Final Report

CBD Chemical Production Building

Virginia, USA

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CBD Chemical

Production Building Virginia, USA

Function/Occupant Type:	High Hazard, Chemical Manufacturing Plant
Size:	55,000 GSF
Stories:	5 floors plus a mezzanine in the first floor and a penthouse roof
Primary Project Team:	Withheld at request of Engineers and Contractors
Dates of Construction:	April 2008 – January 2009
Cost Information:	$125 Million
Project Delivery Method:	Design-Bid-Build with a Negotiated Guaranteed Max Contract

Architecture

The exterior skin is a combination of insulated metal panels and translucent metal panels. The north and south facing walls have horizontal strips of windows, while the west end has a vertical strip of windows. The skin of the building was designed as explosive release wall assemblies in the event of an explosion.

Structural System

The structural system is comprised of is moment frame structural steel. Every girder column connection is a moment connection. The lateral systems in both the north-south and east-west directions are structural steel moment frame systems. The first floor has an 8 inch cast-in-place concrete slab while the other four floors have normal weight concrete on metal deck. The entire building is sitting on precast concrete piles.

Mechanical System

The Production Building has two 5600 MBH has-fired boilers, two 507 ton air cooled screw chillers , and two AHUs. One small AHU serves the boiler building, while the large AHU serves the Production Building. The system is a constant air volume system with terminal air units which condition the air for each space locally. The large AHU uses an energy reclaim coil using a 30% glycol solution.

Electrical System

The electrical system has three service entrances at 480/277V. One powers the motors and control centers, one powers control centers and chillers, and the last controls PBP panels which power the heaters. There are small transformers for lighting/receptacle loads. A UPS at 208/120V serves emergency lighting loads.

Executive Summary

The following report explores the viability of a concrete moment frame structural system instead of the steel moment frame system in CBD Chemical's Production Building. This building is a five story, 55,000 GSF chemical production building with a mezzanine on the first floor, main production floor, and penthouse roof. Due to the heavy loads sustained in the Production Building, a concrete beam and girder system was designed and analyzed. The effects of a concrete frame on the foundations were analyzed as well as a cost and schedule analysis to determine which framing system would ultimately be cheaper.

The gravity beams were designed as 12x22, the lateral beams and girders were determined to have to be 12x30 and the columns are 30x30. Due to the heavier dead loads of the concrete structure compared to the steel structure, earthquake loading controls the lateral design throughout the building. The heavier building would increase the cost of the deep foundation, concrete piles. The total cost of the concrete structure was determined to be less, however it would take longer to construct than the steel structure. This makes the concrete system a viable solution to the existing structure.

On the other hand the existing steel structure could be cheaper if designed compositely. Using the amount of shear studs already placed in the building, it was determined roughly 95,000 pounds of steel could have been saved if the structure were designed to take advantage of the added strength. This would save about $144,000. The recommended structural system for the Production Building is a concrete moment frame.

To determine if some of the large energy usage could be offset, a photovoltaic panel study was performed. The roof would have 60 Solon Black XT 290 Wp panels which would save roughly $7,723.10 in energy costs per year. Due to the price of photovoltaic panels, however, the payback period would be 8.5 years. Therefore, the PV panels are not recommended for this project.

Acknowledgements

First, I would like to thank the Architectural Engineering Faculty and Staff for teaching me a little of the knowledge they have gained over the years. They are the reason that Penn State has such a great AE program and why we are some of the best sought after engineers in the world. These professors not only taught me a bit about engineering but also imparted some lifelong wisdom. I am honored to have learned so much from them and now join many as part of the Penn State Alumni family.

Second, I would like to thank my family for the support they have given me through my lifetime. I would not be anywhere near where I am today without you. Also, my AE friends have helped keep me sane through all those all-nighters and late night study sessions. Thank you for making even Thesis Lab a fun place to be.

Lastly, I would like to thank the project team and owner of my building. Although I cannot name you personally, thank you for answering my questions, going above and beyond to help me get the documents and information I needed, and for even letting me use your building at all. This thesis, while long and often frustrating, has taught me so much and I know will help me when I start on my career in the fall. Thank you!

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1.0 Introduction

The purpose of this report is to explore viable solutions to the design constraints for The Production Building in Virginia, USA. The existing steel structure was compared to both a composite steel floor structure and a concrete beam and girder moment frame system. The cost and schedule of the concrete versus steel options were explored as well as some possible energy saving through the use of photovoltaic panels on the roof.

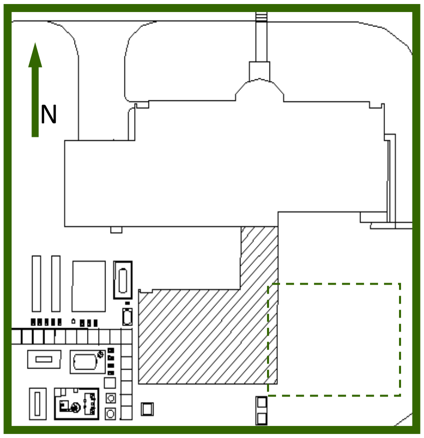


Figure 1.1: Site Plan. Courtesy of Project Engineer. This plan shows a portion of the campus footprint with the Production Building shaded. The future bays will be located in the dashed area.

The Production Building is an addition to an existing campus with laboratory and chemical manufacturing spaces owned by CBD Chemical*. CBD Chemical has occupied the site since 1991 and produces drug substances and intermediates for the pharmaceutical industry. Each facility on site is an FDA inspected cGMP facility. This five-story, $125 Million, 55000 GSF addition includes a penthouse roof as well as a mezzanine level above the first floor. This addition also connects to the existing building at the first floor level. Figure 1 shows the footprint of the existing building campus, the current Production Building addition (shaded area), and the future production building to be built (dashed area). The space was designed to easily expand farther east. Construction started in April 2008 and was completed in January 2009. This project was design-bid-build with a Negotiated Guaranteed Max Contract.

The majority of the chemical production equipment will be located on the first floor, although much of the facility will house additional production spaces, laboratory spaces, and production support. The existing two story building houses the majority of office space; however, the second floor of the new production building incorporates some additional office space.

The Production Building is composed of a steel frame structure with concrete on metal deck for the floor systems. The exterior skin is a combination of insulated metal panels and translucent wall panels. Due to the highly explosive material within, many of the walls must be blast resistant. Some of the factory-insulated metal wall panel systems serve as the explosion release wall assemblies. Each floor has explosion release wall assembly panels as well as translucent pressure venting assembly panels. The north and south facing walls have horizontal strips of windows, while the West end has a vertical strip of windows. The roof is comprised of concrete on metal deck, rigid insulation and an EPDM waterproof membrane covering.

Figure 1.2: Isometric View. Courtesy of Project Architect. The Production Building is the five-story building in the back.

***Name changed for confidentiality**

2.0 General Structural Information

The structural system for the Production Building is moment frame structural steel. The first floor has an 8 inch slab on grade while the other four floors have normal weight concrete on metal deck. The Production Building was designed to IBC 2003, and used ASCE 7–02 and the AISC Steel Manual 3rd edition LRFD; however for the purposes of this report, it will be checked against the most recent ASCE 7–10 and 14th edition of the AISC Steel Manual.

Foundation System

The Production Building was built on site class E soils as noted in the geotechnical report.

The foundation system for CBD Chemical's Production Building is precast concrete piles 12 inch x12 inch that are 80 ft long. Each pile had to be driven to an elevation of 20 feet. On top of the concrete piles are spread footings with piers that extend up to the concrete tie beams that span between each column. Figure 2.1 to the right shows a typical pile cap detail.

Each of the precast concrete piles has 28-day strength of 6000psi and has a 100-ton capacity. The spread footings and strip footings used concrete with 28-day strength of 4000psi. On the first floor, the slab on grade is an 8 inch cast-in-place concrete slab. All rebar is grade 60.

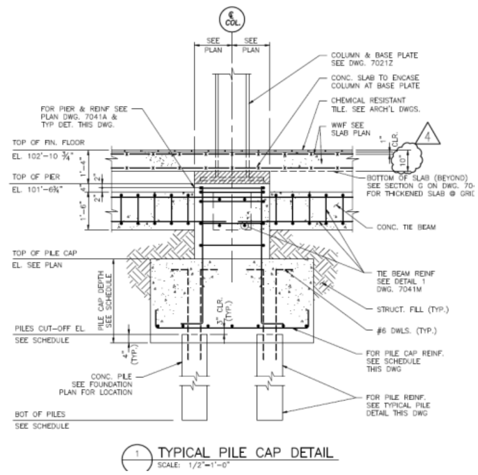


Figure 2.1: Typical Pile Cap Detail. Courtesy of Project Engineer.

Floor System

The floor system is comprised of 7½ inch normal weight concrete on a 2VLI 18 gage composite deck. This forms a one-way slab system running in the east-west direction. The deck must use the three-span condition unless framing does not permit. On the mezzanine level, 1¼ inch steel grating was used.

Framing System

The framing system is composed of W24s for the girders and exterior beams. W12s are used as infill support underneath equipment. Figure 2.2 is the third floor framing plan. In the figure the different spans and infill beams are shown, as well as the equipment framing for the large equipment. The 12 foot girders span the bay from which the pipe racks hang. These are framed with W12s. The beams are framed 3 equal spaces of 6 feet 3 inches, 3 equal spaces of 6 feet 8 inches and 5 equal spaces of 6 feet for the 12 feet 6 inches, 20 feet, and 30 feet East-West bays respectively. The beams included in the lateral system are larger than the infill beams between column lines. However, in locations underneath large equipment loads, the infill beams were increased. In addition, the second floor and fourth floor have equipment built in. Thus, some of the beams had to be spaced slightly differently at those locations. In this case, more framing was necessary to hold the equipment in place. There are W12s framing in between the beams in the East-West direction. The mezzanine level is only special framing to accommodate specific equipment. This framing uses W8s, W10s, and W16s and frames into select columns on the first floor level. The pipe racks on each floor hang from the floor structure above, also utilizing W6s and W8s. Every beam on every floor has ¾ inch diameter steel studs spaced at one foot on center. Each beam works compositely with the slab above. The columns are W14s and are spliced every 2 floors. The floors have large floor to floor heights of 24 feet for the first floor and 18 for subsequent floors. This is because vessels, equipment, and the W24beams and girders must fit above the ceilings. See Appendix A for the additional framing plans. Each floor is slightly altered from the typical framing system in at least one location.

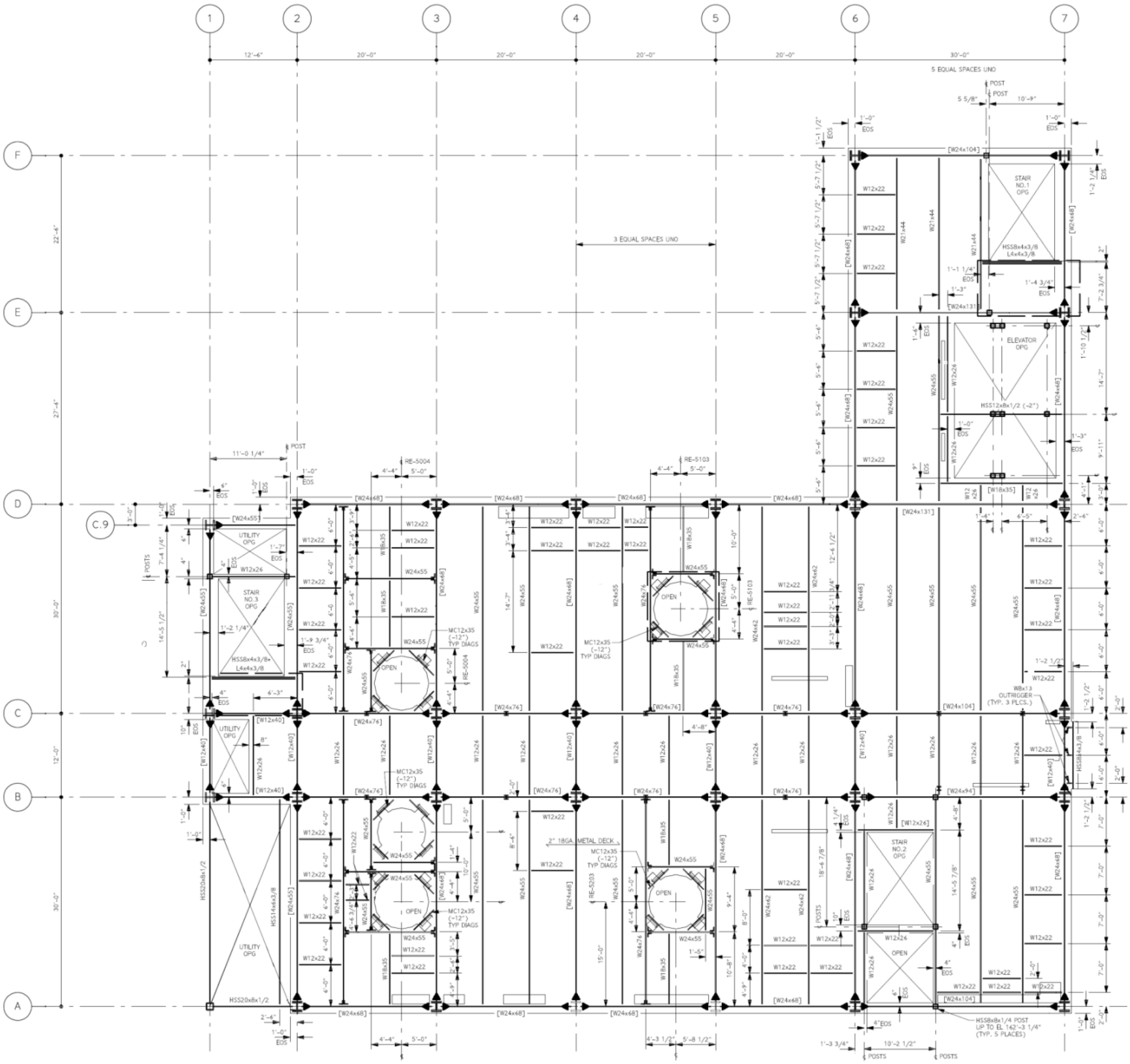


Figure 2.2: Courtesy of Project Engineer. The third floor framing plan.

Lateral System

The lateral system for the Production Building is comprised of steel moment frame connections. Each column has moment connections in both the North-South and East-West directions. Due to CBD Chemical's requests for the Production Building, there was very little room to fit any other kind of lateral system. There simply was no room for any shear walls or even bracing. Due to this constraint, the engineers had still needed extra capacity in the lateral system and needed to turn the columns on the West end 90˚ so the strong axis was along the East-West direction. The out of the ordinary column placement is highlighted in Figure 2.2. The mezzanine does not contribute to the lateral system.

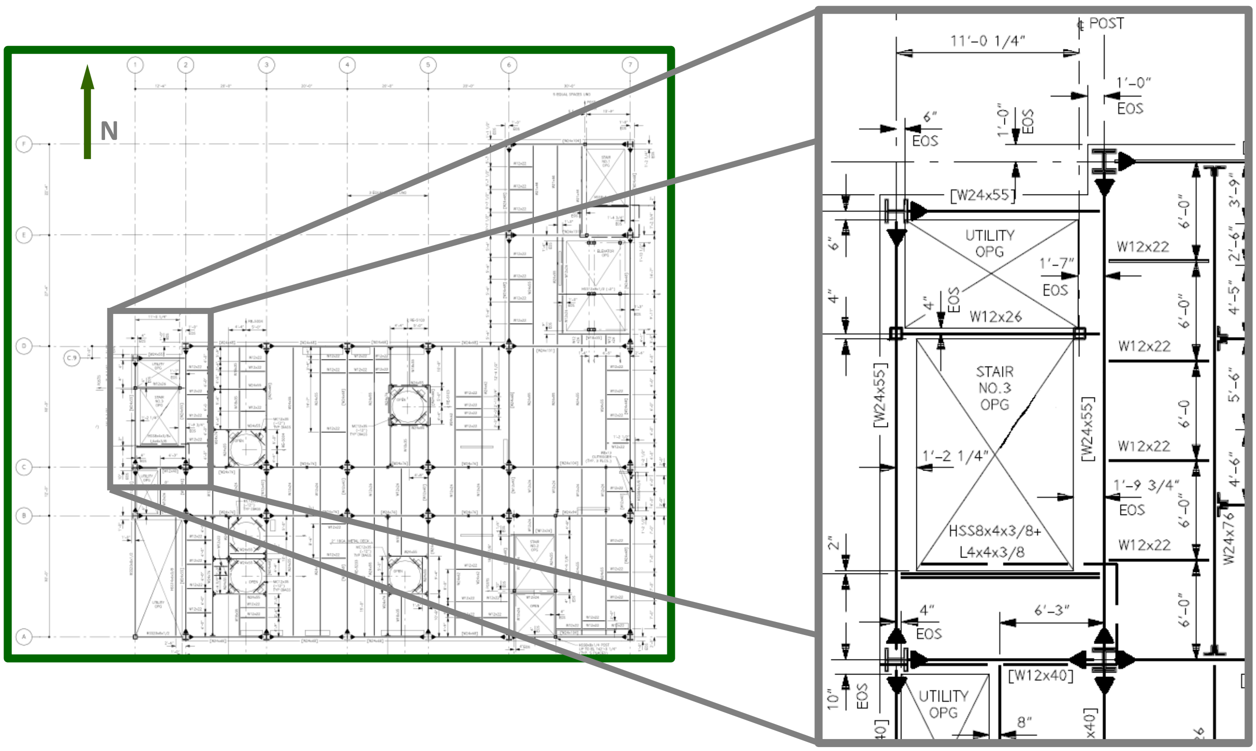


Figure 2.3: Courtesy of Project Engineer. The third floor framing plan showing the odd column rotation on the west end of the building.

With this lateral system any horizontal loads will be caught by the insulated metal panel system. The explosive pressure release panels are tied to the building frame through the use of HSS tubes which then transfer load to the slab system. The slab system works as a rigid diaphragm due to the large amount of concrete from which it is comprised. From the slab system the load is transferred to the foundation through the beams, then to the girders, and lastly to the columns, which sit on pads sitting on concrete piles.

3.0 Thesis Objectives

3.1 Structural Depth

The current Production Building is steel moment frame. Moment frames are extremely expensive in a steel building but are "free" in concrete frames. Due to the availability of concrete in Virginia, USA, concrete could be an extremely viable solution to the design constraints of the Production Building. Because the usable interior space is a large concern special consideration should be taken to ensure CBD Chemical receives the space needed for the chemical production. As stated in section 4.4.5 the cost of a concrete floor system seems to be cheaper.

The production building will be redesigned as a reinforced concrete structure. There is a possibility a concrete moment frame could have been cheaper using the assumptions previously stated in this report. The floor system explored in the redesign will be a one way slab joist system. Although for the initial comparison in section 4.4 the bay sizes were averaged, which largely impacts how the interior spaces are utilized. Therefore, a comparison of the systems with the initial bay spacing seems most appropriate. The gravity system will consist of a 6 inch slab while the beams will be 20 inches deep. The girders will be 28 inches deep. The system originally designed and analyzed in section 4.4.4 will be reviewed to establish if these sizes will still be adequate for the increased spacing. Figure 5.1 shows the layout of the one way slab system. This system will then be compared to a second redesign consisting of a composite steel moment frame structure that is similar to the current design. The new composite floor system would be comprised of 2VLI 18ga composite deck with 6 inches of normal weight concrete. The beams would be W16x31. Comparing these two systems will help determine whether steel or concrete would be a more effective solution given the design assumptions used for this report.

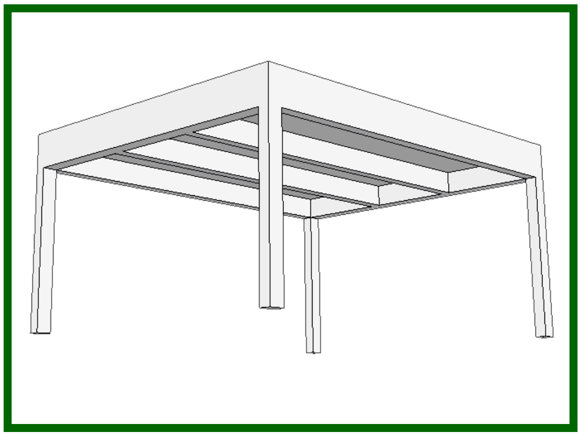


Figure 3.1: A sketchup model of the layout of the one-way slab system.

Lastly, the building is built on a deep foundation system. A heavy building could significantly increase the foundation cost. This once again favors a lighter steel building, which further increases the need to compare the two systems in much more depth. Excluding the foundation impact, a concrete building would be cheaper to construct continuing with the assumptions in this report.

3.2 Breadth Topics

The largest concern of the concrete versus steel building is which would be a more viable solution to the constraints of the Production Building. In order to establish the economy of each option, the cost and schedule were examined. Ultimately, the cheaper and faster option would be the best solution for the Production Building.

Because the Production Building uses so much energy in the everyday production, the possibility of offsetting some of that energy was explored in the use of photovoltaic cells placed on the roof. The production building is not surrounded by any taller buildings, allowing the sun to reach the roof at all times throughout a sunny day. The impact the photovoltaic cells could have to the electricity use of the building will be studied.

3.3 MAE Requirement

Using material learned in AE 597A the lateral system was modeled in ETABS. This model was used to analyze and design the concrete moment frames fulfilling the MAE requirement.

4.0 Steel Optimization

In order to have a comparison based on the same assumptions, I redesigned the steel system by just sizing the members to take advantage of the shear studs already in place throughout the building. The drawings called for one shear stud placed every foot along all the beams in the Production Building. This allowed for the beams to be downsized from the non-composite design used by the engineers.

In order to optimize the steel design according to the same assumptions used in the concrete redesign, a composite system was designed. 2VLI18 decking with 4 inch topping thickness. The whole lateral system will remain the same as the original design. Because the girders and specific beams are part of the lateral moment frame system, composite construction will not add to any strength because the maximum controlling moment is negative. The beams not included in the lateral system were able to be downsized to 16x31 and 12x14 saving 95,040 pounds in steel weight. These beams use the same amount of shear studs as the Production Building has currently. The table below, figure 4.1 shows the potential benefit of using the shear stud strength. The prices were taken from RSMeans, Building Construction Cost Data 2011.

Size	# of studs / ft	plf	linear feet	Total price/ft	Total price	total wt
W24x55	1	65	3600	$ 71.41	$ 257,076.00	234000
W16x31	1	41	3600	$ 42.13	$ 151,668.00	147600
W12x26	1	26	720	$ 36.23	$ 26,085.60	18720
W12x14	1	14	720	$ 24.08	$ 17,337.60	10080
				Total Savings	$ 114,156.00	95040

Figure 4.1 This table shows the benefit of using the shear studs already in the building for their added capacity.

5.0 Concrete Redesign

5.1 Gravity Design

The gravity system was redesigned due to the loads calculated in technical report 1. The superimposed dead load on the structure is 80psf while the live load is 200psf. The live load was increased from the 125psf required by code due to the chemical equipment being sulfuric acid vessels which has a higher density than water. Because each column and girder is part of the lateral system as well, only the one way slab and beams in the gravity system were designed first and found preliminary sizes for the lateral system. None of the structural layout was altered. The space in the original design was very limited, so none of the bays were altered. A 6 inch slab would work for the assumed loads as well as the point loads from the equipment. Only the largest span of the slab was designed for due to the slab being the same thickness throughout the floor. The majority of the equipment is located on top of beams. Two beams were designed due to the typical bays. Figure 5.1 shows the typical beams that were designed, as well as the layout for all the beams in the concrete system.

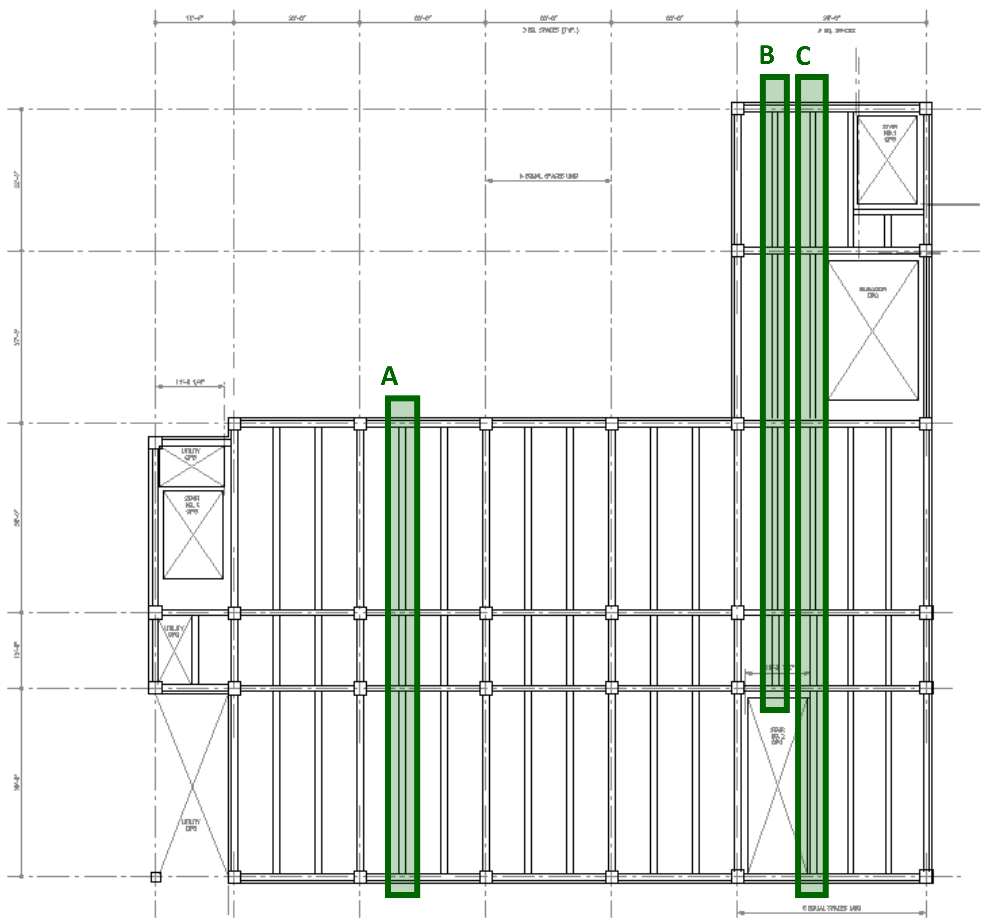


Figure 5.1 The three different typical gravity beams designed.

Using the coefficients of continuous beams, and keeping all the beams the same size for ease of construction, the beams are all 12x22. These beams are shorter than the W24s that were used in the original design, which could allow for a smaller floor to floor height. The beams, girders, and columns all use only #6, #8, or #10 bars. For full gravity calculations please see Appendix B. The rest of the members in the Production Building are part of the lateral system as well as the gravity system so they are redesigned in the lateral design section. Figure 5.2 shows the calculations to determine the reinforcing needed for section A. Figure 5.3 shows the rebar details for one of the gravity beams.

SECTION B												
Trib Width=	6.0 ft.		b = 12		h = 22		d = 19.5					
	Beam 1			Beam 2			Beam 3			Beam 4		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support
Span	12	-	-	30	-	-	27.5	-	-	22.5	-	-
l_n (ft) =	11	11	20	20	29	27.75	27.75	26.5	26.5	24	21.5	21.5
w_u (klf) =	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
$w_u l_n^2$ =	396	396	1310	1310	2755	2523	2523	2301	2301	1887	1514	1514
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.1000	0.07143	-0.0417
$M_u = C_m W_u L_n^2$ =	-16.5	28.3	-131.0	-119.1	172.2	-229.3	-229.3	143.8	-209.1	-188.7	108.2	-63.1
b_{eff} (in) =	41	12	60	60	12	60	60	12	83.25	83.25	12	72
$A_{s(req'd)}$ (in.) =	0.2091358	0.35852	1.65925926	1.50841751	2.18037	2.90393939	2.90393939	1.8206481	2.64821549	2.38933333	1.36963	0.79895062
$A_{s(prov'd)}$ (bars) =	**(2) #6**	**(2) #6**	**(4) #6**	**(4) #6**	**(3) #8**	**(4) #8**	**(4) #8**	**(5) #6**	**(4) #8**	**(4) #8**	**(5) #6**	**(2) #6**
a =	1.29411765	1.29412	2.58823529	2.58823529	3.48529	4.64705882	4.64705882	3.2352941	4.64705882	4.64705882	3.23529	1.29411765
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	0.88	0.88	1.76	1.76	2.37	3.16	3.16	2.2	3.16	3.16	2.2	0.88
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
ρ =	0.00376068	0.00376	0.00752137	0.00752137	0.01013	0.01350427	0.01350427	0.0094017	0.01350427	0.01350427	0.0094	0.00376068
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.22122985	0.37925	0.90879718	0.82617925	0.90924	0.93895257	0.93895257	0.8121866	0.85626744	0.77256112	0.61099	0.84515287
	0.90879718			0.938952573			0.938952573			0.845152874		

Figure 5.2 Calculations to determine reinforcing in continuous beam A. The same method was used throughout all reinforcement calculations.

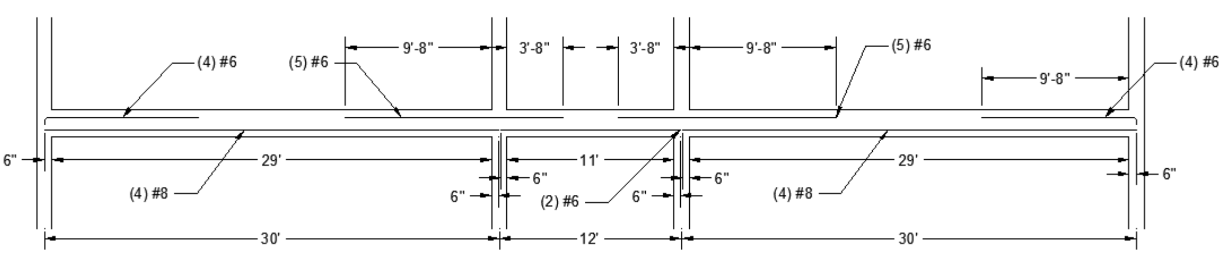


Figure 5.3 Reinforcing required beam A.

5.2 Lateral Design

Lateral Loads

The first step to the lateral design of the Production Building is recalculating the seismic loads. Due to the design category of C, an intermediate concrete moment frame must be designed. This increases the R value to 5. In addition the weight of the building increased which increases the earthquake loads. Using the Equivalent Lateral Force Procedure from ASCE 7-10 the new earthquake loads were determined. Figure 5.4 shows the new earthquake loading, base shear and overturning moment for the concrete frame. Please see Appendix C for full earthquake calculations.

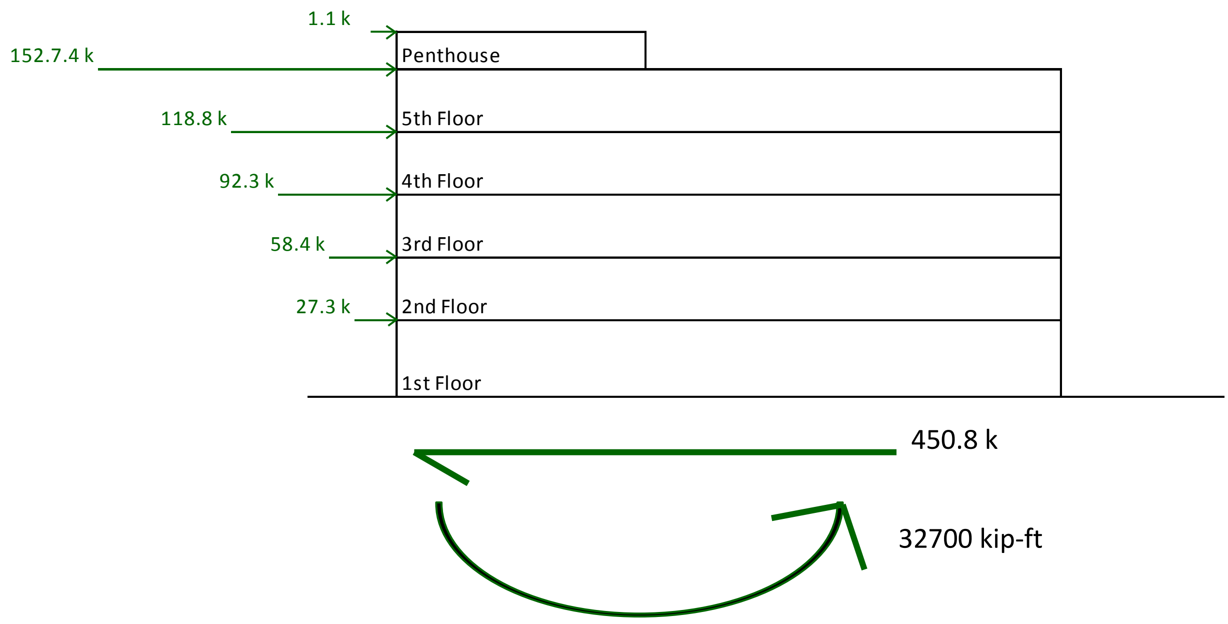


Figure 5.4: The seismic story forces, base shear and overturning moment.

In technical report 1, the wind loads in both directions were calculated for the Production Building. While in the original steel building wind did not control over earthquake, with the intermediate concrete moment frame, the earthquake loads are reduced from a larger R-value. This leads the wind to control throughout the building. The pressure distributions for both the North-South and East-West load cases are included below in figures 5.5 and 5.6.

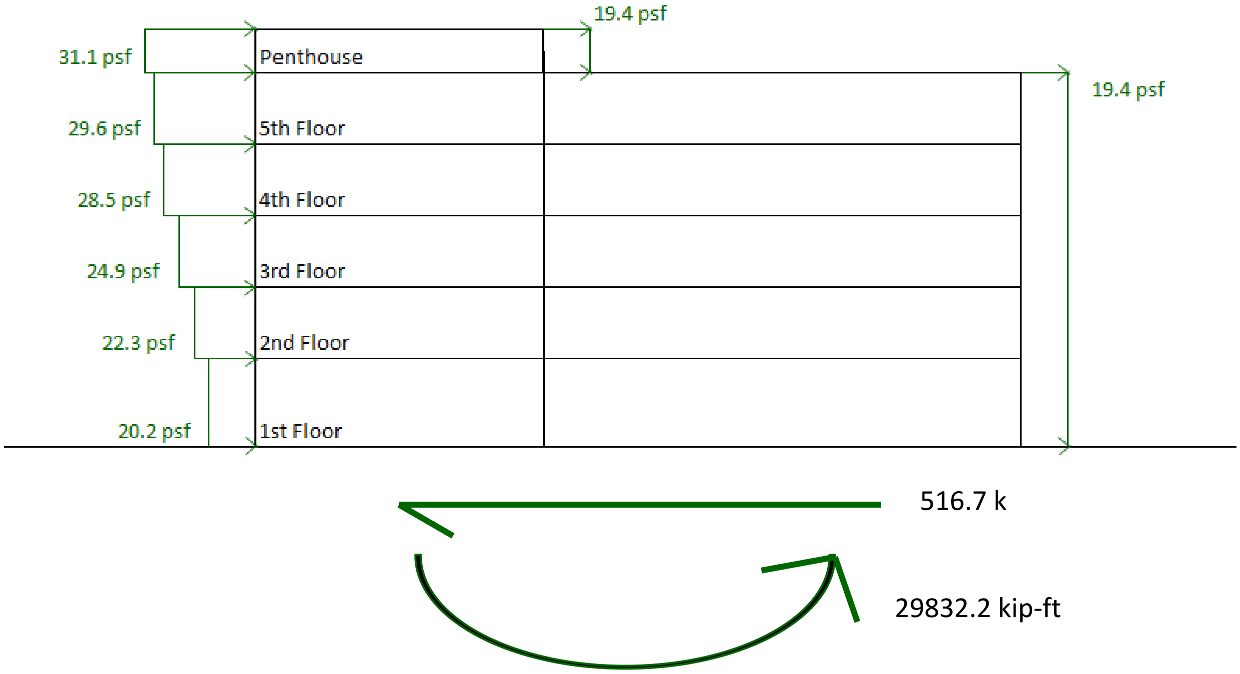


Figure 5.5: The pressure distribution, base shear and overturning moment for the East-West wind load case.

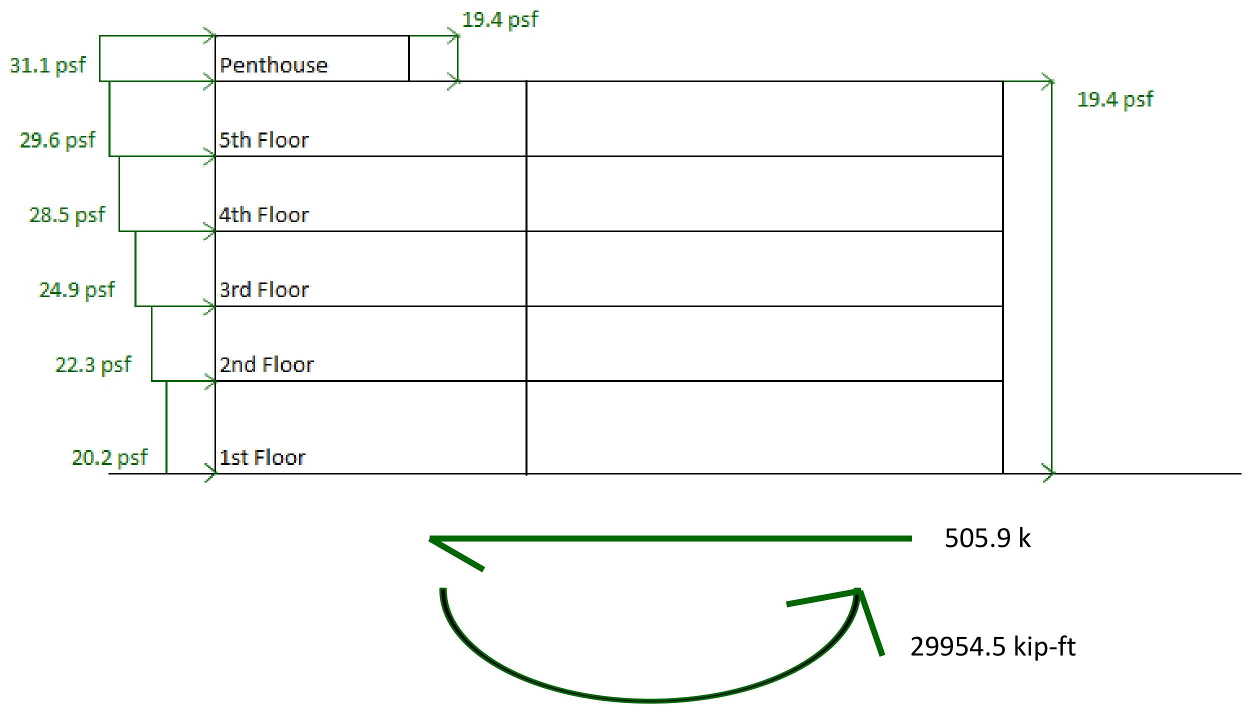
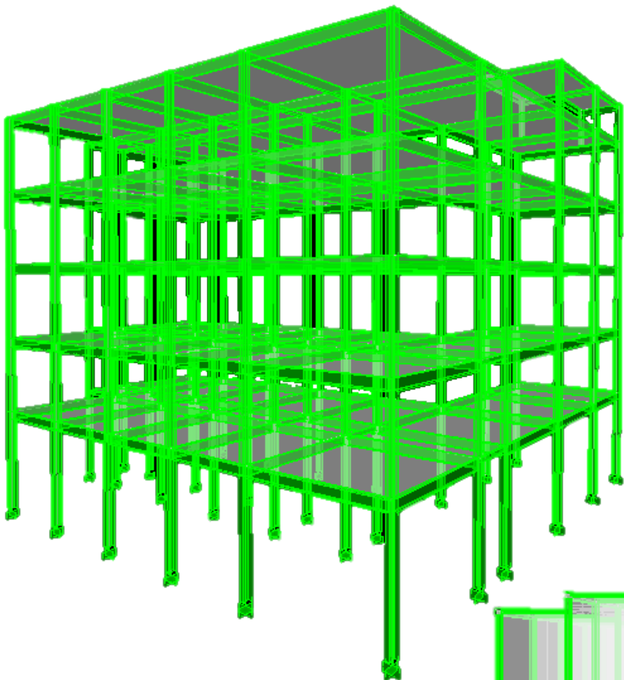


Figure 5.6: The pressure distribution, base shear and overturning moment for the North-South wind load case.

Lateral Beams and Girders

The lateral system of the original building uses each column and girder as part of the moment frame. The new system does the same. The lateral beams and girders are 12x30. The same continuous method was used for the lateral beams as the gravity beams with the new load case. The controlling load case for most members was 1.2D+1.0W+L. The lateral system was modeled in ETABS. Figure 5.5 shows two views of the ETABS model. The slabs were modeled as rigid diaphragms, rigid end zones were applied to all beams with a reduction of 50%, all members were modeled without self-weights which were applied as an additional area mass at the center of gravity of the diaphragms. The moments of inertia for the columns and beams were reduced by factors of 0.7Ig and 0.35Ig respectively. Lastly P-delta effects were considered in the design of the columns.



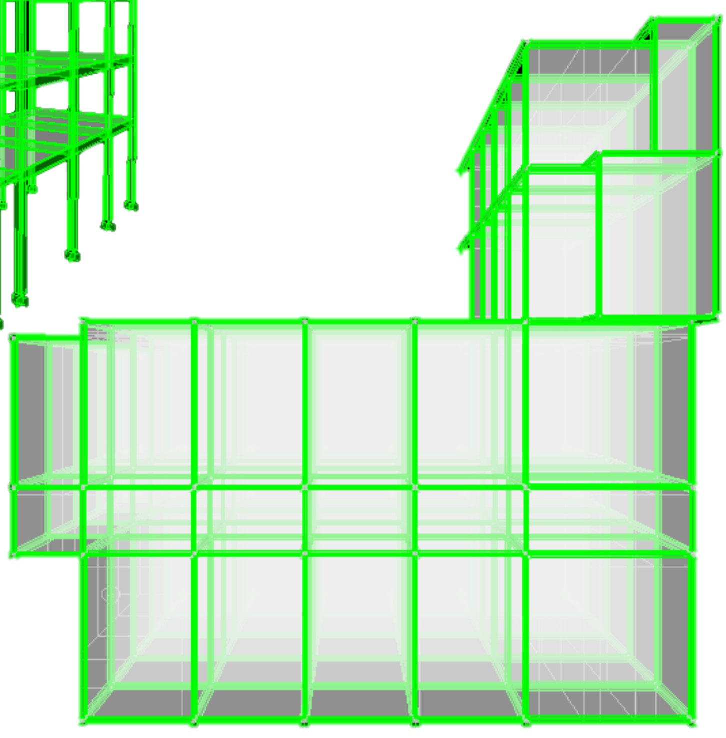


Figure 5.5: Two views of the ETABS model used to determine earthquake and wind loads on individual members. The top is an extruded view fo the structure while the bottom image is a bird's eye view.

The reinforcing was determined in excel similar to the gravity system, for both the beams and girders. The beams and girders were all sized the same to save on construction costs. As concrete is cheap compared to steel the beams were sized up to 12x30 and minimal reinforcing is used. All the reinforcing was #6, #8, or #10 bars. As specified in ACI318-11 the positive moment strength at the end of the beams must be 1/3 the strength of the negative moments as well as neither strengths can be less than 1/5 the maximum moment strength at either joint. Due to this each beam and girder must be designed as doubly reinforced. In addition the shear stirrups must be spaced together closer than in an ordinary moment frame. Figures 5.6 and 5.7 show the calculations for a lateral beam and the reinforcement layout. Please see Appendix D for full calculations of the lateral beams and girders. The different shading of reinforcement in the calculations shows where the steel would be placed. The darker grey is the continuous line on the top of the section and the lighter grey is the continuous line on the bottom of the section.

BEAMS 2/3/4/5									
Trib Width=	6.7 ft.		b = 12		h = 30		d = 27.5		
	Beam 1			Beam 2			Beam 3		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support
Span	30	-	-	12	-	-	30	-	-
l_n (ft) =	27.5	27.5	18.5	18.5	9.5	18.5	18.5	27.5	27.5
w_u (klf) =	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9
$w_u l_n^2$ =	2218	2218	1004	1004	265	1004	1004	2218	2218
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.1000	0.0714286	-0.0417
$M_{uG} = C_m W_u L_n^2$ =	-92.4	158.5	-100.4	-91.3	16.5	-91.3	-100.4	158.5	-92.4
M_{uE} =	-150.6	0.0	-144.5	-282.3	0.0	-282.3	-144.5	0.0	-150.6
M_{utot} =	-243.0	158.5	-244.9	-373.6	16.5	-373.6	-244.9	158.5	-243.0
b_{eff} (in) =	39.5	12	55.5	55.5	12	55.5	55.5	12	40
$A_{s(req'd)}$ (in.) =	2.060865167	1.311277	2.107409095	3.2146967	0.136925	3.2146967	2.10740909	1.3112769	2.06086517
a =	2.588235294	1.294118	3.352941176	3.35294118	1.294118	3.35294118	3.35294118	1.2941176	2.58823529
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	2.2 (5) #6	1.32 (3) #6	2.46 (2)#6, (2)#8	2.46 (2)#6, (2)#8	1.32 (3) #6	2.46 (2)#6, (2)#8	2.46 (2)#6, (2)#8	1.32 (3) #6	2.2 (5) #6
A_s' (in.) =	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	356.1	205.3	413.4	413.4	205.3	413.4	413.4	205.3	356.1
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK
ρ =	0.0067	0.0040	0.0075	0.0075	0.0040	0.0075	0.0075	0.0040	0.0067
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.683	0.772	0.592	0.904	0.081	0.904	0.592	0.772	0.683

Figure 5.6 Calculations for the reinforcing required for column line 3.

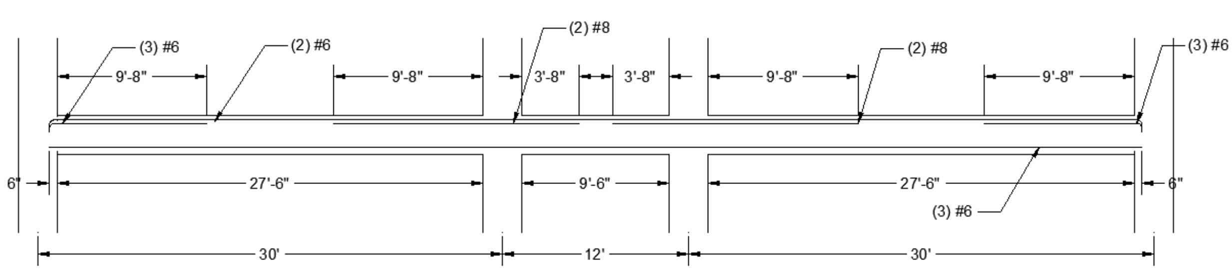


Figure 5.7 Reinforcing required for column line 3.

Column Design

The lateral columns were designed in SpColumn. The controlling load case turned out to be 1.2D+1.0W+L+.5S, although 1.2D+1.6L+.5S was also checked. All the columns were sized the same to save money on formwork and labor. All columns are 30x30. They use either #8 bars or #10 bars. There are three different rebar configurations. Figure 5.8 is the spColumn output of a typical column design. The table in figure 5.9 shows the grouping of the similar columns and the final rebar design for each group. Please see appendix D for the complete table with loads and all calculations.

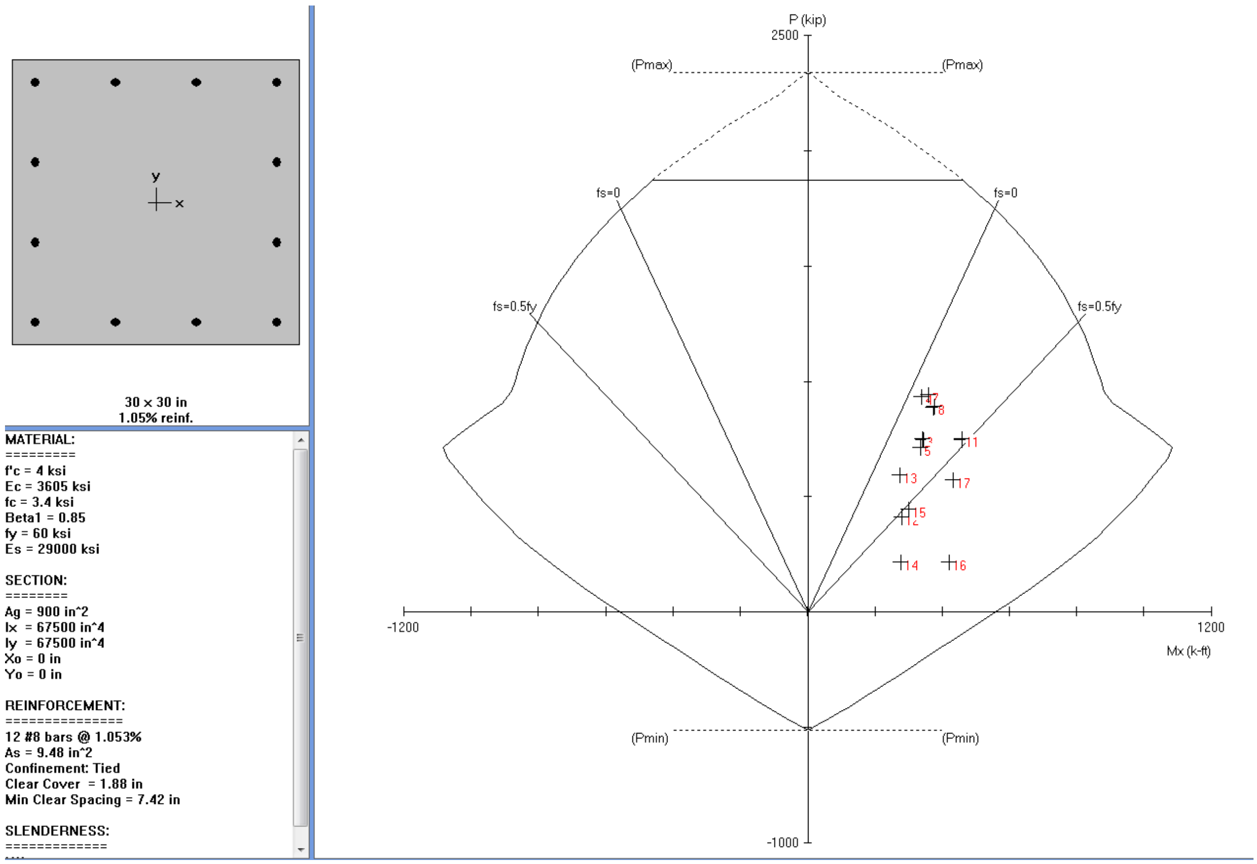


Figure 5.8: The output from spColumn showing the interaction of the beam column.

Column	Trib Area	1.2D+1.0W+L+.5S		1.2D+1.6L+.5S		Rebar
		P_u	M_u	P_u	M_u	
A.2	150	413.4	281.1	448.5	63.4	(12) #8
A.3	300	752.8	343.0	896.9	117.6	(12) #8
A.4	300	754.1	342.3	896.9	117.6	(12) #8
A.5	300	756.3	342.5	896.9	117.6	(12) #8
A.6	375	938.0	339.4	1121.2	117.2	(12) #8
A.7	225	596.0	272.7	672.7	59.2	(12) #8
B.1	37.5	216.3	277.5	112.2	1.6	(12) #8
B.2	247.5	715.2	336.5	739.9	54.0	(12) #8
B.3	420	1128.2	470.3	1255.7	238.5	(12) #10
B.4	420	1120.0	470.6	1255.7	238.5	(12) #10
B.5	420	1128.2	470.8	1255.7	238.5	(12) #10
B.6	525	1390.8	492.5	1569.6	255.0	(12) #10
B.7	315	892.0	374.2	941.8	127.5	(12) #8
C.1	131.25	446.9	299.8	392.4	1.6	(12) #8
C.2	341.25	945.6	360.2	1020.3	54.0	(12) #8
C.3	420	1128.2	493.2	1255.7	238.5	(12) #10
C.4	420	1120.0	493.4	1255.7	238.5	(12) #10
C.5	420	1128.2	493.6	1255.7	238.5	(12) #10
C.6	525	1392.9	499.3	1569.6	255.0	(12) #10
C.7	315	894.3	377.1	941.8	127.5	(12) #8
C9.1	81.25	217.9	421.4	243.0	111.0	(12) #8
D.2	225	576.6	432.5	672.7	117.6	(12) #8
D.3	300	752.8	459.5	896.9	117.6	(12) #8
D.4	300	754.1	458.5	896.9	117.6	(12) #8
D.5	300	755.8	458.8	896.9	117.6	(12) #8
D.6	543.75	1350.2	450.3	1625.8	117.6	(12) #10
D.7	393.75	992.3	640.4	1177.2	398.1	(12) #10
E.6	375	956.8	1118.6	1121.2	942.5	(16) #10
E.7	375	956.8	1118.3	1121.2	942.5	(16) #10
F.6	168.75	458.7	823.7	504.6	518.4	(12) #10
F.7	168.75	458.7	823.8	504.6	518.4	(12) #10

Figure 5.9: Final rebar layout of each column and the loads to which they were designed.

Drift Analysis

All of the drifts of the Production Building under wind and seismic loads are acceptable. The maximum drifts were calculated in ETABS for both wind and earthquake in the North-South (Y) and East-West (X) directions. The wind story drifts were compared to h/400 which although not required by code is commonly used. Due to h/400 being a serviceability check the unfactored wind drifts were used. Story drifts under earthquake loads had to be compared to .015h for category III buildings. The drifts obtained from ETABS are compared to their limits in table 5.10.

Drift (in.)														
Floor	Height (ft)	WIND ANALYSIS						EARTHQUAKE ANALYSIS						
		WIND - E/W		WIND - N/S		Allow	Pass?	EQ - E/W		EQ - N/S		Allow	Pass?
		x-dir	y-dir	x-dir	y-dir			x-dir	y-dir	x-dir	y-dir		
Penthouse	15	0.07	0.04	0.12	0.09	0.45	YES	0.19	0.05	0.06	0.06	2.7	YES
Story 5	18	0.15	0.08	0.10	0.20	0.54	YES	0.59	0.10	0.14	0.61	3.24	YES
Story 4	18	0.25	0.13	0.14	0.32	0.54	YES	0.93	0.17	0.26	0.95	3.24	YES
Story 3	18	0.34	0.16	0.20	0.43	0.54	YES	1.19	0.21	0.30	1.17	3.24	YES
Story 2	18	0.38	0.18	0.24	0.51	0.54	YES	1.27	0.22	0.33	1.27	3.24	YES
Story 1	24	0.44	0.12	0.23	0.66	0.72	YES	0.85	0.13	0.22	0.87	4.32	YES

Table 5.10: Actual drifts of the building compared to allowable drifts for both wind and earthquake

5.3 Foundation Impact

Due to the fact that the Production Building sits on deep foundations increasing the buildings weight could be very costly. The weight of the building went up a total of about 36.4% which would greatly impact the foundations. The piles are called out to be able to support 100 tons each. Roughly using the loads that were calculated for each column, each column would need at least an additional pile. Since the first floor is sitting on grade beams the weight of that floor would also be supported by the piles. Figure 5.11 shows the load of each column on the foundation and roughly how many piles would be needed at that location. Using this method only 13 more piles are required, however this is an extremely simplified approach.

Column	P	P	Existing Cassions in Steel Design	Needed Cassions in Conc Design
	D+.75(.6W)+.75L+.75S	D+L		
A.2	345.5	356.3	3	3
A.3	657.5	712.5	4	4
A.4	658.1	712.5	4	4
A.5	659.1	712.5	4	4
A.6	820.6	890.7	4	5
A.7	507.3	534.4	4	4
B.1	137.2	89.1	3	3
B.2	584.8	587.8	4	4
B.3	954.0	997.5	4	5
B.4	950.3	997.5	4	5
B.5	954.0	997.5	4	5
B.6	1183.8	1246.9	6	7
B.7	736.2	748.2	6	6
C.1	340.6	311.8	3	3
C.2	788.2	810.6	4	5
C.3	954.0	997.5	4	5
C.4	950.3	997.5	4	5
C.5	954.0	997.5	4	5
C.6	1184.7	1246.9	6	7
C.7	737.2	748.2	6	6
C9.1	184.4	193.1	3	3
D.2	498.6	534.4	4	4
D.3	657.5	712.5	4	4
D.4	658.1	712.5	4	4
D.5	658.9	712.5	4	4
D.6	1185.5	1291.6	6	7
D.7	865.0	935.2	6	6
E.6	829.1	890.7	4	5
E.7	829.1	890.7	4	5
F.6	385.8	400.9	4	4
F.7	385.8	400.9	4	4
		Σ =	132	145

Figure 5.11: A simplified approach to the number of piles needed for each column.

6.0 Construction Management Breadth

The most important factor for many owners is cost. Both schedule and actual construction costs effect how much the building will cost the owner. For this report a detailed cost estimate was done for the reinforced concrete structure and compared with the existing structure. Using RS Means Building Construction Cost Data 2011 unit process for the concrete, reinforcement, formwork, labor, equipment, and overhead and profit were estimated. The cost from the project engineer on the original design was $4,599,899 for structural steel and another $537,211 for the concrete slabs totally $5,197,429. The total cost estimated for this redesign structure is $2,81642.24. This is a savings of $2,384786.76. Figure 6.1 shows a simplified cost breakdown of the concrete structure. The full cost analysis can be found in Appendix E.

Concrete Structual Element	Total-O&P	Total Price
Concrete	$ 1,810,613.96	$ 2,161,293.70
Finish	$ 3,612.00	$ 5,882.40
Formwork	$ 493,028.65	$ 748,430.13
Reinforcing	$ 1,273,313.64	$ 1,665,788.74
Total	$ 3,072,127.56	$ 3,930,836.88

Figure 6.1: Total prices for each part of the concrete system.

Both a steel and concrete schedule were created in MS Project based on RSMeans data. The steel schedule was estimated to take 107 days while the concrete schedule would take about 223 days. For the production building, the lost profits due to the extra time for a concrete structure may outweigh the saved construction costs. Please see the full schedules in Appendix E.

7.0 PV/Electrical Breadth

The Production Building uses large amounts of energy every day. An analysis was performed to determine if photovoltaic panels on the roof would help offset energy usage. Initial analysis using PV showed that trying to get the maximum number of panels on the roof in a flat orientation would most likely provide the most energy and there is enough savings to warrant a closer analysis. A large amount of the roof already has equipment; however there were three areas that could each fit 20 Solon Black XT 290 Wp panels flat without interfering with any of the equipment or walkways already on the roof. There will be three Fronius IG Plus 6.0-1_{UNI} PV inverters with two strings of 10 panels per inverter. Figure 8.1 shows the panel chosen for this design while figure 8.2 shows the areas for the PV array on the roof.

Figure 8.1: The panel used in the Production Building array.

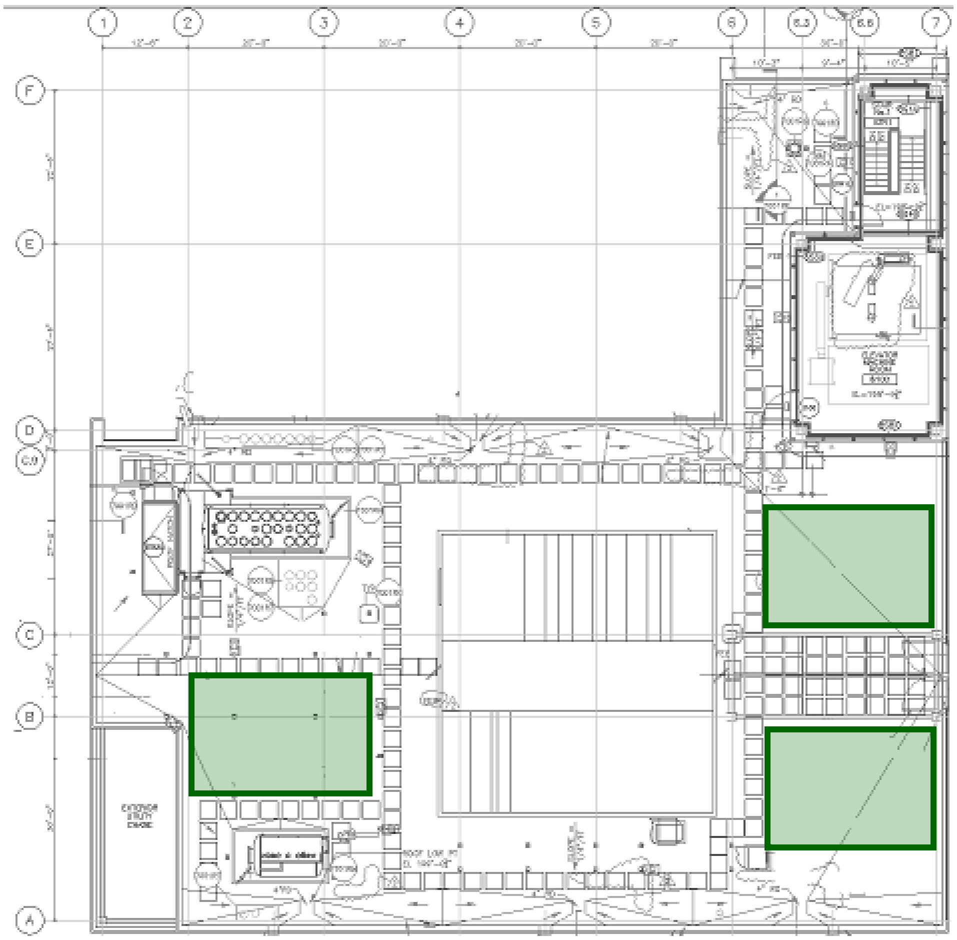


Figure 8.2: The roof plan. The three green boxes show the three areas of panels.

This array is calculated to save the owner about $7,723.10 in energy costs the first year with the maximum output occurring in the summer. The total installed cost would be about $80,171.85. The total installed cost per capacity is $4,599.71 per kW. Figure 8.3 shows the electrical schematic of the PV arrays.

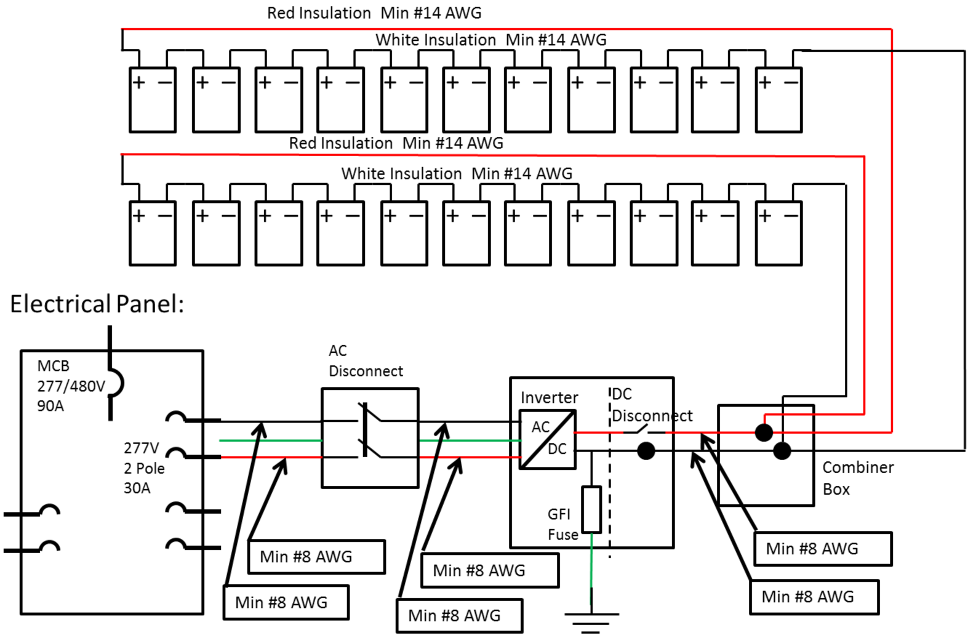


Figure 8.3: Electrical Schematic of one area of panels which consists of two strings on one inverter. Courtesy of Matthew Trethaway.

Although it is clear photovoltaic panels could save the owner money each month on energy bills, the payback period must be determined in order to decide its viability. The product array, location, system costs, inverter information, and incentives were all inputted into System Advisory Model to determine the benefits of this PV system. Using the current energy prices in Virginia, a peak price of .404 cents per kWh and an off-peak price of .272 cents per kWh, the total energy savings per year would be $7,723.10. The only incentive the CBD Chemical would qualify for would be property tax incentive. This would hold the value of the property fixed for property tax purposes. The total payback period would be 8.5 years. If this system were designed for residential or even some commercial this would be acceptable, however, the energy savings is not enough that it is not recommended for the Production Building. Figure 8.4 charts the estimated kWh the system would output each month. Please see appendix F for full PV calculations.

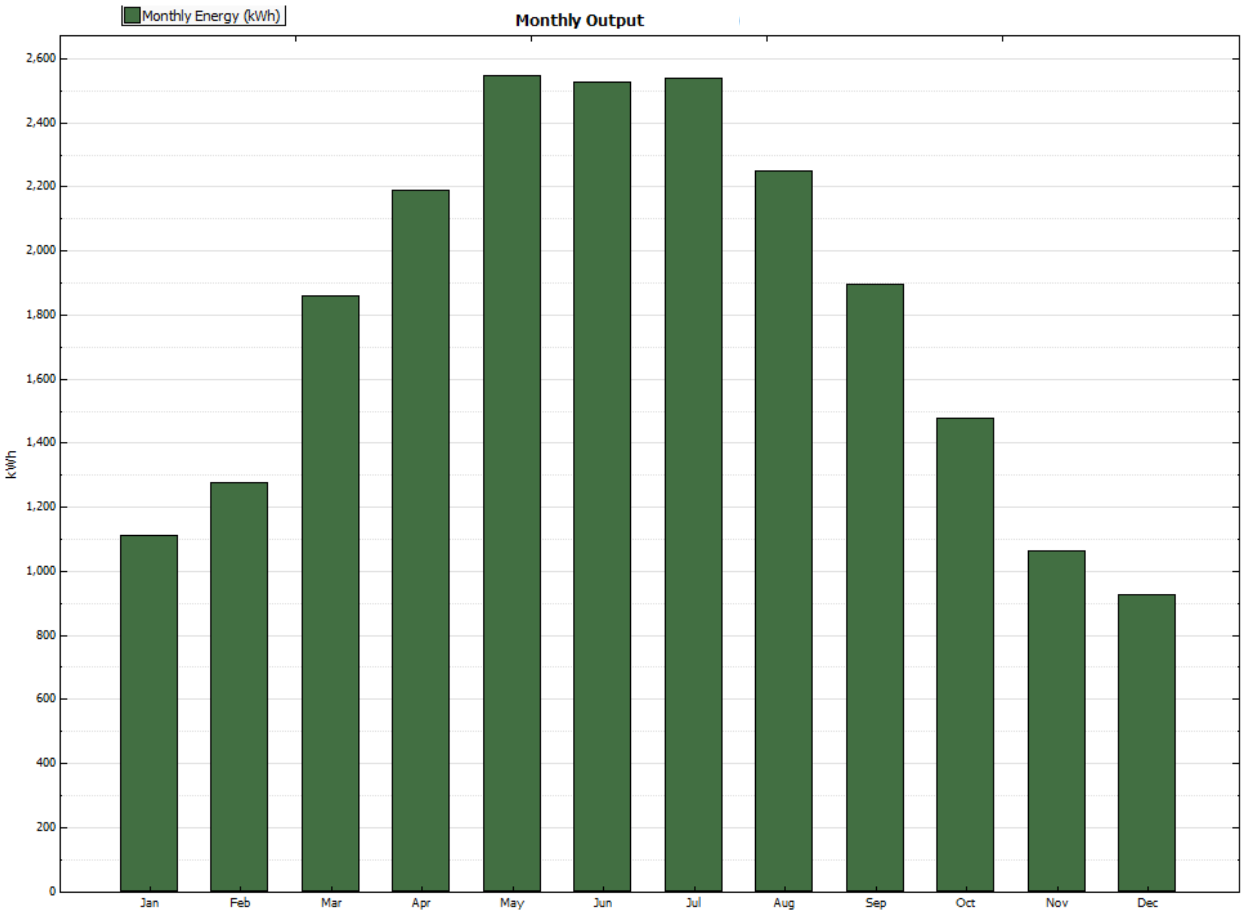


Figure 8.4: Monthly Output of PV array on roof in kWh.

8.0 Conclusions and Recommendations

The production building clearly would have benefitted from a further look at a concrete system. The concrete moment frame system is significantly cheaper than the moment frame steel building, even when the steel is designed compositely as was in the steel optimization. The only significant cost would be in the caisson design; however from a preliminary look at the foundation system, not many more caissons would need to be added. A further look into that would of course be necessary. In addition the concrete moment frame is much stiffer, so the drift does not control the design. The lateral beams turned out to be 30 inches deep, which is shorter than the 24 inch deep steel beams with the 7 ½ inches of concrete on top. This lower floor to floor height could potentially save the client even more money if the building were to be shortened slightly. After the redesign and the cost and schedule analysis, it is determined that a concrete system should be recommended for these design conditions.

The study of photovoltaic panels on the roof was not as promising. While a significant amount of savings could be accomplished each month, an eight and a half year payback period is slightly too long for many commercial owners to seriously consider it as a viable option. While it would be nice to offset some of the usage by the chemical plant, this green is not recommended.

References

AISC. 14th ed. American Institute of Steel Construction, AISC, 2011. Print.

ASCE7-05. *Minimum Design Loads for Buildings and Other Structures.* Published in 2006 by the American Society of Civil Engineers.

American Concrete Institute, *Building Code Requirements for Structural Concrete* (ACI 318-08)

American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10)

International Building Code (IBC), 2006

MacGregor, James and James Wright. *Reinforced Concrete: Mechanics and Design, fifth ed*. Prentice Hall. 2009

Naeim, Farzad. TheSeismic Design Handbook. New York: Springer Science + Buisness Media, LLC,2001. Print.

Waier, P. R., ed. (2011). *RS Means Building Construction Cost Data 2011*, 69th ed. RS Means Company, Kingston, MA.

6.0 Appendices

Appendix A: Framing Plans

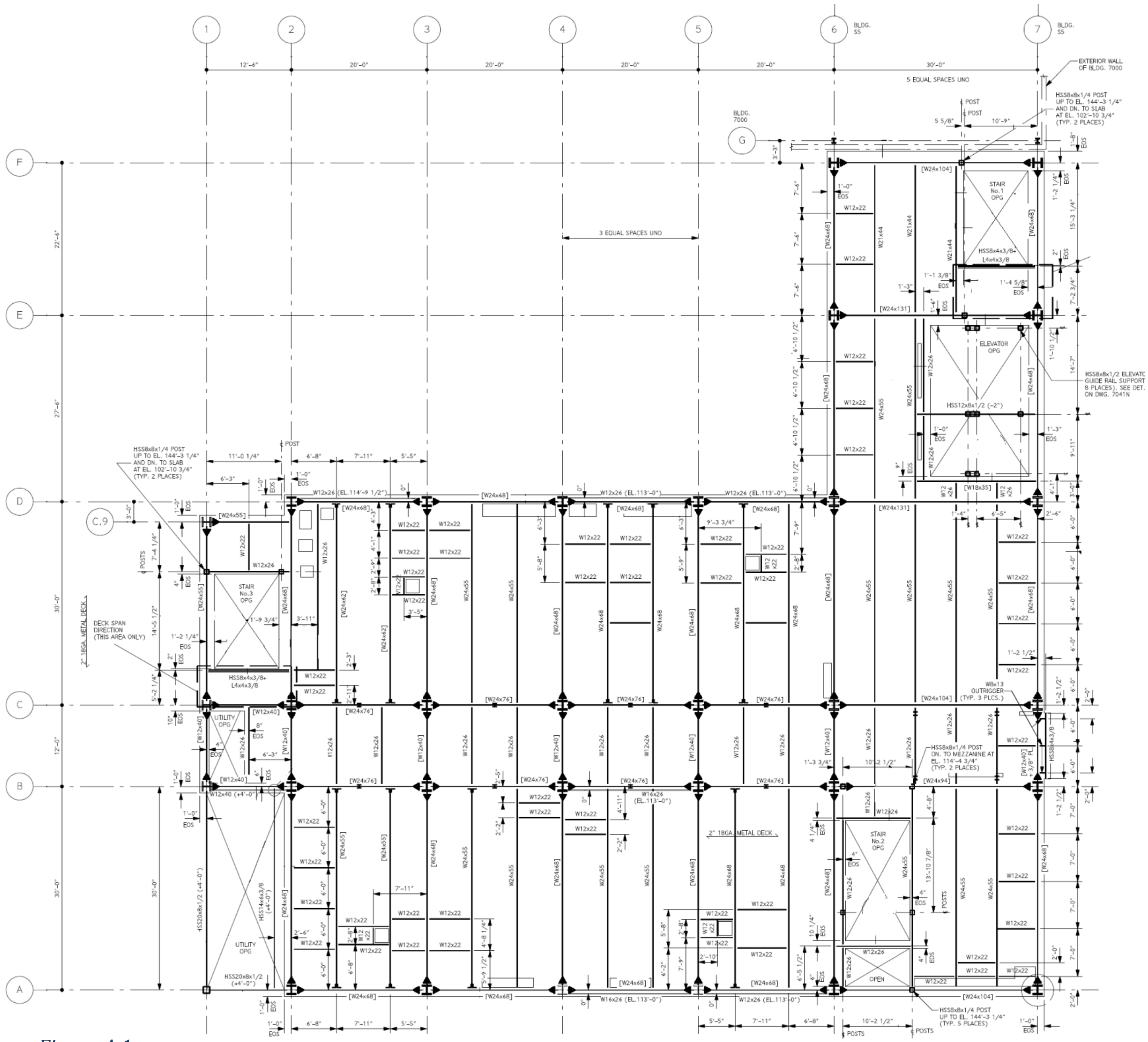


Figure A.1: Courtesy of Project Engineer. The second floor framing plan.

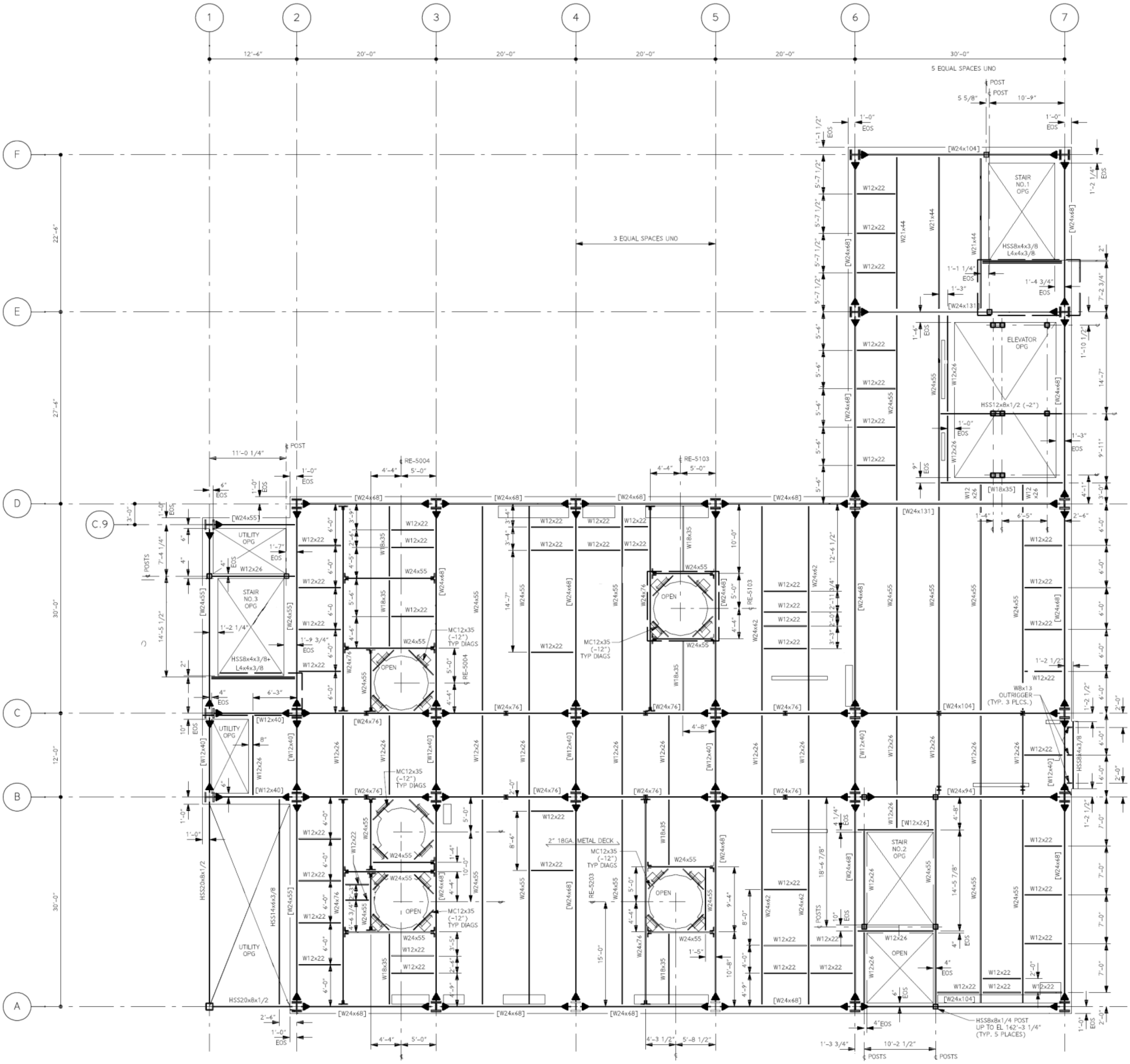


Figure A.2: Courtesy of Project Engineer. The third floor framing plan.

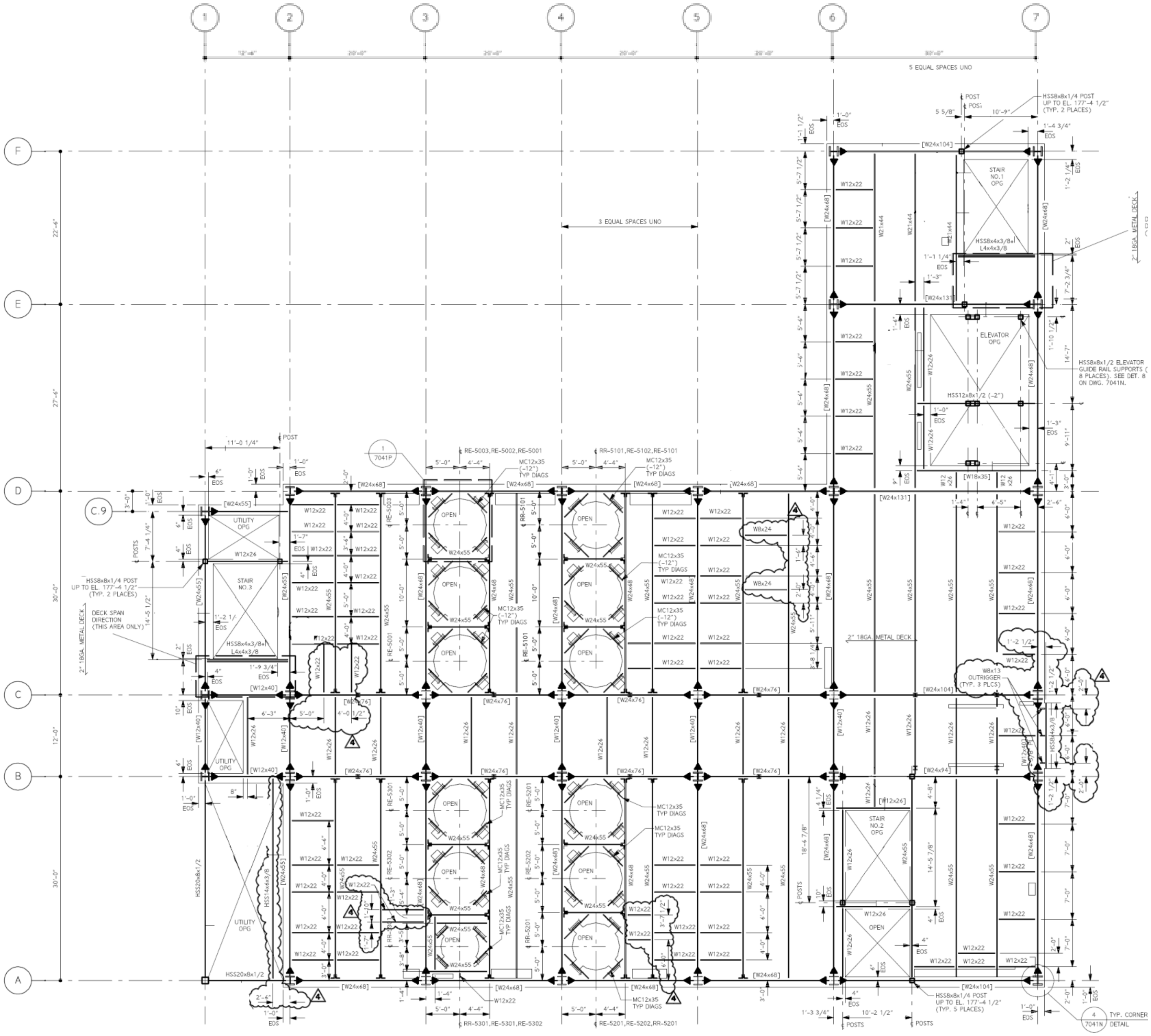


Figure A.3: Courtesy of Project Engineer. The fourth floor framing plan.

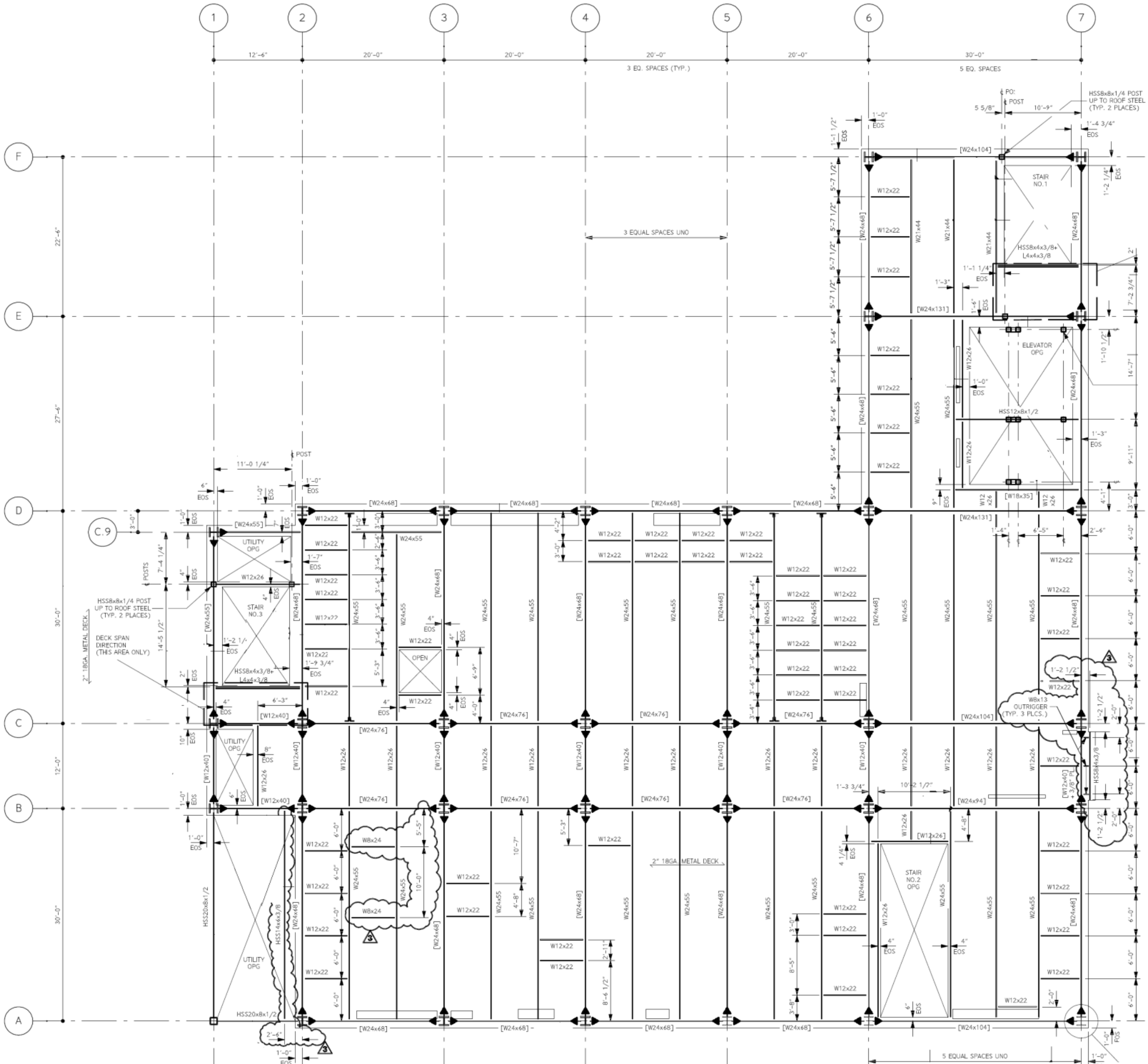


Figure A.4: Courtesy of Project Engineer. The fifth floor framing plan.

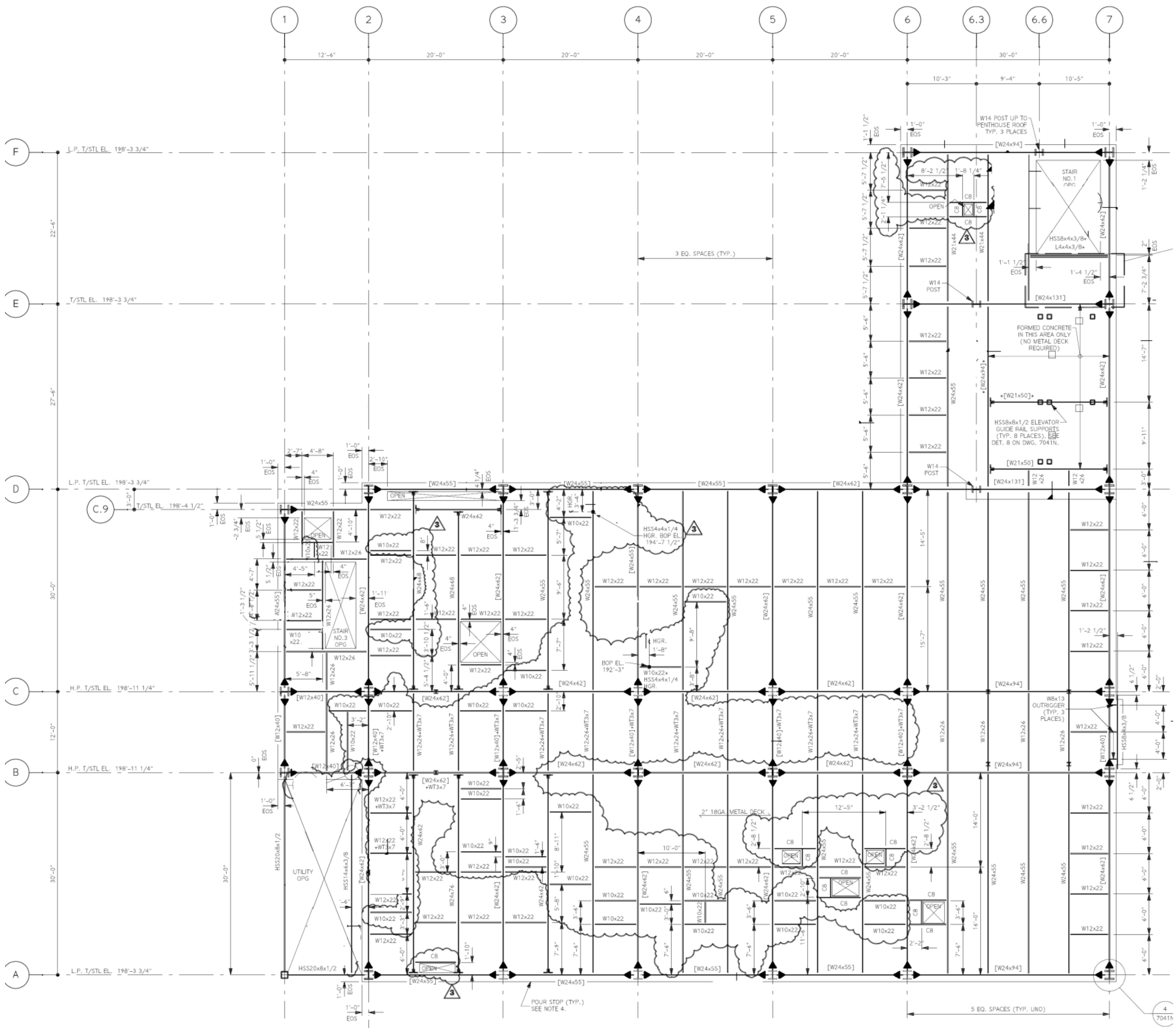


Figure A.5: Courtesy of Project Engineer. The roof framing plan.

Appendix B: Gravity Calculations

Slab Calculations

CONCRETE MIX		
Fy =	60	ksi
f'c =	4	ksi
LOADING		
LL =	200	psf
SDL =	80	psf
Service =	0.28	ksf

SLAB DESIGN						
span	7.5	ft	Mmax	42.7	k-in.	
	12	" Strip	Mn =	81.78529	OK	
wu =	0.506	klf	b =	12	in.	
Mu =	3.6	k-ft	h =	6	in.	
	42.7	k-in	d =	4.625	in.	
Vu =	1.9	k	a =	0.455882	in.	
			As =	0.31	in2/ft width	#5 at 12in.
			ρ =	0.004306	OK	

Figure B.1: Slab design.

Continuous Beam Calculations

CONCRETE MIX			**LOADING**			**Beam Width**		b_w =	12	in.
Fy =	60	ksi	LL =	200	psf	12	in. square			
f'c =	4	ksi	SDL =	80	psf	**Slab thickness**				
			Service =	0.28	ksf	6	in.			

Controlling Load Case: 1.2D+1.6L

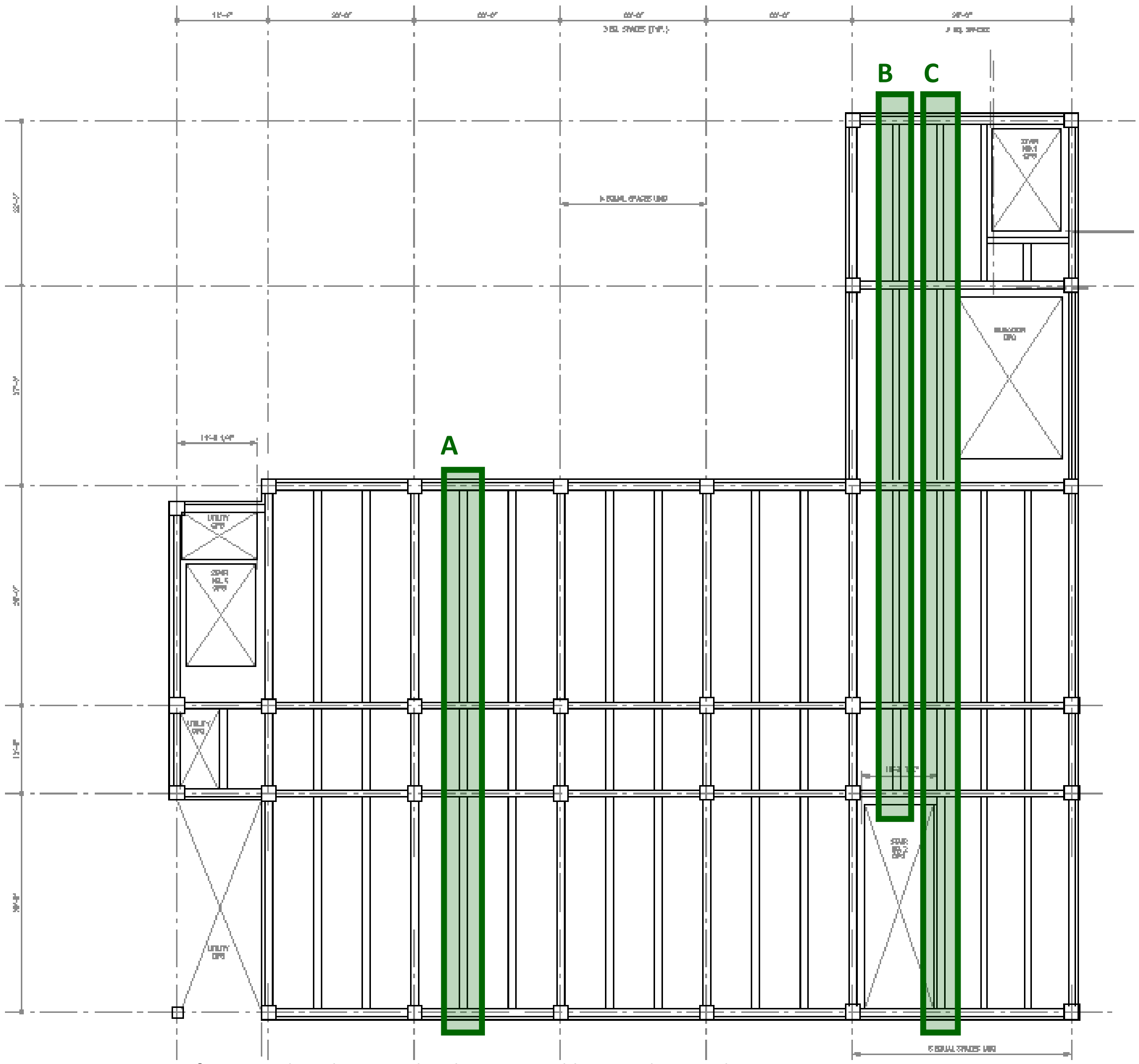


Figure B.2: Concrete framing plan showing the three typical beams designed.

SECTION A									
Trib Width=	6.7 ft.	b = 12		h = 22			d = 19.5		
	Beam 1			Beam 2			Beam 3		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support
Span	30	-	-	12	-	-	30	-	-
l_n (ft) =	29	29	20	20	11	20	20	29	29
w_u (klf) =	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
$w_u l_n^2$ =	3039	3039	1445	1445	437	1445	1445	3039	3039
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.1000	0.0714286	-0.0417
$M_u = C_m W_u L_n^2$ =	-126.6	217.1	-144.5	-131.4	27.3	-131.4	-144.5	217.1	-126.6
b_{eff} (in) =	41	12	60	60	12	60	60	12	83.25
$A_{s(req'd)}$ (in.) =	1.60325701	2.74844	1.83011502	1.66374092	0.34601	1.66374092	1.83011502	2.7484406	1.60325701
$A_{s(prov'd)}$ (bars) =	**(4) #6**	**(4) #8**	**(5) #6**	**(5) #6**	**(2) #6**	**(5) #6**	**(5) #6**	**(4) #8**	**(4) #6**
a =	2.58823529	4.64706	3.23529412	3.23529412	1.29412	3.23529412	3.23529412	4.6470588	2.58823529
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	1.76	3.16	2.2	2.2	0.88	2.2	2.2	3.16	1.76
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK
rho =	0.00752137	0.0135	0.00940171	0.00940171	0.00376	0.00940171	0.00940171	0.0135043	0.00752137
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.87812404	0.88867	0.81640971	0.74219065	0.36602	0.74219065	0.81640971	0.888674	0.87812404
	0.888673973			0.742190646			0.888673973		

Figure B.3: Flexural design for typical beam A.

SECTION B												
Trib Width=	6.0 ft.	b = 12		h = 22		d = 19.5						
	Beam 1			Beam 2			Beam 3			Beam 4		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support
Span	12	-	-	30	-	-	27.5	-	-	22.5	-	-
l_n (ft) =	11	11	20	20	29	27.75	27.75	26.5	26.5	24	21.5	21.5
w_u (klf) =	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
$w_u l_n^2$ =	396	396	1310	1310	2755	2523	2523	2301	2301	1887	1514	1514
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.1000	0.07143	-0.0417
$M_u = C_m W_u L_n^2$ =	-16.5	28.3	-131.0	-119.1	172.2	-229.3	-229.3	143.8	-209.1	-188.7	108.2	-63.1
b_{eff} (in) =	41	12	60	60	12	60	60	12	83.25	83.25	12	72
$A_{s(req'd)}$ (in.) =	0.2091358	0.35852	1.65925926	1.50841751	2.18037	2.90393939	2.90393939	1.8206481	2.64821549	2.38933333	1.36963	0.79895062
$A_{s(prov'd)}$ (bars) =	**(2) #6**	**(2) #6**	**(4) #6**	**(4) #6**	**(3) #8**	**(4) #8**	**(4) #8**	**(5) #6**	**(4) #8**	**(4) #8**	**(5) #6**	**(2) #6**
a =	1.29411765	1.29412	2.58823529	2.58823529	3.48529	4.64705882	4.64705882	3.2352941	4.64705882	4.64705882	3.23529	1.29411765
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	0.88	0.88	1.76	1.76	2.37	3.16	3.16	2.2	3.16	3.16	2.2	0.88
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
ρ =	0.00376068	0.00376	0.00752137	0.00752137	0.01013	0.01350427	0.01350427	0.0094017	0.01350427	0.01350427	0.0094	0.00376068
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.22122985	0.37925	0.90879718	0.82617925	0.90924	0.93895257	0.93895257	0.8121866	0.85626744	0.77256112	0.61099	0.84515287
	0.90879718			0.938952573			0.938952573			0.845152874		

Figure B.4: Flexural design for typical beam B.

SECTION C															
Trib Width=	6 ft.		b = 12		h = 22			d = 19.5							
	Beam 1			Beam 2			Beam 3			Beam 4			Beam 5		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	ext. support
Span	30	-	-	12	-	-	30	-	-	27.5	-	-	22.5	-	-
l_n (ft) =	29	29	20	20	11	20	20	29	27.75	27.75	26.5	24	24	21.5	21.5
w_u (klf) =	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
$w_u l_n^2$ =	2755	2755	1310	1310	396	1310	1310	2755	2523	2523	2301	1887	1887	1514	1514
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.100	0.0714	-0.0417
$M_u = C_m W_u L_n^2$ =	-114.8	196.8	-131.0	-119.1	24.8	-119.1	-119.1	172.2	-229.3	-229.3	143.8	-171.5	-188.7	108.2	-63.1
b_{eff} (in) =	41	12	60	60	12	60	60	12	83.25	83.25	12	72	72	12	33.5
$A_{s(req'd)}$ (in.) =	1.40121379	2.54604	1.59948304	1.45407549	0.29202	1.45407549	1.45407549	2.1549101	2.96709013	2.96709013	1.79939	2.17102969	2.38813266	1.29723	0.75672037
$A_{s(prov'd)}$ (bars) =	**(4) #6**	**(4) #8**	**(4) #6**	**(4) #6**	**(2) #6**	**(4) #6**	**(4) #6**	**(3) #8**	**(4) #8**	**(4) #8**	**(3) #8**	**(6) #6**	**(6) #6**	**(3) #6**	**(3) #6**
a =	2.58823529	4.64706	2.58823529	2.58823529	1.29412	2.58823529	2.58823529	3.4852941	4.64705882	4.64705882	3.48529	3.88235294	3.88235294	1.94118	1.94117647
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	1.76	3.16	1.76	1.76	0.88	1.76	1.76	2.37	3.16	3.16	2.37	2.64	2.64	1.32	1.32
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
rho =	0.00752137	0.0135	0.00752137	0.00752137	0.00376	0.00752137	0.00752137	0.0101282	0.01350427	0.01350427	0.01013	0.01128205	0.01128205	0.00564	0.00564103
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.7961442	0.80571	0.90879718	0.82617925	0.33184	0.82617925	0.82617925	0.9092448	0.93895257	0.93895257	0.75924	0.82235973	0.90459571	0.98275	0.57327301
	0.90879718			0.826179255			0.938952573			0.938952573			0.982753728		

Figure B.5: Flexural design for typical beam C.

Appendix C: Earthquake Loads

Floor	Dead Load (psf)	Area (SF)	Exterior Wall (k)	Equipment PL (k)	Total Weight (k)
1	248.9	10320	59	449	3076
2	269.3	10320	103	143	3025
3	262.5	10320	88	347	3144
4	262.5	10320	88	337	3134
5	262.5	10320	88	79	2876
ROOF	242.1	10320	45	285	2828
Penthouse	20	750	3.6	0	19
				Σ =	18102

Table C.1: The excel calculations for floor weight.

Floor	Total Weight (k)	z (ft)	$w_x h_x^k$	C_{vx}	F_x (k)
1	3076	0	0	0	0
2	3025	24	182468	0.06	27.3
3	3144	42	390370	0.13	58.4
4	3134	60	616477	0.20	92.3
5	2876	78	793586	0.26	118.8
ROOF	2828	96	1020065	0.34	152.7
Penthouse	18.6	106	7624	0.00	1.1
		Σ =	3010590	1.0	450.8
			Overturning Moment =		32700

Table C.2: The excel calculations for story shear and overturning moment.

SEISMIC Zip = 23805 Christina DiPaolo 1

$S_s = .155$ (USGS) $F_a = 2.5$
$S_1 = .059$ (USGS $F_v = 3.5$

Soil site class E

$S_{MS} = F_a S_S = (2.5)(.155) = .3875$
$S_{M1} = F_v S_1 = (3.5)(.059) = .2065$

$S_{DS} = (2/3) S_{MS} = 2/3(.388) = .260 \Rightarrow B$
$S_{D1} = (2/3) S_{M1} = 2/3(.207) = .138 \Rightarrow C$

∴ Design Category C
↳ must use intermediate moment frame

$C_S = \dfrac{.26}{\left(\frac{5}{1.25}\right)} = .04$

$T_a = C_t h_n^x = (.016)(96)^{.9} = .97$
$T_L = 8$
$T_a < T_L$

$C_S = \dfrac{.138}{1.08\left(5/1.25\right)} = .03$ controls

$C_S = .044(.26)(1.25) \geq .01$
$= .0143 \geq .01 < .03$ ✓ OK

$V = C_S W$

$W = 18102$
$V = .03(15026) = 450.8$

Appendix D: Lateral Design

Lateral Beams

The following tables are the calculations for the reinforcing of the girders. The shading shows which reinforcing runs into each other. The dark grey blocks are the reinforcing at the top of the section while the light grey is for the bottom of the section. The framing plan below shows along which column lines the beams and girders run.

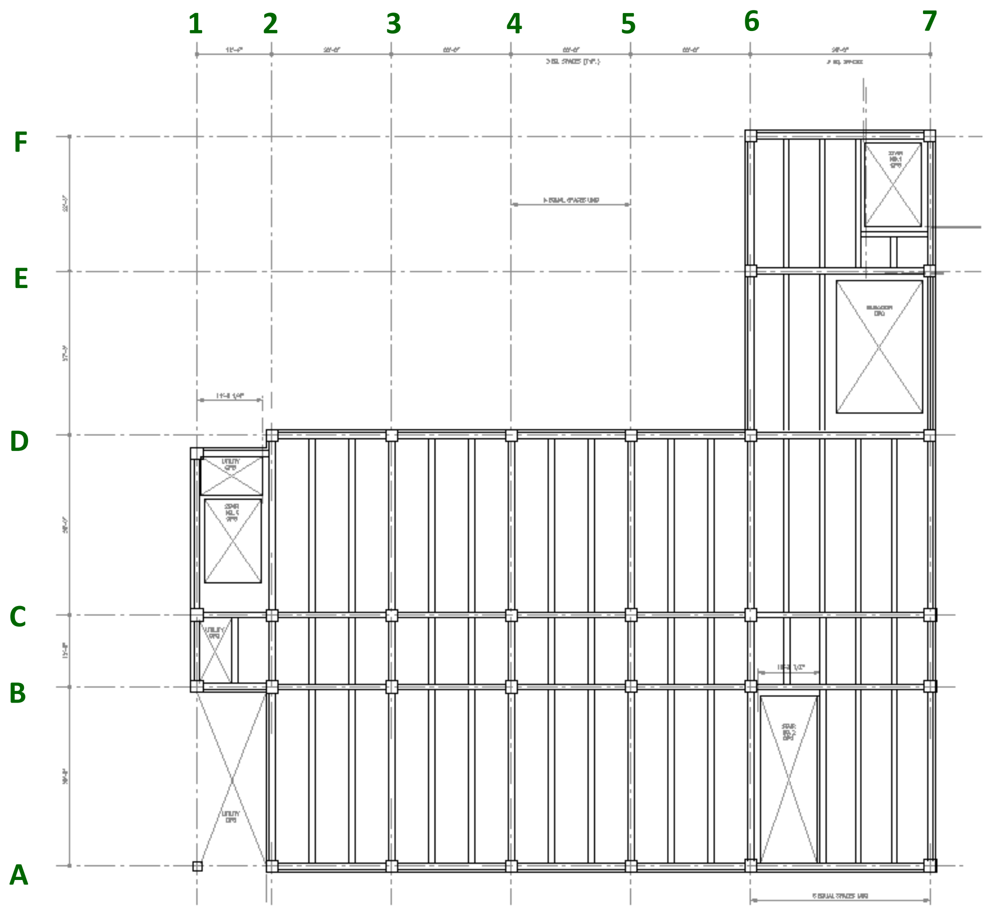


Figure D.1: Concrete framing plan.

BEAM 1						
Trib Width= 6.3	b = 12		h = 30		d = 27.5	
	Beam 2			Beam 3		
	int. support	midspan	int. support	int. support	midspan	int. support
Span	12	-	-	30	-	-
l_n (ft) =	9.5	9.5	18.5	18.5	27.5	27.5
w_u (klf) =	2.8	2.8	2.8	2.8	2.8	2.8
$w_u l_n^2$ =	250	250	949	949	2097	2097
C_m =	-0.0417	0.0625	-0.0909	-0.1000	0.071429	-0.0417
$M_{uG} = C_m W_u L_n^2$ =	-10.4	15.6	-86.3	-94.9	149.8	-87.4
M_{uE} =	-206.3	0.0	-201.0	-112.1	0.0	-116.6
M_{utot} =	-216.7	15.6	-287.3	-207.0	149.8	-204.0
b_{eff} (in) =	28.5	12	55.5	55.5	12	30
$A_{s(req'd)}$ (in.) =	1.837804134	0.129418	2.43594636	1.75523622	1.23938	1.729574617
a =	2.588235294	1.294118	2.58823529	2.588235294	1.294118	2.588235294
t-beam?	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	1.76 (4) #6	1.32 (3) #6	1.76 (4) #6	1.76 (4) #6	1.32 (3) #6	1.76 (4) #6
A_s' (in.) =	0.88 (2) #6	0.88 (2) #6	0.88 (2) #6	0.88 (2) #6	0.88 (2) #6	0.88 (2) #6
Φ =	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	306.5505882	205.3376	306.550588	306.5505882	205.3376	306.5505882
OK?	OK	OK	OK	OK	OK	OK
ρ =	0.005333333	0.004	0.00533333	0.005333333	0.004	0.005333333
OK?	OK	OK	OK	OK	OK	OK
capacity ratio:	0.707	0.076	0.937	0.675	0.729	0.665

Figure D.2: Flexural design for Beam 1.

BEAMS 2/3/4/5									
Trib Width=	6.7 ft.			b = 12		h = 30		d = 27.5	
	Beam 1			Beam 2			Beam 3		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support
Span	30	-	-	12	-	-	30	-	-
l_n (ft) =	27.5	27.5	18.5	18.5	9.5	18.5	18.5	27.5	27.5
w_u (klf) =	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9
$w_u l_n^2$ =	2218	2218	1004	1004	265	1004	1004	2218	2218
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.1000	0.0714286	-0.0417
$M_{uG} = C_m W_u L_n^2$ =	-92.4	158.5	-100.4	-91.3	16.5	-91.3	-100.4	158.5	-92.4
M_{uE} =	-150.6	0.0	-144.5	-282.3	0.0	-282.3	-144.5	0.0	-150.6
M_{utot} =	-243.0	158.5	-244.9	-373.6	16.5	-373.6	-244.9	158.5	-243.0
b_{eff} (in) =	39.5	12	55.5	55.5	12	55.5	55.5	12	40
$A_{s(req'd)}$ (in.) =	2.060865167	1.311277	2.107409095	3.2146967	0.136925	3.2146967	2.10740909	1.3112769	2.06086517
a =	2.588235294	1.294118	3.352941176	3.35294118	1.294118	3.35294118	3.35294118	1.2941176	2.58823529
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	2.2 (5) #6	1.32 (3) #6	2.46 (2)#6, (2)#8	2.46 (2)#6, (2)#8	1.32 (3) #6	2.46 (2)#6, (2)#8	2.46 (2)#6, (2)#8	1.32 (3) #6	2.2 (5) #6
A_s' (in.) =	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	356.1	205.3	413.4	413.4	205.3	413.4	413.4	205.3	356.1
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK
ρ =	0.0067	0.0040	0.0075	0.0075	0.0040	0.0075	0.0075	0.0040	0.0067
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.683	0.772	0.592	0.904	0.081	0.904	0.592	0.772	0.683

Figure D.3: Flexural design for Beams 2/3/4/5.

BEAM 6															
Trib Width=	6 ft.		b = 12			h = 30		d = 27.5							
	Beam 1			Beam 2			Beam 3			Beam 4			Beam 5		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	ext. support
Span	30 -	-		12 -	-		30 -	-		27.5 -	-		22.5 -	-	
l_n (ft) =	27.5	27.5	18.5	18.5	9.5	18.5	18.5	27.5	26.25	26.25	25	22.5	22.5	20	20
w_{uG} (klf) = 1.2D+L	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
$w_{uG}l_n^2$ =	2024	2024	916	916	242	916	916	2024	1844	1844	1673	1355	1355	1070	1070
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.100	0.0714	-0.0417
$M_{uG} = C_mW_uL_n^2$ =	-84.3	144.6	-91.6	-83.3	15.1	-83.3	-83.3	126.5	-167.6	-167.6	104.5	-123.2	-135.5	76.5	-44.6
M_{uE} =	-162.7	0.0	-156.0	-305.0	0.0	-305.0	-153.8	0.0	-157.8	-166.1	0.0	-166.4	-203.0	0.0	-208.0
M_{utot} =	-247.0	144.6	-247.6	-388.3	15.1	-388.3	-237.1	126.5	-325.4	-333.7	104.5	-289.6	-338.5	76.5	-252.6
b_{eff} (in) =	39.5	12	55.5	55.5	12	55.5	55.5	12	78.75	78.75	12	67.5	67.5	12	32
$A_{s(req'd)}$ (in.) =	2.147741657	1.19624211	2.152647334	3.375743023	0.124913381	3.517410896	2.147626723	1.046711847	2.800461621	2.871886576	0.865051114	2.491756978	2.912696786	0.6327231	2.17372817
a =	3.882352941	1.29411765	3.882352941	3.882352941	1.294117647	5.941176471	5.941176471	1.294117647	3.352941176	3.352941176	1.294117647	3.352941176	3.352941176	1.294117647	3.352941176
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	2.64 (6) #6	1.32 (3) #6	2.64 (6) #6	2.64 (6) #6	1.32 (3) #6	3.34 (2)#8, (4)#6	3.34 (2)#8, (4)#6	1.32 (3) #6	2.46 (4)#8, (2)#6	2.46 (4)#8, (2)#6	1.32 (3) #6	2.46 (4)#8, (2)#6	2.46 (4)#8, (2)#6	1.32 (3) #6	2.46 (4)#8, (2)#6
A_s' (in.) =	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	452.14	205.34	452.14	452.14	205.34	594.44	594.44	205.34	413.45	413.45	205.34	413.45	413.45	205.34	413.45
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
ρ =	0.008	0.004	0.008	0.008	0.004	0.010121212	0.010121212	0.004	0.007454545	0.007454545	0.004	0.007454545	0.007454545	0.004	0.007454545
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.546	0.704	0.548	0.859	0.074	0.653	0.399	0.616	0.787	0.807	0.509	0.700	0.819	0.372	0.611

Figure D.4: Flexural design for Beam 6.

BEAM 7															
Trib Width=	6 ft.		b = 12			h = 30		d = 27.5							
	Beam 1			Beam 2			Beam 3			Beam 4			Beam 5		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	ext. support
Span	30 -	-		12 -	-		30 -	-		27.5 -	-		22.5 -	-	
l_n (ft) =	27.5	27.5	18.5	18.5	9.5	18.5	18.5	27.5	26.25	26.25	25	22.5	22.5	20	20
w_{uG} (klf) = 1.2D+L	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
$w_{uG}l_n^2$ =	1148	1148	520	520	137	520	520	1148	1046	1046	949	768	768	607	607
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.100	0.0714	-0.0417
$M_{uG} = C_mW_uL_n^2$ =	-47.8	82.0	-52.0	-47.2	8.6	-47.2	-47.2	71.7	-95.1	-95.1	59.3	-69.9	-76.8	43.4	-25.3
M_{uE} =	-18.6	0.0	-173.2	-338.6	0.0	-338.7	-170.7	0.0	-174.5	-189.4	0.0	-188.2	-226.1	0.0	-231.0
M_{utot} =	-66.4	82.0	-225.2	-385.8	8.6	-385.9	-217.9	71.7	-269.6	-284.5	59.3	-258.1	-302.9	43.4	-256.3
b_{eff} (in) =	39.5	12	55.5	55.5	12	55.5	55.5	12	78.75	78.75	12	67.5	67.5	12	32
$A_{s(req'd)}$ (in.) =	0.55647096	0.67050737	1.96212244	3.362357298	0.070015377	3.363228758	1.899176471	0.586693948	2.25821169	2.383020756	0.484871032	2.204415552	2.587841844	0.354648526	2.136226022
a =	1.9412	0.6471	4.0000	4.0000	0.6471	4.0000	4.0000	0.6471	1.9412	1.9412	0.6471	2.9706	2.9706	0.6471	1.6765
t-beam?	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
$A_{s(prov'd)}$ (in.) =	2.64 (2) #6, (2) #8	1.32 (3) #6	4.04 (2) #6, (4) #8	4.04 (2) #6, (4) #8	1.32 (3) #6	4.04 (2) #6, (4) #8	4.04 (2) #6, (4) #8	1.32 (3) #6	2.64 (2) #6, (2) #8	2.64 (2) #6, (2) #8	1.32 (3) #6	3.34 (4) #6, (2) #8	3.34 (4) #6, (2) #8	1.32 (3) #6	2.46 (2) #6, (2) #8
A_s' (in.) =	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6	1.32 (3) #6	0.88 (2) #6	1.32 (3) #6
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	306.08	152.81	460.62	460.62	152.81	460.62	460.62	152.81	306.08	306.08	152.81	384.97	384.97	152.81	285.27
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
ρ =	0.0120	0.0067	0.0162	0.0162	0.0067	0.0162	0.0162	0.0067	0.0120	0.0120	0.0067	0.0141	0.0141	0.0067	0.0115
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.217	0.537	0.489	0.838	0.056	0.838	0.473	0.470	0.881	0.929	0.388	0.670	0.787	0.284	0.898

Figure D.5: Flexural design for Beam 7.

Lateral Girders

The following tables are the calculations for the reinforcing of the girders. The shading shows which reinforcing runs into each other. The dark grey blocks are the reinforcing at the top of the section while the light grey is for the bottom of the section.

GIRDER A															
Trib Width=	0 ft.		b = 12		h = 30		d = 27.5								
	Beam 1			Beam 2			Beam 3			Beam 4			Beam 5		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	ext. support
Span	20	-	-	20	-	-	20	-	-	20	-	-	30	-	-
l_n (ft) =	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	22.5	22.5	27.5	27.5
w_{uSW} (klf) = 1.2D+L	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$w_{uG}l_n^2$ =	92	92	92	92	92	92	92	92	92	92	92	152	152	227	227
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.100	0.0714	-0.0417
$M_{uSW} = C_m W_u L_n^2$ =	-3.8	6.6	-9.2	-8.4	5.7	-8.4	-8.4	5.7	-8.4	-8.4	5.7	-13.8	-15.2	16.2	-9.5
M_{Dpt} =	-23.6	55	-70.6	-69	41.9	-61.6	-67.1	45.2	-57.6	-53.9	38.3	-97.1	-173	127.2	-99.1
M_{Lpt} =	-25.1	58.6	-75.2	-73	44.6	-65.6	-71.4	48	-61.3	-57.5	40.8	-103	-183.3	134.6	-104.9
M_{uLat} =	-169.2	0.0	-164.4	-162.0	0.0	-162.0	-162.0	0.0	-162.0	-162.5	0.0	-164.0	-118.3	0.0	-121.6
M_{utot} = 1.D+E+L =	-227.2	132.5	-335.3	-327.8	101.8	-311.5	-323.9	109.1	-302.4	-294.7	93.7	-400.1	-527.4	306.7	-356.8
$A_{s(req'd)}$ (in.) =	1.903244026	1.083249158	2.843678763	2.779890941	0.832179233	2.641838815	2.746989083	0.892362013	2.533395597	2.468561894	0.765781926	3.489625957	4.600275774	2.639164407	3.010104225
a =	1.941176471	0.647058824	2.588235294	2.588235294	0.647058824	2.588235294	2.588235294	0.647058824	1.941176471	1.941176471	0.647058824	4.044117647	4.044117647	3.352941176	2.323529412
$A_{s(prov'd)}$ (in.) =	2.64	1.32	3.08	3.08	1.32	3.08	3.08	1.32	2.64	2.64	1.32	4.95	4.95	3.16	3.16
	(6) #6	(3) # 6	(7) #6	(7) #6	(3) # 6	(7) #6	(7) #6	(3) # 6	(6) #6	(6) #6	(3) # 6	(4)#6, (4)#8	(4)#6, (4)#8	(4) #8	(4) #8
A_s' (in.) =	1.32	0.88	1.32	1.32	0.88	1.32	1.32	0.88	1.32	1.32	0.88	2.2	2.2	0.88	1.58
	(3) # 6	(2) #6	(3) # 6	(3) # 6	(2) #6	(3) # 6	(3) # 6	(2) #6	(3) # 6	(3) # 6	(2) #6	(5)#6	(5)#6	(2) #6	(2) #8
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	306.08	152.81	356.05	356.05	152.81	356.05	356.05	152.81	306.08	306.08	152.81	562.79	562.79	363.95	365.01
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
rho =	0.008	0.004	0.009333333	0.009333333	0.004	0.009333333	0.009333333	0.004	0.008	0.008	0.004	0.015	0.015	0.009575758	0.009575758
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.742	0.867	0.942	0.921	0.666	0.875	0.910	0.714	0.988	0.963	0.613	0.711	0.937	0.843	0.977

Figure D.6: Flexural design for girder A.

GIRDER B/C																		
Trib Width=	0 ft.		b = 12		h = 30		d = 27.5											
	Beam 1			Beam 2			Beam 3			Beam 4			Beam 5			Beam 6		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	ext. support	int. support	midspan	ext. support
Span	12.5	-	-	20	-	-	20	-	-	20	-	-	20	-	-	30	-	-
l_n (ft) =	10	10	13.75	13.75	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	22.5	22.5	27.5	27.5
w_{uSW} (klf) = 1.2D+L	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$w_{uG}l_n^2$ =	30	30	57	57	92	92	92	92	92	92	92	92	92	92	152	152	227	227
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.100	0.0714	-0.0417
$M_{uSW} = C_m W_u L_n^2$ =	-1.3	2.1	-5.7	-5.2	5.7	-8.4	-8.4	5.7	-8.4	-8.4	5.7	-8.4	-8.4	5.7	-13.8	-15.2	16.2	-9.5
M_{Dpt} =	0	0	-35.6	-72	69.7	-93.7	-95.8	59.4	-86.1	-95.4	63.7	-79.9	-73.9	53.5	-134.8	-242.2	177.6	-137.2
M_{Lpt} =	0	0	-36.9	-76.4	73.9	-99.4	-101.6	63	-91.3	-101.6	67.6	-84.7	-81.5	56.7	-143.1	-257.5	188.8	-145.9
M_{uLat} =	-169.2	0.0	-164.4	-162.0	0.0	-162.0	-162.0	0.0	-162.0	-162.5	0.0	-164.0	-118.3	0.0	-121.6	-118.3	0.0	-121.6
M_{utot} = 1.2D+E+L =	-170.7	2.6	-250.8	-331.0	164.4	-383.9	-388.6	141.2	-366.6	-388.6	150.9	-354.6	-298.5	127.8	-443.0	-684.7	421.4	-443.5
$A_{s(req'd)}$ (in.) =	1.429859571	0.021026592	2.126971568	2.806727772	1.36074997	3.238745016	3.278568826	1.168261227	3.093455606	3.278737571	1.249030364	2.95556603	2.487980077	1.099691534	3.913854476	6.048552405	3.535433049	3.953706326
a =	1.941176471	0.647058824	2.588235294	2.588235294	1.294117647	2.323529412	2.323529412	1.294117647	2.323529412	2.323529412	1.294117647	1.676470588	1.676470588	3.352941176	4.691176471	4.691176471	2.029411765	5.147058824
$A_{s(prov'd)}$ (in.) =	2.64	1.32	3.08	3.08	1.76	3.34	3.34	1.76	3.34	3.34	1.76	3.34	3.34	3.16	6.35	6.35	3.95	5.08
	(6) #6	(3) # 6	(7) #6	(7) #6	(4) # 6	(4)#6, (2)#8	(4)#6, (2)#8	(4) # 6	(4)#6, (2)#8	(4)#6, (2)#8	(4) # 6	(4)#6, (2)#8	(4)#6, (2)#8	(4) #8	(5) #10	(5)#10	(5) #8	(4) #10
A_s' (in.) =	1.32	0.88	1.32	1.32	0.88	1.76	1.76	0.88	1.76	1.76	0.88	2.2	2.2	0.88	3.16	3.16	2.57	1.58
	(3) # 6	(2) #6	(3) # 6	(3) # 6	(2) #6	(4) # 6	(4) # 6	(2) #6	(4) # 6	(4) # 6	(2) #6	(5)#6	(5)#6	(2) #6	(4) #8	(4) #8	(2) #10	(2) #8
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	306.08	152.81	356.05	356.05	205.34	385.26	385.26	205.34	385.26	385.26	205.34	384.27	384.27	363.95	716.59	716.59	453.60	570.34
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
rho =	0.008	0.004	0.009333333	0.009333333	0.005333333	0.010121212	0.010121212	0.005333333	0.010121212	0.010121212	0.005333333	0.010121212	0.010121212	0.009575758	0.019242424	0.019242424	0.011969697	0.015393939
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.542	0.016	0.691	0.911	0.773	0.970	0.982	0.664	0.926	0.982	0.710	0.885	0.745	0.348	0.616	0.953	0.895	0.778

Figure D.7: Flexural design for girders B and C.

GIRDER D															
Trib Width=	0 ft.		b = 12		h = 30		d = 27.5								
	Beam 1			Beam 2			Beam 3			Beam 4			Beam 5		
	ext. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	int. support	int. support	midspan	ext. support
Span	20 -		-	20 -		-	20 -		-	20 -		-	30 -		-
l_n (ft) =	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	22.5	22.5	27.5	27.5
w_{uSW} (klf) = 1.2D+L	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$w_{uG}l_n^2$ =	92	92	92	92	92	92	92	92	92	92	92	152	152	227	227
C_m =	-0.0417	0.0714	-0.100	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.0909	0.0625	-0.0909	-0.100	0.0714	-0.0417
$M_{uSW} = C_m W_u L_n^2$ =	-3.8	6.6	-9.2	-8.4	5.7	-8.4	-8.4	5.7	-8.4	-8.4	5.7	-13.8	-15.2	16.2	-9.5
M_{Dpt} =	-25.1	51.6	-65.2	-67.5	40.5	-57.7	-68.4	47.2	-47.4	-38.6	32.5	-134.6	-291.6	224.3	-176.3
M_{Lpt} =	-27.6	57.5	-72.7	-74.8	45	-64.2	-75.5	52	-53.6	-44.4	36.6	-144.9	-310	237.8	-189.8
M_{uLat} =	-220.2	0.0	-213.9	-209.6	0.0	-209.7	-209.7	0.0	-209.7	-210.3	0.0	-212.4	-153.2	0.0	-157.6
M_{utot} = 1.2D+E+L =	-282.5	127.3	-375.9	-375.4	100.5	-353.2	-377.3	115.5	-330.2	-311.0	82.5	-535.4	-831.3	526.4	-570.3
$A_{s(req'd)}$ (in.) =	2.413282457	1.040892	3.21070536	3.206927386	0.821713	2.979721503	3.18339667	0.956076731	2.78600229	2.62434462	0.669814616	4.978652494	7.730797949	4.46761361	4.836096147
a =	2.970588235	0.647059	2.97058824	2.970588235	0.647059	2.323529412	2.32352941	1.294117647	2.32352941	2.32352941	0.264705882	7.205882353	7.205882353	2.63235294	2.588235294
$A_{s(prov'd)}$ (in.) =	3.34	1.32	3.34	3.34	1.32	3.34	3.34	3.34	4.92	4.92	3.34	8.24	8.24	4.95	4.92
	(2)#6,(4)#6	(3) # 6	(2)#8,(4)#6	(2)#8,(4)#6	(3) # 6	(2)#8,(4)#6	(2)#8,(4)#6	(2)#8,(4)#6	(4)#6,(4)#8	(4)#6,(4)#8	(2)#8,(4)#6	(4)#8,(4)#10	(4)#8,(4)#10	(4)#6,(4)#8	(4)#6,(4)#8
A_s' (in.) =	1.32	0.88	1.32	1.32	0.88	1.76	1.76	2.46	3.34	3.34	3.16	3.34	3.34	3.16	3.16
	(3) # 6	(2) # 6	(3) # 6	(3) # 6	(2) # 6	(4) # 6	(4) # 6	(2)#8,(2)#6	(2)#8,(4)#6	(2)#8,(4)#6	(4)#8	(2)#8,(4)#6	(2)#8,(4)#6	(4)#8	(4)#8
Φ =	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ΦMn =	384.9736765	152.8094	384.973676	384.9736765	152.8094	385.2648529	385.264853	383.0876471	563.014853	563.014853	377.6677941	902.6801471	902.6801471	566.410699	563.0505882
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
rho =	0.010121212	0.004	0.01012121	0.010121212	0.004	0.010121212	0.01012121	0.010121212	0.01490909	0.01490909	0.010121212	0.024969697	0.024969697	0.015	0.014909091
OK?	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
capacity ratio:	0.722539658	0.788555	0.96128903	0.9601579	0.62251	0.892132187	0.95311278	0.286250518	0.56626063	0.53340338	0.200543298	0.6042054	0.938203635	0.9025482	0.982946371

Figure D.8: Flexural design for girder D.

GIRDER E			
Trib Width=	6 ft.		
b = 12	h = 30		d = 27.5
	Beam 1		
	ext. support	midspan	int. support
Span	30 -		-
l_n (ft) =	28	28	28
w_{uSW} (klf) = 1.2D+L	0.3	0.3	0.3
$w_{uG}l_n^2$ =	235	235	235
C_m (Table 3-22a)=	-0.0830	0.0420	-0.083
$M_{uSW} = C_m W_u L_n^2$ =	-19.5	9.9	-19.5
P_{Dpt} =	30.125	30.125	30.125
P_{Lpt} =	30	30	30
C_m (Table 3-22a)=	-0.4	0.2	-0.4
$M_{uG} = C_m P_u L_n$ =	-740.88	370.44	-740.88
M_{uE} =	-174.2	0.0	-174.2
M_{utot} = 1.2D+E+L =	-934.6	380.3	-934.6
$A_{s(req'd)}$ (in.) =	8.832313085	3.26536	8.83231308
a =	7.970588235	3.2352941	7.97058824
$A_{s(prov'd)}$ (in.) =	10.16	4.74	10.16
	(8) #10	(6)#8	(8) #10
A_s' (in.) =	4.74	2.54	4.74
	(6)#8	(2) #10	(6)#8
Φ =	0.9	0.9	0.9
ΦMn =	1106.773676	541.98529	1106.77368
OK?	OK	OK	OK
rho =	0.030787879	0.0143636	0.03078788
OK?	OK	OK	OK
capacity ratio:	0.869322154	0.6888945	0.86932215

Figure D.9: Flexural design for girder E.

GIRDER F				
Trib Width	6 ft.			
b = 12		h = 30		d = 27.5
		Beam 1		
	ext. support	midspan	int. support	
Span	30	-	-	
l_n (ft) =	28	28	28	
w_{uSW} (klf) = 1.2D+L	0.3	0.3	0.3	
$w_{uG}l_n^2$ =	235	235	235	
C_m (Table 3-22a)=	-0.0830	0.0420	-0.083	
$M_{uSW} = C_m W_u L_n^2$ =	-19.5	9.9	-19.5	
P_{Dpt} =	16.6	16.6	16.6	
P_{Lpt} =	16.5	16.5	16.5	
C_m (Table 3-22a)=	-0.4	0.2	-0.4	
$M_{uG} = C_m P_u L_n$ =	-407.484	203.742	-407.484	
M_{uE} =	-89.3	20.0	-44.4	
M_{utot} = 1.2D+E+L =	-516.3	233.6	-471.4	
$A_{s(req'd)}$ (in.) =	4.378199376	1.961258	3.99702965	
a =	2.588235294	2.058824	2.58823529	
$A_{s(prov'd)}$ (in.) =	4.92 (4)#6,(4)#8	3.16 (4) #8	4.92 (4)#6,(4)#8	
A_s' (in.) =	3.16 (4) #8	1.76 (4)#6	3.16 (4) #8	
Φ =	0.9	0.9	0.9	
ΦMn =	563.0505882	364.7647	563.050588	
OK?	OK	OK	OK	
rho =	0.014909091	0.009576	0.01490909	
OK?	OK	OK	OK	
capacity ratio:	0.889877922	0.620651	0.8124044	

Figure D.10: Flexural design for girder F.

Columns

The following table displays the axial loads and moments on each column at the first floor. The color coding is the grouping of columns with the same reinforcement.

Column	Trib Area	P_D	P_L	P_W	P_S	$M_{D\,(unbalaced)}$	$M_{L\,(unbalaced)}$	M_W	1.2D+1.0W+L+.5S		1.2D+1.6L+.5S		Rebar
									P_u	M_u	P_u	M_u	
A.2	150	187.5	138.8	48.2	2.9	21.6	23.4	231.8	413.4	281.1	448.5	63.4	(12) #8
A.3	300	375.0	277.5	22.4	5.8	42.0	42.0	250.6	752.8	343.0	896.9	117.6	(12) #8
A.4	300	375.0	277.5	23.7	5.8	42.0	42.0	249.9	754.1	342.3	896.9	117.6	(12) #8
A.5	300	375.0	277.5	25.9	5.8	42.0	42.0	250.1	756.3	342.5	896.9	117.6	(12) #8
A.6	375	468.8	346.9	24.9	7.2	47.3	37.8	244.8	938.0	339.4	1121.2	117.2	(12) #8
A.7	225	281.3	208.1	48.2	4.3	24.1	18.9	224.9	596.0	272.7	672.7	59.2	(12) #8
B.1	37.5	46.9	34.7	125.0	0.7	1.3	0.0	275.9	216.3	277.5	112.2	1.6	(12) #8
B.2	247.5	309.4	228.9	112.6	4.8	15.0	22.5	296.0	715.2	336.5	739.9	54.0	(12) #8
B.3	420	525.0	388.5	105.6	8.1	78.8	90.0	285.8	1128.2	470.3	1255.7	238.5	(12) #10
B.4	420	525.0	388.5	97.4	8.1	78.8	90.0	286.1	1120.0	470.6	1255.7	238.5	(12) #10
B.5	420	525.0	388.5	105.6	8.1	78.8	90.0	286.3	1128.2	470.8	1255.7	238.5	(12) #10
B.6	525	656.3	485.6	112.6	10.1	82.5	97.5	296.0	1390.8	492.5	1569.6	255.0	(12) #10
B.7	315	393.8	291.4	125.0	6.1	41.3	48.8	275.9	892.0	374.2	941.8	127.5	(12) #8
C.1	131.25	164.1	121.4	127.3	2.5	1.3	0.0	298.2	446.9	299.8	392.4	1.6	(12) #8
C.2	341.25	426.6	315.7	114.7	6.6	15.0	22.5	319.7	945.6	360.2	1020.3	54.0	(12) #8
C.3	420	525.0	388.5	105.6	8.1	78.8	90.0	308.7	1128.2	493.2	1255.7	238.5	(12) #10
C.4	420	525.0	388.5	97.4	8.1	78.8	90.0	308.9	1120.0	493.4	1255.7	238.5	(12) #10
C.5	420	525.0	388.5	105.6	8.1	78.8	90.0	309.1	1128.2	493.6	1255.7	238.5	(12) #10
C.6	525	656.3	485.6	114.7	10.1	82.5	97.5	302.8	1392.9	499.3	1569.6	255.0	(12) #10
C.7	315	393.8	291.4	127.3	6.1	41.3	48.8	278.8	894.3	377.1	941.8	127.5	(12) #8
C9.1	81.25	101.6	75.2	20.0	1.6	40.0	39.4	334.0	217.9	421.4	243.0	111.0	(12) #8
D.2	225	281.3	208.1	28.8	4.3	42.0	42.0	340.1	576.6	432.5	672.7	117.6	(12) #8
D.3	300	375.0	277.5	22.4	5.8	42.0	42.0	367.1	752.8	459.5	896.9	117.6	(12) #8
D.4	300	375.0	277.5	23.7	5.8	42.0	42.0	366.1	754.1	458.5	896.9	117.6	(12) #8
D.5	300	375.0	277.5	25.4	5.8	42.0	42.0	366.4	755.8	458.8	896.9	117.6	(12) #8
D.6	543.75	679.8	503.0	26.2	10.5	42.0	42.0	357.9	1350.2	450.3	1625.8	117.6	(12) #10
D.7	393.75	492.2	364.2	33.7	7.6	137.2	145.9	329.9	992.3	640.4	1177.2	398.1	(12) #10
E.6	375	468.8	346.9	43.7	7.2	337.4	336.0	377.7	956.8	1118.6	1121.2	942.5	(16) #10
E.7	375	468.8	346.9	43.7	7.2	337.4	336.0	377.4	956.8	1118.3	1121.2	942.5	(16) #10
F.6	168.75	211.0	156.1	47.8	3.3	185.6	184.8	416.2	458.7	823.7	504.6	518.4	(12) #10
F.7	168.75	211.0	156.1	47.8	3.3	185.6	184.8	416.3	458.7	823.8	504.6	518.4	(12) #10

Figure D.11: Load determination of the lateral columns.

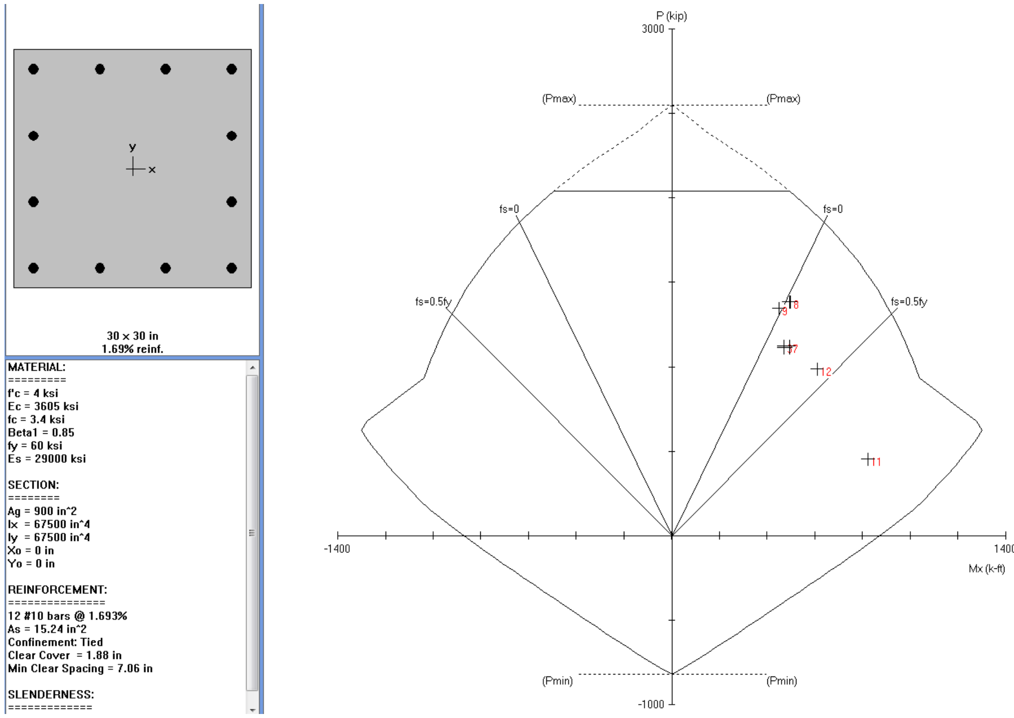


Figure D.12: Design of interior columns (shaded blue above) in spColumn.

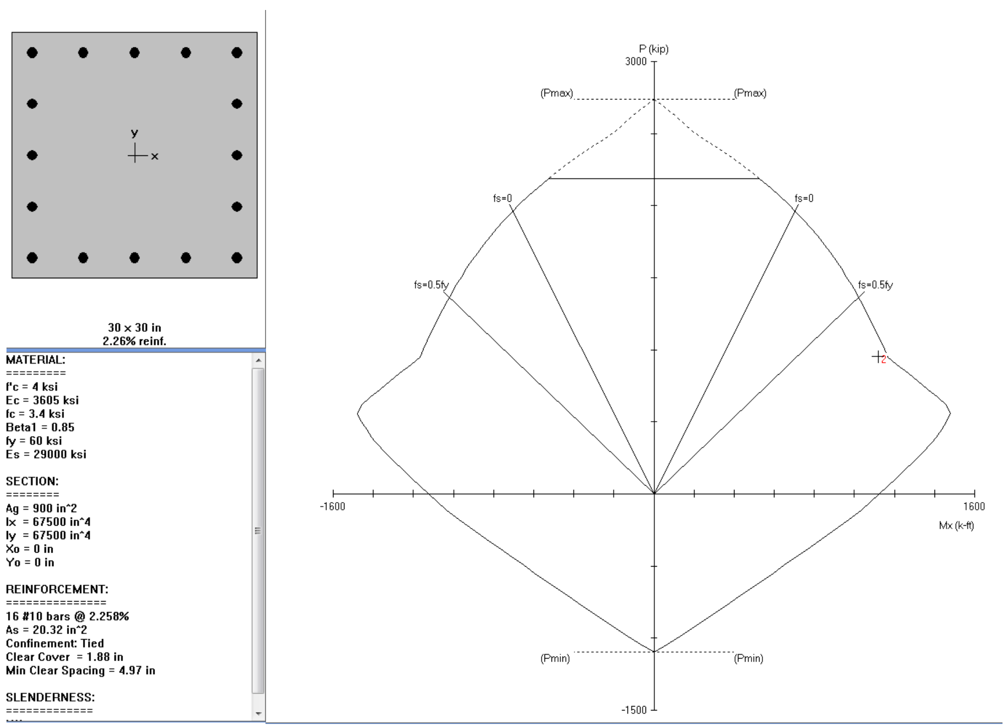


Figure D.13: Design of interior column line E columns (shaded purple above) in spColumn.

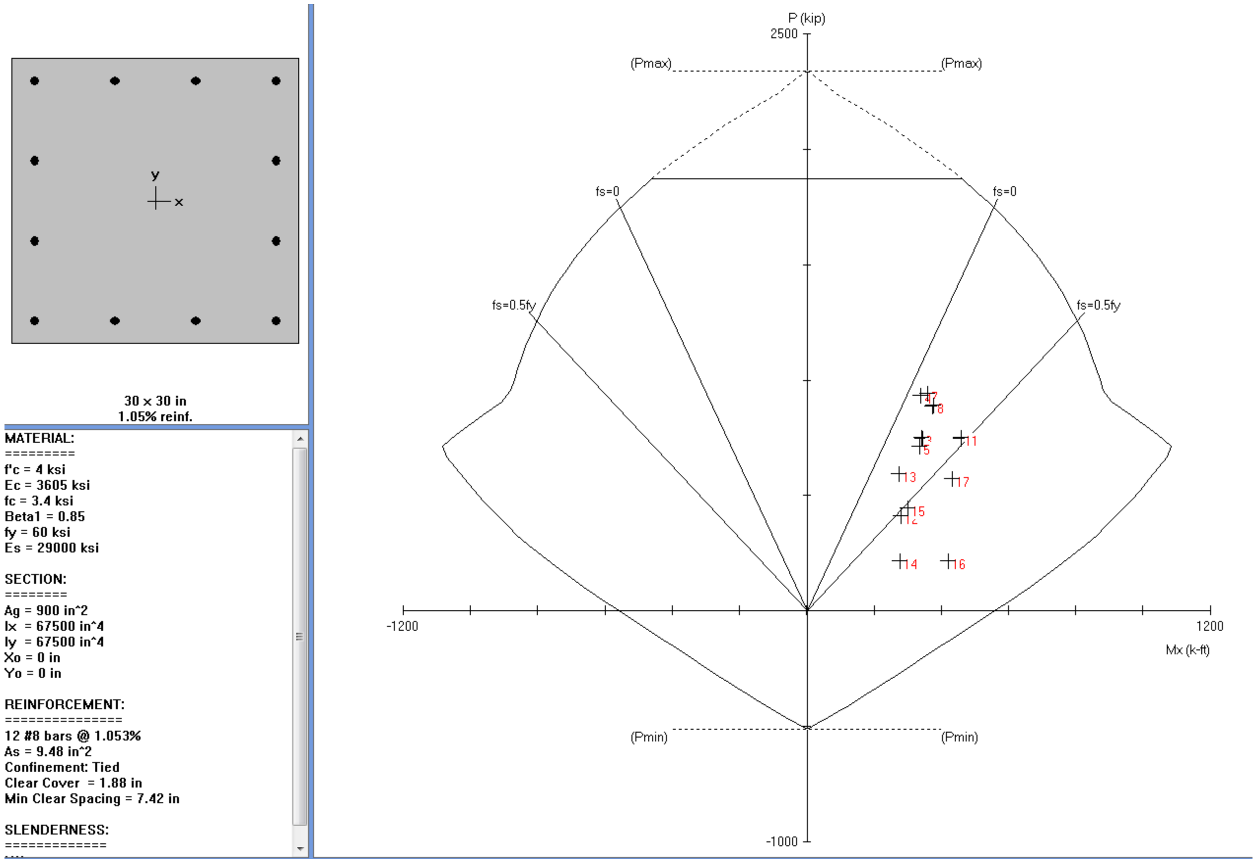


Figure D.14: Design of interior column line E columns (shaded orange above) in spColumn.

Appendix E: Construction Management Breadth

Concrete Structual Element	Size	Unit	Material	Labor	Equipment	Total	Total Incl. O&P	Amount	Total-O&P	Total Price
4000psi Concrete		C.Y.	$ 103.00	-	-	$ 103.00	$ 113.00	13,259.30	$ 1,365,707.90	$ 1,498,300.90
Concrete Finish	Bull Float	S.F.	-	$ 0.35	-	$ 0.35	$ 0.57	10,320.00	$ 3,612.00	$ 5,882.40
Concrete Labor/Equip	6" Slab	C.Y.	-	$ 15.10	$ 4.82	$ 15.95	$ 23.50	9,440.90	$ 150,582.36	$ 221,861.15
Equipment Pads	4" Slab	C.Y.	-	$ 16.00	$ 0.50	$ 16.50	$ 25.00	3,818.40	$ 63,003.60	$ 95,460.00
Slab Reinforcing		Ton	$ 850.00	$ 385.00	-	$1,235.00	$ 1,625.00	48.08	$ 59,381.64	$ 78,133.74
Slab Form	4 use	SFCA	$ 1.32	$ 2.48	-	$ 3.80	$ 5.60	61,920.00	$ 235,296.00	$ 346,752.00
Edge Form	4 use	L.F.	$ 0.12	$ 1.84	-	$ 1.96	$ 3.22	3,842.50	$ 7,531.30	$ 12,372.85
Concrete Beams	12x22	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	7,294.22	$ 206,791.14	$ 310,004.35
Beam Reinforcing		Ton	$ 800.00	$ 415.00	-	$1,215.00	$ 1,600.00	240.80	$ 292,572.00	$ 385,280.00
Beam Form	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	24,764.00	$ 103,183.33	$ 157,870.50
Concrete Beams	12x30	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	264.40	$ 7,495.74	$ 11,237.00
Beam Reinforcing		Ton	$ 800.00	$ 415.00	-	$1,215.00	$ 1,600.00	232.60	$ 282,609.00	$ 372,160.00
Beam Form	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	3,997.00	$ 16,654.17	$ 25,480.88
Concrete Girder	12x30	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	48.56	$ 1,376.68	$ 2,063.80
Girder Reinforcing		Ton	$ 800.00	$ 415.00	-	$1,215.00	$ 1,600.00	180.40	$ 219,186.00	$ 288,640.00
Girder Form	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	3,905.00	$ 16,283.85	$ 24,913.90
Concrete Column	30x30	C.Y.	-	$ 17.25	$ 5.50	$ 22.75	$ 32.50	688.20	$ 15,656.55	$ 22,366.50
Column Reinforcing		Ton	$ 1,175.00	$ 510.00	-	$1,685.00	$ 2,175.00	249.00	$ 419,565.00	$ 541,575.00
Column Form	4 use	SFCA	$ 0.62	$ 3.22	-	$ 3.83	$ 6.08	29,760.00	$ 114,080.00	$ 181,040.00
									$ 3,072,127.56	$ 3,930,836.88

Figure E.1: Concrete detailed estimate for the new structural system using RSMeans.

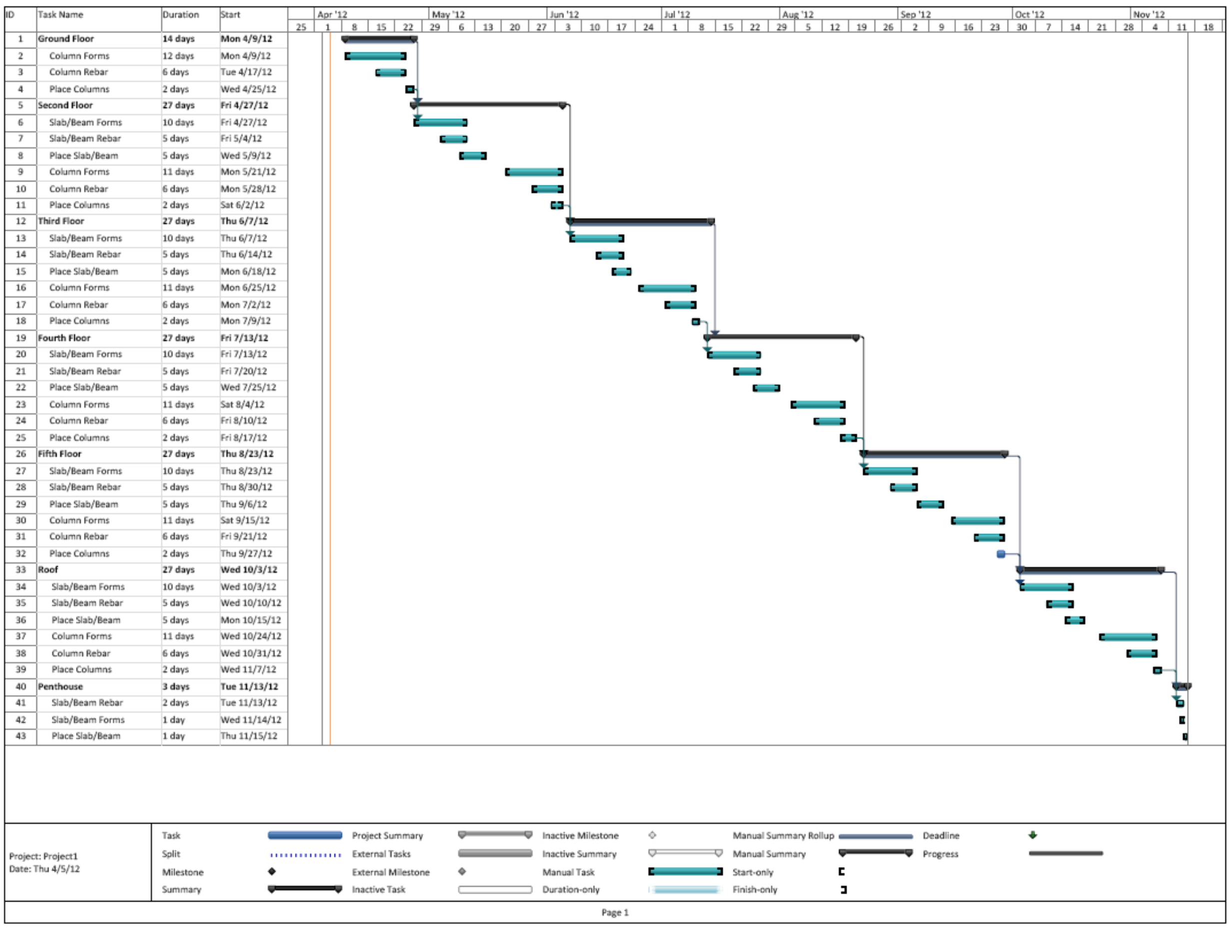
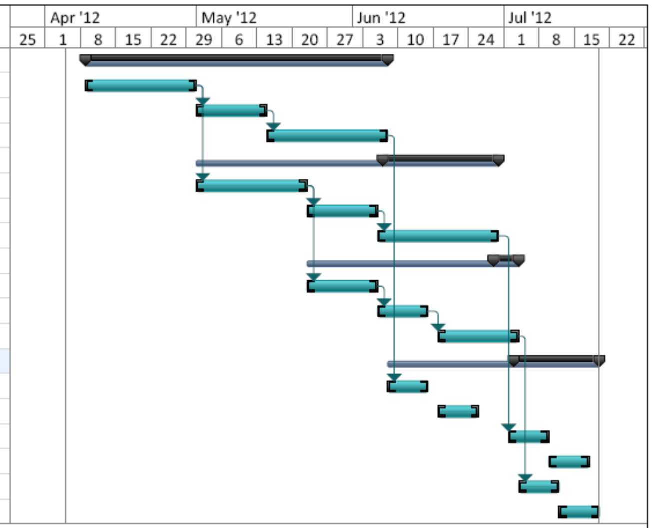


Figure E.2: Estimated concrete schedule based on RSMeans data.

ID	Task Name	Duration	Start	Finish
1	**Ground Floor -Second Floor**	**44 days**	**Mon 4/9/12**	**Thu 6/7/12**
2	Set Structural Steel	16 days	Mon 4/9/12	Mon 4/30/12
3	Detail Steel	10 days	Tue 5/1/12	Mon 5/14/12
4	Install Decking	18 days	Tue 5/15/12	Thu 6/7/12
5	**Third Floor-Fourth Floor**	**17 days**	**Thu 6/7/12**	**Fri 6/29/12**
6	Set Structural Steel	16 days	Tue 5/1/12	Tue 5/22/12
7	Detail Steel	10 days	Wed 5/23/12	Tue 6/5/12
8	Install Decking	18 days	Wed 6/6/12	Fri 6/29/12
9	**Fifth Floor-Penthouse**	**3 days**	**Fri 6/29/12**	**Tue 7/3/12**
10	Set Structural Steel	10 days	Wed 5/23/12	Tue 6/5/12
11	Detail Steel	8 days	Wed 6/6/12	Fri 6/15/12
12	Install Decking	12 days	Mon 6/18/12	Tue 7/3/12
13	**Concrete Pour**	**13 days**	**Tue 7/3/12**	**Thu 7/19/12**
14	Pour 1st floor	6 days	Fri 6/8/12	Fri 6/15/12
15	Pour 2nd floor	6 days	Mon 6/18/12	Mon 6/25/12
16	Pour 3rd floor	6 days	Mon 7/2/12	Mon 7/9/12
17	Pour 4th floor	6 days	Tue 7/10/12	Tue 7/17/12
18	Pour 5th floor	6 days	Wed 7/4/12	Wed 7/11/12
19	Pour Roof floor	6 days	Thu 7/12/12	Thu 7/19/12



Project: Project2
Date: Thu 4/5/12

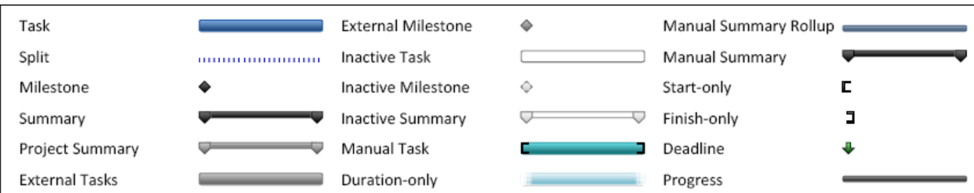


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Figure E.3: Estimated steel schedule based on RSMeans data.

Appendix F: Photovoltaic Analysis Breadth

Location Inputs		
Min Temp	-22	°C
Max Temp	41	°C
Roof	30	°C
Array Size	6000	W
Max Roof Temp	**71**	**°C**

Pronius IG Plus 6.0		
PV Array	5100-6900	W
MPPT Min Voltage	230	V
Max Voltage	600	V
Solon Black XT 290 Wp		
Pmax	290	W
Vmpp	35.6	V
Impp	8.16	A
Voc	44.23	V
ISC	8.56	A
Temp Coeff	-0.36	%/°K

Solon Black XT 290 Wp		
Modules Per String	10	
Number of Strings	2	
Number of Panles	20	
Array Rated Power	5800	W
Max Voltage	51.714	V
High Temp V	29.705	V
Array Output Voltage		
Max Voltage	517.13716	OK
High Temp V	247.53867	OK

Figure F.1: Preliminary voltage calculations to determine how many strings per inverter.

System Summary

Nameplate Capacity	17.4298	kW
Total Direct Cost	72,370.20	$
Total Installed Cost	80,171.85	$
Total Installed Cost per Capacity	4,599.71	$/kW
Analysis Period	30	years
Inflation Rate	2.5	%
Real Discount Rate	5.2	%

Figure F.2: System Summary of the PV array from SAM.

PV Analysis

20 panels / space

→ 1 inverter, 2 strings *see excel chart for calcs

$I_{sc} = 8.56A$

25% SF

25% wire + fuse SF

1 string = (8.56)(1.25)(1.25) = 13.4A ⇨ #14AWG @ 90°C 25(.58) = 14.5

2 strings = 13.4 (2) = 26.75 ⇨ #8AWG @ 90°C 55(.58) = 31.9 > 26.8 ✓

Conduit 1" above roof ⇨ .58 correction factor at 63°C

Out of inverter = 277V → 21.7A

21.7(1.25) = 27.125 ∴ 30A Breaker 2 pole

27.125 ⇨ #8 AWG @ 90°C 31.9 > 26.8 ✓

(27.125) 3 = 81.375 ⇨ 90A