \frac{0.51}{1 - \frac{2{,}376}{.75(19{,}710)}} = 0.61 \ngeq 1.0$$

$\therefore$ MOMENT MAGNIFICATION DOES NOT APPLY
CHECK M_2 AGAINST INTERACTION DIAGRAM

REINFORCEMENT CHECK:

$$e = \frac{M_u}{P_u} = \frac{(359)(12)}{2376} = 1.81''$$

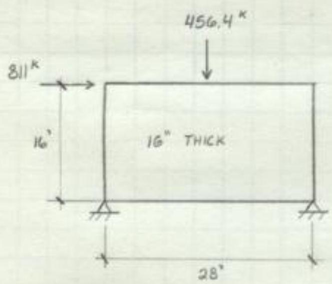
$$\gamma = \frac{h - 2d}{h} = \frac{30 - 2(1.69)}{30} = 0.89 \qquad e/h = 0.06$$

$$\phi P_n / h^2 = 2376/30^2 = 2.64 \qquad \phi M_n/bh^2 = 359(12)/(30 \cdot 30^2) = 0.16$$

$\rho_g = 0.03$ (FROM MACGREGOR DESIGN AIDS) < 0.05 OK ✓

$A_{s,REQ} = \rho bh = 0.03(30^2) = 27\text{ in}^2 < 36\text{ in}^2$ (FROM SP COLUMN) OKAY ✓

APPENDIX | SHEAR WALL REINFORCEMENT



SIMPLIFIED METHOD:

$V_c \leq 2\sqrt{f'_c}\,hd$

$\leq 2\sqrt{6000}\,(16)(268.8)$

≤ 666 kips ⟵ USE TO DESIGN

$d = 0.8 l_w = 0.8(28 \times 12) = 268.8$ in

METHOD 2:

MIN of:

$V_c \leq 3.3\sqrt{f'_c}\,hd + \dfrac{N_u d}{4 l_w} = 3.3\sqrt{6000}\,(16)(268.8) + \dfrac{(456.4)(268.8)}{4(28 \times 12)} = 1{,}099$ kips

$V_c \leq \left[0.6\sqrt{f'_c} + \dfrac{l_w(1.25\sqrt{f'_c} + 0.2\, N_u / l_w h)}{M_u/V_u - l_w/2}\right] hd$

$\leq \left[0.6\sqrt{6000} + \dfrac{(28 \times 12)(1.25\sqrt{6000} + 0.2(456.4/(28 \times 12 \times 16)))}{(811 \times 16 \times 12)/811 - (28 \times 12)/2}\right](16)(268.8) = 6{,}031$ kips

HORIZONTAL REINFORCEMENT:

$\tfrac{1}{2}\phi V_c = \tfrac{1}{2}(0.75)(666) = 250^k < 811^k$ ∴ NEEDS REINFORCEMENT

$V_u \leq \phi(V_c + V_s)$

$811 = (0.75)(666 + V_s)$

$V_{s,min} = 415.3$ kips

$V_s = \dfrac{A_v f_y d}{s}$

$415.3 = \dfrac{A_v (60)(268.8)}{18}$

$A_v = 0.464$ in^2

$\rho_t = \dfrac{A_v}{sh}$

$0.0025 = \dfrac{A_v}{18(16)}$

$A_v = 0.72$ in^2 ⟵ CONTROLS

$s =$ MIN of: $l_w/5 = (28 \times 12)/5 = 67.2"$; $3h = 3(16) = 48"$; 18 in

USE (2) #6's @ 18" o.c. $A_v = 0.88 > 0.72$ in^2 ok ✓

VERTICAL REINFORCEMENT:

$\rho_l = \dfrac{A_v}{sh} \geq 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_t - 0.0025)$

$\dfrac{A_v}{(18)(16)} = 0.0025 + 0.5(2.5 - 16/28)(0.88/(18 \times 16) - 0.0025)$

$A_v = 0.87$ in^2

USE (2) #6's @ 18" o.c. $A_v = 0.88 > 0.87$ in^2 ok ✓

APPENDIX H: SPOT CHECKS – SHEAR WALL

APPENDIX | SHEAR WALL REINFORCEMENT

FLEXURE:

$jd = 0.9d = 0.9(268.8) = 241.9$ in
$M_u = 811(16) = 12{,}976$ ft·k

$M_u = \varnothing M_n = \varnothing A_s f_y jd$
$12{,}976(12) = (0.9)A_s(60)(241.9)$
$A_s = 11.9$ in^2

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{(11.9)(60{,}000)}{0.85(6000)(16)} = 8.75$

$jd = d - a/2 = 268.8 - (8.75/2) = 264.4$

$A_s = \dfrac{12{,}976(12)}{0.9(60)(264.4)} = 10.9$ in^2

USE (14) #8's $\qquad A_v = 11.06 > 10.9$ in^2 ✓

TENSION CONTROLLED SECTION:

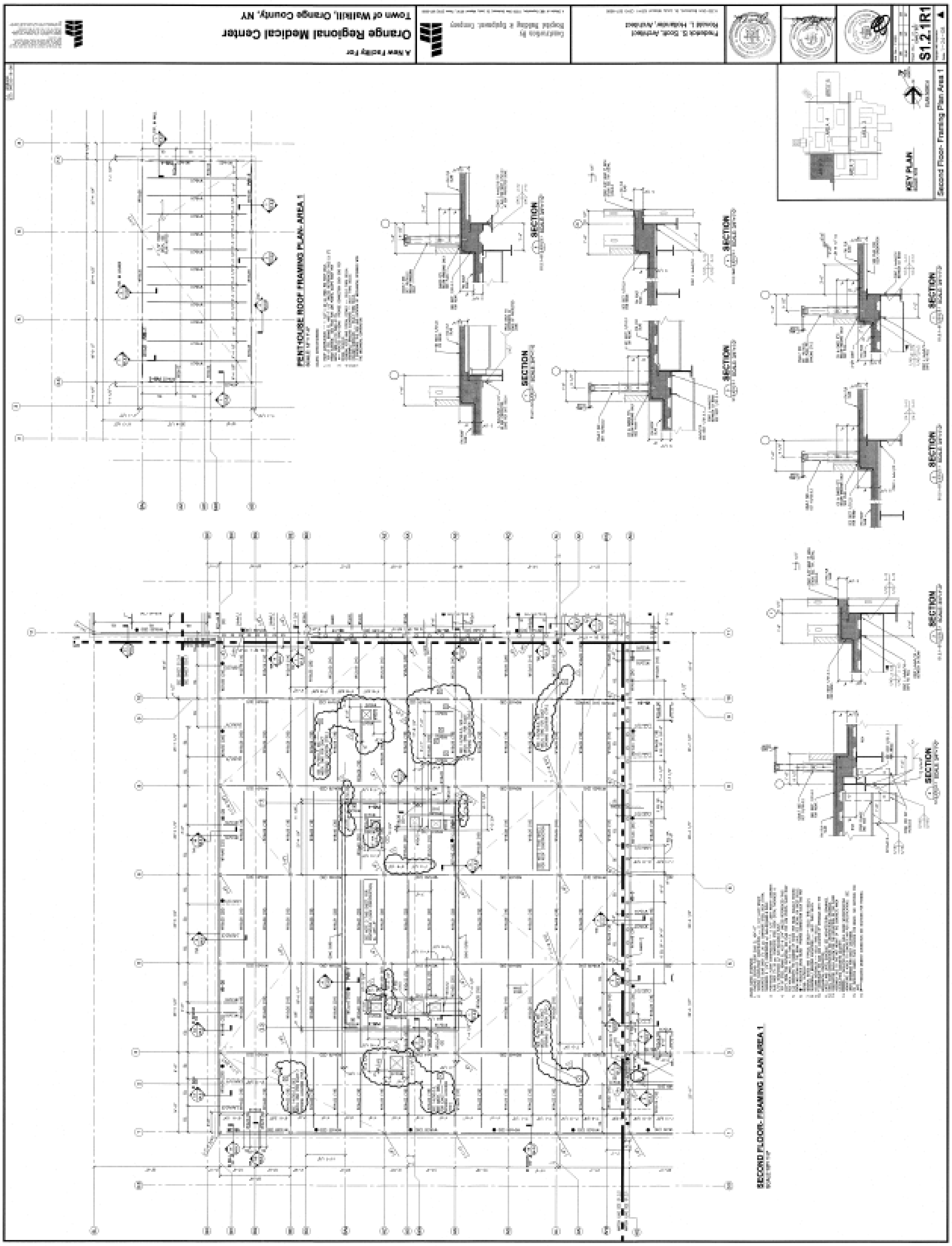
$d_t = 28(12) - 1.5''(2) = 333''$

$\varepsilon_t = \varepsilon_u\left(\dfrac{d_t - c}{c}\right) = 0.003\left(\dfrac{333 - 9.57}{9.57}\right)$
$= 0.101 > 0.005$ ok ✓

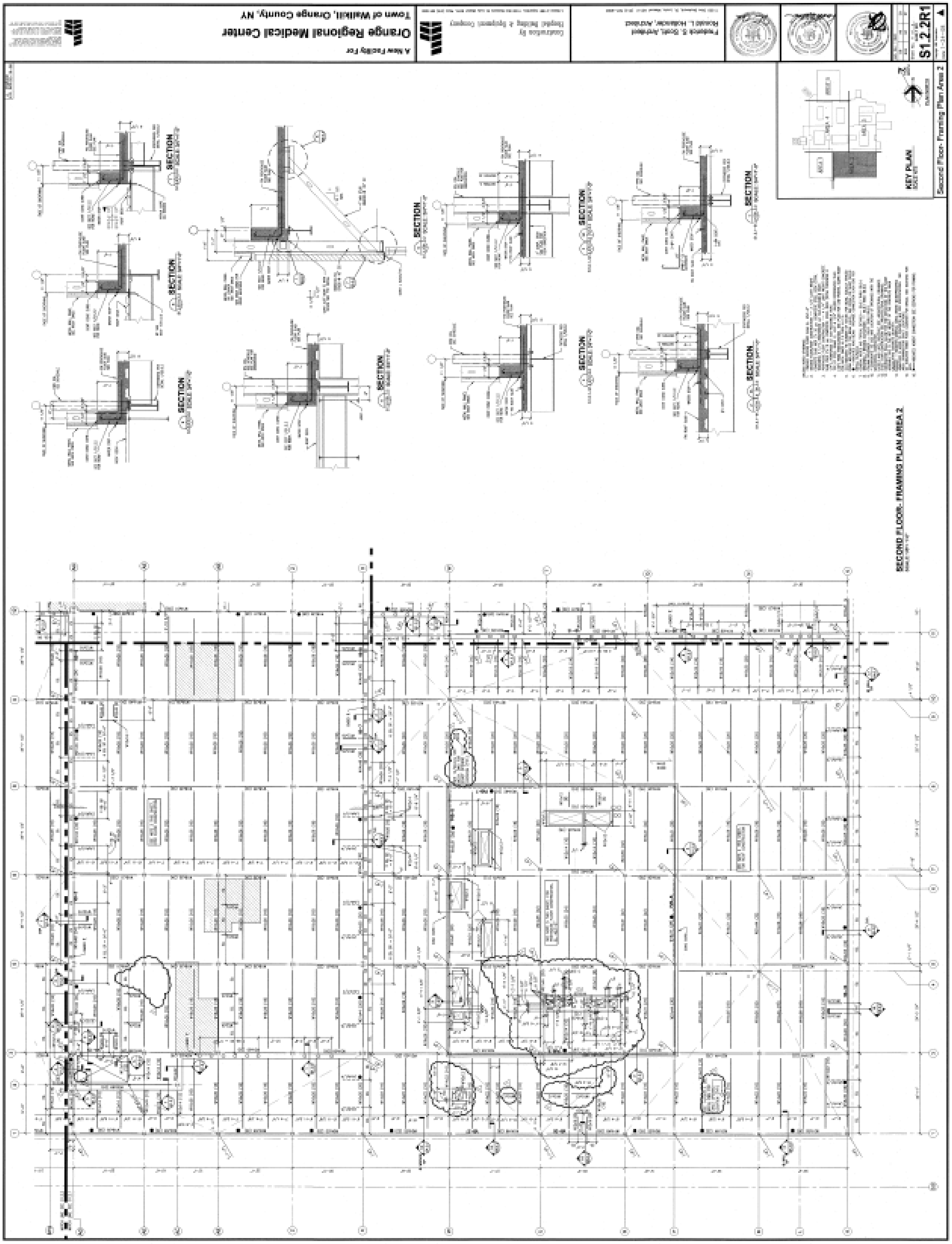
$a = \dfrac{(11.06)(60{,}000)}{0.85(6000)(16)} = 8.13''$

$c = a/\beta = 8.13/0.85 = 9.57''$

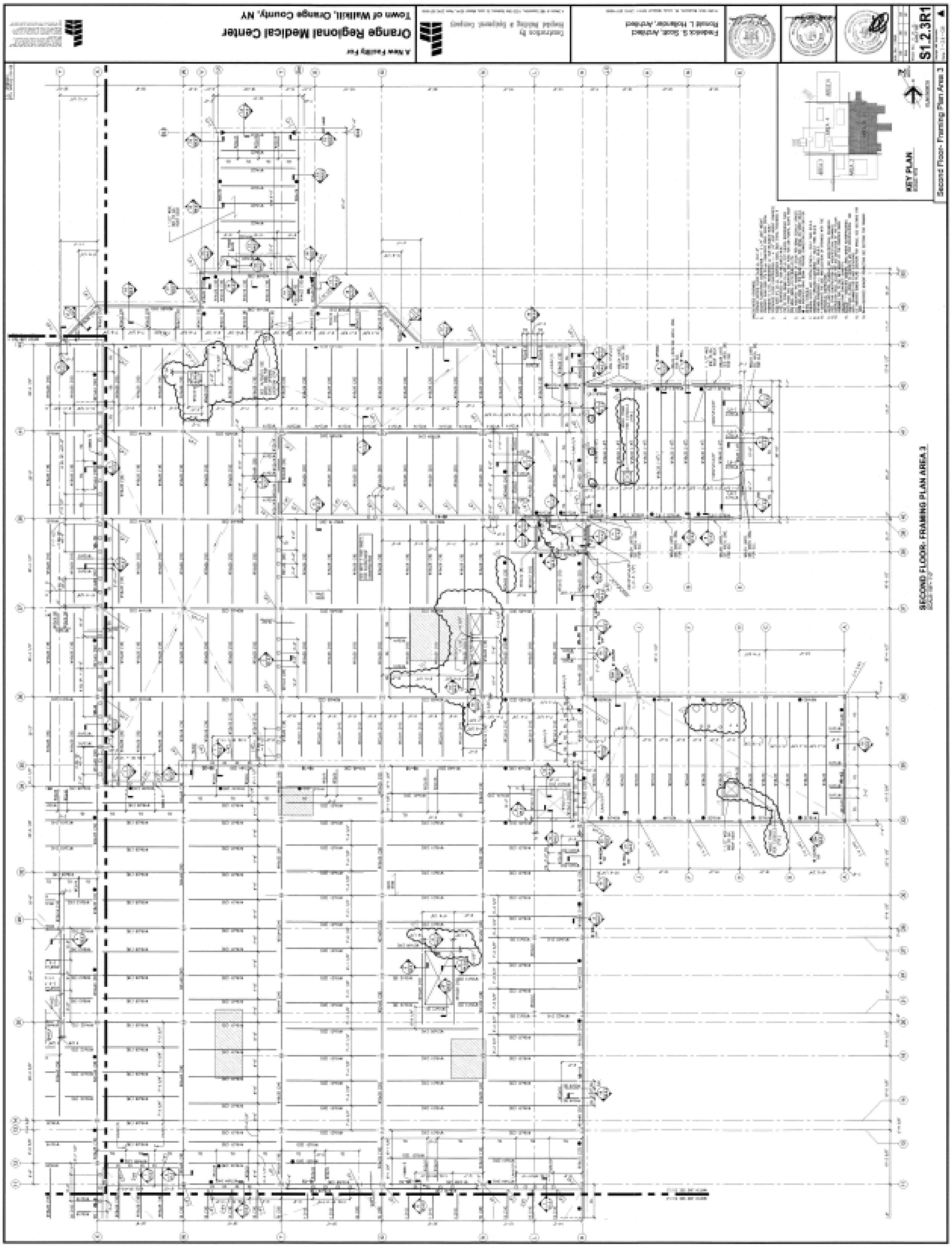
APPENDIX I: TYPICAL FLOOR PLANS – 2ND



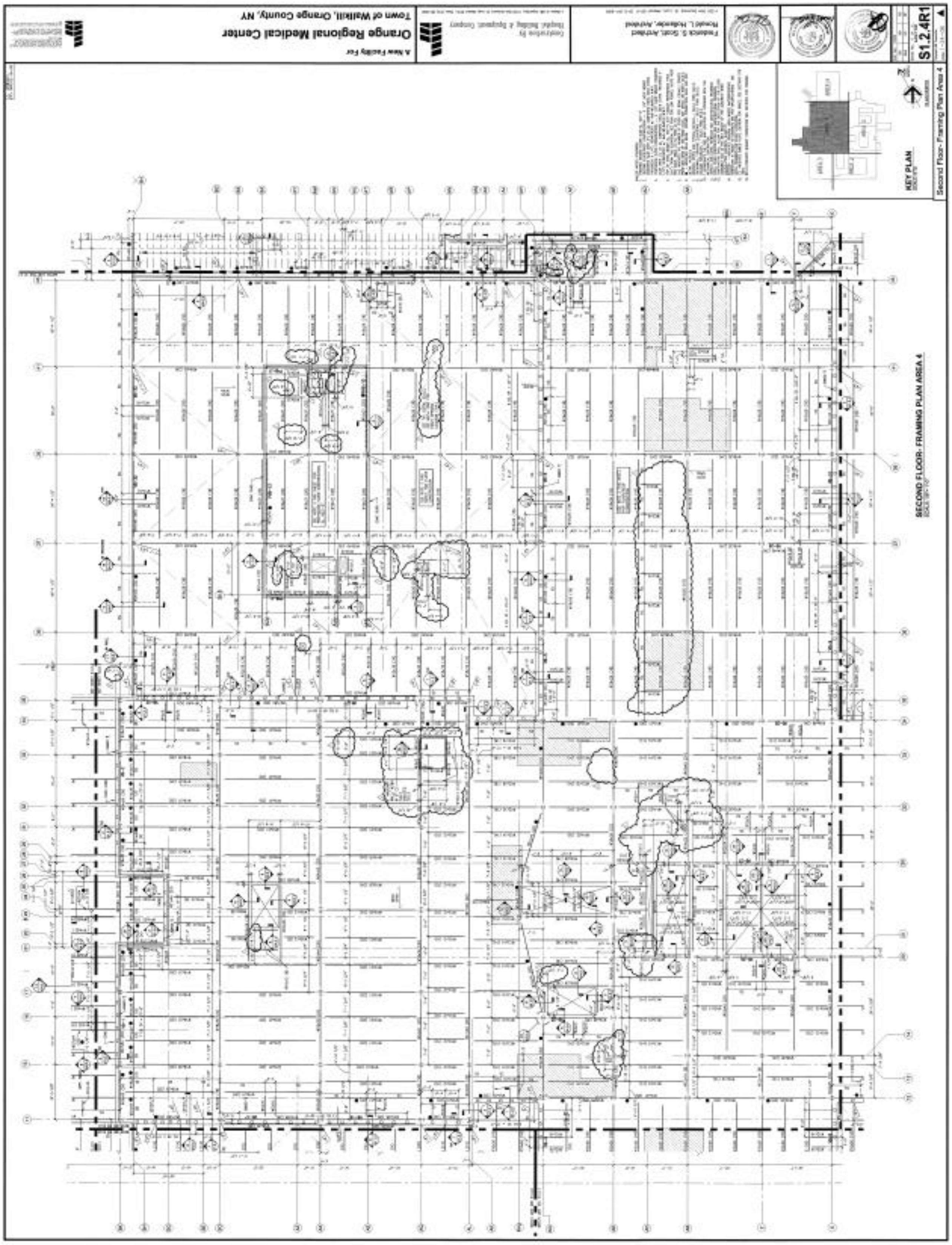
APPENDIX I: TYPICAL FLOOR PLANS – 2ND



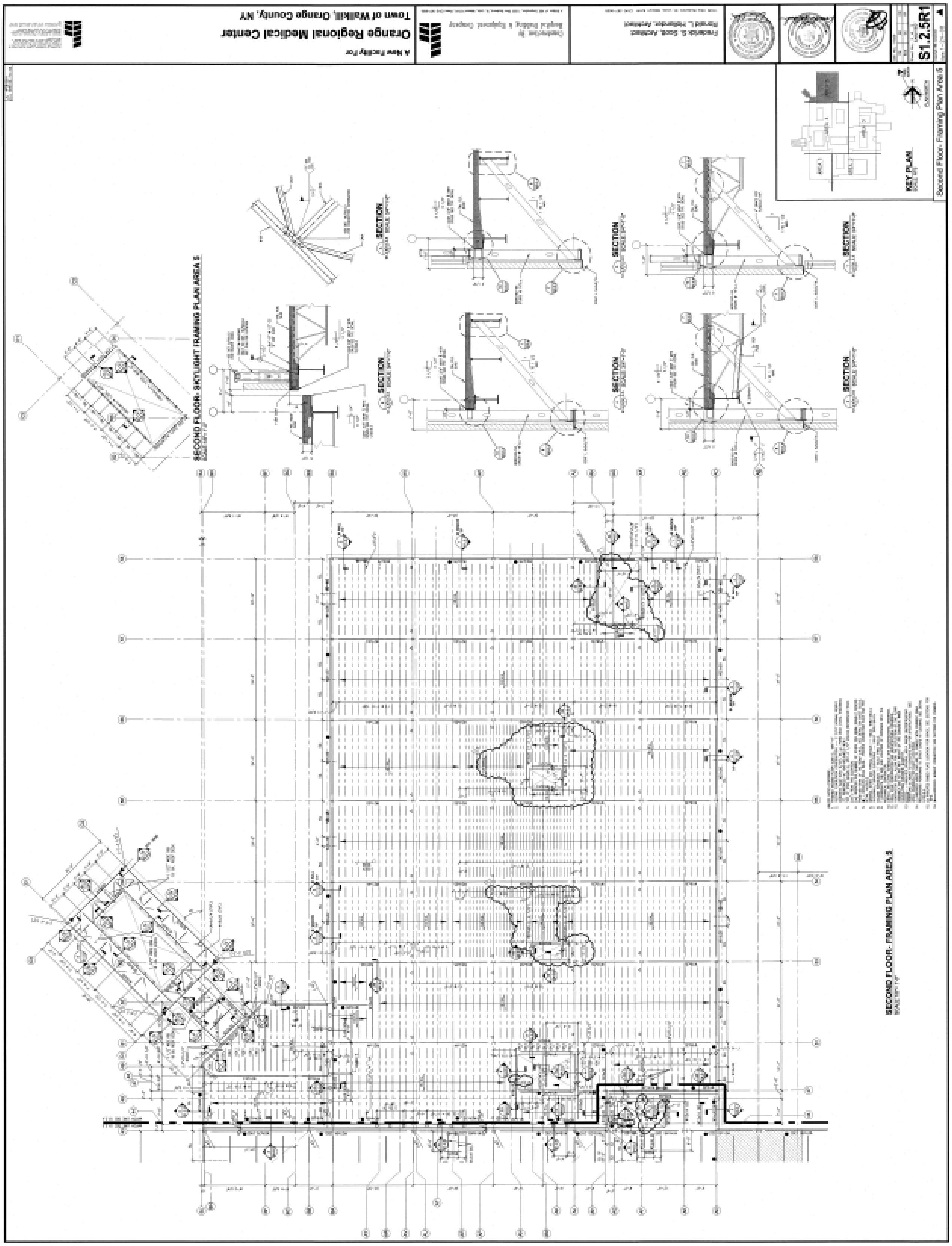
APPENDIX I: TYPICAL FLOOR PLANS – 2ND



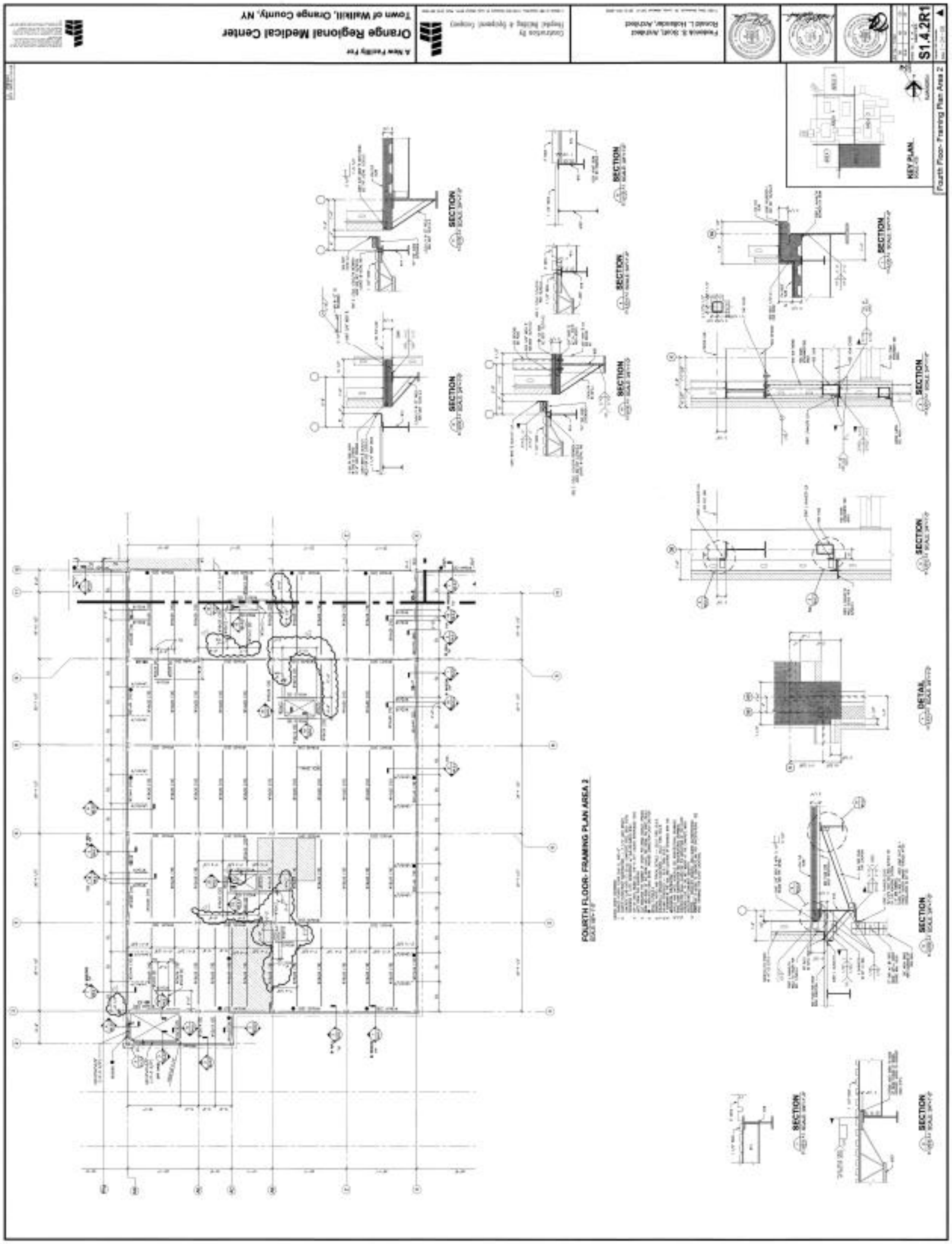
APPENDIX I: TYPICAL FLOOR PLANS – 2ND



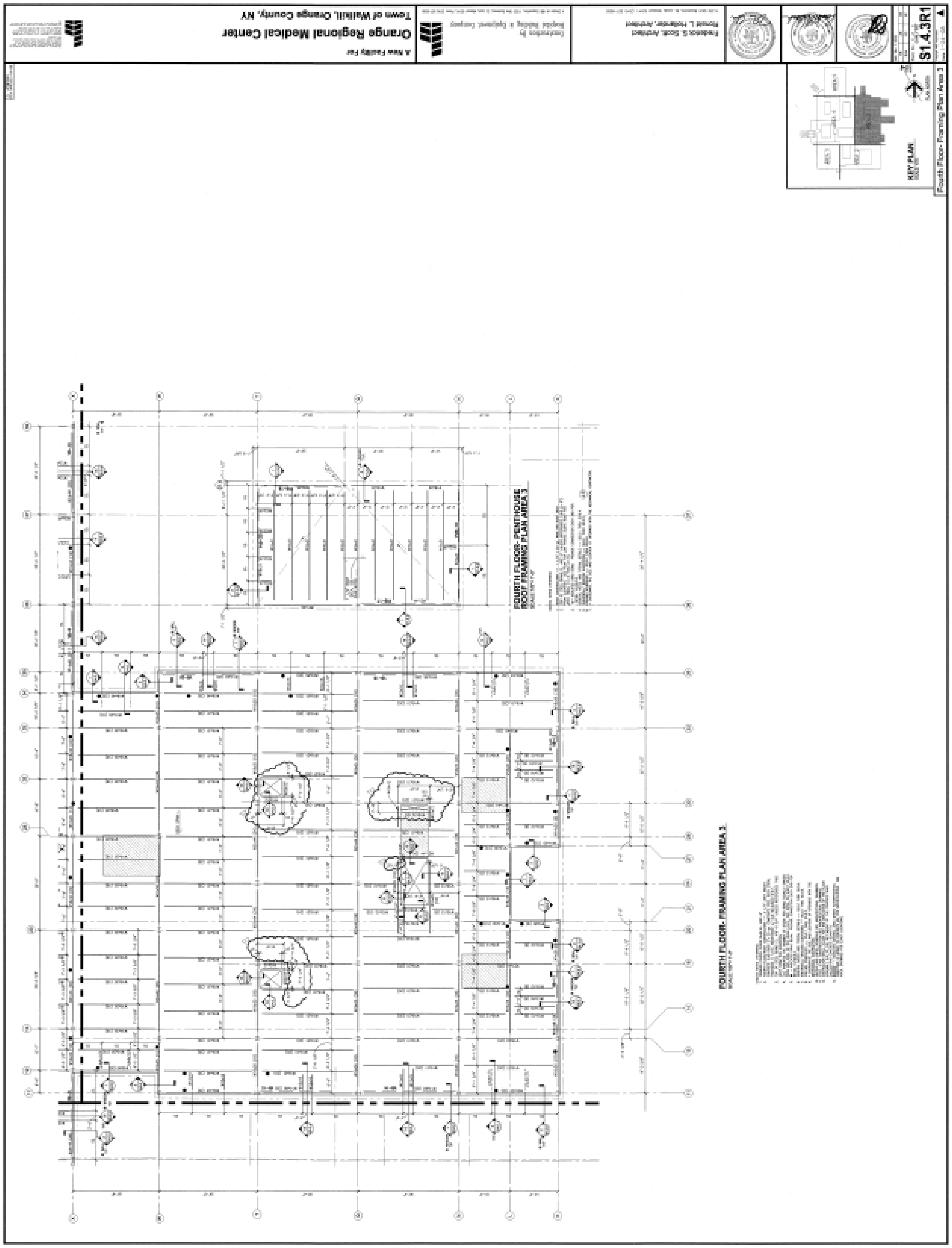
APPENDIX I: TYPICAL FLOOR PLANS – 2ND



APPENDIX I: TYPICAL FLOOR PLANS – 4TH



APPENDIX I: TYPICAL FLOOR PLANS – 4TH



APPENDIX I: TYPICAL FLOOR PLANS – 4TH