Technical Assignment I

CBD Chemical Production Building

Virginia, USA

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Executive Summary

The following technical report analyzes the existing conditions and structural design of CBD Chemical's Production Building located in Virginia. 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. The analysis of the structural systems included verifying the loads used by the structural engineers on the project and spot checking various beams and columns.

Using ASCE7-10 to determine the loads on the Production Building, it was determined that earthquake loads control for both base shear in the North-South direction and overturning moment in both directions, while the wind loads control the base shear in East-West direction. The controlling base shear calculated is 516.7 kips in the North-South direction and 514.4 kips in the East-West. The controlling overturning moment was calculated to be 37282.3 kip-ft.

Select spot checks were performed in the Production Building to determine the efficiency of the existing structural system. Checks were done for the composite deck, a composite beam, and a girder on the third floor. Both the deck and beam were found to have extra capacity for the loads used in this evaluation. The floor was designed as a 5.5 inch slab and the decking and two inches of concrete in the decking were not accounted for in added strength. The beam was designed as a non-composite beam but built as a composite beam, meaning it has more capacity than needed according to assumptions used in this report. The girder spot checked was found to be adequate. Two columns were checked, one interior and one exterior. Both were also found to be acceptable.

Table of Contents

1.0 Introduction

The purpose of this report is to analyze the existing conditions and explore other viable solutions to the design constraints for The Production Building in Virginia, USA. All of the structural loads on The Production Building were calculated, including dead, live, snow, wind, seismic and blast. The existing structure was analyzed and compared to four other systems to determine feasibility.

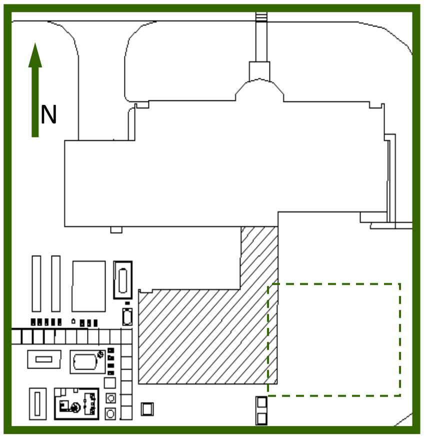


Figure 1.1: Site Plan. Courtesy of Project Engineer. This plan shows a portion of the campus footprint with the Production Building shaded. The future bays will be located in the dashed area.

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.

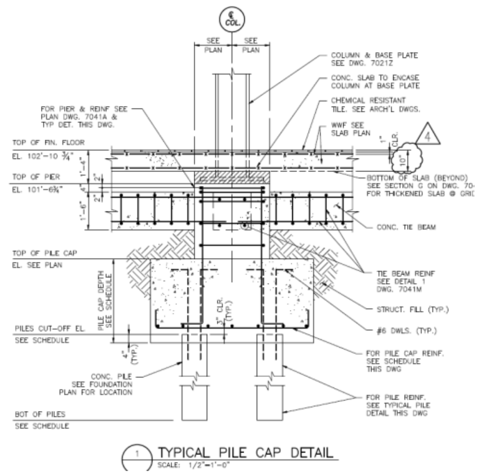


Figure 2.1: Typical Pile Cap Detail. Courtesy of Project Engineer.

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.

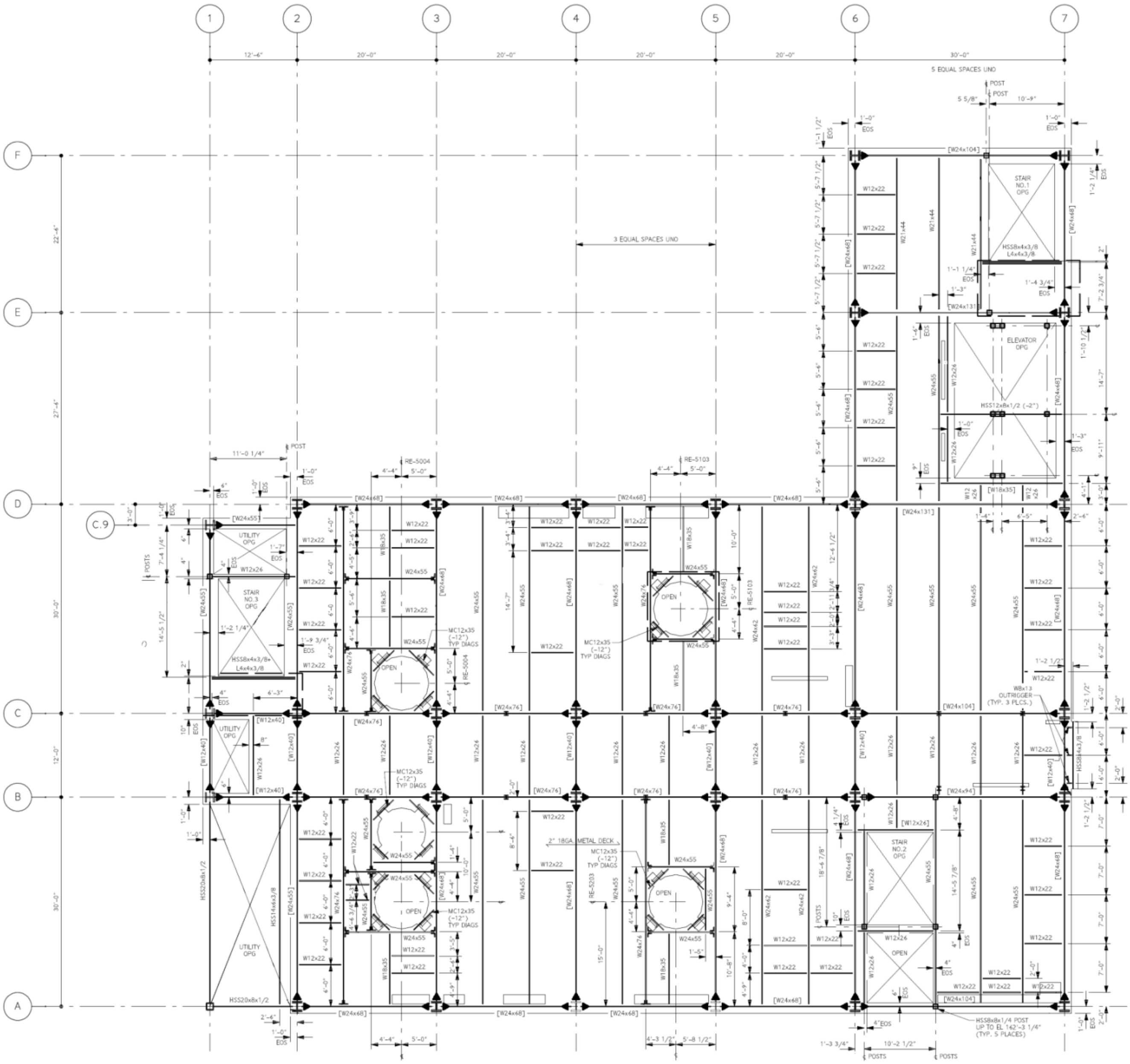


Figure 2.2: Courtesy of Project Engineer. The third floor framing plan.

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.

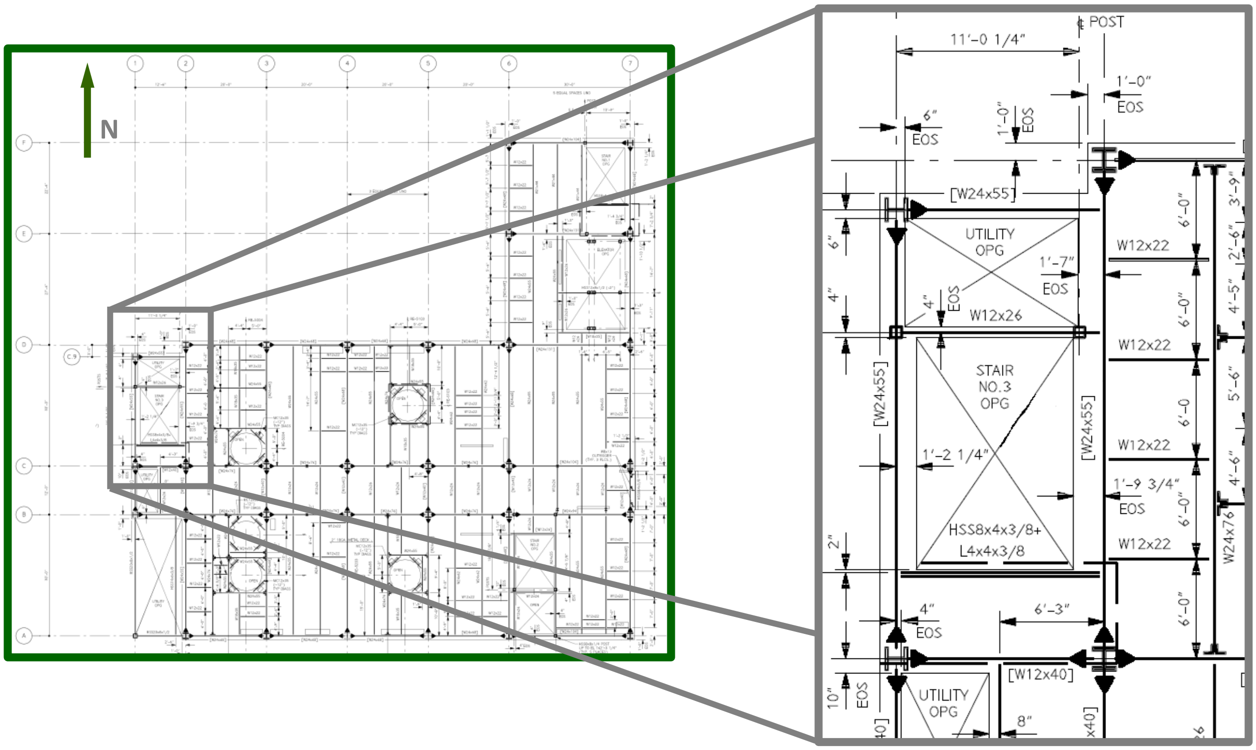


Figure 2.3: Courtesy of Project Engineer. The third floor framing plan showing the odd column rotation on the west end of the building.

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 these 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 Determination of Loads

3.1 Gravity Loads

3.1.1 Dead and Live Loads

The dead loads used for the Production Building are relatively high due to the heavy equipment supported on each floor. The live loads plus superimposed dead loads on the second through fifth floor of the production building include the live load of 200 psf and the equipment pads, steel framing, MEP, and partitions, totaling 298 psf. The steel framing seems high at first glance but due to the framing in the pipe rack and around equipment there are many beams in a relatively small area in many parts of the building. Also, because the Production Building is for the pharmaceutical industry most of the rooms are clean rooms. Therefore there will be many partitions between clean room production areas and the equipment. There are equipment loads on each of these floors. The slab was increased to a 7½ inch depth (larger than specified in the deck manual) on 2VLI composite deck. The slab was designed as a 5½ inch concrete slab. The additional two inches of concrete in the deck and the decking itself were considered arbitrary and were not designed to contribute to the strength of the system. A summary of dead loads is included below, as well as a table of the equipment point loads per floor. For the purposes of this report equipment will be considered dead load. Most of this equipment is built into the framing or bolted to the equipment pads. Therefore, it will act as dead load on the structure for the majority of the building life. The only equipment loads listed in table 3.2 are those that exceed the live loads per floor. Please see Appendix B for the location of the equipment point loads on the floor plans per floor.

First Floor Dead Load	
Equipment Pad (NWC)	100 psf
Total	100 psf
Second through fifth floor Dead Load	
7½" slab on 2VLI 18 ga Deck (NWC)	82 psf
Equipment Pads (NWC)	50 psf
Steel Framing	18 psf
MEP	20 psf
Partitions	10 psf
Total	180 psf
Penthouse Roof Dead Load	
6" slab on 2VLI 18 ga Deck (NWC)	63 psf
Equipment Pads (NWC)	50 psf
Steel Framing	18 psf
MEP	20 psf
Roofing	4 psf
Misc Dead	5 psf
Total	160 psf

Table 3.1: Dead Loads

Equipment Loads per Floor											
First Floor		Second Floor		Third Floor		Fourth Floor		Fifth Floor		Roof Level	
No.	Operational Weight	No.	Operational Weight	No.	Operational Weight	No.	Operational Weight	No.	Operational Weight	No.	Operational Weight
1	47 k	1	31 k	1	44 k	1	44 k	1	11 k	1	20 k
2	56 k	2	31 k	2	40 k	2	25 k	2	3 k	2	102 k
3	50 k	3	27 k	3	36 k	3	23 k	3	6 k	3	126 k
4	25 k	4	27 k	4	51 k	4	23 k	4	2 k	4	26 k
5	58 k			5	21 k	5	51 k	5	2 k	5	11 k
6	36 k			6	23 k	6	44 k				
				7	11 k	7	21 k				
						8	29 k				

Table 3.2: Equipment dead loads per floors. The only equipment loads listed are those that exceed the live loads per floor. Appendix B shows the layout of the equipment for design purposes (not the equipment layout plan).

3.1.2 Snow Loads

The ground snow loads for Virginia, USA are 25 psf. The pressure on the flat roof without drift was calculated to be 19.3 psf. Because there is a penthouse, drift loads had to be considered as well as just snow loads. The penthouse is 15 feet by 50 feet and is located above the elevator and stairs on the Northeast corner of the Production Building. The drift on the penthouse was calculated to be 39.7psf. The drift was also accounted for on the 4 foot 6 inch parapets on the building. The parapet condition produced the highest drift weight of 48.3 psf. The figure below shows the loading produced by the snow load and drift against the penthouse. This figure is not drawn to scale. For the full calculations for snow loads please see Appendix C.

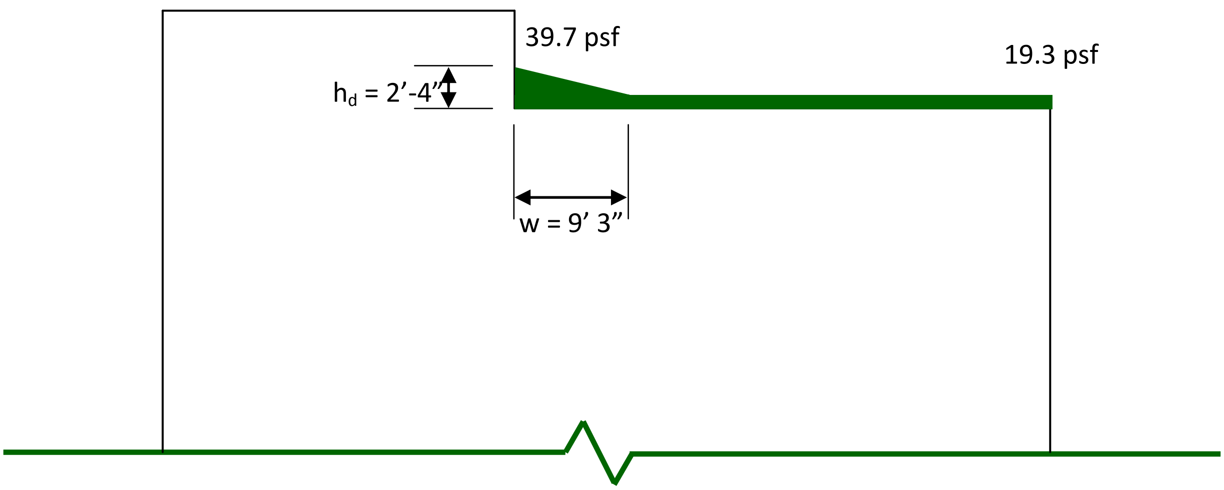


Figure 3.1: Snow load and drift up to the penthouse.

3.2 Wind Loads

To determine the wind pressures on the Production Building, ASCE 7-10 was used. Both the North-South and East-West directions were analyzed. For simplification the assumed layout of the building was the entire outline of the footprint as shown in figure 3.2 below. To calculate the pressures, the penthouse was assumed to act as an extension of the building due to the columns continuing up through the penthouse level without splices beyond the fifth floor.

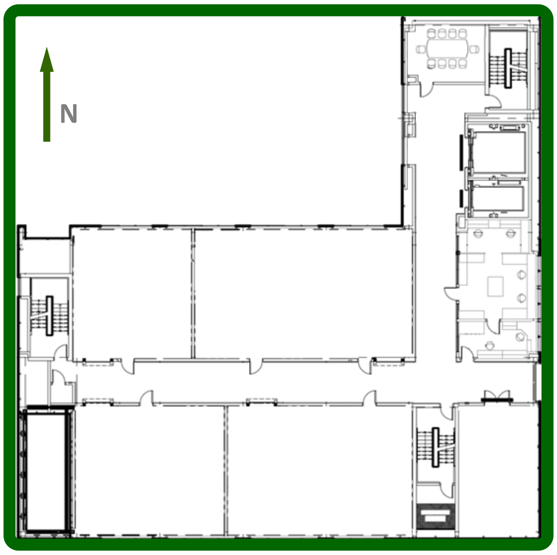


Figure 3.2: Courtesy of Project Engineer. Layout of the building footprint. The building is 122 feet by 122.5 feet.

The building footprint is 122 feet by 122.5 feet. Therefore the base shears and overturning moments are not much different for the two directions. The Production Building is located in an area with very little surrounding it; therefore the exposure is Exposure C. This was confirmed with the engineers involved. Throughout the entire site the elevation remains constant. Therefore, the K_{zt} factor is 1.0. In tables 3.5 and 3.6 below the East-West and North-South wind pressures and forces were calculated as well as the base shear and overturning moment each way. Neither of these base shears or overturning moments control over the earthquake loading. Figures 3.4 and 3.5 below show the pressures acting on the Production Building. For full wind calculations please see Appendix D.

East - West Wind				
Floor	h_i	z	Windward Force (k)	Leeward Force (k)
1	0	0	29.5	-28.4
2	24	24	54.0	-49.7
3	18	42	51.8	-42.6
4	18	60	56.8	-42.6
5	18	78	60.7	-42.6
Roof	18	96	58.4	-39.1
PH Roof	15	111	11.1	-7.3
			Σ = 292.7	Σ = -224.0
			Base Shear = 516.7 k	
			Overturning Moment = 29832.2 k-ft	

Table 3.4: East-West wind loading.

North-South Wind				
Floor	h_i	z	Windward Force (k)	Leeward Force (k)
1	0	0	29.6	-28.5
2	24	24	54.2	-49.9
3	18	42	52.0	-42.8
4	18	60	57.0	-42.8
5	18	78	60.9	-42.8
ROOF	18	96	58.6	-39.2
PH Roof	15	111	3.3	-2.2
			Σ = 286.1	Σ = -219.8
			Base Shear = 505.9 k	
			Overturning Moment = 29954.5 k-ft	

Table 3.5: North-South wind loading.

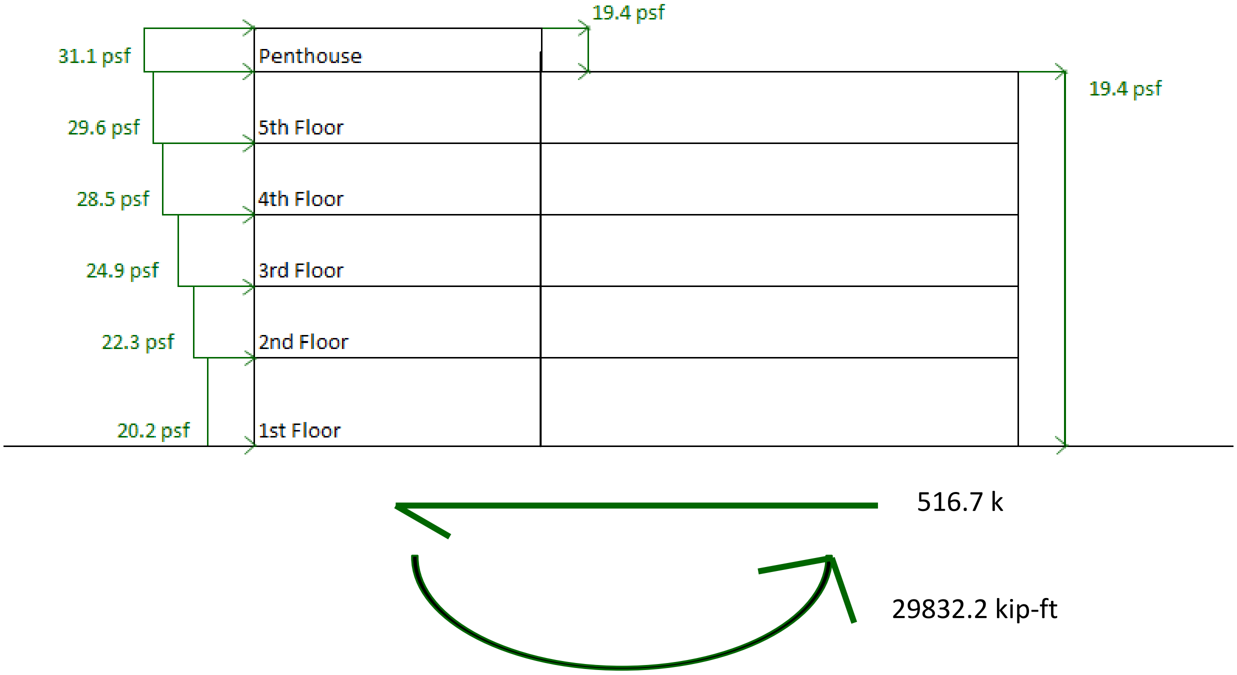


Figure 3.3: The pressure distribution, base shear and overturning moment for the East-West wind load case.

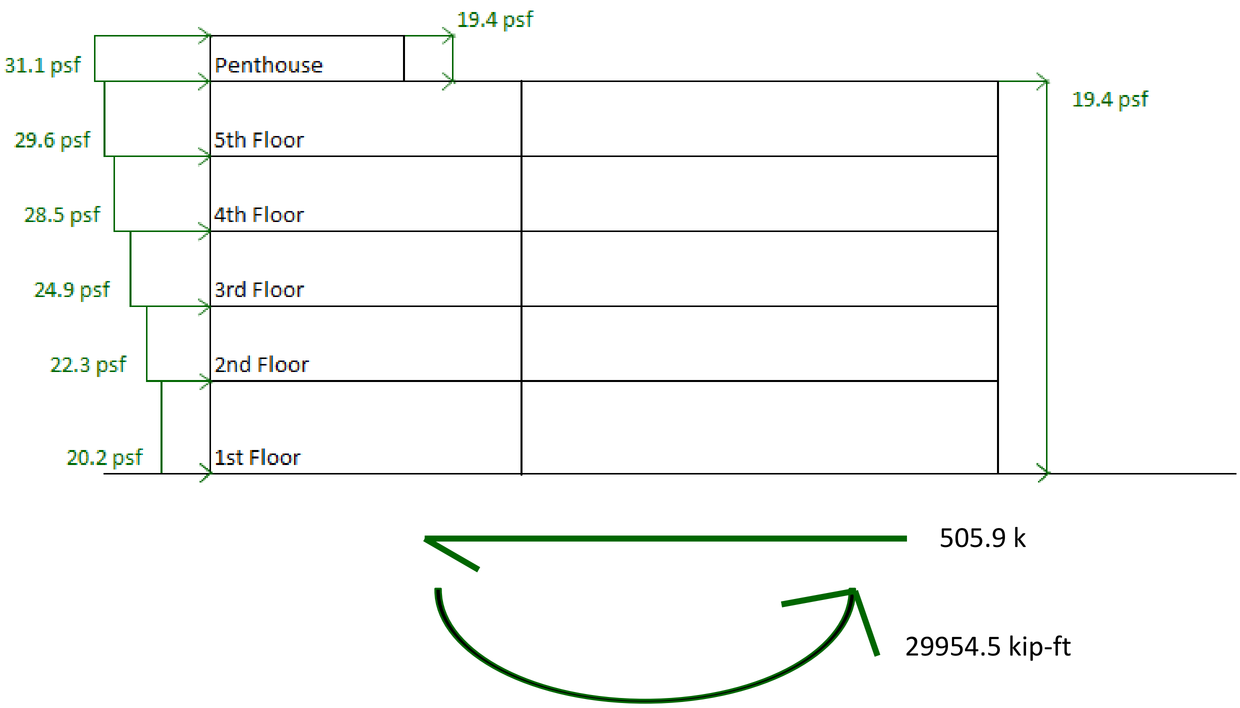


Figure 3.4: The pressure distribution, base shear and overturning moment for the North-South wind load case.

3.3 Seismic Loads

To calculate the seismic loads for the Production Building, ASCE 7-10 was used. The geotechnical report classified the site soils as site class E. Because a more recent code was used to check, some of the seismic response coefficients are slightly different from the designers. Using the USGS website to pinpoint the seismic region, S_{DS} = .26g and S_{D1} = .138g were calculated by inputting the address of the site and performing subsequent calculations. The seismic data may have changed from ASCE 7-02 to ASCE 7-10. The designers for the Production Building calculated SDS = .40g and SD1 = .18g. These two numbers do not match, however the USGS website has been updated since the building was designed in 2002. Design category C was the more conservative site classification. This category was confirmed by the structural engineers of the Production Building.

To calculate the building weights, the equipment loads should be considered dead load. Most of the equipment will be bolted to the equipment pads or framed into the floor itself. Therefore, for the purposes of earthquake engineering these loads will be adding to the mass of the building that will increase the base shear and moment to be resisted. For this reason, when calculating the floor weights of each level, the equipment point loads per floor were added as dead load. The dead loads used were the same calculated in section 3.1.1. For the penthouse roof level 8 psf was used for framing, 5 psf for roofing/insulation, 2 psf for roof deck, and 5 psf for miscellaneous dead load. In addition the exterior wall weight was added to each floor. For the full weight calculations please see Appendix E. The following table shows the floor weights calculated.

Floor	Total Weight (k)
1	2572
2	2103
3	2293
4	2283
5	2025
ROOF	1981
Penthouse	19

Table 3.6: These are the total dead loads per floor used in the seismic procedure.

The earthquake base shear and overturning moment controlled over wind. The base shear to resist seismic loads was 514.4 kips, while the overturning moment was 37,214 ft-kips. The figure below shows the load on each floor as well as the base shear and overturning moment for the earthquake loading. Please see Appendix E for complete calculations and tables.

Floor	Total Weight (k)	z (ft)	$w_x h_x^k$	C_{vx}	F_x (k)
1	2572	0	0	0	0
2	2103	24	126874	0.06	30.5
3	2293	42	284658	0.13	68.4
4	2283	60	449002	0.21	107.9
5	2025	78	558657	0.26	134.2
ROOF	1981	96	714527	0.33	171.6
Penthouse	18.6	106	7624	0.00	1.8
		Σ =	2141342	1.0	514.4
			Overturning Moment =		37214

Table 3.7: The table used to calculate story forces and overturning moment.

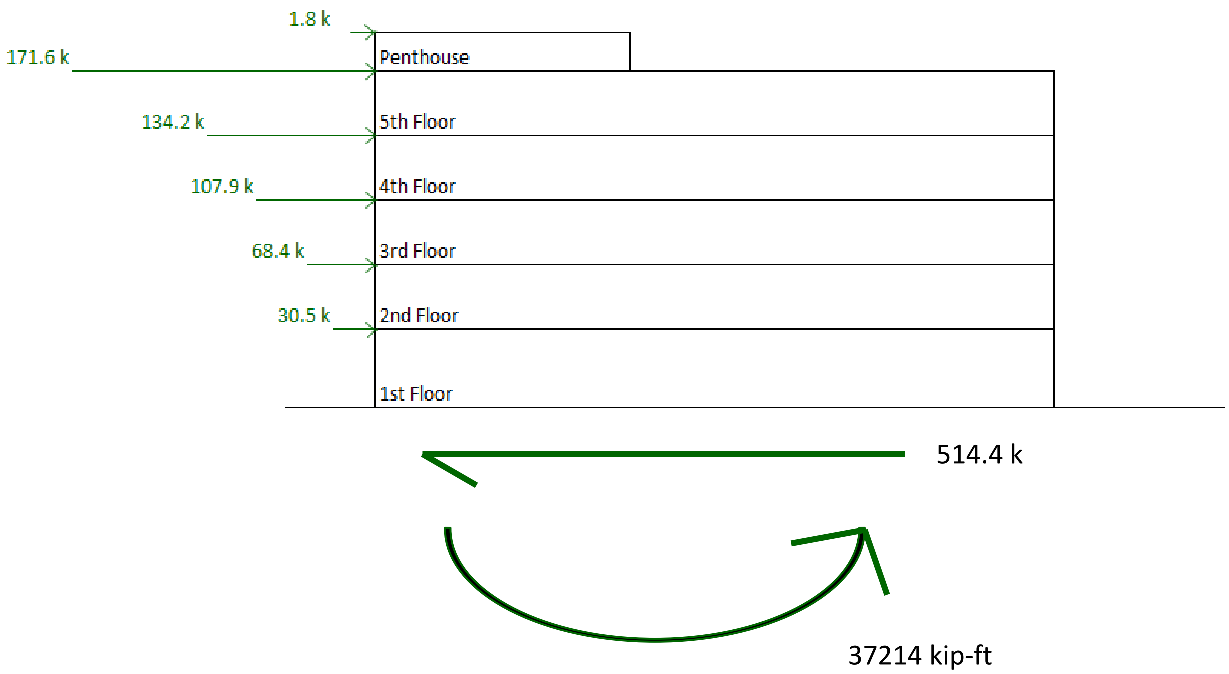


Figure 3.5: The seismic story forces, base shear and overturning moment.

3.4 Blast Loads

Due to the close regulation of their systems, CBD Chemical determined that 40psf would be the over pressure that could be caused by an explosion. The engineers used this overpressure to design their blast resistant system. Rather than designing the building to stand with parts of the structural system removed to account for an explosion, the walls were designed to fail first. At 40psf the connections of the fabricated panels will fail causing the panels to fall out onto the ground below.

4.0 Evaluation of Systems

Spot checks were performed on a beam, girder, and two columns (one exterior and one interior). The figures below show the area of the building chosen to complete these spot checks. The green box outlines the bay and the gray boxes show exactly which beam, girder, and columns were spot checked. Complete spot check calculations can be found in appendix F.

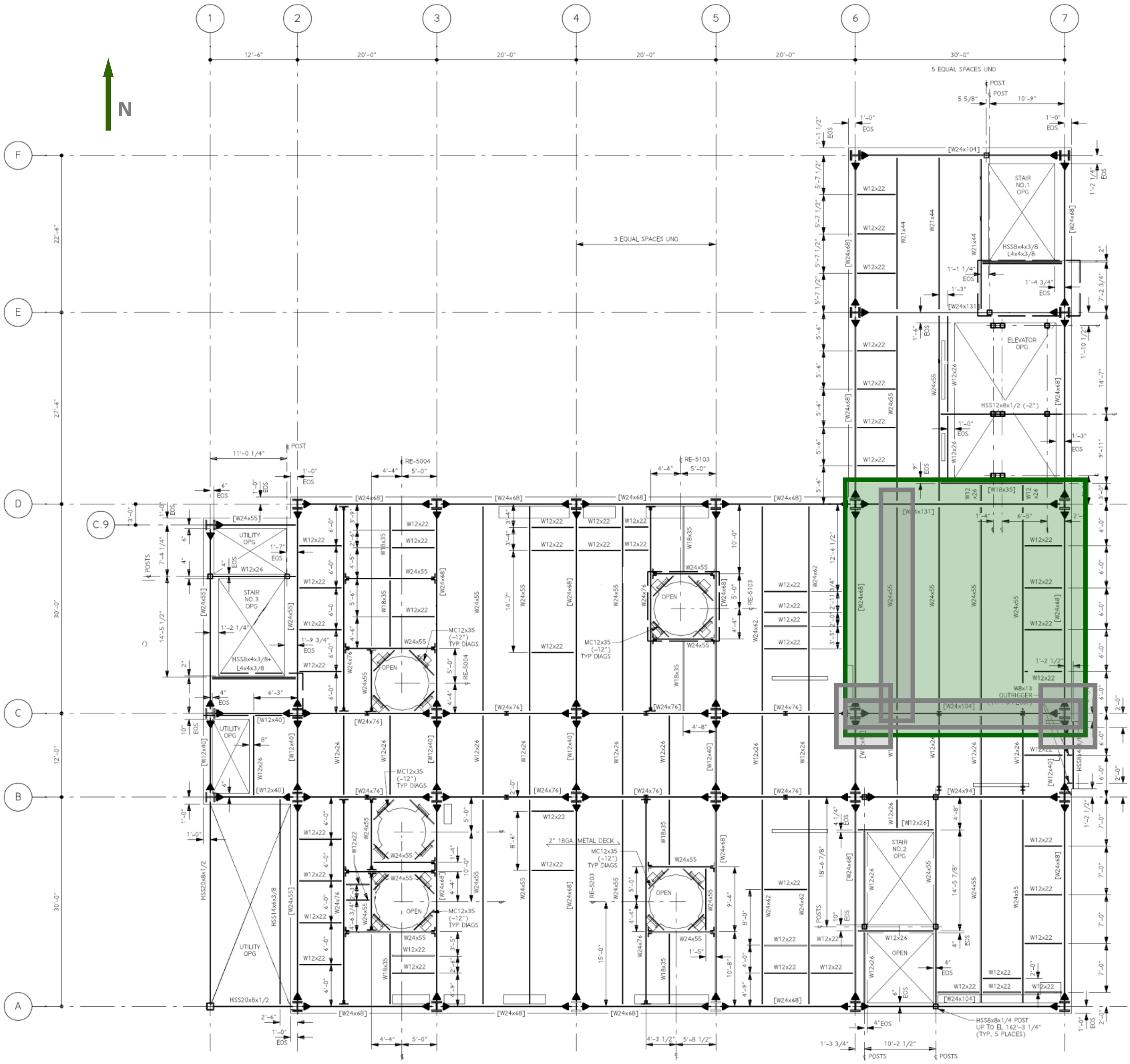


Figure 4.1: Courtesy of Project Engineer. The third floor plan with the green box locates the area where spot checks will be performed. The gray boxes outline which beam, girder and columns were spot checked.

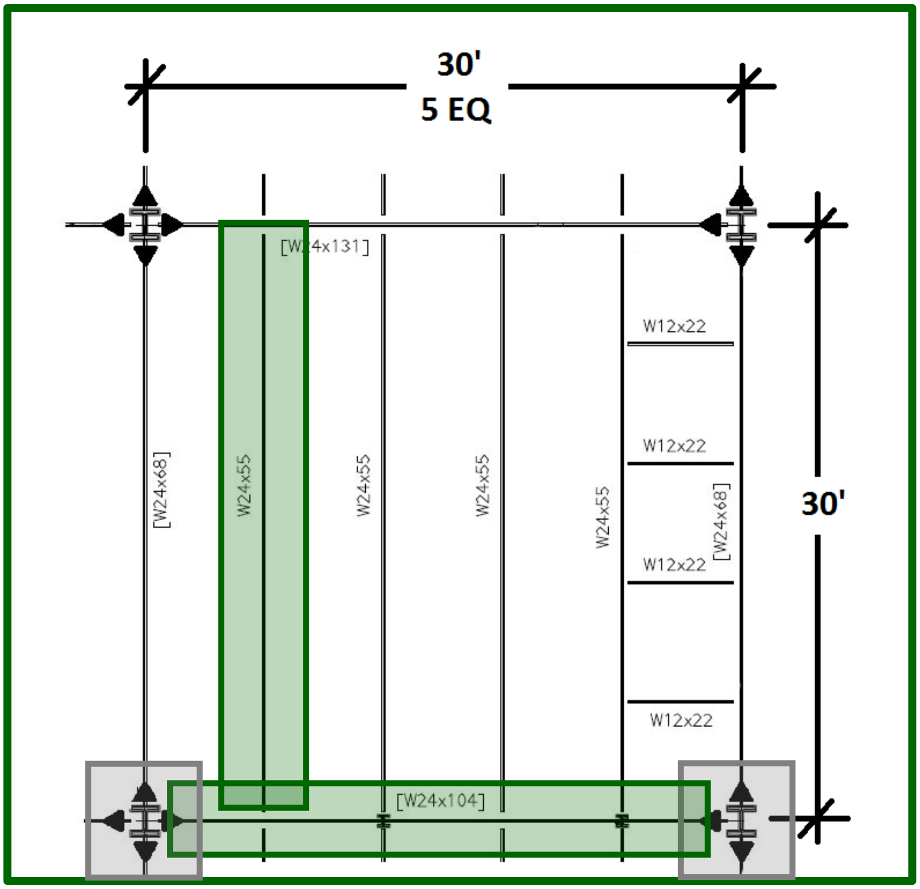


Figure 4.2: A framing plan of the third floor 30 foot by 30 foot bay all of the spot checks will be in. The green boxes show which beam and girder analyzed. The gray boxes show the two columns analyzed.

4.1 Floor System for Typical Bay

The floor check will be performed on the third floor. The area in question is within the green box in Figure 4.1 above. The dead and live loads calculated in section 3.1.1 and 3.1.2 were used. The drawings specify 2VLI 18 gage deck with 7 ½ inches of normal weight concrete. This specification with 7 ½ inches of normal weight concrete is not listed in the Vulcraft Steel Deck catalog. However, the table states that for any loads above 200 the manufacturer must be contacted due to the majority of those cases resulting from high point loads. The designers however designed the slab as a 5½ inch slab and considered the deck and concrete underneath arbitrary. The full calculations for the decking spot check can be found in Appendix F.

4.2 Typical Beam and Girder Check

4.2.1 Beam Check

Figure 4.3 below shows the beam that was analyzed in the typical beam check. The structural cover sheet notes that every beam shown in the plans should have ¾ inch shear studs spaced every foot on center. Calculating the capacity of the beam that was spot checked revealed that much of the capacity of the composite beam is not needed. Because it actually acts as a composite beam, the capacity was calculated to be 910 kip-ft even though the load it needs to hold is only 361.8 kip-ft.

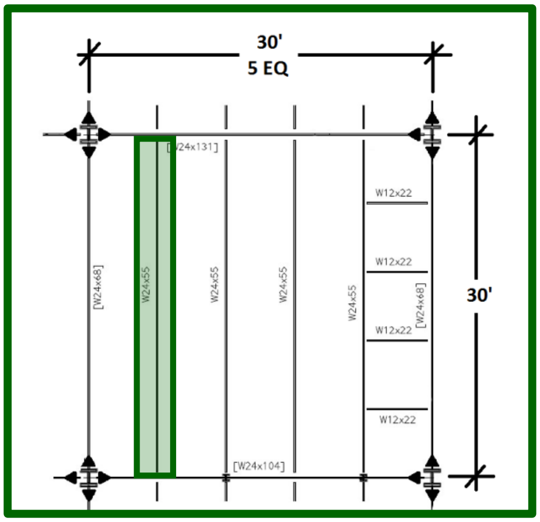


Figure 4.3: Floor plan courtesy of Project Engineer. The bay chosen to spot check. The beam being checked is highlighted in green.

Comparing these calculations with the engineer's calculations, it was discovered that the beam was never designed as a composite beam. Figure 4.4 shows the output of the designer's final RAM model. The value shown for ΦMn is equal to 502.5 kip-ft which is the capacity of the W24x55 without acting compositely. Comparing the engineers Mu to the output ΦMn the beam is still only using 72% of its capacity. This is probably to accommodate for future use of the space. CBD Chemicals would eventually like to expand and therefore the engineers were mindful to design the building for enough capacity that it would still hold if production were increased. For complete calculations and a suggested beam calculation please see Appendix G.

RAM

Gravity Beam Design

RAM Steel v14.03.02.00
DataBase: BICI BLDG S5 (FINAL DESIGN) 09/23/11 02:01:45
Building Code: IBC Steel Code: AISC LRFD

Floor Type: 2ND FL **Beam Number = 80**

SPAN INFORMATION (ft): I-End (96.00,42.00) J-End (96.00,72.00)

Beam Size (User Selected) = W24X55 Fy = 50.0 ksi
Total Beam Length (ft) = 30.00
Mp (kip-ft) = 558.33

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	1.080	1.200	0.0%	Red	0.000
	30.000	1.080	1.200			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 48.24 kips 0.90Vn = 251.69 kips

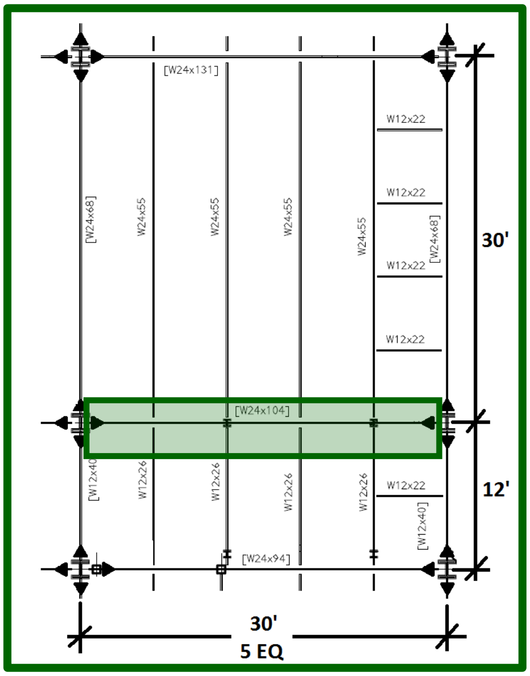
MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	361.8	15.0	0.0	1.00	0.90	502.50
Controlling		1.2DL+1.6LL	361.8	15.0	0.0	1.00	0.90	502.50

Figure 4.4: Courtesy of Project Engineer. The output from the engineers' calculations in RAM. The dashed line shows that the capacity of the beam is the non-composite capacity of a W24x55 rather than the composite action of the constructed beam.

4.2.2 Girder Check

Because this girder is part of the lateral system it is connected to both columns with moment connections. To simplify calculations, fixed beam coefficients from ACI continuous beam moment coefficients used. Because the bay sizes are different, the average bay length was calculated and used in the tables. The W24x55 on the right end of this bay has W12x22 beams framing every 6 feet. These beams are already accounted for in the steel allowance. The controlling moment was calculated as -561.9. Because this largest moment is negative, the beam will not work compositely. A W21x68 was determined to be the most economical. The larger beam chosen by the designer is due to the lateral analysis. Each girder is part of the lateral system and therefore could have more moment when the lateral loads are applied. The designer chose a W24 for the ease of the connection with the W24 beams that would be framing into the girder. For complete calculations please see Appendix G.



4.3 Typical Column Check

For capacity of the columns, the 14th edition of the AISC Steel Manual was used to calculate the interaction between the bending and axial loading of the column. The figure 4.6 below represents the columns checked. Using pattern loading, the unbalanced moments were calculated for each floor level, and then added together down the length of the column. The columns in the Production building are only spliced once in the third floor level. At this splice the column size changes from W14x370 to W14x176. Also these columns are only braced at the floor levels. Since the greatest loading on these columns will be at the base right before the splice, only two checks per column had to be performed. Using combined loading the interaction for the first floor interior column was found to be .95. The interior 3rd floor column interaction was .64. The interactions for the exterior columns were calculated to be .86 and .57 for the first floor and third floor respectively. These numbers seem correct as the wind and earthquake loading will increase the moment in the columns. Although the first floor columns seem to be loaded close to capacity, the earthquake and wind loading would increase the moment at the base by a smaller percentage than the top. The W14x370s used on the first floor are mostly controlled by the Pu not the Mu. Please see appendix H for full calculations and tables.

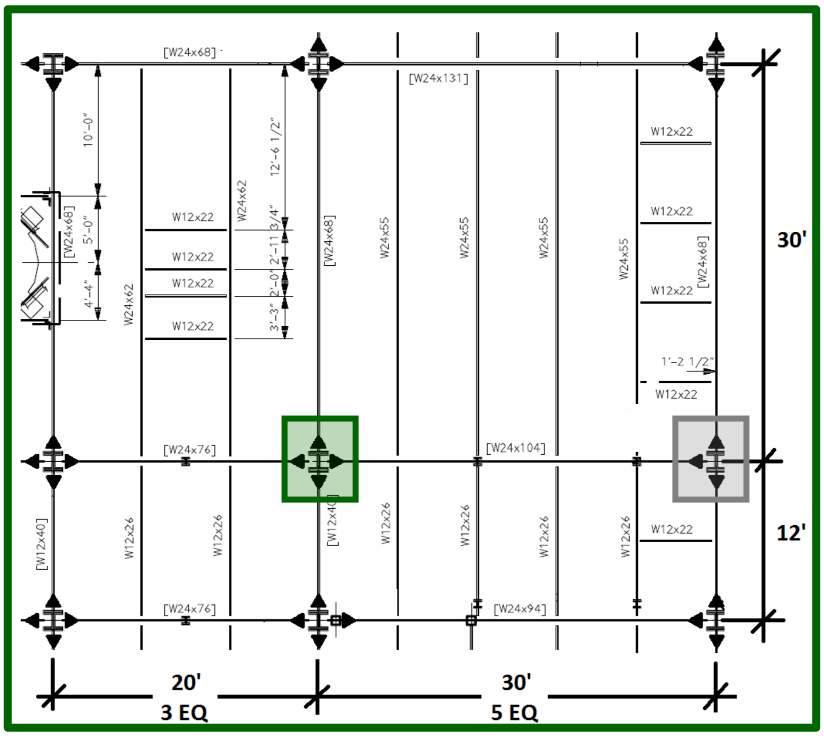


Figure 4.6: Floor plan courtesy of Project Engineer. The bay chosen to spot check. The interior column checked is highlighted in green. The exterior column checked is highlighted in grey.

5.0 Conclusions

By analyzing each part of the structural system, it was clear how each individual system works together in the structural integrity of the Production Building. By verifying each load, a greater understanding was gained for the considerations that designers must address. Using spot checks, the entire structure was determined to have adequate strength.

The lack of space in the building footprint drove the majority of design decisions for the Production Building. Engineers had to design the entire lateral system from moment connections at every girder and beam framing into the columns. The possibility of attempting to redesign the structure in concrete could be explored. In addition, the lateral system will be further explored in technical assignment three.

The spot checks performed also revealed the beams and girders were not designed to take advantage of the large amount of concrete on top of the composite deck. A study to determine is money could have been saved based on the assumptions used for this report.

6.0 Appendices

Appendix A: Framing Plans

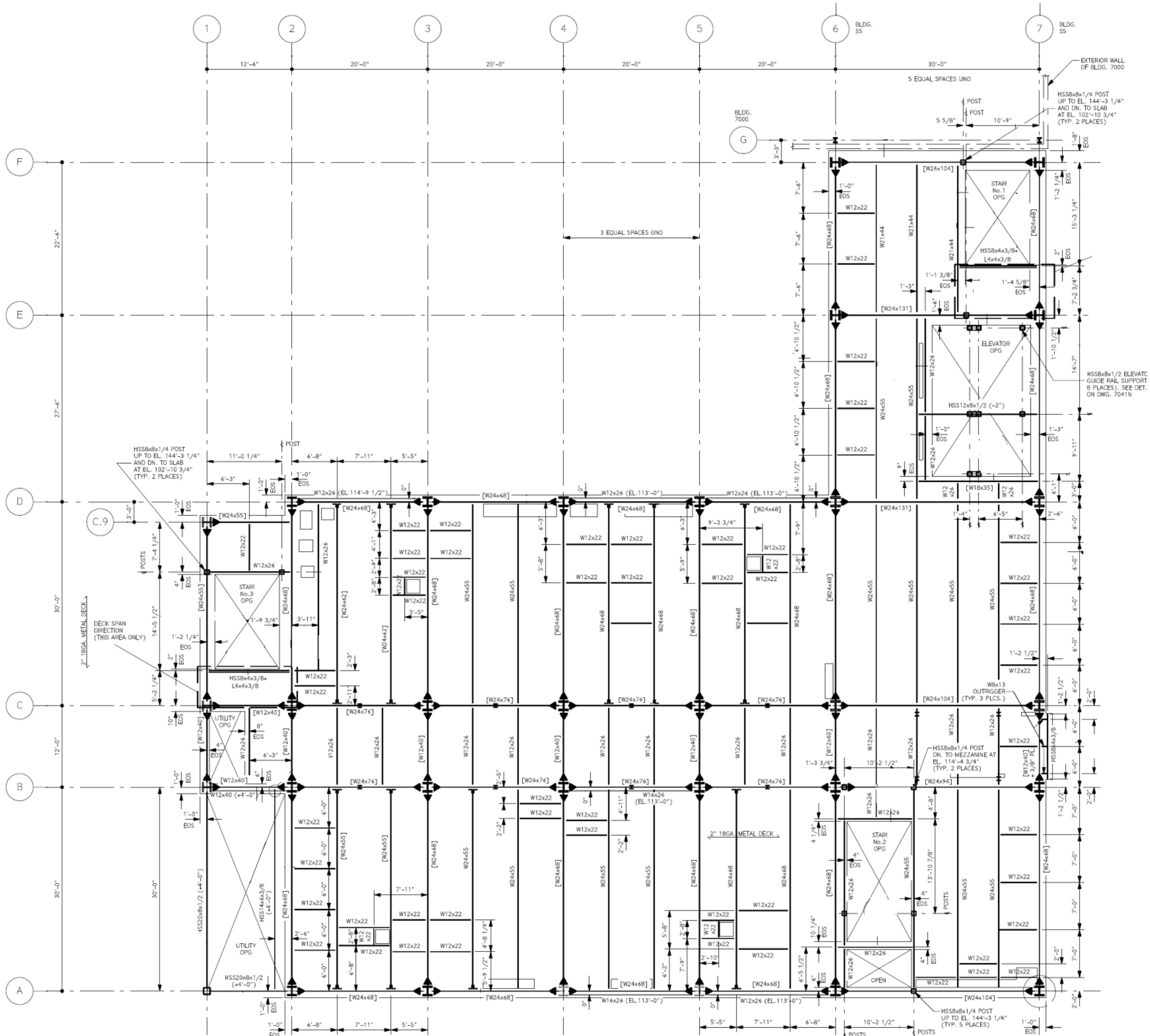


Figure A.1: Courtesy of Project Engineer. The second floor framing plan.

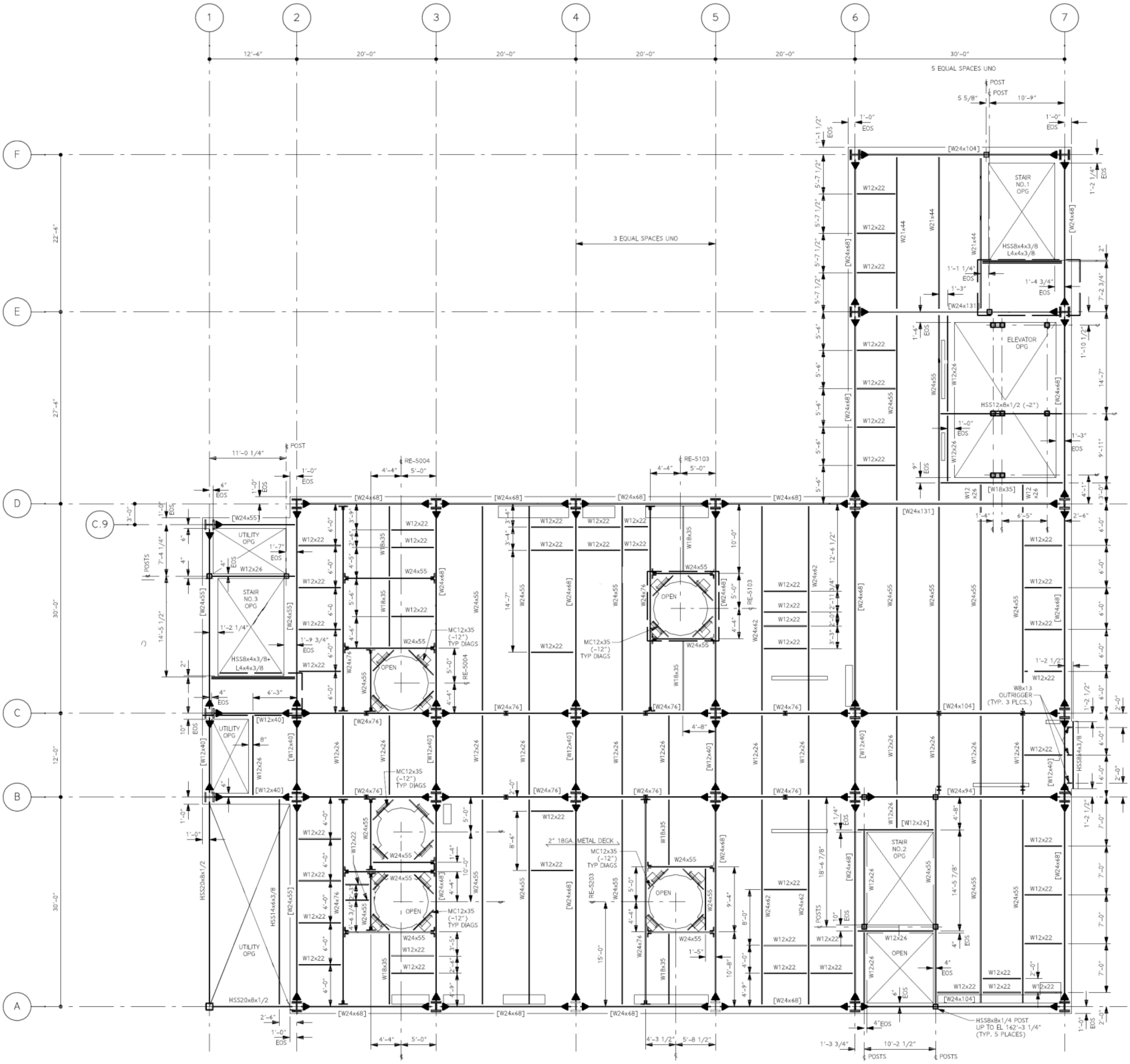


Figure A.2: Courtesy of Project Engineer. The third floor framing plan.

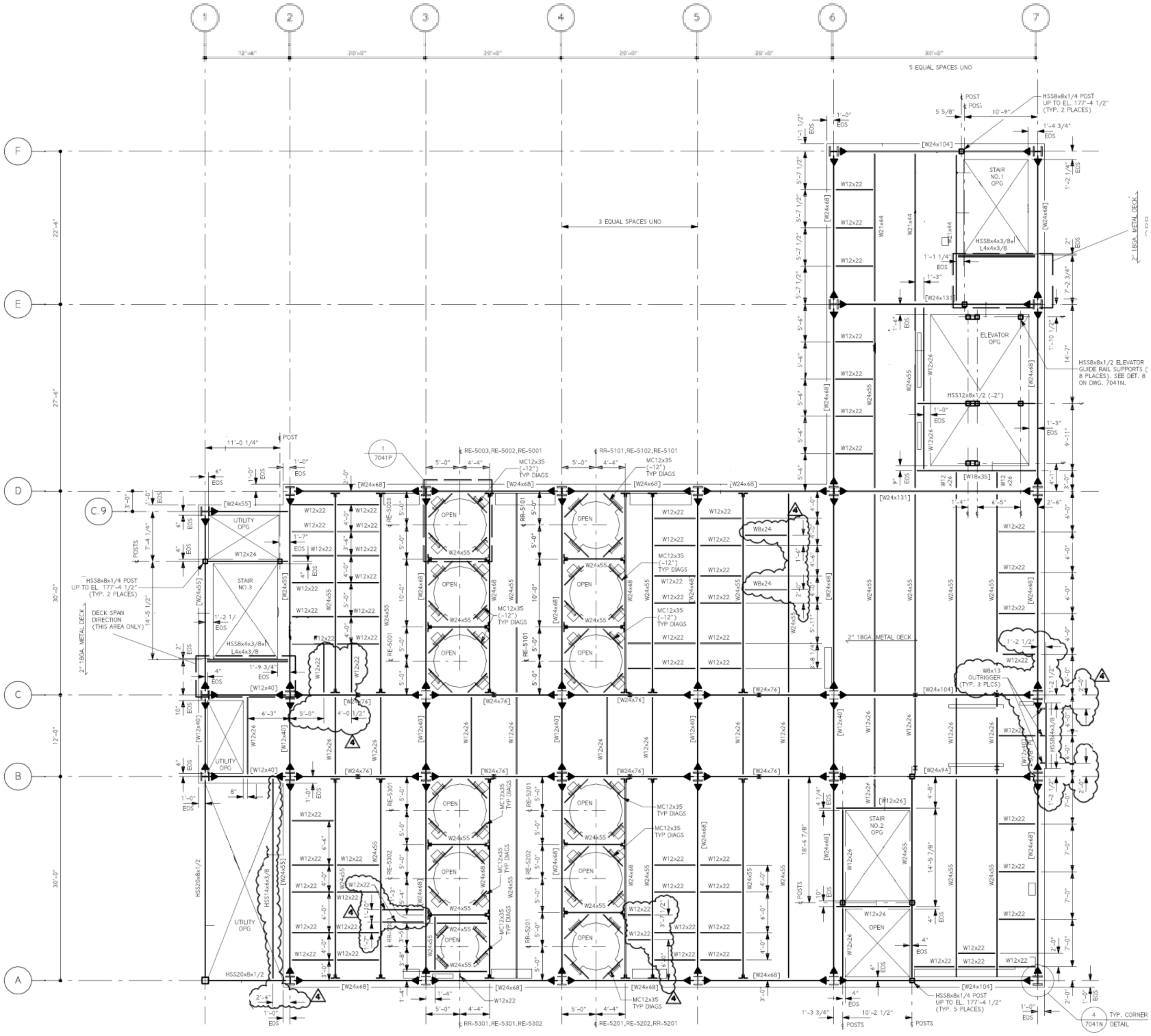


Figure A.3: Courtesy of Project Engineer. The fourth floor framing plan.

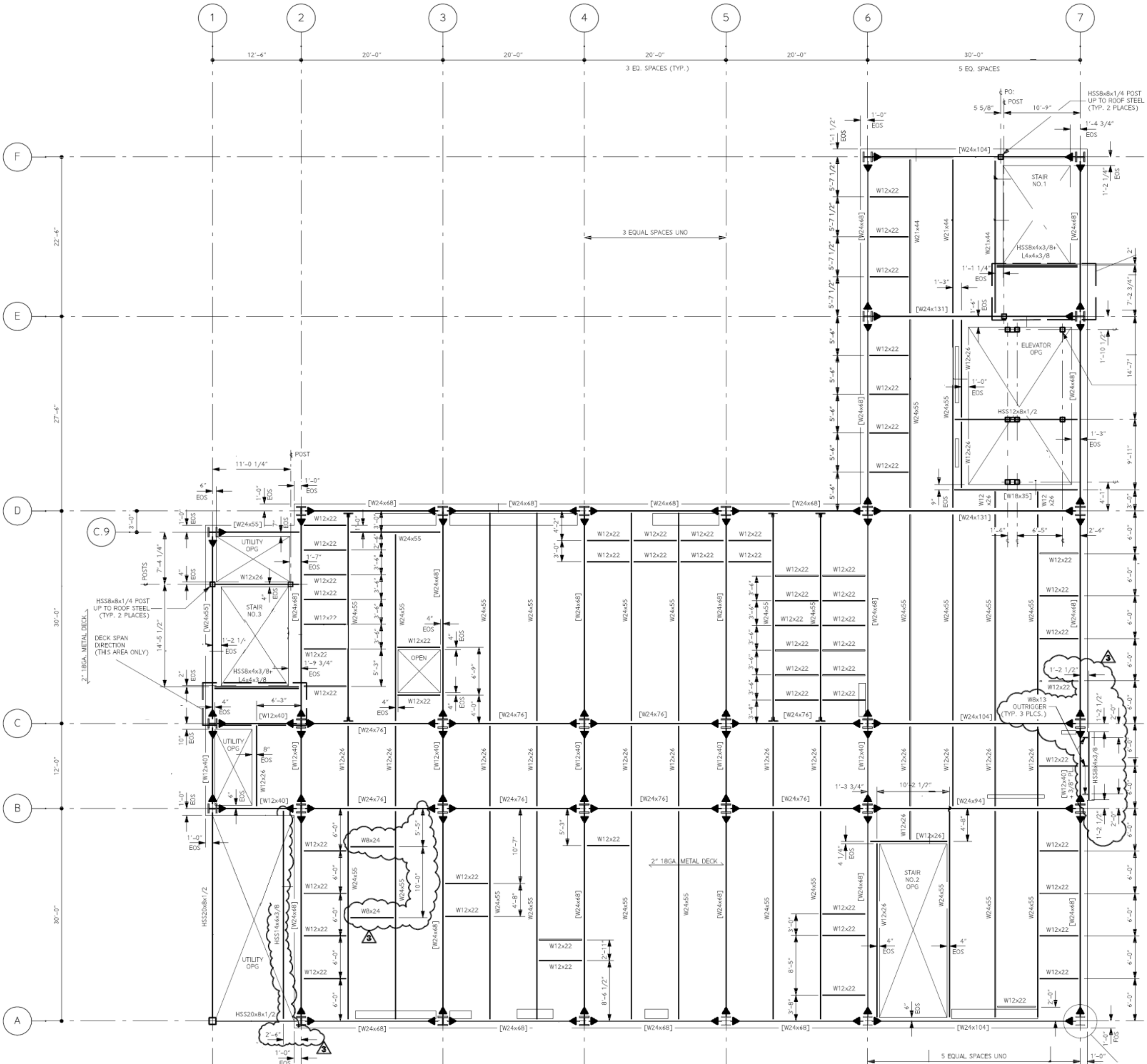


Figure A.4: Courtesy of Project Engineer. The fifth floor framing plan.

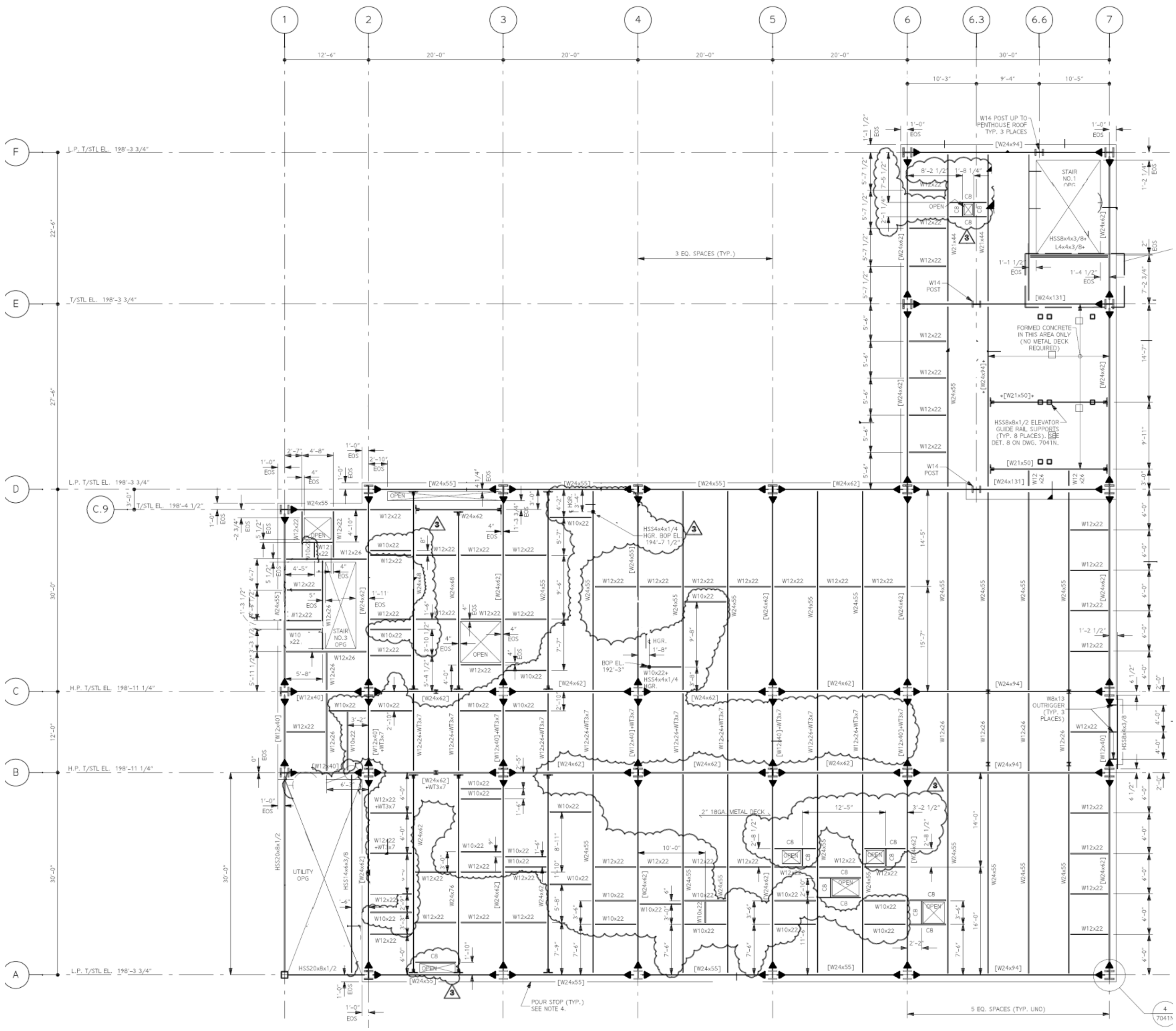


Figure A.5: Courtesy of Project Engineer. The roof framing plan.

Appendix B: Equipment Loads per Floor

The following table is a copy of the table shown in the dead loads section of the main report. These equipment loads are only the loads that exceed the live load for the floor. The following images show the general location of equipment, but are for design purposes only. The equipment numbers assigned in the table correspond to the numbers on the plans.

Equipment Loads per Floor											
First Floor		Second Floor		Third Floor		Fourth Floor		Fifth Floor		Roof Level	
No.	Operational Weight	No.	Operational Weight	No.	Operational Weight	No.	Operational Weight	No.	Operational Weight	No.	Operational Weight
1	47 k	1	31 k	1	44 k	1	44 k	1	11 k	1	20 k
2	56 k	2	31 k	2	40 k	2	25 k	2	3 k	2	102 k
3	50 k	3	27 k	3	36 k	3	23 k	3	6 k	3	126 k
4	25 k	4	27 k	4	51 k	4	23 k	4	2 k	4	26 k
5	58 k			5	21 k	5	51 k	5	2 k	5	11 k
6	36 k			6	23 k	6	44 k				
				7	11 k	7	21 k				
						8	29 k				

Table B.1: Equipment dead loads per floors. These point loads are only the loads that exceed the live load for the floor.

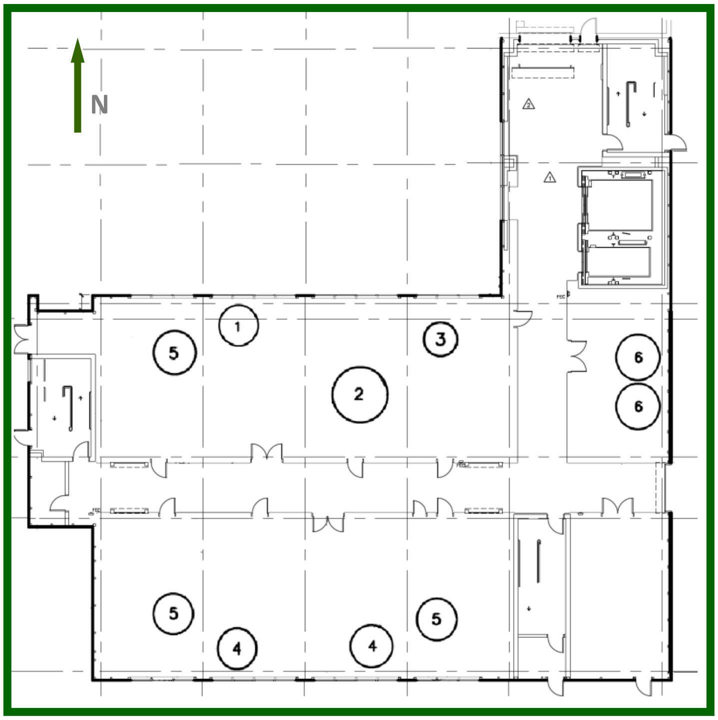


Figure B.1: Equipment dead loads on the First Floor.

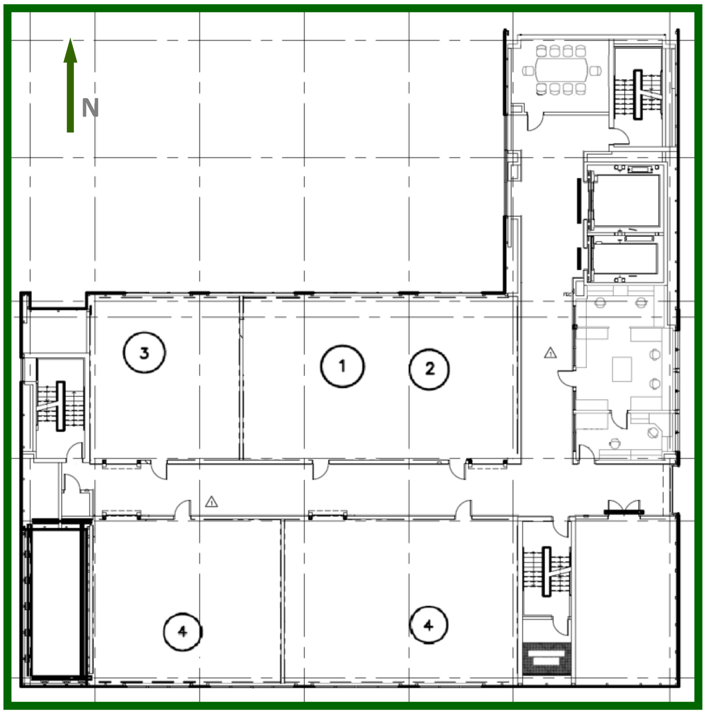


Figure B.2: Equipment dead loads on the Second Floor.

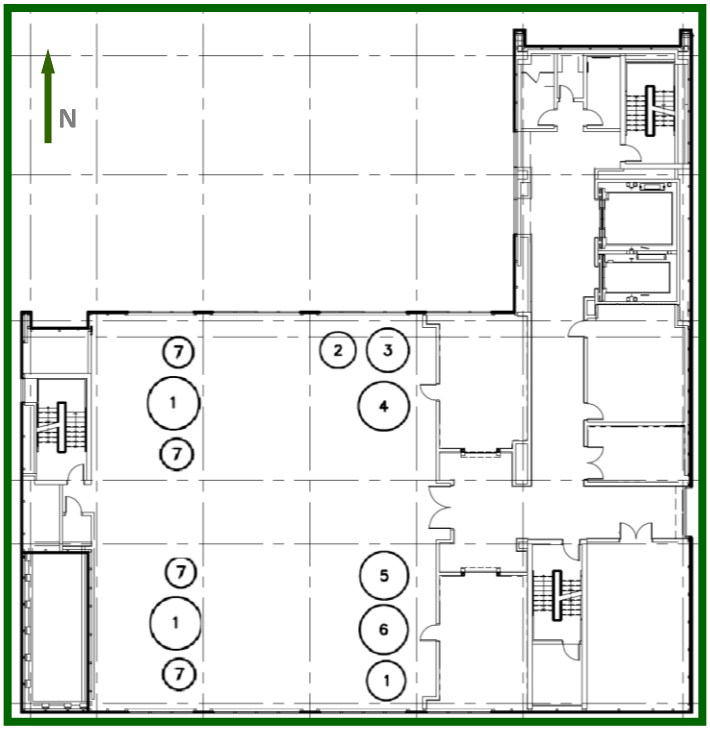


Figure B.3: Equipment dead loads on the Third Floor.

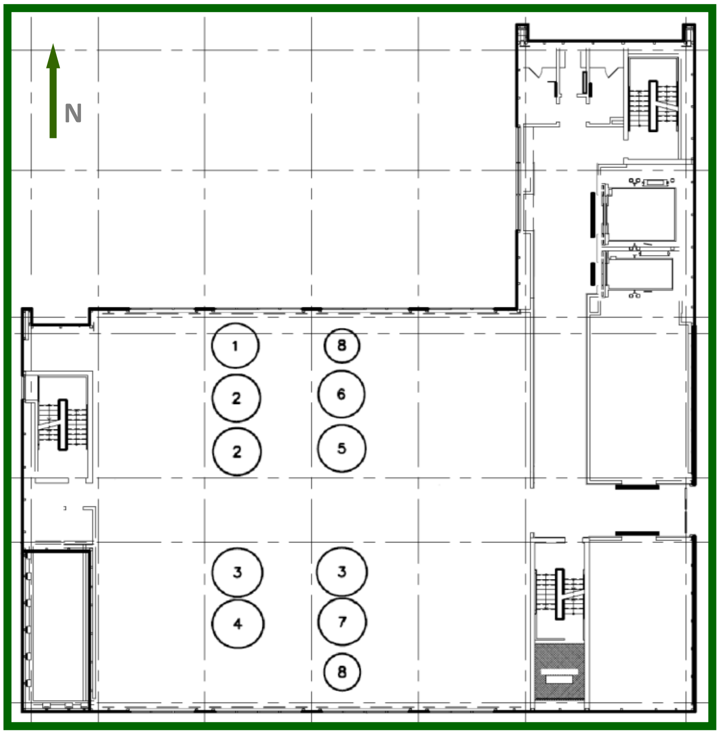


Figure B.4: Equipment dead loads on the Fourth Floor.

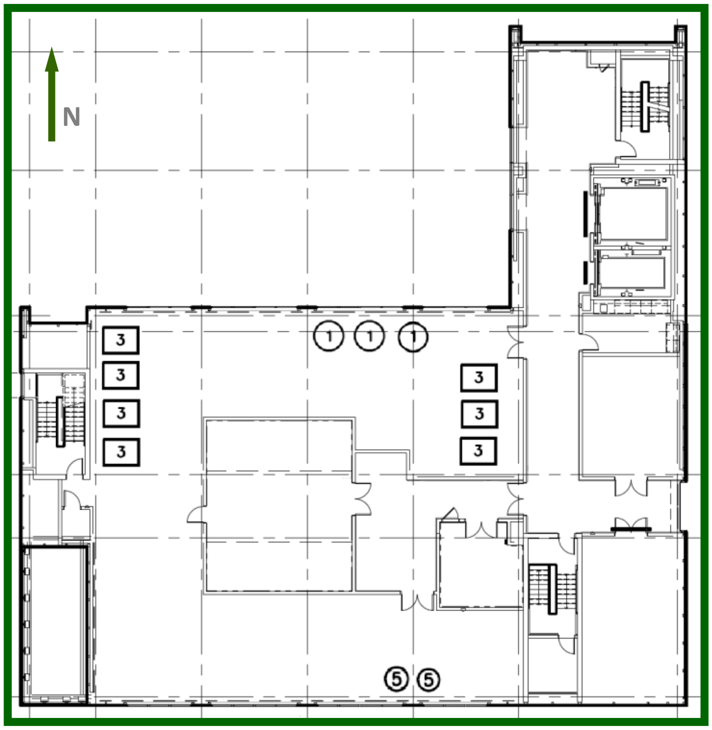


Figure B.5: Equipment dead loads on the Fifth Floor.

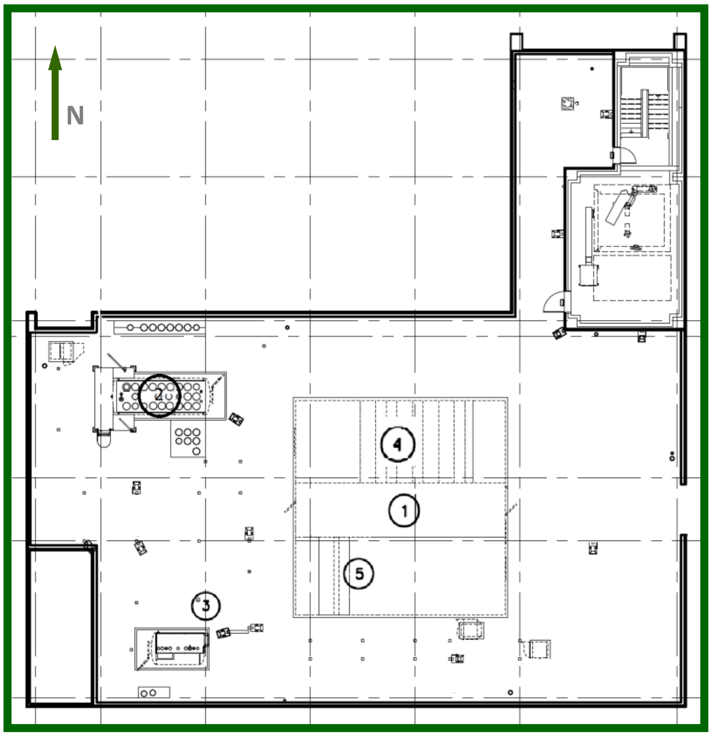


Figure B.6: Equipment dead loads on the Roof.

Appendix C: Snow Load Calculations

SNOW | Christina DiPaolo | 1

Ground Snow Load = 25psf

Flat Roof

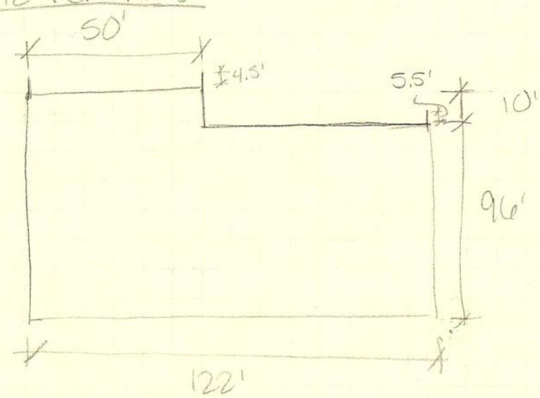
$P_f = .7 C_e C_t I_s P_g = .7(1.0)(1.0)(1.1)(25) = 19.3\text{psf}$

$C_e = 1.0$
(partially exposed + exposure C)

$I_s = 1.10$

$C_t = 1.0$

Drift to Penthouse



$h_b = \frac{P_s}{\gamma} = \frac{19.3}{17.25} = 1.1 > .2$
∴ calculate drift

$\gamma = .13(25) + 14 = 17.25$
(eq 7.7-1)

$l_u = 50\text{ft}$

$W = 4h_d = 4(2.3) = 9.2$

$h_d = 2.3$ (from Fig 7-9)

$P_d = h_d \gamma = 2.3(17.25) = \boxed{39.7\text{psf}}$

4.5' tall parapet of penthouse (1.1 + 2.3 = 3.4' < 4') Don't need to check leeward wind.

Drift on Penthouse due to Parapets

$l_u = 50\text{ft}$

$h_d = 2.3(.75) = 1.73$

$P_d = h_d \gamma = 29.8\text{psf}$

Drift on Roof due to Parapets

$l_u = 122 ft$

$.75\ h_d = [.43 \sqrt[3]{l_u}\ \sqrt[4]{p_g + 10} - 1.5].75$

$= [.43 \sqrt[3]{122}\ \sqrt[4]{35} - 1.5].75$

$= 3.7(.75) = 2.8$

$P_d = 2.8(17.25) = \boxed{48.3\,psf}$

Appendix D: Wind Load Calculations

East-West Wind									
Floor	Elev	h_i	z	k_z	q_z	Windward Pressure, p (psf)	Windward Force (k)	Leeward Pressure, p (psf)	Leeward Force (k)
1	102.9	0	0	0.85	24.5	20.2	29.5	-19.4	-28.4
2	126.9	24	24	0.94	27.1	22.3	54.0	-19.4	-49.7
3	144.9	18	42	1.05	30.2	24.9	51.8	-19.4	-42.6
4	162.9	18	60	1.13	32.5	26.8	56.8	-19.4	-42.6
5	180.9	18	78	1.20	34.5	28.5	60.7	-19.4	-42.6
Roof	198.9	18	96	1.25	36.0	29.6	58.4	-19.4	-39.1
PH	208.9	15	111	1.31	37.7	31.1	11.1	-19.4	-7.3
						Σ =	292.7	Σ =	-224.0
								Base Shear =	516.7 k
								Overturning Moment =	29832.2 k-ft

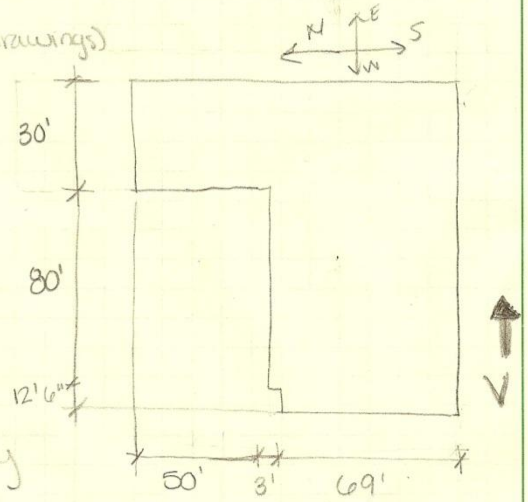
Table D.1: The East-West wind Excel calculations for the windward and leeward pressures and forces per floor level.

North - South Wind									
Floor	Elev	h_i	z	k_z	q_z	Windward Pressure, p (psf)	Windward Force (k)	Leeward Pressure, p (psf)	Leeward Force (k)
1	102.9	0	0	0.85	24.5	20.2	29.6	-19.4	-28.5
2	126.9	24	24	0.94	27.1	22.3	54.2	-19.4	-49.9
3	144.9	18	42	1.05	30.2	24.9	52.0	-19.4	-42.8
4	162.9	18	60	1.13	32.5	26.8	57.0	-19.4	-42.8
5	180.9	18	78	1.20	34.5	28.5	60.9	-19.4	-42.8
Roof	198.9	18	96	1.25	36.0	29.6	58.6	-19.4	-39.2
PH	208.9	15	111	1.31	37.7	31.1	3.3	-19.4	-2.2
						Σ =	286.1	Σ =	-219.8
								Base Shear =	505.9 k
								Overturning Moment =	29954.5 k-ft

Table D.2: The North-South wind Excel calculations for the windward and leeward pressures and forces per floor level.

WIND LOADS

$V = 115$ mph (Figure 26.5-1A)
Exposure C (from drawings)
Importance Factor(?) = 1.15 (from drawings)



$K_d = .85$ (Table 26.6-1)
$K_{zt} = 1.0$ (26.8.2)
$GC_{pi} = \pm .18$
- enclosed building

Approximate Natural Frequency

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \frac{11937}{111} = 107.5$$

$h = 111 \ll 4(107) = 430$ ✓ yes
approx Freq applies

Structural steel moment frame

$$n_a = 22.2/h^{.8} = 22.2/(111)^{.8} = .53 < 1 \quad \therefore \text{Flexible}$$

East-West

Gust Factor

$$G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$g_Q = 3.4$
$g_v = 3.4$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .20\left(\frac{33}{66.6}\right)^{1/6} = .161$$

$\bar{z} = .6h = .6(111) = 66.6 > 15$ ✓ OK

$c = .20$ (Table 26.9-1)

$$g_R = \sqrt{2\ln(3600(.53))} + \frac{.577}{\sqrt{2\ln(3600(.53))}} = 4.035$$

$$Q=\sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar z}}\right)}}=\sqrt{\frac{1}{1+.63\left(\frac{122+111}{570.1}\right)}}=.893$$

$B = 122$

$h = 96$

$$L_{\bar z}=\ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon}=500\left(\frac{63.6}{33}\right)^{.2}=570.1$$

$\ell = 500$ (Table 26.9-1)

$\bar\epsilon = 1/5.0$ (Table 26.9-1)

$$R=\sqrt{\frac{1}{\beta}R_nR_hR_B(.53+.47R_L)}=\sqrt{\frac{1}{.01}(.0775)(.357)(.321)(.53+.47(.112))}$$
$$=.719$$

$$R_n=\frac{7.47N_1}{(1+10.3N_1)^{5/3}}=\frac{7.47(2.53)}{(1+10.3(2.53))^{5/3}}=.0775$$

$$N_1=\frac{n_1L_{\bar z}}{\bar V_{\bar z}}=\frac{(.58)(570.1)}{119.4}=2.53$$

$$\bar V_{\bar z}=\bar b\left(\frac{\bar z}{33}\right)^{\bar\alpha}\left(\frac{88}{60}\right)V=.65\left(\frac{57.6}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)115=119.4$$

$\bar b = .65$ (table 26.9-1)

$\bar\alpha = 1/6.5 = .154$ (table 26.9-1)

$$R_h=\frac{1}{\eta}-\frac{1}{2\eta^2}(1-e^{-2\eta})=\frac{1}{2.16}-\frac{1}{2(2.16)^2}(1-e^{-2(2.16)})=.357$$

$$\eta=4.6n_1h/\bar V_{\bar z}=4.6(.53)(111)/(119.4)=2.16$$

$$R_B=\frac{1}{\eta}-\frac{1}{2\eta^2}(1-e^{-2\eta})=\frac{1}{2.49}-\frac{1}{2(2.49)^2}(1-e^{-2(2.49)})=.321$$

$$\eta=4.6n_1B/\bar V_{\bar z}=4.6(.53)(122)/119.4=2.49$$

$$R_L=\frac{1}{\eta}-\frac{1}{2\eta^2}(1-e^{-2\eta})=\frac{1}{8.37}-\frac{1}{2(8.37)^2}(1-e^{-2(8.37)})=.112$$

$$\eta=15.4n_1L/\bar V_{\bar z}=15.4(.53)(122.5)/119.4=8.37$$

β = damping ratio = .01 for steel building

$$G_f = .925\left(\frac{1 + 1.7(.161)\sqrt{(3.4)^2(.893)^2 + (4.03)^2(.719)^2}}{1 + 1.7(3.4)(.161)}\right) = 1.03$$

K_h

$h = 106$, exposure C $\Rightarrow K_h = 1.31$

Velocity Pressure

$q_z = .00256 K_h K_{zt} K_d V^2 = .00256(1.31)(1.0)(.85)(115)^2 = 37.7$ psf

C_p (Figure 27.4-1)

Windward $C_p = .8$

Leeward $L/B = 122.5/122 = 1.0$ $C_p = -.5$

Side Wall $C_p = -.7$

Roof (flat)

$h/L = .78$, $\theta < 10°$

1.0	-1.3
.5	-.9

$$\frac{1.0 - .78}{1.0 - .5} = \frac{-1.3 - C_p}{-1.3 - -.9}$$

$C_p = -1.124$

Wind pressure

$p = q G_f C_p - q_i \underbrace{(G_{pi})}_{\text{not needed fully enclosed}}$

Windward

$p = q_z(1.03)(.8) = .824\, q_z$ psf

* see table

Leeward

$p = (37.7)(1.03)(-.5) = -18.9$ psf

*repeated for North South wind

Appendix E: Seismic Load Calculations

Floor	Dead Load (psf)	Area (SF)	Equipment PL (k)	Total Weight (k)
1	200	10320	449	2513
2	180	10320	143	2001
3	180	10320	347	2205
4	180	10320	337	2195
5	180	10320	79	1937
ROOF	160	10320	285	1936
Penthouse	20	750	0	15
			Σ =	12801

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

Floor	Total Weight (k)	z (ft)	$w_x h_x^k$	C_{vx}	F_x (k)
1	2513	0	0.0	0	0
2	2000.6	24	120679.3	0.058	30.1
3	2204.6	42	273729.4	0.133	68.2
4	2194.6	60	431688.5	0.209	107.5
5	1936.6	78	534369.9	0.259	133.1
ROOF	1936.2	96	698361.2	0.338	174.0
Penthouse	15	106	6148.0	0.003	1.5
		Σ =	2064976.3	1.0	514.4
			Overturning Moment =		37282 k-ft

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

SEISMIC | ZIP = 23805 | Christina DiPaolo | 1

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER
COMET

$S_s = .155$ (USGS) $F_a = 2.5$ (Table 11.4-1)
$S_1 = .059$ (USGS) $F_v = 3.5$ (Table 11.4-2)

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 (worse of two)

$I = 1.25$ $R = 3.5$ (steel mom. frame)

Equivalent Lateral Force Analysis

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{.26}{\left(\frac{3.5}{1.25}\right)} = .09$

$C_s = \frac{S_{D1}}{T(R/I)} = \frac{.138}{1.08(3.5/1.25)} = .05 \Leftarrow$ Controls

$C_s = .044(.26)(1.25) \geq .01$
$= .0143 \geq .01 < .05$ ✓ OK

$T_a = C_t h_n^x = .028(96)^{.8}$
$= 1.08$
$C_t = .028$ (steel mom frame
$x = .8$ Table 12.8-2)

$T_L = 8$ (Fig 22-12)

$T_a < T_L$

$V = C_s W$

* Weight by floor + total calculated in excel
$W = 12800$ K
$V = .05(12800) = 640^K$

Vertical Distribution of Seismic forces

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ $k = 1.29$ (Section 12.8.3)

* see excel for calculation of F_x

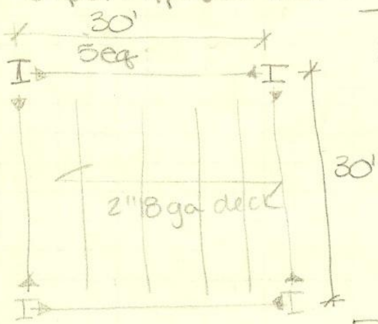
Appendix F: Floor Spot Check Calculations

Floor Check		Christina DiPaolo	1

3rd floor

Live Load = 200psf
Superimposed dead = 98psf

298psf (load greater than 200 contact Vulcraft)



30/5 = 6' ⇨ use $\boxed{\text{2VLI18 w/ 4" NWC}}$

324psf > 298psf

Check unshored length:
3span = 12'-9" >> 6'

also I am ignoring point loads located in other parts of the building and the largest span is 7'-11".

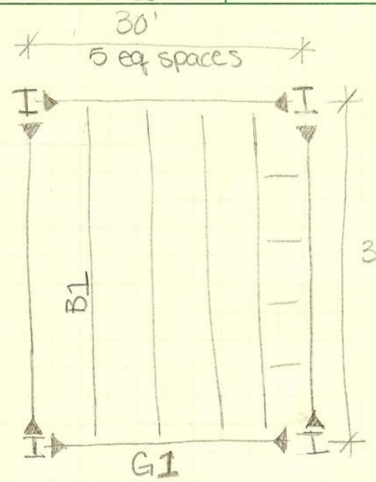
→ 7½" NWC on 2VLI 18 → NOT even in the deck manual

used // 6" NWC on 2VLI8 hold 400psf >> 298psf

Appendix G: Beam and Girder Spot Check Calculations

Beam Calculations

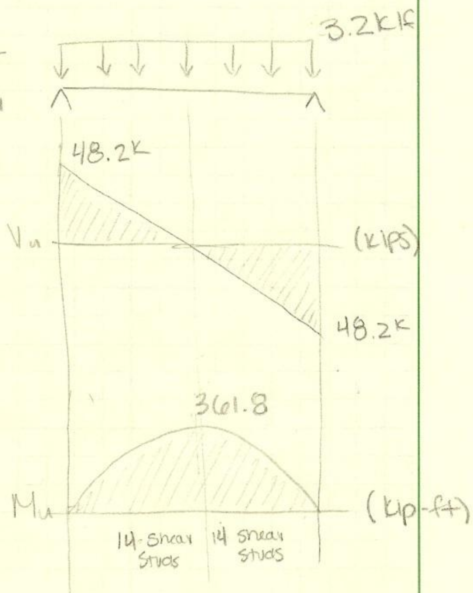
Beam Check | Christina DiPaolo | 1



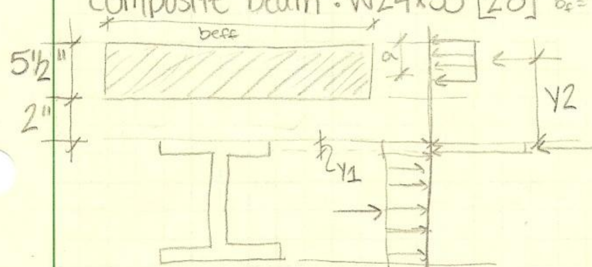
LL = 200
DL = 180
380

(Note: No equipment called out in this area)

$W_u = 1.2D + 1.6L = 536\text{psf}\,(6')$
$W_u = 3216\text{plf} = 3.2\text{klf}$



Composite beam: W24x55 [28]
t = .505
A = 16.2
b_f = 7.01



$Q_n = 21.5^k$ (Table 3-21)
$\Sigma Q_n = 21.5(14)$ 3/4" Φ Studs
$= 301^k$
5" rib → strong position
deck ⊥
f'c = 4000psi (NWC)

$b_{eff} = \min \begin{cases} 30(12)/8 = 45'' \\ 6(12)/2 = 36'' \Leftarrow \text{CONTROLS} \end{cases}$

$V = \frac{wL}{2} = \frac{3.2(30)}{2} = 48.2$

$M = \frac{wL^2}{8} = \frac{3.2(30)^2}{8} = 361.8'k$

$a = \frac{\Sigma Q_n}{.85(f'c)(b_{eff})} = \frac{301}{.85(4)(36)} = 2.5''$

$a/2 = \frac{2.5''}{2} = 1.3''$

$7.5 - 1.3 = 6.2'' = Y2$

Y1

$X = \frac{A_s F_y - \Sigma Q_n}{2(b_f)(F_y)} = \frac{(16.2)(50) - 301}{2(7.01)(50)} = .726 > .505$ in WEB

Table 3-19

Through Interpolation

	$Y1$	ϕM_n
$Y2 = 6"$		
$Y1 = .726"$	.505	916
		910
	3.46	840

$\phi M_n = 912 \gg 362$

$325.6/910 = \boxed{.36}$

Way overdesigned!

<u>My Design</u>

$\Sigma Q_n = 301$

$b_{eff} = 36$

$a = 2.8"$

$Y2 = 6"$

Table 3-19 ⇨ TRY W16x31

$\phi M_n = 393 > 362$

$Y2 = 6"$

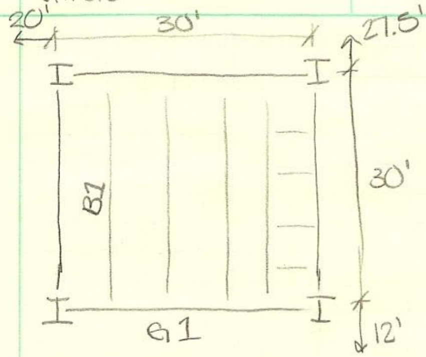
$Y1 = .330$

$$\Delta_{LL} = l/360 = \frac{30(12)}{360} = 1"$$

$$\Delta_{LL} = \frac{5 w_L l^4}{384 E I_{LB}} = \frac{5(.2\,ksf)(6')(30)^4(1728)}{(384)(29000)(1040)} = .73" \checkmark \underline{OK}$$

Girder Calculations

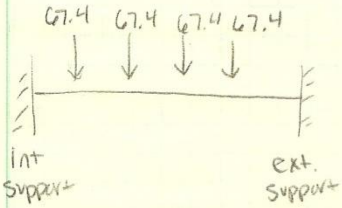
Girder check — Christina DiPaolo — 1



$V = \frac{(3.2klf)(30')}{2} = 48.2$

$V = \frac{(3.2klf)(12')}{2} = 19.2$

$P_u = 48.2 + 19.2 = 67.4$



$(67.4)4 / 30 = 8.99^{klf}$

Using ACI Moment coefficients for end span

↳ span not equal use average

$\ell = 25'$

$M^+ = \frac{w_u l_n^2}{11} = \frac{(8.99)(25)^2}{14} = 401.3'^k$

$M^-_{int} = \frac{w_u l_n^2}{10} = \frac{8.99(25^2)}{10} = -561.9'^k$

$M^-_{ext} = \frac{w_u l_n^2}{16} = \frac{8.99(25)^2}{16} = -351.2'^k$

* Neg moment → doesn't work compositely

UBL = 6'

try W21x68 @ 6'

$\phi M_n = 600 > 561.9$ ✓OK

or W24 for ease of construction

W24x68 @ 6'

$\phi M_n = 644 > 561.9$ ✓OK

They use W24x104 ⇒ $\phi M_n = 1080$ → also part of the lateral system that will increase Moments.

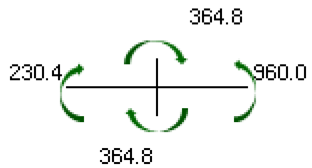
Appendix H: Column Spot Check Calculations

Interior Column

Fifth Floor

Trib Area	525	ft^2	LL =	200	psf
LL=	105	k	DL =	160	psf
DL=	84	k	SL =	19.3	psf
SL=	10.13	k			
Pu=	273.87	k			

$W_{u(TL)}$=	12.8	klf
$W_{u(DL)}$=	4.8	klf
M_{right}=	960	'k
M_{left}=	230.4	'k



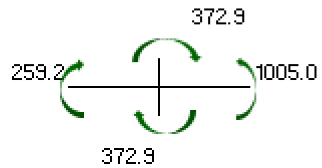
Pu= 273.87
Mu= 364.8

Peq= 760.27

Fourth Floor

Trib Area	525	ft^2	LL =	200	psf
LL=	210	k	DL =	180	psf
DL=	189	k	SL =	19.3	psf
SL=	10.13	k			
Pu=	567.87	k			

$W_{u(TL)}$=	13.4	klf
$W_{u(DL)}$=	5.4	klf
M_{right}=	1005	'k
M_{left}=	259.2	'k



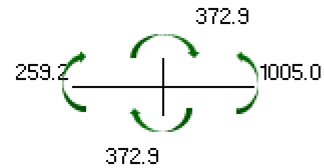
Pu= 567.87
Mu= 372.9

Peq= 1065.07

Third Floor

Trib Area	525	ft^2	LL =	200	psf
LL=	315	k	DL =	180	psf
DL=	283.5	k	SL =	19.3	psf
SL=	10.13	k			
Pu=	849.27	k			

$W_{u(TL)}$=	13.4	klf
$W_{u(DL)}$=	5.4	klf
M_{right}=	1005	'k
M_{left}=	259.2	'k



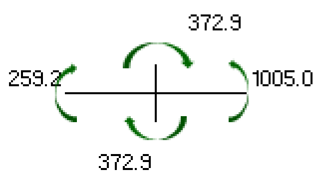
Pu= 849.27
Mu= 372.9

Peq= 1346.47

Second Floor

Trib Area	525	ft^2	LL =	200	psf
LL=	420	k	DL =	180	psf
DL=	378	k	SL =	19.3	psf
SL=	10.13	k			
Pu=	1130.7	k			

$W_{u(TL)}$=	13.4	klf
$W_{u(DL)}$=	5.4	klf
M_{right}=	1005	'k
M_{left}=	259.2	'k



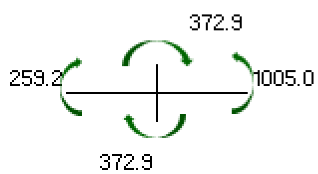
Pu= 1130.7
Mu= 372.9

Peq= 1627.87

First Floor

Trib Area	525	ft^2	LL =	200	psf
LL=	525	k	DL =	180	psf
DL=	472.5	k	SL =	19.3	psf
SL=	10.13	k			
Pu=	1412.1	k			

$W_{u(TL)}$=	13.4	klf
$W_{u(DL)}$=	5.4	klf
M_{right}=	1005	'k
M_{left}=	259.2	'k



Pu= 1412.1
Mu= 372.9

Peq= 1909.27

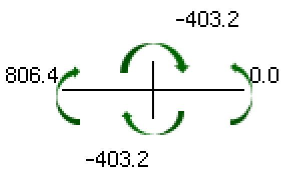
Figure H.1: The Excel calculations for unbalanced moment on the interior column per floor level.

Exterior Column

Fifth Floor

Trib Area	315	ft^2	LL =	200	psf
LL=	63	k	DL =	160	psf
DL=	50.4	k	SL =	19.3	psf
SL=	6.08	k			
Pu=	164.32	k			

$W_{u(TL)}$=	10.752	klf
$W_{u(DL)}$=	4.032	klf
M_{right}=	0	'k
M_{left}=	806.4	'k

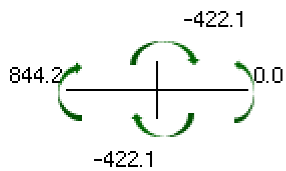


Pu=	164.32
Mu=	403.2
Peq=	701.92

Fourth Floor

Trib Area	315	ft^2	LL =	200	psf
LL=	126	k	DL =	180	psf
DL=	113.4	k	SL =	19.3	psf
SL=	6.08	k			
Pu=	340.72	k			

$W_{u(TL)}$=	11.256	klf
$W_{u(DL)}$=	4.536	klf
M_{right}=	0	'k
M_{left}=	844.2	'k

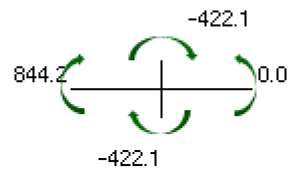


Pu=	340.72
Mu=	422.1
Peq=	903.52

Third Floor

Trib Area	315	ft^2	LL =	200	psf
LL=	189	k	DL =	180	psf
DL=	170.1	k	SL =	19.3	psf
SL=	6.08	k			
Pu=	509.56	k			

$W_{u(TL)}$=	11.256	klf
$W_{u(DL)}$=	4.536	klf
M_{right}=	0	'k
M_{left}=	844.2	'k

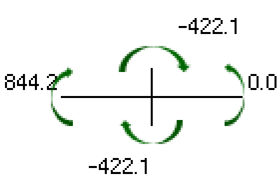


Pu=	509.56
Mu=	422.1
Peq=	1072.36

Second Floor

Trib Area	315	ft^2	LL =	200	psf
LL=	252	k	DL =	180	psf
DL=	226.8	k	SL =	19.3	psf
SL=	6.08	k			
Pu=	678.4	k			

$W_{u(TL)}$=	11.256	klf
$W_{u(DL)}$=	4.536	klf
M_{right}=	0	'k
M_{left}=	844.2	'k

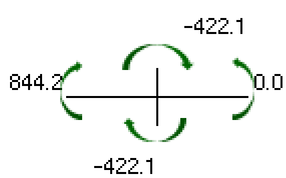


Pu=	678.4
Mu=	422.1
Peq=	1241.20

First Floor

Trib Area	315	ft^2	LL =	200	psf
LL=	315	k	DL =	180	psf
DL=	283.5	k	SL =	19.3	psf
SL=	6.08	k			
Pu=	847.24	k			

$W_{u(TL)}$=	11.256	klf
$W_{u(DL)}$=	4.536	klf
M_{right}=	0	'k
M_{left}=	844.2	'k

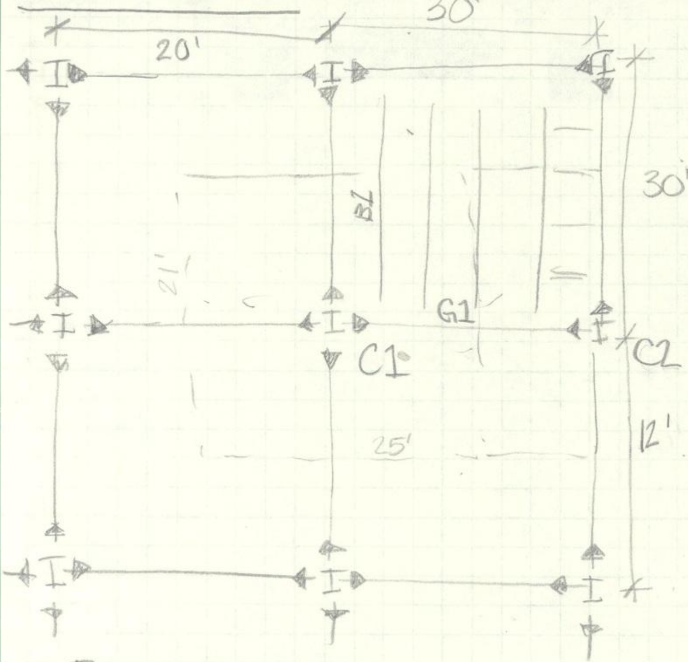


Pu=	847.24
Mu=	422.1
Peq=	1410.04

Figure H.2: The Excel calculations for unbalanced moment on the exterior column per floor level.

Column Check		Christina DiPaolo	1

Interior column (C1)



Typical Floor Plan

$L = 200$

$D = 180$

Note: this area has no additional equipment loads

$A_T = \left(\frac{30}{2} + \frac{12}{2}\right) \times \left(\frac{30}{2} + \frac{20}{2}\right)$

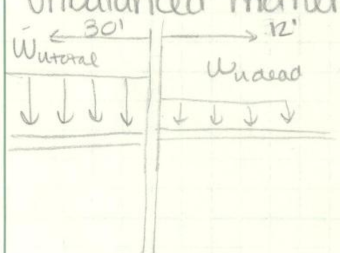
$= 525$

LL Not reducable

$L = 200 > 100$

* all loads tabulated in excel

unbalanced moments (strong axis)

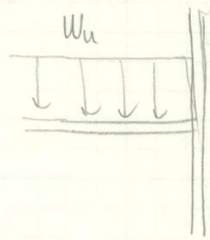


See table from excel

Exterior column (C2)

$A_t = (15) \times \left(\frac{30}{2} + \frac{12}{2}\right) = 315$

trib width $= 15 + 6 = 21$



See table from excel

Both columns checked:

W14x370 for 1st floor + 2nd floor
↳ braced at floor levels ⇒ 24'
$K = 1.0$
$\phi P_n = 3520^k > 1909^k$ ✓OK

combo loading

$p = .284 \Rightarrow \phi P_n = 3520^k$

$b_x = .331 \Rightarrow \phi M_n = 3020'^k$

INT $\quad \Sigma M = \Sigma M_{unbal\,1-5} = 1856$

$\frac{P_r}{P_c} = \frac{1412}{3520} = .4 > .2$

$\frac{1412}{3520} + \frac{8}{9}\left(\frac{1856}{3020}\right) = .95 < 1.0$

EXT $\quad \Sigma M = 2091$

$\frac{P_r}{P_c} = \frac{847}{3520} = .24 > .2$

$\frac{847}{3520} + \frac{8}{9}\left(\frac{2091}{3020}\right) = .86 < 1.0$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

COMET

Column Check | Christina DiPaolo | 3

W14x176 for 3^{rd} through 5^{th} floor

↳ braced at floor levels ⇒ 18'

$K=1.0$

$\phi P_n = 3390^k > 1346^k$ ✓ OK

Combo loading

$p = .295 \Rightarrow \phi P_n = 3390^k$

$b_x = .398 \Rightarrow \phi M_n = 2510'^k$

INT

$\Sigma M = M_{unbal}$ 3-5. $= 1111$

$$\frac{P_r}{P_c} = \frac{849}{3390} = .25 > .2$$

$$\frac{849}{3390} + \frac{8}{9}\left(\frac{1111}{2510}\right) = .64 < 1.0$$ ✓ OK

EXT

$\Sigma M = 1247$

$$\frac{P_r}{P_c} = \frac{510}{3390} = .15 < .2$$

$$\frac{510}{2(3390)} + \left(\frac{1247}{2510}\right) = .57 < 1.0$$ ✓ OK