Final Report

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GENERIC BUILDING NAME

NOWHERE, USA

I EXECUTIVE SUMMARY

The purpose of this report is to discuss the structural systems of the Generic Building Name. The Generic Building Name is a Cancer research and treatment center. It is 10 floors and 10800 sf total. The main architectural feature is a two floor canopy on the northern side of the building. In this report the main gravity system, lateral system, loads and their paths and other structural features will be detailed.

The Generic Building name is a concrete building. The floor slabs are reinforced two way concrete slabs. These slabs are supported by reinforced concrete columns. The canopy is supported by four large concrete post tensioned beams. These beams are supported on both sides by large circular columns. The lateral system is ordinary reinforced shear walls. An auger cast piles system is the foundation system.

An alternate composite steel system was designed and analyzed for cost and schedule. The steel redesign includes a structural design of beams, columns and girders, special vibration bays and transfer trusses to replace the transfer girders. Also included in the alternate system is an analysis of the new structure on the mechanical system.

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1 INTRODUCTION

1.1 Purpose

The purpose of this report is to describe the existing structural conditions of the Generic Building Name and complete a redesign of the building in composite steel. A deeper understanding of the structural systems and how they interact, will be achieved by the depth of the analysis in this report. The redesign will be completed for educational purposes to gain a better understanding of composite steel framing and how it differs from concrete framing. An improvement of the existing structure is not the purpose of this report.

1.2 Scope

Included in the existing structural assignment is a description of the gravity structural system, lateral structural system, loads used, load paths, joint details, and design codes and standards.

Included in the steel alternate system design is a gravity design, lateral design, cost and schedule assessment and a mechanical system and structure conflict analysis. The Gravity design includes beam and girder framing, special vibration area framing, transfer truss framing and column design. The lateral design includes a trial brace frame design and a shear wall design.

1.3 General Building Description

The Generic Building Name is being built in a larger Medical campus of a notable university. Its purpose is to provide clinical services to cancer patients with solid tumors and provide diagnostic and treatment planning to new patients. The tenth floor will house a special breast cancer center. It is entirely funded by philanthropic donations.

In form, The Generic Building Name is a rectangular box. It's most prominent architectural feature is the large, 2 story canopy below the building on its north edge. This creates a covered space for drop off and an access road to the parking garage on the other side. The main entrance is a two-story mezzanine. Above the two bottom floors, the floor plans maintain a consistent rectangular shape with little variation. Refer to figure 1.3-1 for a typical architectural floor plan. The second floor features a connection bridge to the adjacent parking garage. Refer to Figure 1.3-2 for the space programing per floor. It is zoned in a community business district. No set backs are required in the zoning laws. The plot is 302,607 square feet. The building is total 108,000 square feet.

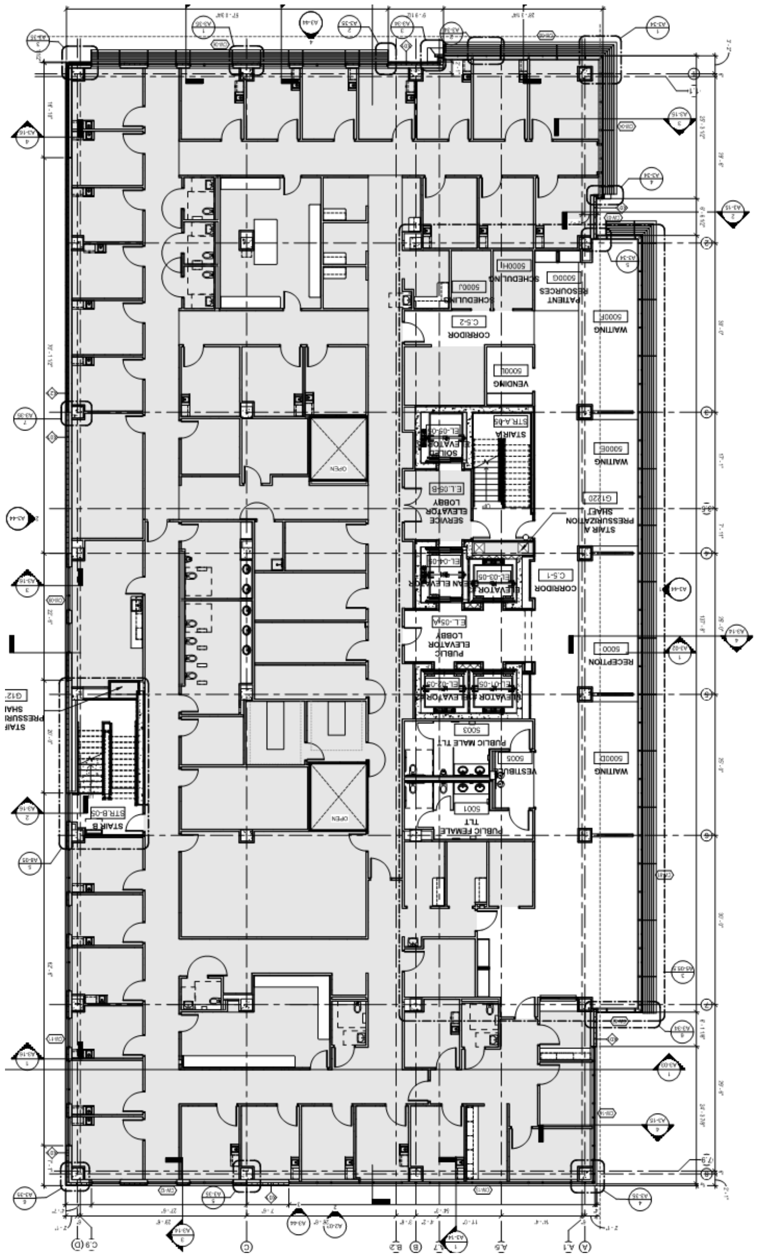


Figure 1.3-1: Typical Architectural floor plan

1st	Intake, Pharmacy, Phlebotomy
2nd	Patient Services, Clinical Research
3rd	Mechanical
4th	Imaging, Parking Garage Connection
5th	Clinic, Administration
6th	Infusion
7th	Clinical Research, Clinic
8th	Clinical Research
9th	Clinical Research
10th	Breast Clinic, Food Service
Roof	Elevator Rooms

Figure 1.3-2: Programing by floor

1.4 Brief Structural Framing Description

The Generic Building Name is a reinforced concrete building with two way reinforced concrete slabs. The Lateral system is ordinary reinforced concrete shear walls. Post tensioned beams act as transfer girders to create the canopy on the norther edge of the building. The foundation system is reinforced concrete auger piles.

2 Existing Structure

2.1 Full Structural Description

The Generic Building Name is a reinforced concrete structure. Figure 2.1-1 shows a full 3D view of the structure and Figure 2.1-2 shows a typical floor plan. The gravity system is made of concrete columns and concrete flat slabs. All floors above grade and the roof are reinforced two way slabs with a 30”x30” typical bay. Every column has a 10’x10’x8” drop cap at its connection to the slab above in order to provide punching shear capacity. Due to the building medical use, some floor areas include thicker slab to carry a heavy equipment load.

Columns above the canopy come down on to four large concrete post tensioned beams. These beams are braced by smaller concrete infill beams. The beams are supported on either side by larger columns.

The lateral system is ordinary reinforced concrete shear walls. There are 9 individual shear walls placed around fire stairs and elevators. Most shear walls extend the height of the building while some taper after the third floor.

The first floor is a slab on grade. The buildings foundation system is reinforced concrete auger cast piles. Gravity load piles sit under the columns and lateral load piles sit underneath the shear walls.

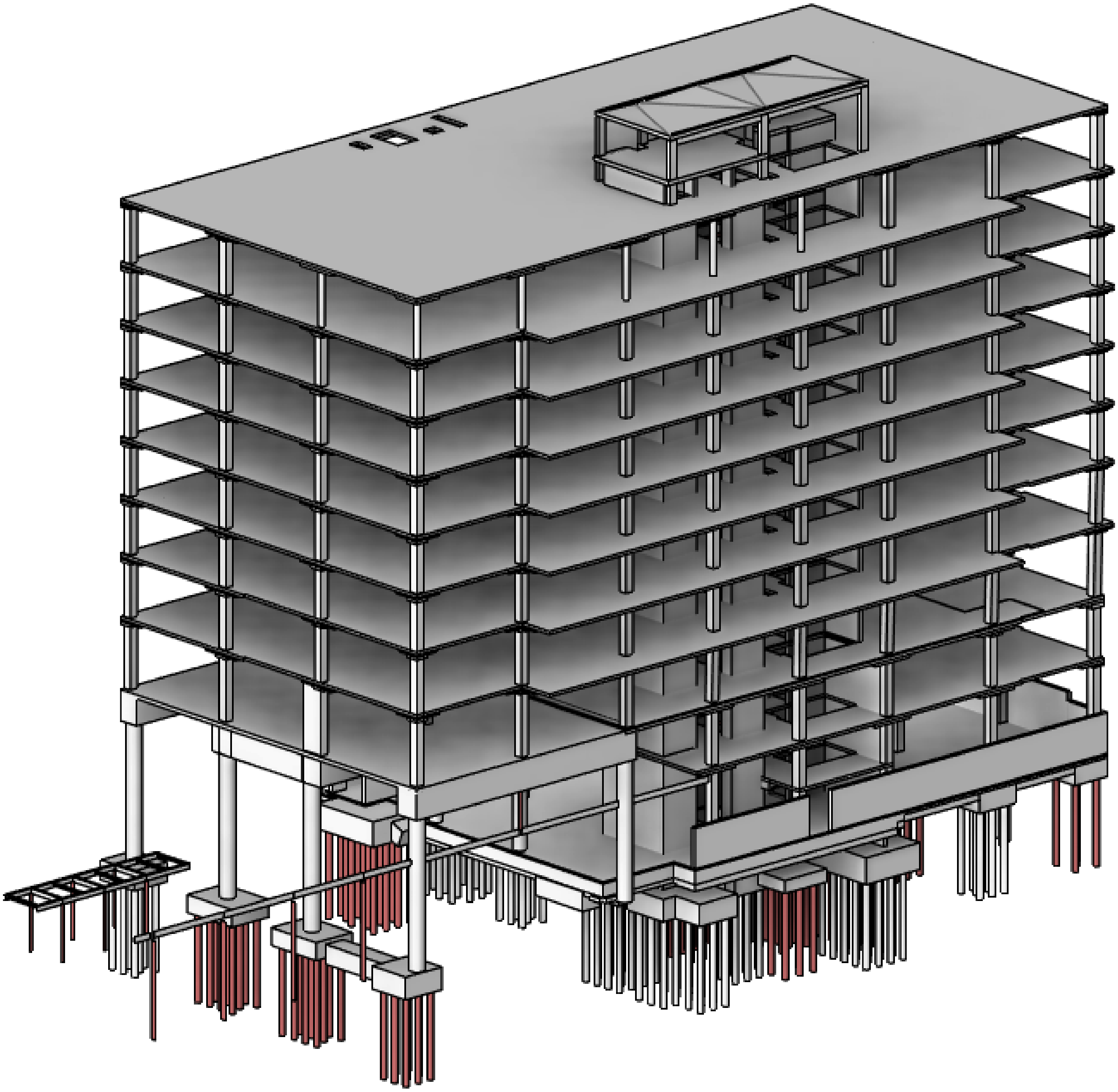
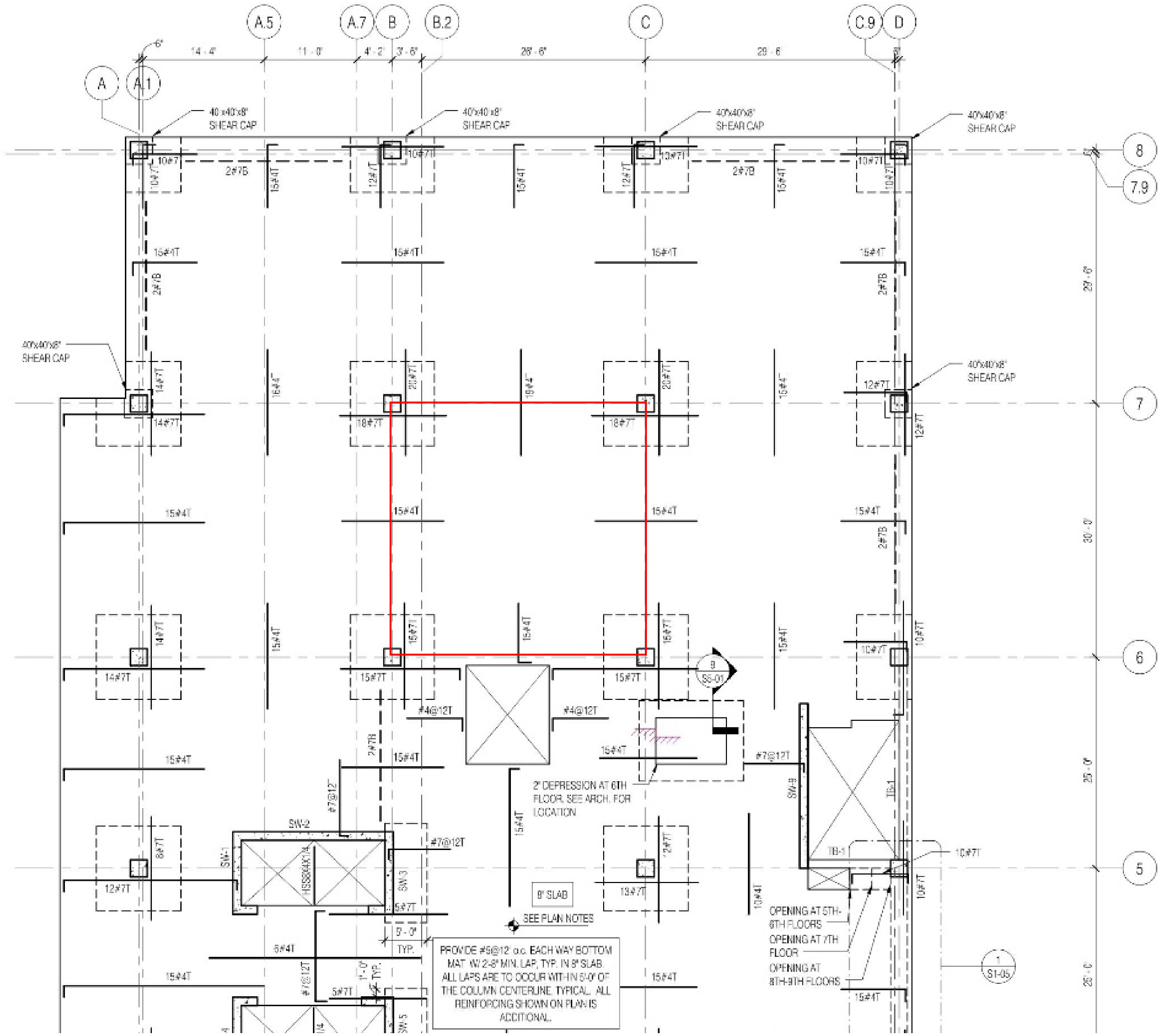


Figure 2.1-1: 3D analytical view of concrete structure



Typical Bay used in Section 2.2

Figure 2.1-2: Typical floor plan (rebar excluded for clarity)

2.2 Typical Bay

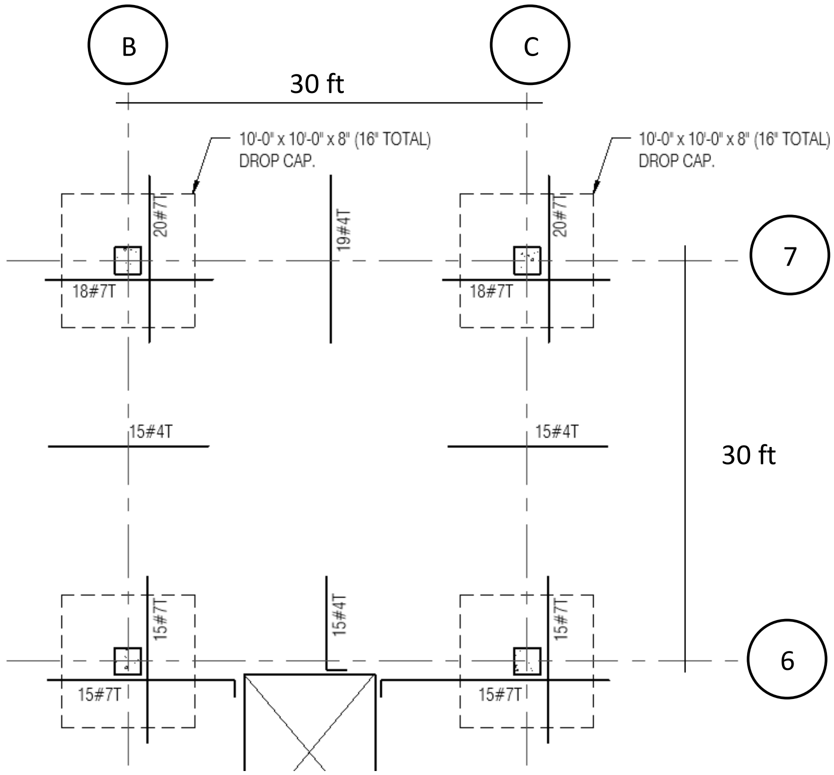


Figure 2.2-1: Typical floor bay

A typical bay in Generic Buildings Name is a 30' by 30' by 8 inch Concrete slab. All reinforced floors are 5000 psi concrete. See Appendix B for complete concrete strength chart. The slab is a two way system, therefore the slab must transfer loads in each direction to the columns. A uniform mat of #5 bars at 12" each way covers the entire bay. A minimum of two bottom bars continue through the columns. Additional bottom bars exist in special cases in other bays. See Figure 2.2-1 for rebar locations.

Top bar reinforcing is shown in Figure 2.2-1. The middle strip is 15' wide each way and the column strip is 15' each way. Top bars in the middle strip are typically #4 bars that spaced at least 12". Top bars in the column strip are typically #7 bars spaced at least 12". Each column has a 10'x10'x8" drop cap to help with punching shear.

2.3 Columns

The columns in Generic Building Name are assumed to only contribute to the gravity system; they are not designed to take any lateral load. All columns are reinforced. Five thousand psi or 6000 psi concrete mix was used. The majority of columns are 24"x24" with 8 #8 rebar in equally spaced along all sides. Forty two inch and thirty two inch circular columns support the transfer girders on either side. The circular columns have a corresponding circular rebar layout with # 10 rebar. More in depth information about the columns and their reinforcing can be found in the column schedule in Appendix B. Number 4 or #3 ties were used on all columns with between 10” and 22” spacing.

2.4 Lateral System

Lateral Loads applied to Generic Building name are resisted by ordinary reinforced concrete shear walls. Shear wall are constructed using 5000 psi concrete. See figure 2.4-1 for location of shear walls. The lengths of shear walls 8 and 9 decreases above the 3rd floor.

Shear walls are reinforced using #5 bars at 12” horizontally and vertically. Rebar chords are included in the ends of all shear walls. Six, eight and ten bar patterns of #8 bars are used in chords. Rebar in shear walls is to be spliced with a 2 foot minimum staggered splice.

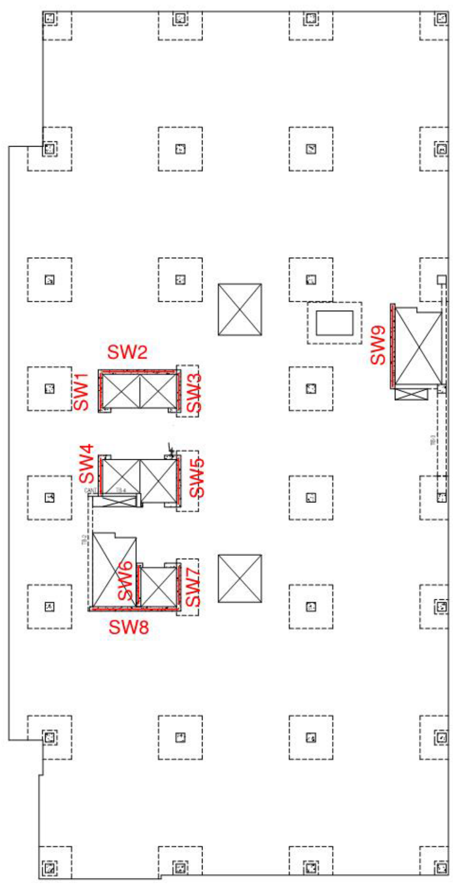


Figure 2.4-1: First Floor plan with highlighted shear wall locations

2.5 Load Paths

Gravity loads, such as dead, live, and snow loads are transferred from the two way slabs directly to the columns. The columns then take the loads down to the pile caps. Pile caps transfer the loads to the piles. Loads are transferred out of the building finally from friction between the piles and earth. In the case of columns above the canopy, loads are taken from the columns then transferred to the larger column by way of the large PT beams on the third floor (Figure 2.5-1 A). In the case of the column along column line D, the columns transfer the load to a grade beam then to the foundation system (Figure 2.5-1 B).

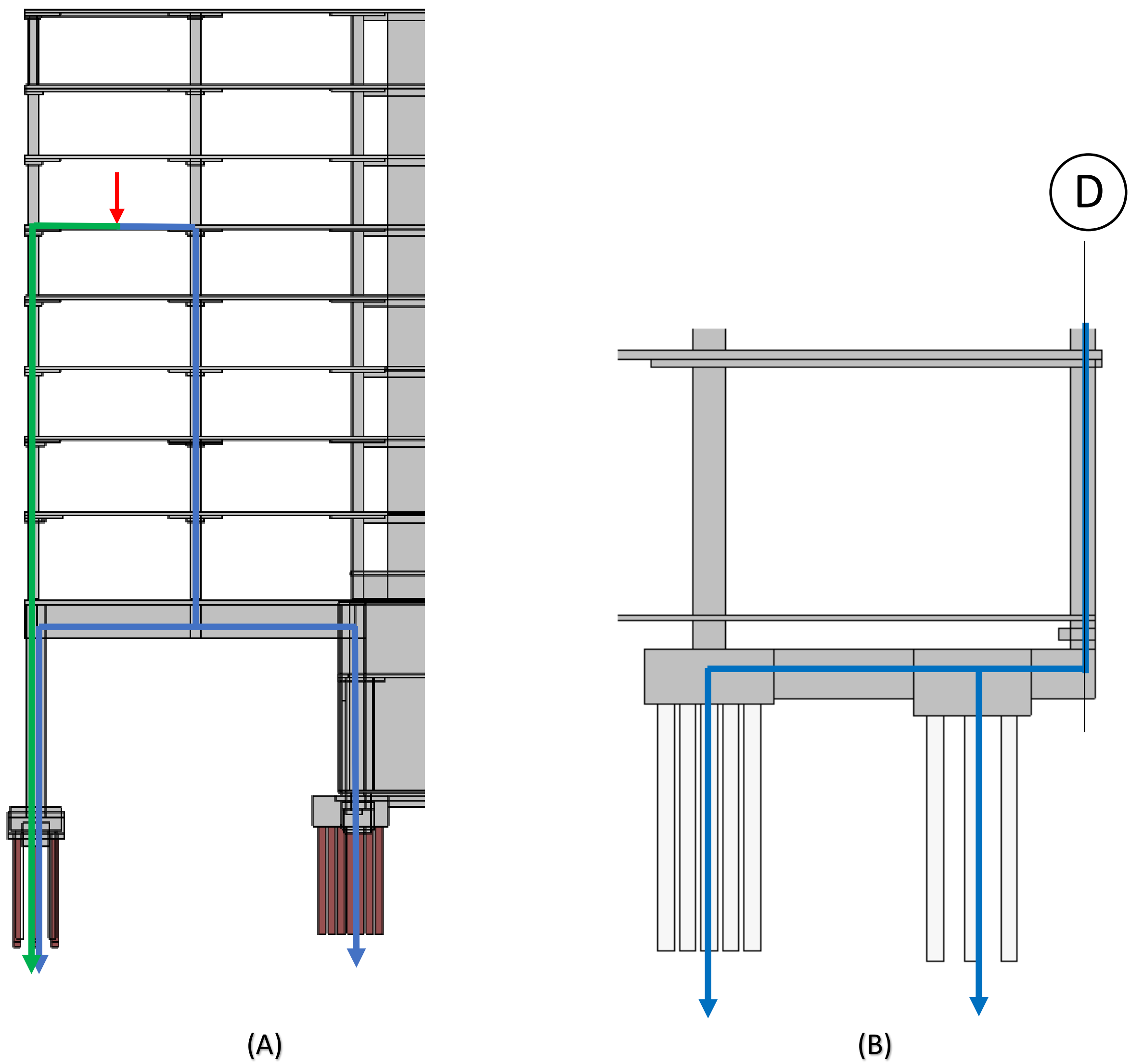


Figure 2.5-1: Special gravity load Paths for a sample load

Lateral wind pressure loads are transferred from the building envelope to the slabs. The slabs then transfer the loads to the shear walls which carry the loads down to the lateral foundations. Seismic equivalent loads are transferred from the slabs to the shear walls and from the shear walls down to the lateral foundations.

2.6 Other Structural Elements

The most notable additional structural elements in the Generic Building Name are the Post Tensioned concrete beams above the canopy. These beams act as transfer girders from the column above to the six columns below. In order to accommodate the loads from the upper floors, the beams are 72”x84” and 60”x84”. Figure 2.6-1 is a typical detail view of a PT beam.

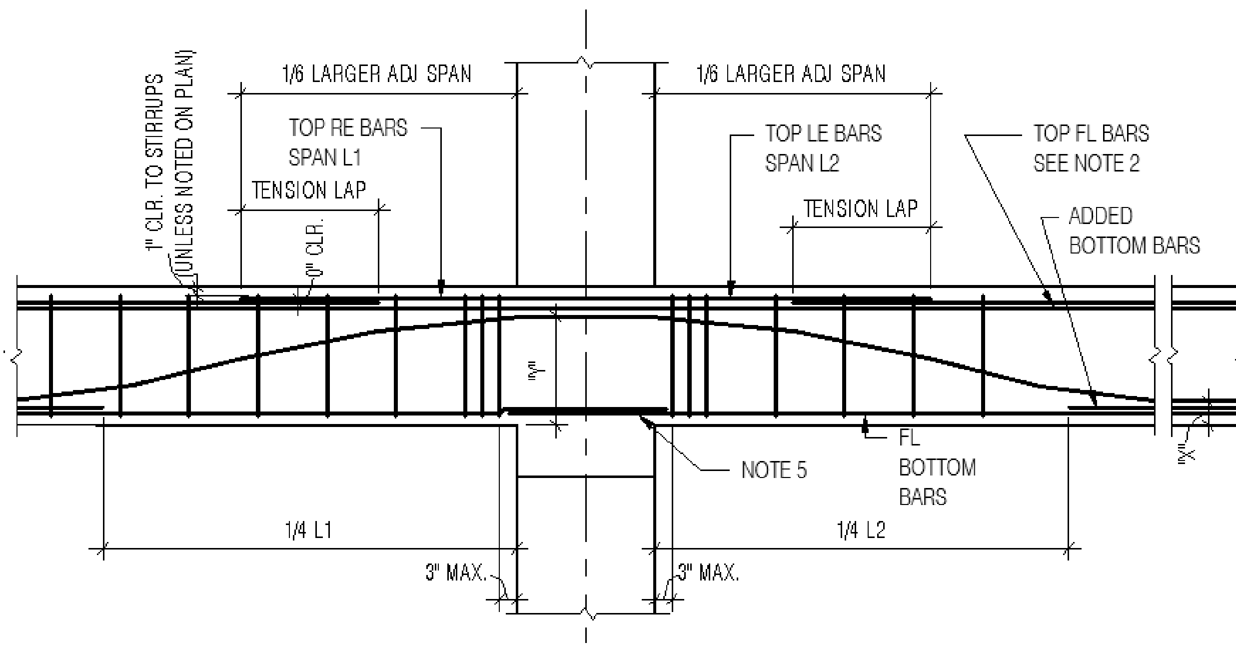


Figure 2.6-1: PT Beam detail

In order to support heavy medical equipment and HVAC equipment, there are drops in the 2nd ,3rd, 4th, and Roof Slabs. The Second floor connection bridge is an 7” and 8” cantilevered slab. The bridge roof is a similar 7” cantilevered slab.

2.7 Foundation System

The first floor of the Generic Building Name is a 5” slab on grade. The geotechnical report shows no suitable bed rock before 99 feet. All Foundations under the Generic Building Name are auger cast piles. All piles extend 46 feet into the ground. Each pile cap has between four and 21 piles. Two separate pile details are used, one for the lateral system and one for the gravity system. Lateral piles are reinforced with 6 #6 rebar. The reinforcing in the lateral piles extend 30 feet into the ground. Gravity piles are reinforced with 4 #5 vertical bars. The reinforcing in the gravity piles extend 12 feet into the ground. The piles have an axial capacity of 120 tons and a 28 kip lateral capacity.

Columns along column line D and south of column line 5 do not sit directly on pile caps. In these cases, grade beams are used to connect the column to the foundation system. A grade beam also extends underneath the retaining wall along southern side, see Figure 2.7-1.

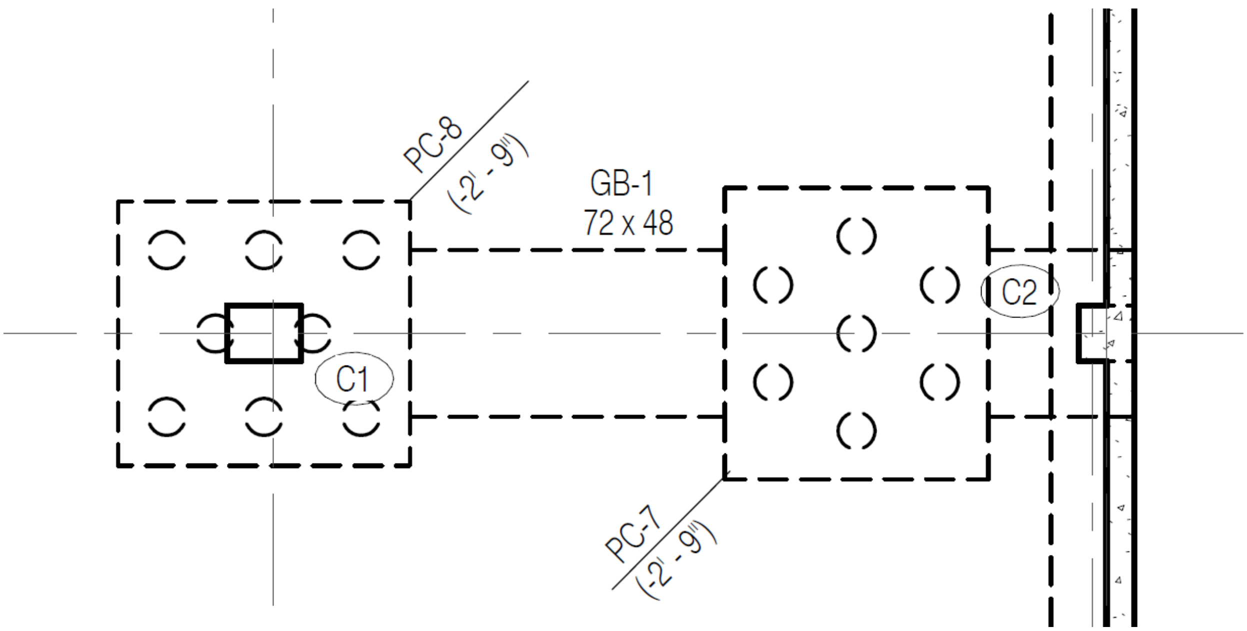


Figure 2.7-1: Grade Beam in Plan

2.8 Design Loads

Live loads per area type are as follows in Figure 2.8-1. The live loads were determined using ASCE 7-10. Loads followed by a [U] are unreducible. All other loads are reducible. A minimum of 30 psf roof live load was used.

AREA	LIVE LOAD	PARTITIONS
ASSEMBLY AREAS - LOBBIES	100 PSF [U]	N / A
BALCONIES - EXTERIOR	100 PSF	N / A
CORRIDORS	100 PSF	N / A
CORRIDORS - ABOVE 1ST FLOOR	80 PSF	N / A
HOSPITALS - PATIENT ROOMS	40 PSF	15 PSF
MECHANICAL ROOMS	150 PSF [U]	N / A
OFFICES	80 PSF	15 PSF
STAIRS & EXITWAYS	100 PSF [U]	N / A

Figure 2.8-1: Live Loads per area type

Superimposed Dead loads per area type are as follows. These loads were used in addition to self-weight of the structure.

AREA	SD LOAD
TYP. FLOORS	40 PSF
ROOF	30 PSF
MECHANICAL - 3RD	30 PSF
IMAGING - 4TH	30 PSF

Figure 2.8-2: Dead Loads per area type

Wind Loads were determined in accordance with ASCE 7-10. The design criteria is labeled in the Figure 2.8-3. See Appendix D for Cladding and components wind pressures.

Exposure Category	B
Ultimate Design Wind speed	115
Risk Category	III
Internal Pressure Coefficient	+/- 0.18

Figure 2.8-3: Wind load Design parameters

Seismic loads were determined using the equivalent force method. Figure 2.8-4 contains all factors used to determine the seismic loads on the structure. The seismic loads resulted in a base shear of 425 Kips.

Risk Category	III
Seismic Importance Factor, Ie	1.0
Mapped short Period spectral response acceleration, Ss	0.160
Mapped 1-second period spectral response acceleration, S1	0.053
Short period design spectral response coefficient, Sd1	0.085
Soil Site Class	D
Seismic design category	B
Response modification factor, R	4.5

Figure 2.8-4: Seismic load design parameters

In calculating the flat roof snow load conservative values of 1.0 were used for the exposure, and thermal factor. The Flat roof factor is 18 psf. Drifting and sliding loads were applied where applicable in the building. Figure 2.8-5 contains variables used in calculating flat roof snow load.

Ground Snow Load, Pg	25 psf
Snoe exposure factor	1.0
Snow Load importance Factor	1.0
Thermal Factor	1.0
Flat Roof Load	18 psf

Figure 2.8-5: Snow load design parameters

2.9 Design Codes and Standards

Generic Building name was built to the 2012 version of International Building Code (IBC). The concrete was designed to ACI 318 and ACI 117. ACI 336.1 was consulted in the design of the drilled piers. All welding was done to ANSI/AWS welding code. Additional standards used were: Manual of standard practice concrete reinforcing, Post tensioning manual from the Post tensioning institute, PCI design hand book, Steel Construction Manual, 14^{th} edition, 2010, and Detailing for Steel construction from AISC.

2.10 Joint Details

Shown in Figure 2.10-1 is a typical detail of a floor slab to column connection. A minimum of two rebar must continue through all interior columns in both directions and a minimum of two rebar must terminate in 90 degree hook in all eternal columns.

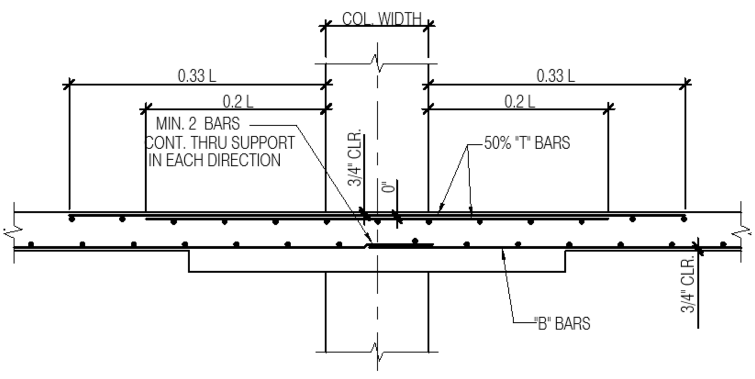


Figure 2.10-1: Slab to column connection detail

Figure 2.10-2 shows the joint detail between two slabs. The slab reinforcing should continue through the joint. Note 2 in the detail refers to additional required 4 foot #4 dowels placed at 18” on center in the middle of the joint.

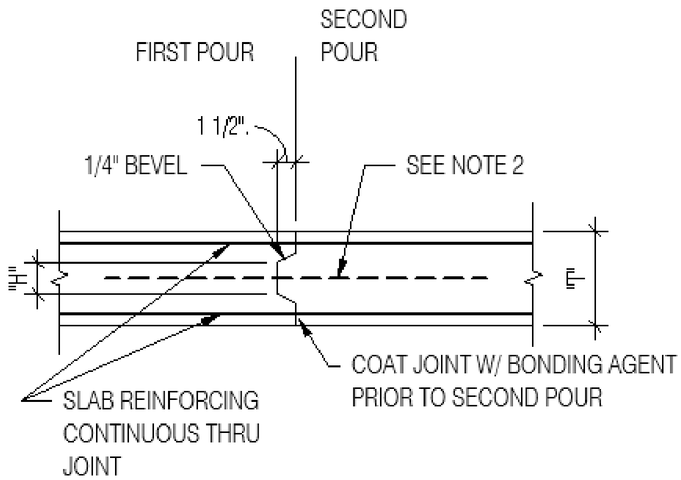


Figure 2.10-2: Slab joint detail

Figure 2.10-3 shows the Connection of the columns to pile caps. Additional top reinforcement is required at tension pile caps.

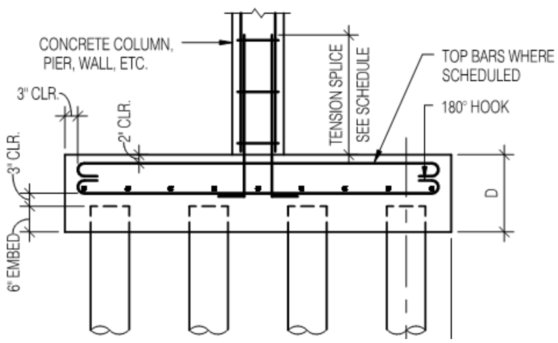
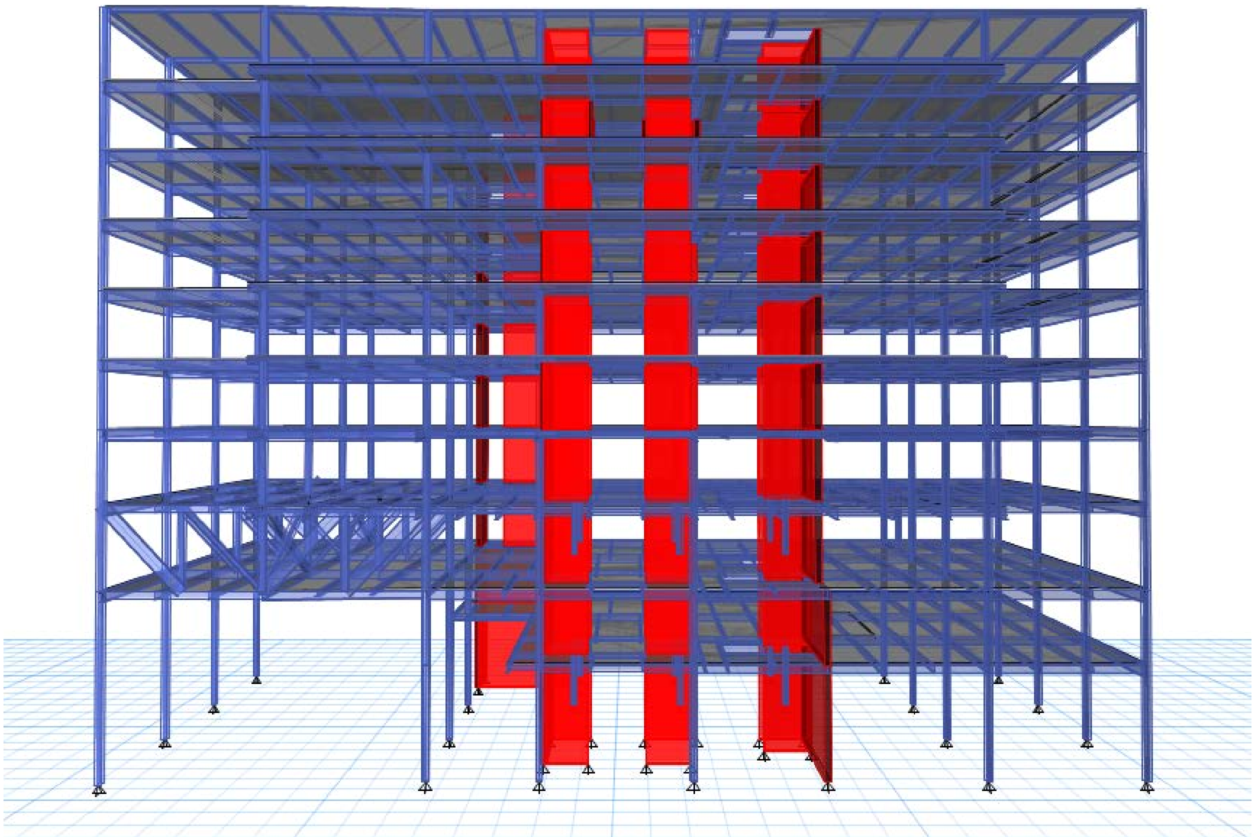


Figure 2.10-3: Column to foundation connection

3 Alternate Gravity Design



The alternate design chosen for The Generic Building Name, was a steel system with composite beams and girders. A vibration analysis and design of the MRI imaging room framing, and a design of full story transfer trusses was also completed. Columns were designed but the effects of the new system was on the foundations was out of the scope of this thesis.

3.1 Typical Framing

The Same bay size was use except the edge columns on the east and west Facades were moved outward 1' 6" to decrease the deck over hang. The corresponding footing will have to be moves as well to decrease the eccentricity on the footings. A typical interior bay is 30"x30" and a typical exterior bay is 31.5' x 30'. Vulcraft 2VLI steel decking was used on all floors, the max span is 10'. This creates an even three span typical bay with beam spacing at 10', as seen in figure 3.1.

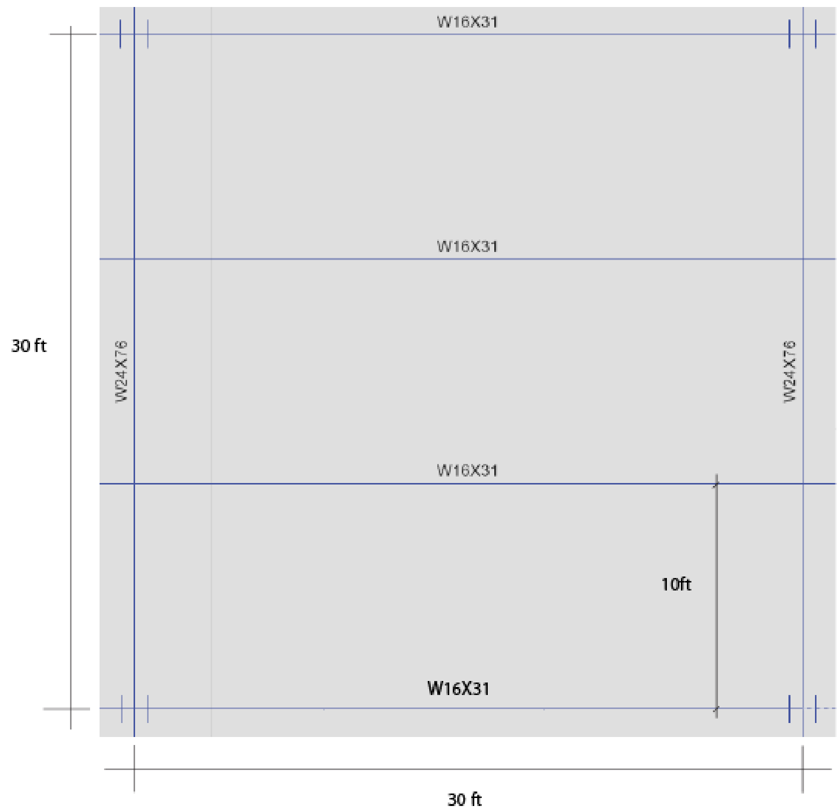
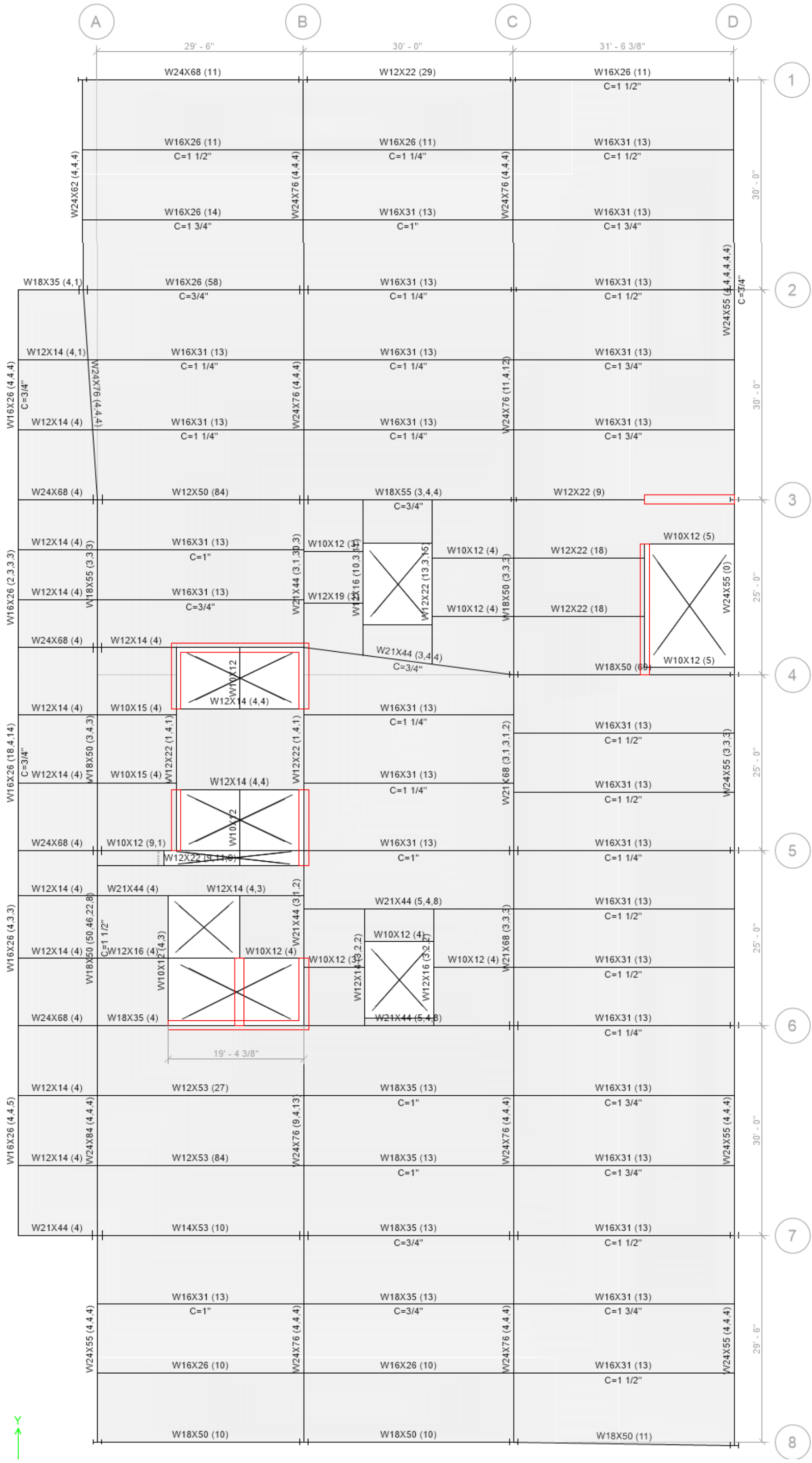


Figure 3.1 -2: *Typical bay*

The loading on a typical bay is plan is 95 live load (office) and a (40 dead load). Typical beam sizes are W16x31 and girder sizes W24X76. The beam design moment is 298 kft and the capacity is 309 kft. The Girder design moment is 677 kft and the reduced moment capacity is 750 kft. Edge beams were assigned an extra .36 klf line load to account for slab overhang and façade weight. A typical edge beam is W16x26. Studs and camber are not shown here but are shown in the floor plans in the appendix.

On the plan east side of the building is a cantilevered floor bay. The east west beams were designed with fixed connections. The adjacent bay was also modeled with fixed connection to control torsion on the girder running parallel to the overhang. The typical sizes of the cantilevered beams are W24X68.

Floor plans were design for varying loading based on area type. See Section 2.8 for loading types. See Appendix A for all floor plans floor plans and their framing. See Appendix B for gravity design hand checks of a typical beam, girder and cantilevered beam.


A
B
C
D
29' - 6"
30' - 0"
31' - 6 3/8"
1
2
3
4
5
6
7
8
W24X68 (11)
W12X22 (29)
W16X26 (11)
C=1 1/2"
W16X26 (11)
C=1 1/2"
W16X26 (11)
C=1 1/4"
W16X31 (13)
C=1 1/2"
W24X62 (4,4,4)
W24X76 (4,4,4)
W24X76 (4,4,4)
W16X26 (14)
C=1 3/4"
W16X31 (13)
C=1"
W16X31 (13)
C=1 3/4"
W18X35 (4,1)
W16X26 (58)
C=3/4"
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/2"
W12X14 (4,1)
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 3/4"
W16X26 (4,4,4)
C=3/4"
W24X76 (4,4,4)
W24X76 (4,4,4)
W24X76 (11,4,12)
W12X14 (4)
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 3/4"
W24X68 (4)
W12X50 (84)
W18X55 (3,4,4)
C=3/4"
W12X22 (9)
W12X14 (4)
W16X31 (13)
C=1"
W10X12 (4)
W10X12 (4)
W12X22 (18)
W10X12 (5)
W16X26 (2,3,3,3)
W18X55 (3,3,3)
W21X44 (3,1,30,3)
W18X50 (3,3,3)
W24X55 (0)
W12X14 (4)
W16X31 (13)
C=3/4"
W10X12 (4)
W12X22 (18)
W24X68 (4)
W12X14 (4)
W21X44 (3,4,4)
C=3/4"
W18X50 (66)
W10X12 (5)
W12X14 (4,4)
W12X14 (4)
W10X15 (4)
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/2"
W16X26 (18,4,14)
C=3/4"
W18X50 (3,4,3)
W12X22 (1,4,1)
W12X22 (1,4,1)
W21X68 (3,1,3,1,2)
W24X55 (3,3,3)
W12X14 (4)
W10X15 (4)
W12X14 (4,4)
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/2"
W24X68 (4)
W10X12 (9,1)
W16X31 (13)
C=1"
W16X31 (13)
C=1 1/4"
W12X14 (4)
W21X44 (4)
W12X14 (4,3)
W21X44 (5,4,8)
W16X31 (13)
C=1 1/2"
W16X26 (4,3,3)
C=1 1/2"
W12X14 (4)
W12X16 (4)
W10X12 (4)
W10X12 (4)
W10X12 (4)
W21X68 (3,3,3)
W16X31 (13)
C=1 1/2"
W24X68 (4)
W18X35 (4)
W21X44 (5,4,8)
W16X31 (13)
C=1 1/4"
19' - 4 3/8"
W12X14 (4)
W12X53 (27)
W18X35 (13)
C=1"
W16X31 (13)
C=1 3/4"
W16X26 (4,4,5)
W24X84 (4,4,4)
W24X76 (9,4,13)
W24X76 (4,4,4)
W24X55 (4,4,4)
W12X14 (4)
W12X53 (84)
W18X35 (13)
C=1"
W16X31 (13)
C=1 3/4"
W21X44 (4)
W14X53 (10)
W18X35 (13)
C=3/4"
W16X31 (13)
C=1 1/2"
W16X31 (13)
C=1"
W18X35 (13)
C=3/4"
W16X31 (13)
C=1 3/4"
W24X55 (4,4,4)
W24X76 (4,4,4)
W24X76 (4,4,4)
W24X55 (4,4,4)
W16X26 (10)
W16X26 (10)
W16X31 (13)
C=1 1/2"
W18X50 (10)
W18X50 (10)
W18X50 (11)
Y

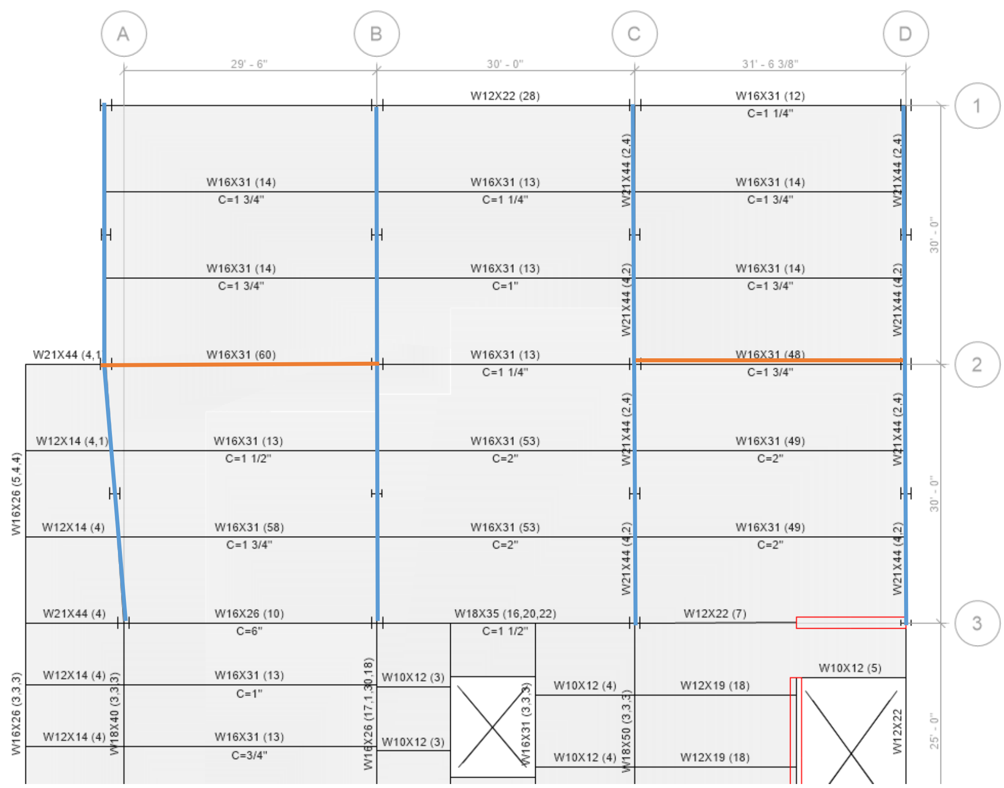
3.2 MRI Vibration Design

Three MRI imaging rooms are on the west side of the fourth floor, see fourth floor plan in appendix A. AISC Design Guide 11 was followed to calculate the vibration limits and check the capacity. No corridors go through the bay being designed, it is entirely a MRI room there for a very slow walking speed was used. A class C for vibration limit was used. The maximum one third octave constraint for very slow, class C is 500 mips. A dampening factor of .007 was used due to the partitions around and through the bay. After analyzing the bay layout with full beam lengths of 30' it was determined that the spans were too long. So intermediate columns, and girders were added to reduce the beam length to 15', see figure 3.2. The floor below is a mechanical floor so the added columns will not interrupt any architecture below. This reduced the panel frequency, fn, to 10.22. The designed one third octave for the new bay was calculated to be 490 mips, less than the 500 mips constraint. See appendix E for hand calculations and vibration bay location on architectural floor plans.

Figure 3.2: *MRI Vibration Control Framing*

3.4 Transfer Trusses Design

To transfer the loads from the columns along grid line 2 to the column lines 1 and 3. A full floor height transfer truss will be used in the 3rd floor. Secondary trusses were attached from the interior trusses to the exterior to reduce the steel sizes of the interior trusses. Two different truss designs were tested Truss A (figure 3.4-3) and Truss B (figure 3.4-4). Panel length are half bay length equaling 15'. Panel height are the full Story height of 13 ft. A 1500 kip test load was applied to the center joint to text the deflection and preliminary brace sizes. Truss A yielded less deflection than truss B. Truss A put the cross bracing into compression so it yielded much larger design sections than truss B. When modeled in the entire building the deflection of Truss A was 1.5” under the L/360 deflection limit of 2” so truss configuration A was selected. A secondary truss was added along grid line 2 to help create a uniform flexure between the trusses.



Truss Locations

Secondary Truss Locations

Figure 3.4-2: *4th floor plan*

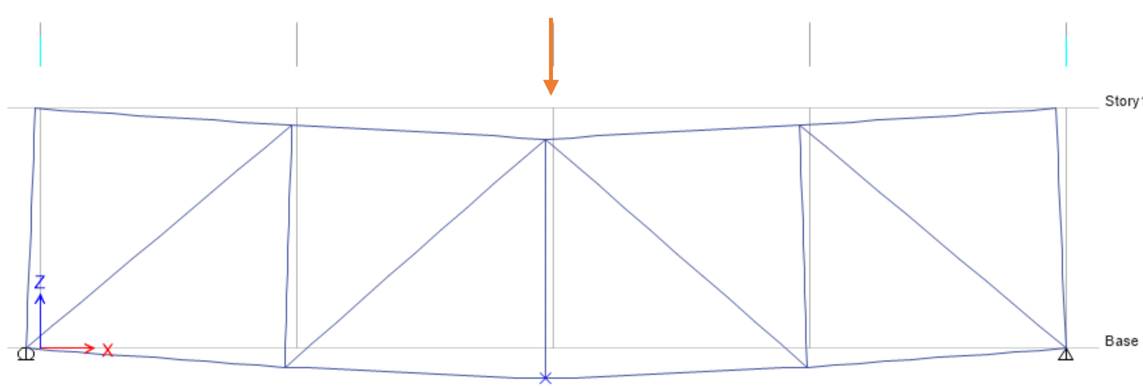


Figure 3.4-3: *Truss Configuration A with Deflection shown*

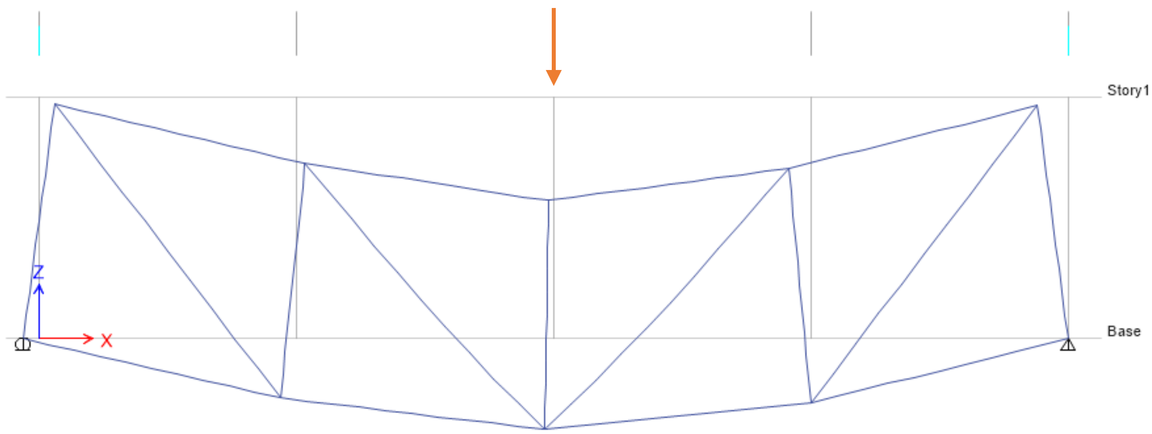


Figure 3.4-4: *Truss Configuration with Deflection shown*

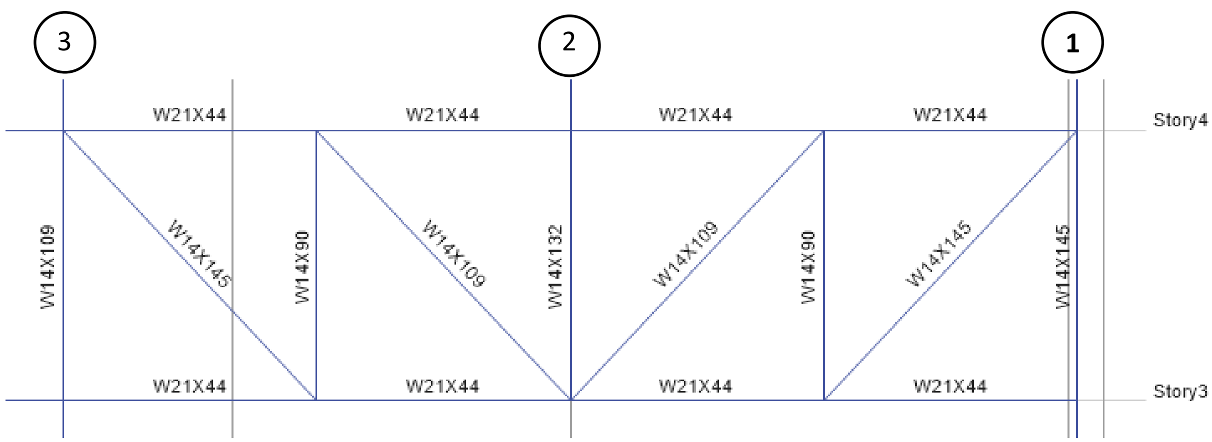
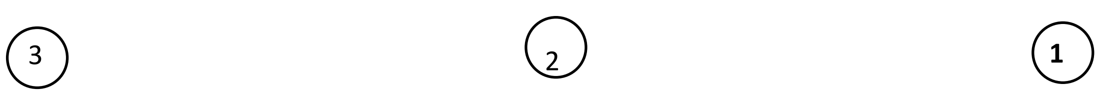


Figure 3.4-5: *Interior Truss Elevation*



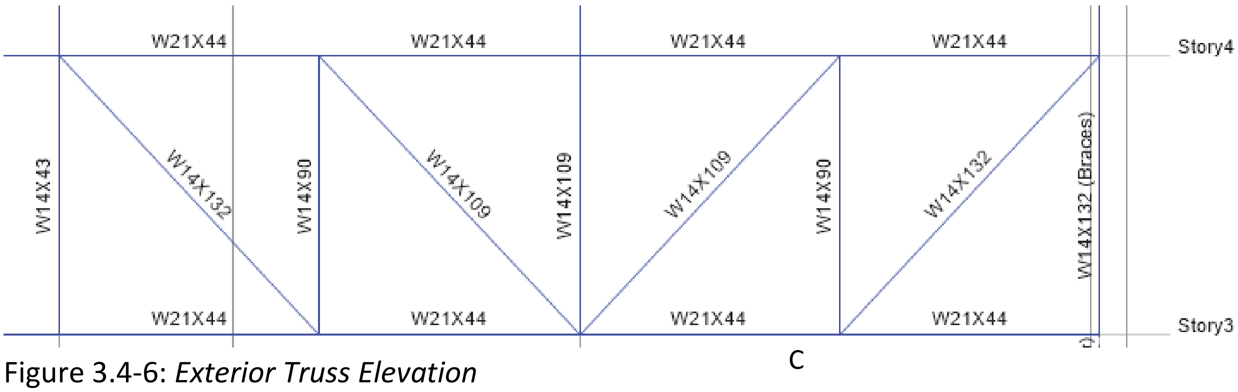


Figure 3.4-6: *Exterior Truss Elevation*

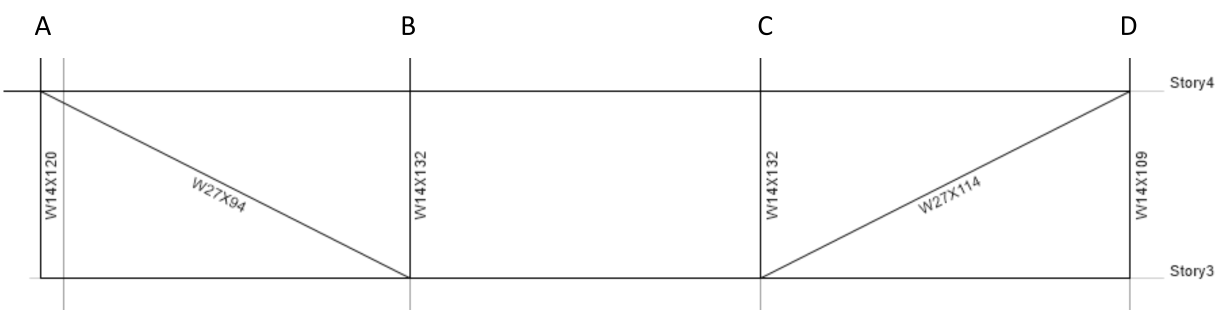


Figure 3.4-7: *Secondary Bracing*

3.5 Columns

See the column schedule, figure 3.5-1, for column sizes. Columns were spliced every 2 floors. All columns we designed with full floor to floor unbraced lengths.

	1D, 1A,	1B, 1C	2A,2D,	2B,2C	3A ,4A,4D, 5A, 6A,7A, 5D, 6D 7D	3B, 3C,4B,4C, 5B, 6B,7B, 5C, 6C, 7C	8D, 8A	8B, 8C
1	W14X233	W14X342			W14X342	W14X311	W14X82	W14X109
2								
3	W14X132	W14X145	W14X109	W14x132	W14x283	W14X159	W14X61	W14X90
4	W14X53	W14X74	W14X90	W14X109	W14X193	W14X99	W14X43	W14X69
5								
6	W14X43	W14X43	W14X90	W14X90	W14X84	W14X74	W14X34	W14X61
7								
8	W14X34	W14X342	W14X61	W14X61	W14X53	W14X53	W14X30	W14X43
9								
10	W14X30	W14X30	W14X30	W14X30	W14X30	W14X30	W14X30	W14X30

Figure 3.5-1: *Column Schedule*

4 Alternate Lateral System

From the existing lateral analysis it was determined that wind forces would govern over seismic therefore, during this design only wind forces were considered. During the hand verification of the lateral wind forces on the building the hand calculated wind pressure were found to me more conservative than the Etabs calculated pressures. The hand calculated wind pressures were manually inserted in to the model. The 4 cases ASCE 7-10 prescribed cases were modeled as different wind loading.

The original proposal of replacing shear walls with brace frames in the lateral system was abandoned for a redesign of shear walls instead. A mixture of K braces and X-frame braces were used in similar places to the shear walls in the original design to not disrupt the architecture. When the model was run a total displacement of 24” was found at the roof. With and additional introduction of moment frames at along the north and south side of the building a displacement of 16” was found. These displacements were far greater than the h/400 constraint of 5.5” there for the model was switched back to a shear wall system.

The shear wall system is shown in figure 4-4. Shear walls were modeled with a cracked moment of inertia (.35Ig). During the existing lateral calculations the building showed large deflection in the x directions. One Extra shear wall was added at gridline 9 to stiffen the building in the x direction. Case 3 wind loading gave the largest deflection of 3.4 inches less than the h/400 deflection criteria of 4.5 inches. Shear walls were designed with uniform reinforcing. See Appendix D for shear wall hand calculations. The additional Shear walls were designed with #8 corner bars, #4 bars at 12” vertical and #5 bars @ 12” horizontal.

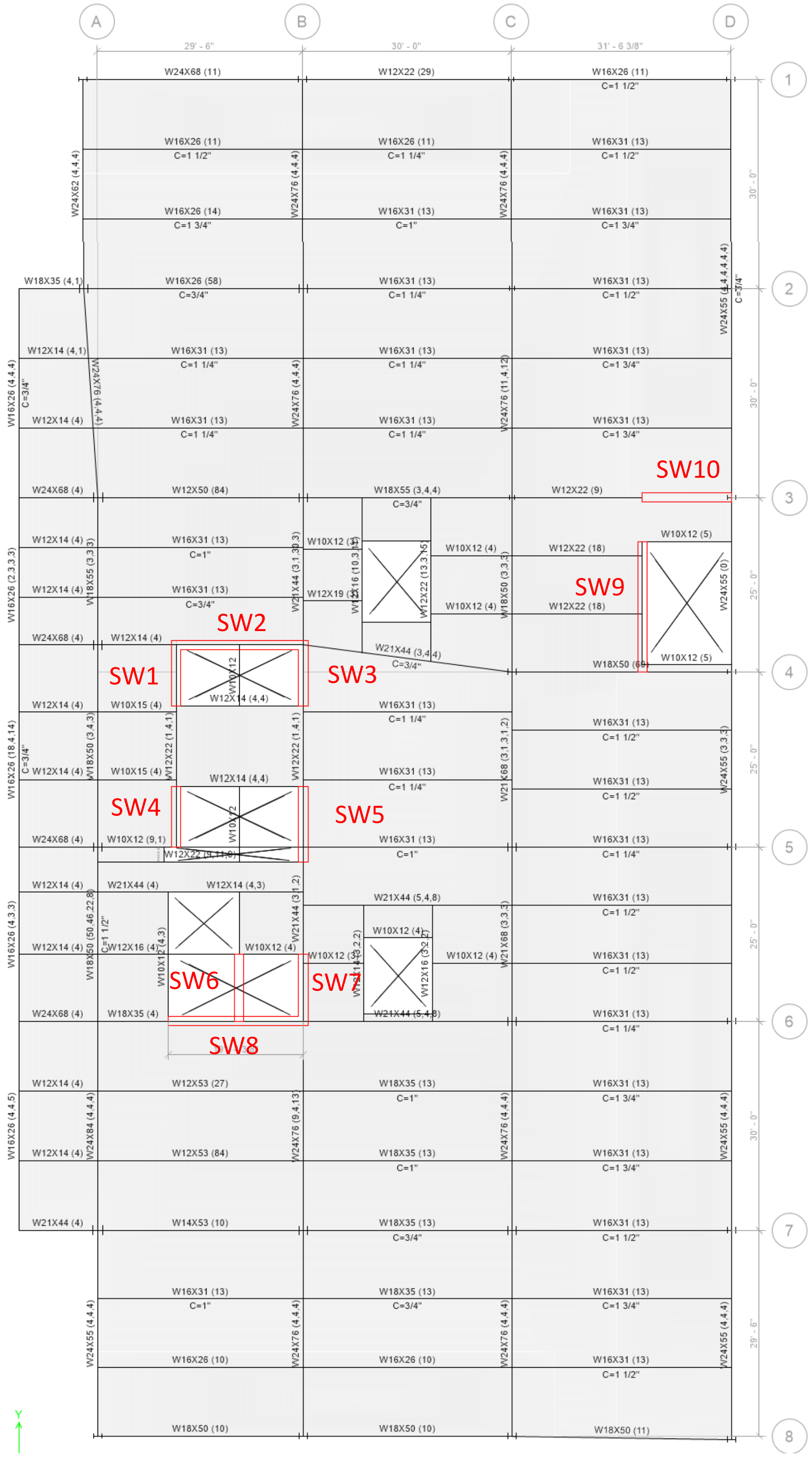


Figure 4-1: Shear Wall Floor Plan

Table 4-1: *Etabs Verification COM Third Floor*

	Calculated	Model
x [ft]	58.3	58
y [ft]	63.7	62

Table 4-6: *Etabs verification COR Third Floor*

Column1	Calculated	Model
x [ft]	64.6	63
y [ft]	97.5	92

5 Cost Estimate and Critical Path Duration

The purpose this construction breath was to analyze the impact on the cost and schedule of the change in system. A takeoff and estimate was completed of a typical floor for both the existing and new framing. R.S. Means was used to compare the two systems prices. Because of the repetitive nature of the floor plans only one floor was estimated. To keep the estimate concise only items that would differ between the two structural systems were included in the estimate. See table ## and table ## for the existing and redesigned takeoffs and estimations respectively. The existing concrete system cost of one typical floor was estimated as $860,000 and the cost of the redesigned composite steel system was estimated as $1.1 million. The redesigned framing would be 27% more expensive than the existing framing.

R.S. Means was also utilized to estimate the critical path construction duration of a single floor. Table 5-1 and table ## contain the critical paths and total durations of the existing concrete system and redesigned steel system respectively.

Table 5-1: *Existing concrete take off, typical floor*

Item	Unit		# of Crews	Daily Outpt	Days	Total O&P	Cost
Slab Concrete	CY	457.913333				121	$ 55,407.51
Slab Form Work	SF	18545.49	4	560	8	8.2	$ 152,073.02
Placing Concrete	CY	457.913333	2	160	2	31.5	$ 14,424.27
Rebar high chairs	EA	64				65	$ 4,160.00
rebar slab NS	LB	21008.75	1	5800	4	1	$ 21,008.75
rebar slab EW	Ton	21866.25	1	5800	4	1	$ 21,866.25
Columns Formwork	LF	364	1	238	2	10.9	$ 3,967.60
Columns Concrete	CF	1456				121	$ 176,176.00
Placing Column	CF	1456	2	92	8	87	$ 126,672.00
Rebar Columns	LB	4253.2	1	4600	1	1.13	$ 4,806.12
Shear Walls Formwork	SFCA	2992.16658	1	395	8	9.95	$ 29,772.06
Walls Concrete	CF	1505.83329				121	$ 182,205.83
Shear walls Rebar	LB	7736.89583	1	8000	1	0.87	$ 6,731.10
Placing Walls	CF	1505.83329	2	120	6	42	$ 63,245.00

Total Cost = $862,515.50

Table 5-2: *Redesigned steel takeoff, typical floor*

Item	Unit	Column2	# of Crews	Daily Output	Days	Total O&P	Cost
Slab Concrete	CY	457.913333				121	$ 55,407.51
Slab Form Work	SF	18545.49	4	560	8	8.2	$ 152,073.02
Placing Concrete	CY	457.913333	2	160	2	31.5	$ 14,424.27
High chairs	EA	64				65	$ 4,160.00
rebar slab NS	LB	21008.75	1	5800	4	1	$ 21,008.75
rebar slab EW	Ton	21866.25	1	5800	4	1	$ 21,866.25
Columns Formwork	LF	364	1	238	2	10.9	$ 3,967.60
Columns Concrete	CF	1456				121	$ 176,176.00
Placing Column	CF	1456	2	92	8	87	$ 126,672.00
Rebar Columns	LB	4253.2	1	4600	1	1.13	$ 4,806.12
SW Formwork	SFCA	2992.16658	1	395	8	9.95	$ 29,772.06
SW Concrete	CF	1505.83329				121	$ 182,205.83
Shear walls Rebar	LB	7736.89583	1	8000	1	0.87	$ 6,731.10
Placing SW	CF	1505.83329	2	120	6	42	$ 63,245.00

Total Cost = 1,100,000

Table 6-3: Concrete Framing Critical Path, typical floor

Critical Path Items	Days
Slab Formwork	8
Slab Rebar	8
Slab Placment	1
Slab Curing	1
Shear Wall/Columns Formwork	8
Shear Wall/Columns Rebar	1
Shear Wall/Columns Placment	8
Shear Wall/Columns Cuing	1
Total Days	36

Table 6-4: Steel Framing Critical Path, typical floor

Critical Path	Days
Framing	14
Shear Wall Formwork	9
Shear Wall Reinforcing	2
Slab Pour	1

Curing	1
Shear Wall Placement	1
Total Days	28

6 Mechanical Conflict Analysis

Two problems that have arisen with the change in framing system, the beams and girder will disrupt the duct work that runs in the ceiling cavity and the air handling systems on the 4th floor will need to moved or altered to fit around the added transfer trusses. This breath looks to preserve the original architecture as much as possible, therefore floor to ceiling heights are being preserved.

The clear space between the ceiling and the floor is shown in Table 6-1. Floor thickness, member height, and a 6" allowance were subtracted from the ceiling to floor height to obtain the clear space for a duct. The 6" allowance was included for installation and maintenance.

New Duct Systems Were designed for a .03" friction head loos per 100 feet of area. A slide rule, Ductulator, was used to find new sizes of duct work. See Table 6-2 for duct work changes. See appendix ## for locations of old duct systems.

Table 6-1: *Cavity Clearance*

Story	Ceiling to Floor [in]	Beam Height [in]	Girder Height [in]	Clear Space Beam [in]	Clear Space Girder [in]
1	96	16	21	68	63
2	54	18	24	24	18
3					
4	54	16	24	26	18
5	54	16	24	26	18
6	54	16	24	26	18
7	54	16	24	26	18
8	54	16	24	26	18
9	47	16	24	19	11
10	54	16	24	26	18

Table 6-2: *Ductwork Resizing*

Floor	Nearest Grid Lines	Top Duct size	Bottom Duct Size	Target Height [in]	Redesign Top size	Redesign Bottom Size
2	C4	18"X12"	34"X16"	24	8"X30"	34"x16"
4	C6	32"X14"	26"X14"	26	12"X49"	30"X12"
9	C6	14"X12"	30"x16"	26	8"X25"	30"X16"
9	B6	18"X10"	18"X16"	24	12"X15"	12"X25"

ꟼORT

Figure 6-1: Truss Conflict Plan

Figure ##: Truss and Air handling Unit Conflict Plan

As seen in Figure 6-1, the full story trusses cross directly through two of the four air handler units that are housed in the mechanical floor. In order to maintain the same number of air handlers the two disrupted air handlers must be either relocated or modified to fit in the space. By rotating the air handling systems by 90 degrees and placing one custom compact fit air handling unit in to the reduced space between column lines C and D.

7 Conclusion

4.1 Summary

The Generic Building name has a concrete column and flat slab system that sits on auger cast piles. A typical bay in Generic Buildings Name is a 30’ by 30’ by 8 inch two way Concrete slab. The lateral system is ordinary reinforced concrete shear walls. Post tensioned concrete beams hold up the canopy. Lateral forces such as Wind and seismic forces were determined using ASCE 7-10. Seismic loads were determined using the equivalent force method.

The report has explored a full redesign of the Generic Project Names structural system to a composite steel system. The design yielded 30’x 30’ typical bay with W16x31 beams and W24x72 girders. The bays containing MRI imaging equipment were designed with sing AISC design guide 11 as a guide. The Rooms were designed for class C, slow impulse vibrations. The canopy was redesigned using full floor height transfer trusses. The Steel Redesign cased conflicts with mechanical system duct work and with the air handling units on the mechanical floor. And a redesign of some of the duct work and air handling units was necessary.

4.2 Conclusion

The proposed redesign of The Generic Building will be a steel frame with composite steel beams and girders. The lateral system will be designed as either a moment frame or a braced frame. The more efficient of the two lateral systems will be chosen.

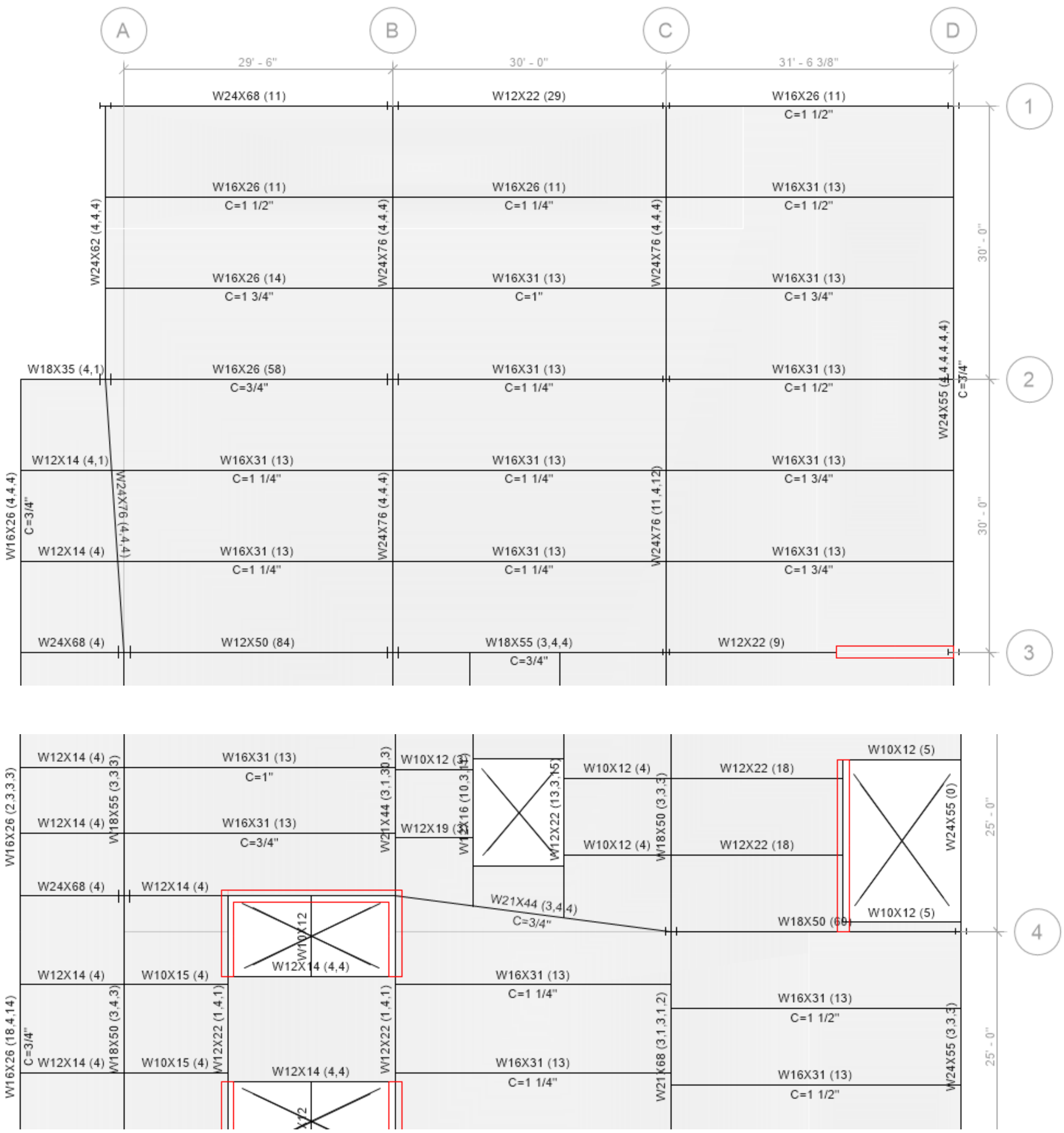
The Large transfer girders will pose the biggest challenge in a steel redesign. From communication with the design team the girders were barley with in service limits. A switch from concrete to steel could possibly require more redesign to keep those girders within service limits. The top heavy form of the building could propose a challenge in future seismic designs.

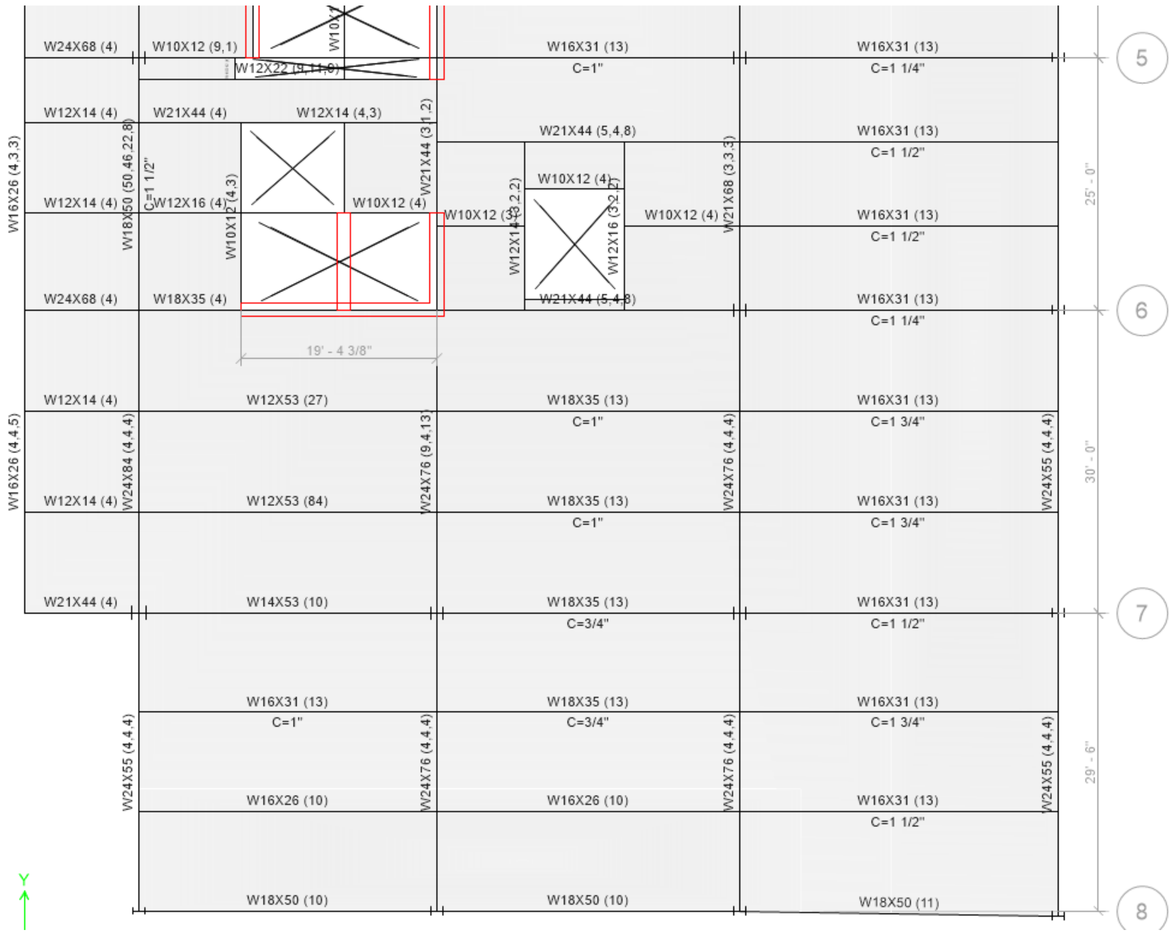
The in addition the main redesign two breath topics have been chosen. A study on the mechanical system will be conducted to analyze the introduction of beams and girders on the duct work. A study on construction will be conducted to analyze the effect of the change in construction type on the schedule.

4 APENDIX

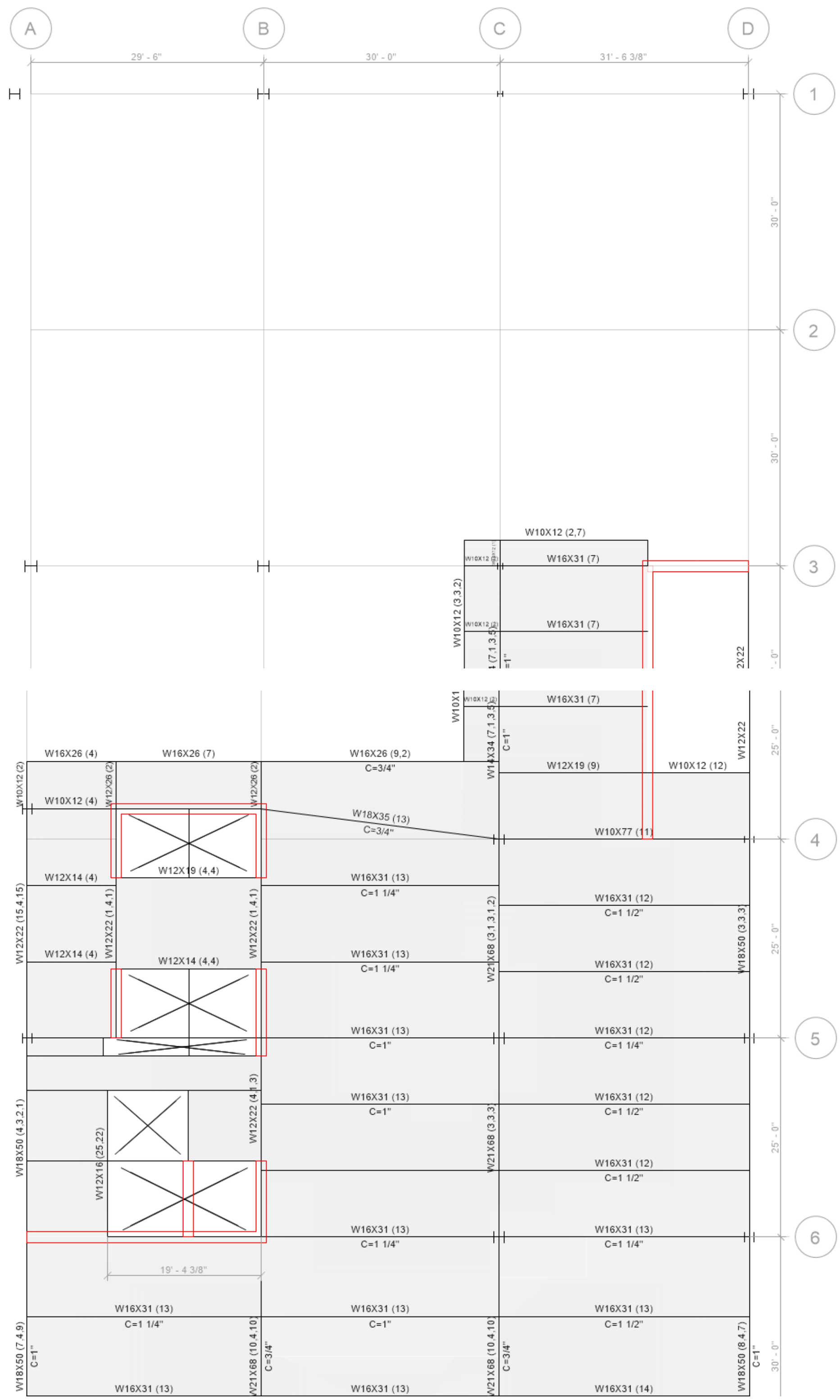
Appendix A: *Floor Plans*

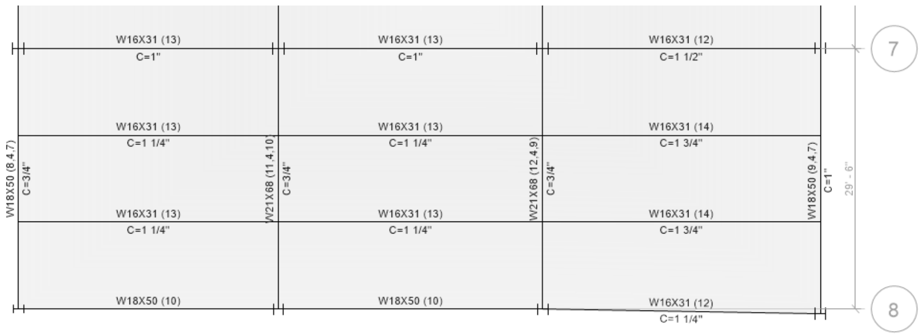
A-1 Typical Floor 5-10



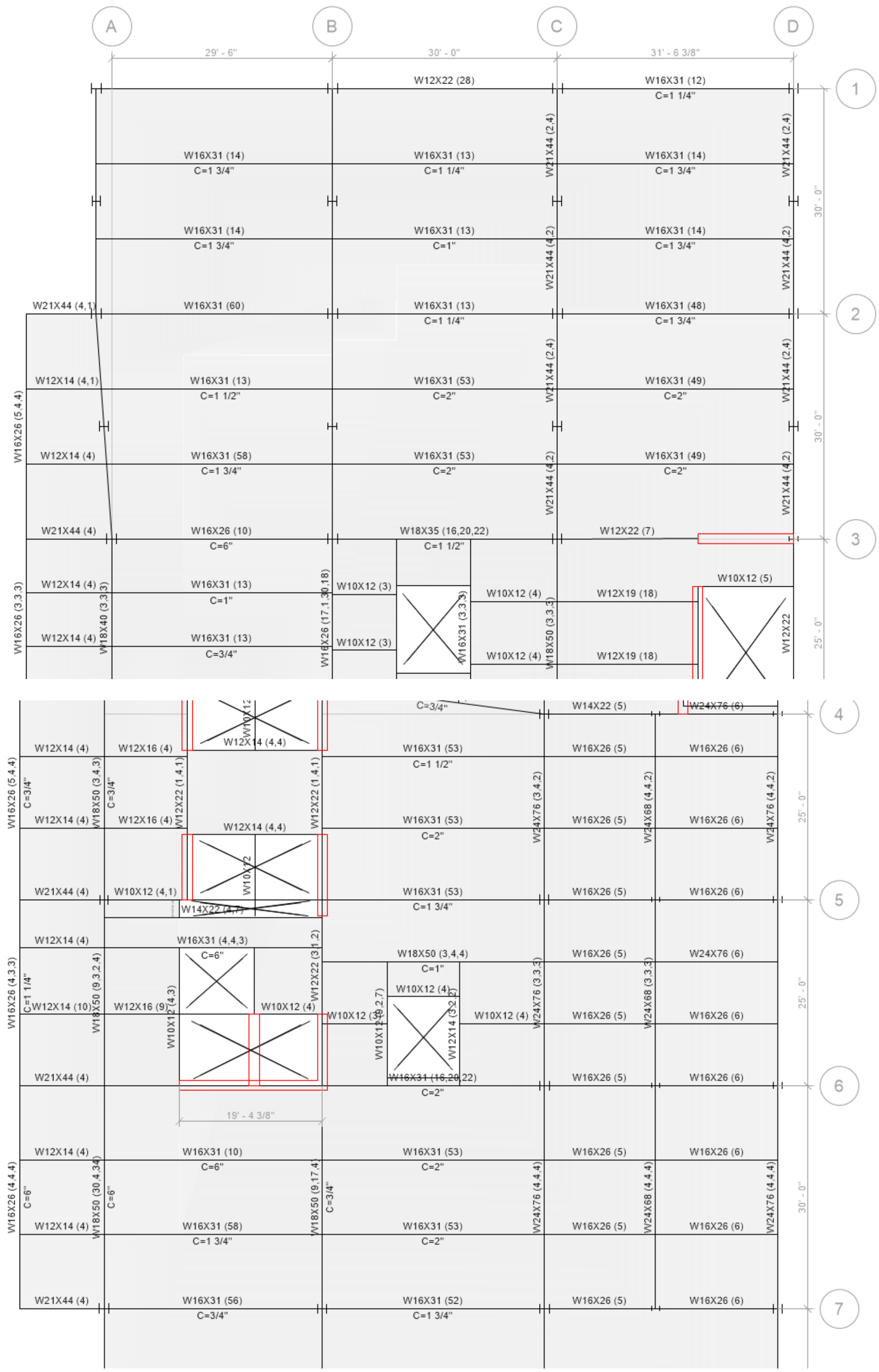


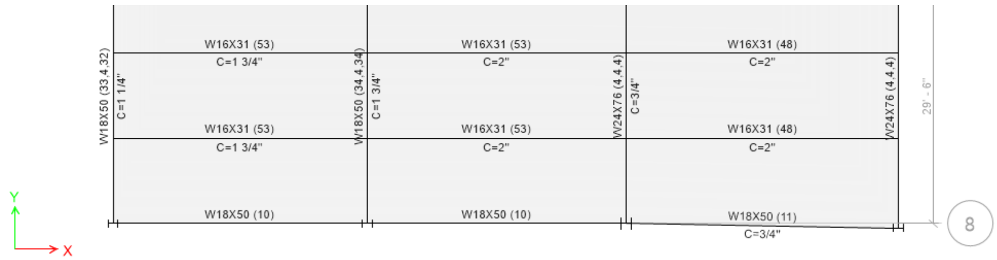
A-2 Second Floor



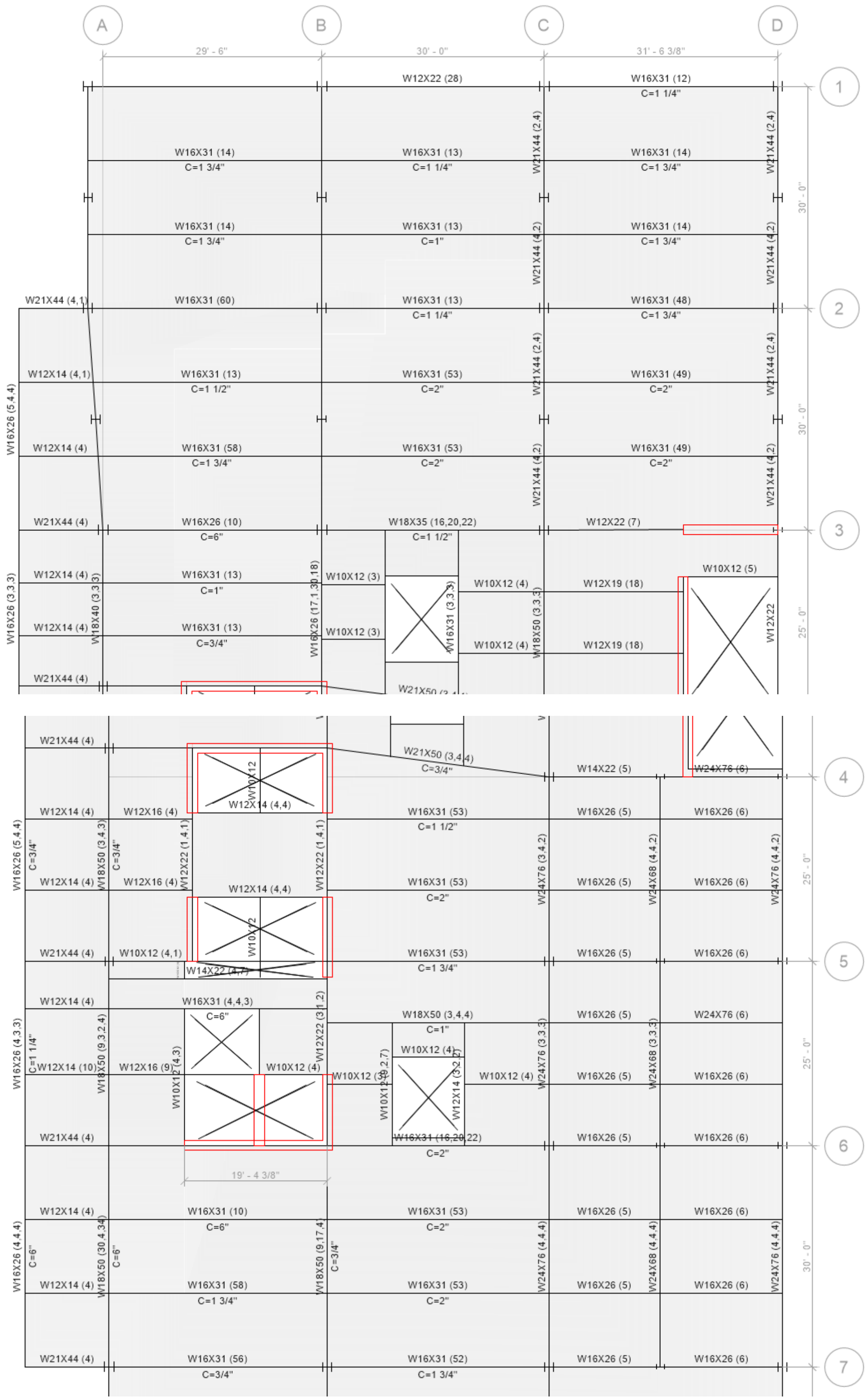

W16X31 (13)
C=1"
W16X31 (13)
C=1"
W16X31 (12)
C=1 1/2"
7
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/4"
W16X31 (14)
C=1 3/4"
W18X50 (8,4,7)
C=3/4"
W21X68 (11,4,10)
C=3/4"
W21X68 (12,4,9)
C=3/4"
W18X50 (9,4,7)
C=1"
29' - 6"
W16X31 (13)
C=1 1/4"
W16X31 (13)
C=1 1/4"
W16X31 (14)
C=1 3/4"
W18X50 (10)
W18X50 (10)
W16X31 (12)
C=1 1/4"
8

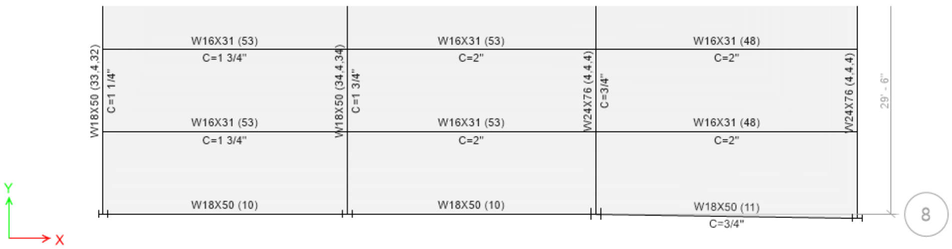
A-3 3rd Floor




W16X31 (53)
C=1 3/4"
W16X31 (53)
C=2"
W16X31 (48)
C=2"
W18X50 (33.4.32)
C=1 1/4"
W18X50 (34.4.34)
C=1 3/4"
W24X76 (4.4.4)
C=3/4"
W24X76 (4.4.4)
29' - 6"
W16X31 (53)
C=1 3/4"
W16X31 (53)
C=2"
W16X31 (48)
C=2"
W18X50 (10)
W18X50 (10)
W18X50 (11)
C=3/4"
8
Y
X

A-4 4th Floor Plan




W16X31 (53)
C=1 3/4"
W16X31 (53)
C=2"
W16X31 (48)
C=2"
W18X50 (33,4,32)
C=1 1/4"
W18X50 (34,4,34)
C=1 3/4"
W24X76 (4,4,4)
C=3/4"
W24X76 (4,4,4)
W16X31 (53)
C=1 3/4"
W16X31 (53)
C=2"
W16X31 (48)
C=2"
W18X50 (10)
W18X50 (10)
W18X50 (11)
C=3/4"
29' - 6"
8
Y
X

Appendix B

Composite Beam spot check

Beam: W 16 x 31 (interior), 16x26 (exterior)

Spacing: 10' Length: 30'

Area: Typ floor, office

Loading: SDL = 40 psf

LL = 95 psf

Slab / Deck weight = 69 psf Beam weight = 31 plf

LL Reduction: $K_{LL} = 2$

$LL = 95 \left| \begin{array}{l} .25 + \frac{15}{\sqrt{30(10)(2)}} = \underline{0.86} \quad LL = 81.7 \\ .5 \end{array} \right.$

$w_u = 1.2(40+69) + 1.6(81.7) = 261.52$ psf

Interior Beam: W 16x31 [13]

$10(261.52) + 1.2(31) = 2.65$ k/ft

$M_u = \frac{2.65 \; (30)^2}{8} = 298.12$ kft

$\Sigma Q_n = \frac{13(21.5)}{2} = 139.75$ kips => 3/4" ϕ Studs

$b_{eff} = \min \left| \begin{array}{l} 10(12) = 120'' \\ \frac{30(12)}{4} = 90'' \end{array} \right.$

$a = \frac{111.8}{.85(5)(90)} = 0.29 \quad Y_2 = 6.5 - \frac{.29}{2} = 6.355''$

Table 3-19 $Y_2 = 6.5$ $Q_n = 114$

$\phi M_n = 309 > 298.12$ ok

Construction Loading

$1.2(69) + 1.6(20) = 1161.8$

$1161.8(16) + 1.2(31) = 1185.2 \text{ k/ft}$

$M_u = 133.3 \text{ kft} \quad \phi M_n = 203 \text{ kft}$ ok

Δ_{LL}

$W = 81.7(10) = 817$

$$\Delta_{LL} = \frac{5(.817)(30)^4(1728)}{384(29000)(523)} = 0.98'' < \frac{12(30)}{360} = 1'' \text{ ok}$$

Δ_{WC}

$$\frac{5(.721)(30)^4(1728)}{384(29000)(375)} = 1.2''(.8) = .96'' \Rightarrow 1''$$

$$\Delta_{TL} = \frac{5(1.938)(30)^4(1728)}{384(29000)(523)} = 2.3'' - 1'' = 1.3'' < \frac{30 \times 12}{24} = 1.5''$$

ok

Girder design 24 x 76

$LL_R = 95 \quad \Big| \quad .25 + \dfrac{15}{\sqrt{30(30)(2)}} = .6(95) = 57 \text{ k/sf}$

$.5$

$P = \left[1.2(31) + 10(1.6(57) + 1.2(40+69))\right]30$

$= 67.716 \text{ k}$

$M_u = 677.16 \text{ k ft}$

$\phi M_n = 750 \Rightarrow$ table 3-2 ok

$\underline{\Delta_{LL}}$

$\Delta = \dfrac{23 D l^3}{648 E I} = \dfrac{23(28500)\,30^3\,(1728)}{648(29000000)(2100)} = 0.02''$

$\dfrac{30(12)}{360} = 1''$ ok

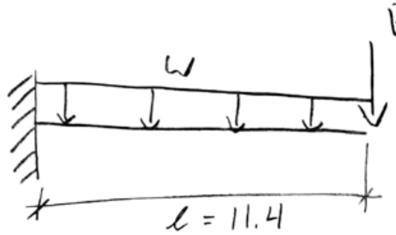
$P = 95(10)(30) = 28500$

$\underline{\Delta_{DL}}$

$\Delta = 0.05''$

Ney Bending Beam

W24 x68



$W = 8.5\left(1.2(40+6) + 1.6(95)\right) + 68(1.2) = 2485 \text{ plf}$

$L_r = 95 \Big| \; .25 + \frac{15}{\sqrt{(11.8)(8.5)(1)}} = 1.77 \quad K_{LL} = 1$ ASCE Table 4-2

$\quad .5$

$P = 12.5\left(1.2(40+6) + 1.6(95)\right)\left(\frac{11.8}{2}\right) + 8.25(19+26)(1.2)$

$= 20.6 \text{ k}$

$M_u = \frac{w\ell^2}{2} + P\ell = \frac{2.5(11.4)^2}{2} + 20.6(11.4)$

$= 397.29 \text{ kft}$

$L_b = 11.4$ Table 3-10

$\phi M_n = 570 \text{ kft}$

LL Deflection

$I = 1830$

$\Delta_{LL} = \frac{w\ell^4}{8EI} + \frac{P\ell^3}{3EI} = \frac{(8.5)(95)(11.4)^4(1728)}{8(29{,}000k)(1830)} + \frac{6766.75(11.4)^3(1728)}{3(29{,}000k)(1830)}$

$\Delta = .18'' < \frac{11.4(12)}{360} = .38$ ok

Const Δ

$$\Delta = \frac{654(11.8)^4(1728)}{8(29{,}000k)(177)} + \frac{5287.5(11.8)^3(1728)}{3(29{,}000k)(177)} = 0.46 < 1''$$ ok

$W_{CL} = 69(8.5) + 68 = 654 \text{ lb/ft}$

$P_{CL} = 12.5(69)\left(\frac{11.4}{2}\right) + 8.23(19+28) = 5287.5 \text{ lb}$

Total Deflection

$w = 1802 \text{ lb/ft}$

$p = 14{,}906 \text{ lb}$

$\Delta = .39 < \frac{11.8(12)}{240} = .59$ ok

Shear

$\phi V_n = 295 \qquad V_u = 50$ ok

Appendix C

Column1	DL	LL	kLL	TL	Column2
2	114	150	150	336859.2	2013.288
3	104	95	47.5	179515.2	1676.4288
4	114	95	47.5	190243.2	1496.9136
5	114	95	47.5	190243.2	1306.6704
6	114	95	47.5	190243.2	1116.4272
7	114	95	47.5	190243.2	926.184
8	114	95	47.5	190243.2	735.9408
9	114	95	47.5	190243.2	545.6976
10	114	95	47.5	190243.2	355.4544
Roof	114	30	30	165211.2	165.2112
			Pu	2013.288	

Column Hand Check

Column 6 8

$A_{trib} = 30 \times 30 = 900$

$DL = 69 + 410 + (3)31\frac{(30)}{900} + \frac{76(30)}{900} = 114.0$

$\underbrace{}_{\text{Beam}}$ $\underbrace{}_{\text{Girder}}$

$LL = 95,\ 100$

$$LLR = \left| \begin{array}{l} .25 + \dfrac{15}{\sqrt{(30)(30)(4)}} = .5 \\ .5 \end{array} \right.$$

$P_u = 2026.8$ $\ell_b = 21.5$

Etabs $P_u = 1900$ kips

$W14 \times 211$ $\phi P_n = 2310$ ok

Scanned by CamScanner

Appendix D

Second Floor Stifness							
Wall #	**Length [ft]**	**Height [ft]**	**Thickness [in]**	**E**	**v**	**G**	**k**
1	8.75	21.5	12	4030000	0.15	1752174	296
2	18	21.5	12	4030000	0.15	1752174	608
3	8.75	21.5	12	4030000	0.15	1752174	296
4	8.92	21.5	12	4030000	0.15	1752174	301
5	11.375	21.5	12	4030000	0.15	1752174	384
6	10	21.5	12	4030000	0.15	1752174	338
7	10	21.5	12	4030000	0.15	1752174	338
8	20.5	21.5	12	4030000	0.15	1752174	693
9	30.5	21.5	12	4030000	0.15	1752174	1031
10	12.8	21.5	12	4030000	0.15	1752174	433

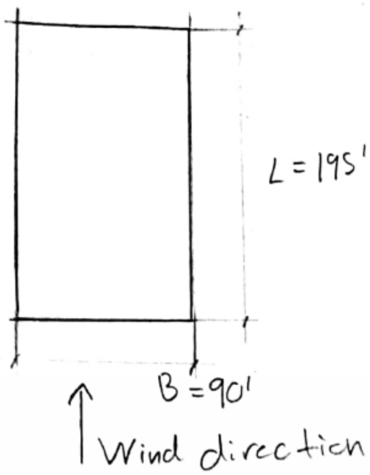
COR					
Wall #	**k**	**xi [ft]**	**yi [ft]**	**kxi [lb*ft]**	**kyi [lb*ft]**
1	296	11.625		3437.094143	0
2	608		113.833	0	69235.8614
3	296	29.625		8759.046365	0
4	301	11.625		3503.871972	0
5	384	29.625		11386.76027	0
6	338	20.5		6926.985432	0
7	338	29.62		10008.64919	0
8	693		59.5	0	41215.56332
9	1031	79.76		82200.67766	0
10	577		126	0	72662.38767
			Sum	126223	183114

COM						
x bar [ft]	**y bar [ft]**	**DL [psf]**	**Area [sf]**	**Weight [lb]**	**xw**	**yw**
58	91	109	6362	693458	40220564	63104678
56	30	109	4919	536171	30025576	16085130
31	112	0	158	0	0	0
31	91	-109	158	-17222	-533882	-1567202
30	86	-109	90	-9810	-294300	-843660
31	67	-109	185	-20165	-625115	-1351055

26	76	-109				
30	86	-109	43	-4687	-140610	-403082
			Sum	1177745	68652233	75024809

Note Book A Baker

Wind Loads: NS (ASCE 7-10)



Exposure B § 26.7.3

h = 144.5

β = 0.02 pg. 521

L/B = 2.167

C_{pw} = .8 Figure 27.4-1

C_{pL} = -.29

Kd = 0.85 Table 26.6-1

Kzt = 1 §26.8

kz: Table 27.3-1

Gcpi = ± 0.18 Table 26.11-1

Note Book A | | Baker

Natural frequency (ASCE 7-10)

§ 26.9.2.1 Limitations

1. 144.5 ft < 300 ft OK

2. $L_{eff} = \dfrac{(195)(\Sigma h_{3-10} = 819.5) + (138.75)(\Sigma h_{1-2} = 21.5)}{\Sigma h_{1-10} = 841}$

$= 193.5$ ft

$4 L_{eff} = 774$ ft > 144.5 ft OK

Apreximate Natural Frequency § 26.9.3

$$C_w = \frac{100}{A_B} \Sigma \left(\frac{h}{h_i}\right)^2 \frac{A_i}{1+0.83\left(\frac{h_i}{D_i}\right)^2}$$

$$= \frac{100}{17,550}\left(\frac{19}{1+0.83\left(\frac{144.5}{19}\right)^2} + \frac{28.25}{1+0.83\left(\frac{144.5}{28.25}\right)^2}\right)$$

$= 0.045$

$n_a = \dfrac{385(0.045)^{.5}}{144.5} = 0.565 < 1$

therefore $G_f > 0.85$

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Note Book A | Baker

Gust Factor NS ASCE 7-10 § 26.9.5

$$g_R = \sqrt{2\ln(3600\, n_1)} + \frac{0.577}{\sqrt{2\ln(3600\, n_1)}}$$

$$= \sqrt{2\ln(3600(.565))} + \frac{0.577}{\sqrt{2\ln(3600(.565))}}$$

$$= 4.048$$

$$\bar{z} = .6h = .6(144.5) = 86.7$$

$\bar{b} = 0.45$ $\bar{\alpha} = .25$ Table 26.9-1

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{86.7}{33}\right)^{.25}\left(\frac{88}{60}\right)115$$

$$= 96.63$$

$\ell = 320$ $\bar{\epsilon} = 1/3$ Table 26.9-1

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{86.7}{33}\right)^{1/3} = 441.6$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.565)(441.6)}{96.63} = 2.58$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = 0.765$$

Note Book A | | Baker

R :

$$\eta = \frac{(4.6)(0.565)(144.5)}{96.63} = 3.88$$

$$R_h = \frac{1}{3.88} - \frac{1}{2(3.88)^2}\left(1-e^{-2(3.88)}\right)$$
$$= 0.225$$

R_B :

$$\eta = \frac{4.6(90)(.565)}{96.63} = 4.28$$

$$R_B = \frac{1}{4.28} - \frac{1}{2(4.28)^2}\left(1-e^{-2(4.28)}\right)$$
$$= 0.206$$

R_L :

$$\eta = \frac{15.4(195)(.565)}{96.63} = 17.55$$

$$R_L = \frac{1}{17.55} - \frac{1}{2(17.55)^2}\left(1-e^{-2(17.55)}\right)$$
$$= 0.055$$

$\beta = 0.02$ pg 521

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$
$$= \sqrt{\frac{1}{0.02}(0.225)(0.765)(0.206)(0.53 + 0.47(0.055))}$$
$$= .99$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{90+144.5}{441.6}\right)}}$$
$$= 0.866$$

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Note Book A | Baker

$c = 0.30 \quad 26.9\text{-}1$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{86.7}\right)^{1/6} = 0.255$

$G_f = 0.95\left(\frac{1+1.7(I_{\bar{z}})\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$

$= 0.95\left(\frac{1+1.7(0.225)\sqrt{3.4^2(0.886)^2 + 4.06^2(.99)^2}}{1+1.7(3.4)(0.255)}\right)$

$= 1.12$

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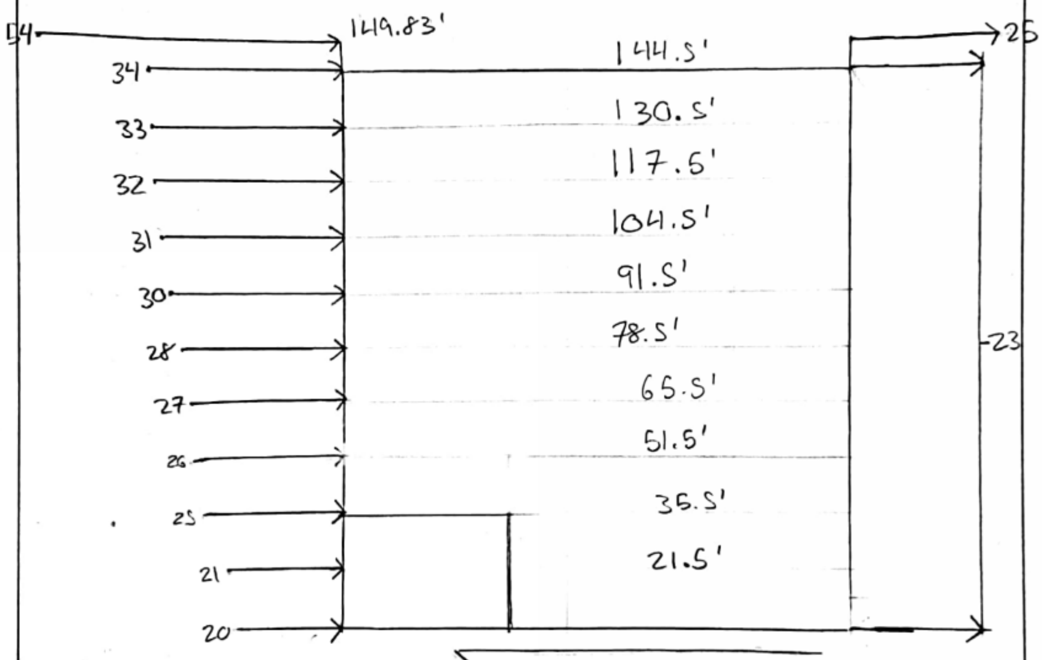
NS Story Foce														
Floor	z [ft]	kz	kzt	kd	qz	Gf	Cpw	Cpl	GCpw	GCpl	GCpi	Pw [psf]	Pl [psf]	Fx [kips]
1	0.00	0.57	1.0	0.85	16.40	0.86	0.8	-0.5	0.688	-0.43	0.18	16.83	-18.78	53.12
2	21.50	0.63	1.0	0.85	18.13	0.86	0.8	-0.5	0.688	-0.43	0.18	18.02	-18.78	145.52
3	35.50	0.73	1.0	0.85	21.01	0.86	0.8	-0.5	0.688	-0.43	0.18	20.00	-18.78	166.36
4	51.50	0.82	1.0	0.85	23.60	0.86	0.8	-0.5	0.688	-0.43	0.18	21.78	-18.78	181.92
5	65.50	0.87	1.0	0.85	25.04	0.86	0.8	-0.5	0.688	-0.43	0.18	22.77	-18.78	166.10
6	78.50	0.95	1.0	0.85	27.34	0.86	0.8	-0.5	0.688	-0.43	0.18	24.35	-18.78	164.02
7	91.50	0.96	1.0	0.85	27.63	0.86	0.8	-0.5	0.688	-0.43	0.18	24.55	-18.78	164.77
8	104.50	1.00	1.0	0.85	28.78	0.86	0.8	-0.5	0.688	-0.43	0.18	25.34	-18.78	167.78
9	117.50	1.03	1.0	0.85	29.64	0.86	0.8	-0.5	0.688	-0.43	0.18	25.94	-18.78	170.04
10	130.50	1.07	1.0	0.85	30.79	0.86	0.8	-0.5	0.688	-0.43	0.18	26.73	-18.78	177.49
Roof	144.50	1.10	1.0	0.85	31.66	0.86	0.8	-0.5	0.688	-0.43	0.18	27.32	-18.78	147.59
Parapet	149.33	1.11	1.0	0.85	31.94				1.5	1	0.18	53.46	25.25	26.58

Table 4.1: NS Wind Story Force Calculations

Note Book A | Baker

Wind Pressure Vs Story height

North South:



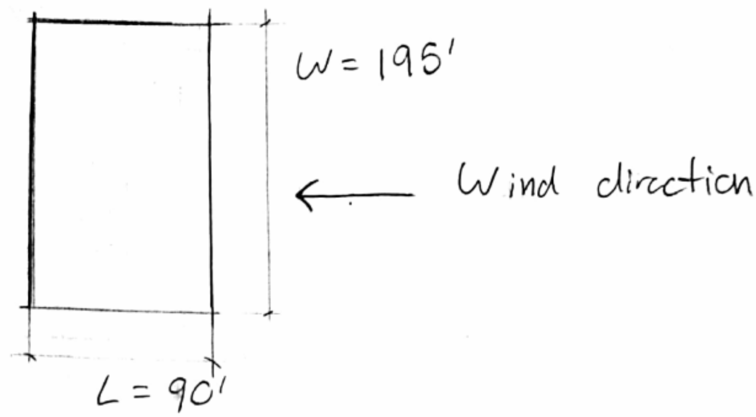
Base shear = 1020 kip

*pressures in psf

Figure 4.1: Wind pressure vs Story height NS

Note Book A | | Baker

Wind Loads: EW



Exposure B § 26.7.3

h = 144.5

β = 0.02 pg 521

L/B =

C_{pw} = .8 Figure 27.4-1

C_{pL} = -0.5

K_d = 0.85 Table 26.6-1

K_{zt} = 1 § 26.8

K_z : Table 27.3-1

GC_{pi} = ± 0.18 Table 26.11-1

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Note Book A | | Baker

Gust Factor EW ASCE 7-10 § 26.9.5

See NS for calculations not shown here.

R_B: $\eta = \frac{4.6(195)}{96.63} = 9.28$

$R_B = \frac{1}{9.28} - \frac{1}{2(9.28)^2}\left(1 - e^{-2(9.28)}\right)$

$= 0.101$

R_L: $\eta = \frac{15.4(.565)(90)}{96.63} = 8.1$

$R_L = 0.11$

$R = \sqrt{\frac{1}{0.02}(0.765)(0.225)(0.101)(0.53 + 0.47(0.11))}$

$= 0.71$

$G_f = 0.925\left(\frac{1 + 1.7(0.265)\sqrt{3.4^2(0.866)^2 + (4.05)^2(.71)^2}}{1 + 1.7(3.4)(.255)}\right)$

$G_f = 0.86$

EW Story Foce														
Floor	z [ft]	kz	kzt	kd	qz	Gf	Cpw	Cpl	GCpw	GCpl	GCpi	Pw [psf]	Pl [psf]	Fx [kips]
1	0.00	0.57	1.0	0.85	16.40	0.86	0.8	-0.29	0.688	-0.2494	0.18	16.83	-13.22	29.07
2	21.50	0.63	1.0	0.85	18.13	0.86	0.8	-0.29	0.688	-0.2494	0.18	18.02	-13.22	80.13
3	35.50	0.73	1.0	0.85	21.01	0.86	0.8	-0.29	0.688	-0.2494	0.18	20.00	-13.22	65.77
4	51.50	0.82	1.0	0.85	23.60	0.86	0.8	-0.29	0.688	-0.2494	0.18	21.78	-13.22	72.45
5	65.50	0.87	1.0	0.85	25.04	0.86	0.8	-0.29	0.688	-0.2494	0.18	22.77	-13.22	66.40
6	78.50	0.95	1.0	0.85	27.34	0.86	0.8	-0.29	0.688	-0.2494	0.18	24.35	-13.22	65.94
7	91.50	0.96	1.0	0.85	27.63	0.86	0.8	-0.29	0.688	-0.2494	0.18	24.55	-13.22	66.29
8	104.50	1.00	1.0	0.85	28.78	0.86	0.8	-0.29	0.688	-0.2494	0.18	25.34	-13.22	67.68
9	117.50	1.03	1.0	0.85	29.64	0.86	0.8	-0.29	0.688	-0.2494	0.18	25.94	-13.22	68.72
10	130.50	1.07	1.0	0.85	30.79	0.86	0.8	-0.29	0.688	-0.2494	0.18	26.73	-13.22	71.91
Roof	144.50	1.10	1.0	0.85	31.66	0.86	0.8	-0.29	0.688	-0.2494	0.18	27.32	-13.22	59.90
Parapet	149.33	1.11	1.0	0.85	31.94				1.5	1	0.18	53.46	25.25	12.27

Table 4.2: EW Wind Story Force Calculations

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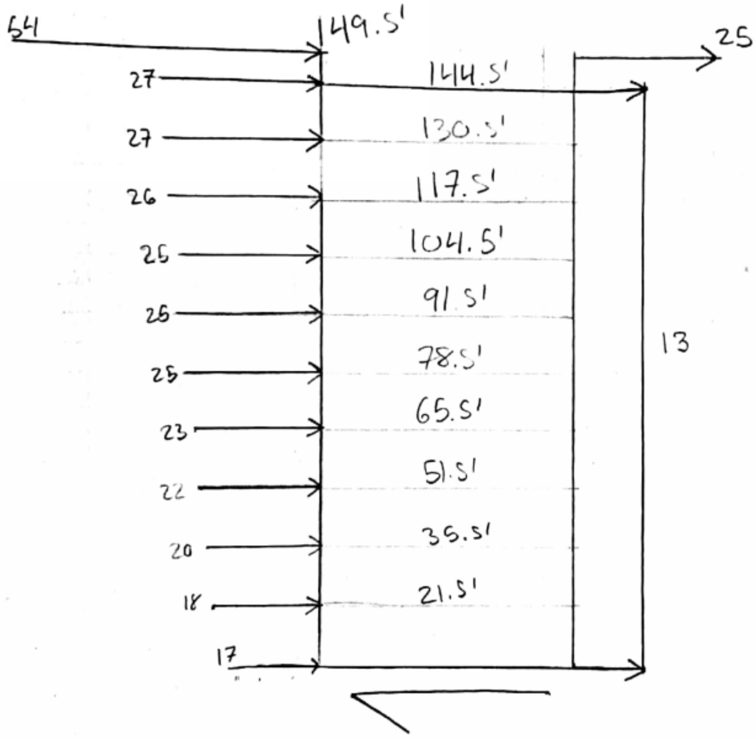
Note Book A Baker

Wind Pressure Vs Story height



Base shear = 1506 kip

* pressures in psf

Figure 4.2: Wind pressure vs Story height EW

Appendix E

Shear Wall hand Check

SW 10

$\ell = 12.8'$

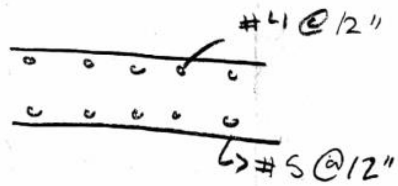
Controlling load case from Etabs:

1.2D + 1LL − Wind case 3 , .9D − W_3

$V_u = 260.86$ $P_u = -1393.7$ $M_u = 2397$

Min Reinforcement:

$\rho_\ell = \frac{2(.2)}{16''(12)} = .002 \geq \rho_m = .002$ ok



Max horizontal spacing:

$S_{max} = \begin{cases} l_w/3 = 51.2 \\ 3h = 48'' \\ 18'' \end{cases}$ ok

Shear Capacity

$\phi V_c = \phi 2 \sqrt{f'c}\, hd = 2\sqrt{5000}\ 16\,(12.8(12))$
$= 260$ k

$\phi V_s = \phi .2 (2)(60)(12.8)(12)/12 = 307$ k

Scanned by CamScanner

$\phi V_n = 567k > V_u = 260$ ok

Flexural

$\omega = .002\left(\frac{60}{5}\right) = 0.024$

$\alpha = .002 \dfrac{(.1393.7)}{16(12.8)(12)(5)} = 2.2\times10^{-4}$

$c = \left(\dfrac{2.2\times10^{-4} + 0.024}{.85(.85) + 2(.024)}\right)(12)(12.6) = 4.82$

$M_n = .4(60)\left(\dfrac{12.4(12) - 4.82}{12.4(12)}\right)\left(\dfrac{12.4(12)}{2}\right)$

$+ 1393.7\left(\dfrac{12.4(12) - 4.82}{2}\right) = 102006\ kft$

$\phi M_n = 91854 < M_u = 2397$

Scanned by CamScanner

Appendix F

1

Vibration design

Dampening:

β = 0.01 (structure) + .01 (ceiling & Ductwork) + 0.01 (Hospital fitout) + .04 (partitions = 0.07

Deck properties

$E_c = w^{1.5}\sqrt{f'_c} = (145)^{1.5}\sqrt{5} = 3904$

$n = \frac{E_s}{1.35 E_c} = \frac{29000}{1.35(3904)} = 5.5$

Slab + Deck weight = 69 psf

Beam Properties try W16 x 26 => ETABS

D_{eff} = 120"

LL = 11 psf DL = 9 + 69 = 78

$W_b = 10(11 + 69 + 9) + 26 = 916$ plf

$\Delta_b = \frac{5(916)(15)^4(1728)}{384(29,000)(1078)} = .03$

$f_b = 0.18\sqrt{\frac{386}{.03}} = 20$ Hz

2

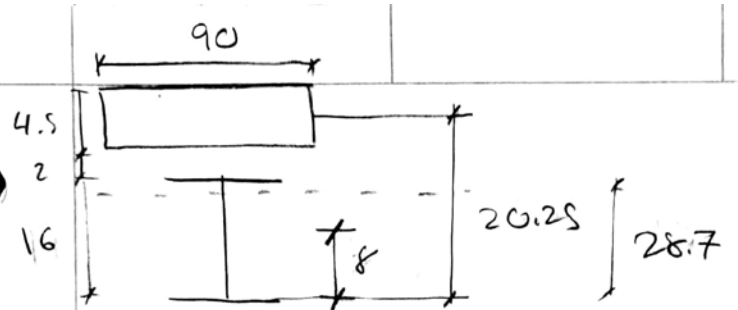
$D_s = 30.28 \text{ in}^4$

$B_j = C_j \left(\frac{D_s}{D_j}\right)^{1/4} L_j = 2.0 \left(\frac{30.28}{142.9}\right)^{1/4} L_j = 20.3 \text{ ft}$

$D_j = \frac{I_j}{s} = \frac{1429}{10} = 142.9 \text{ in}^4/\text{ft}$

$W_j = 1.5\left(\frac{916}{10}\right)(20.3)(15) = 41838 \text{ lb}$

3



Beam:
$d = 8$
$A = 7.68$
$I = 301$

Conc:
$d = 20.25$
$A = 4.5(90)/5.5 = 73.6$
$I = \frac{bh^3}{12n} = \frac{90(4.5)^3}{12(5.5)} = 124$

$$ENA = \frac{8(7.68) + 20.25(73.6)}{7.68 + 73.6} = 19$$

Beam $\bar{Y} = 11$ Conc: $\bar{Y} = 1.25$

$Ay^2 = 929$ $Ay^2 = 75.1$

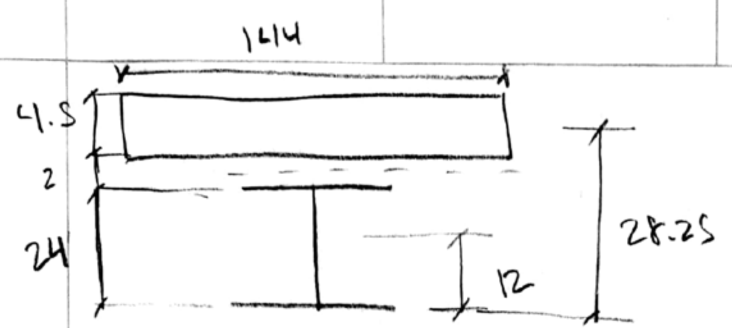
$I = \Sigma(I + Ay^2) = 1429.1$

<u>Girder</u>
W 24 x 76

$$W_g = 15\left(\frac{916}{10}\right) + 76 \doteq 1450$$

$$\Delta g = \frac{5(1450)(30)^4(1728)}{384(29{,}000{,}000)(7406)} = .12$$

$$f_g = 0.18\sqrt{\frac{386}{.12}} = 10.2$$



B:

$d = 15''$

$A = 22.4$

$I = 2100$

C:

$d = 28.25''$

$A = 4.5(144)/5.5 = 117.8$

$I = \dfrac{144(4.5)^3}{12(5.5)} = 198.8$

$$ENA = \frac{12(22.4) + 28.25(117.8)}{26.3 + 117.8} = 24.9''$$

$\bar{y} = 13.0''$ $\bar{y} = 3.35$

$A\bar{y}^2 = 3785.6$ $A\bar{y}^2 = 1322$

$I = 7406$

$$D_s = \frac{12 d_c^3}{12n} = \frac{12(5.5)^3}{12(5.5)} = 30.25 \rightarrow \text{Avc conc } t$$

$$D_g = \frac{I_g}{L_j} = \frac{7406}{30} = 246 \text{ in}^4/\text{ft}$$

$C_g = 1.8$

$$B_g = 1.8\left(\frac{142.9}{246}\right)^{1/4}(30) = 47$$

$$W_g = \left(\frac{1450}{30}\right)(47)(30) = 68150$$

S

$$W = \frac{.03}{.12+.03}(41838) + \frac{.12}{.12+.03}(68150)$$

$$= 55347$$

$$V = \frac{250\times10^6}{(.07)(55347)} \frac{1.25^{(2.43)}}{(10.2)^{1.8}} \left(1 - e^{\frac{-2\pi(.07)}{1.25}}\right)$$

$$= 490 < 500 \quad \underline{\underline{ok}}$$

References

Architectural Floor Plans and exterior rendering created by Wilmot Sanz

Structural diagrams and plans created by Cagley and Associates

ASCE 7-10, ACI 318, IBC