Report

Final Thesis

Source: Oliver, Glidden, Spina

Source: Oliver, Glidden, Spina

Largo Medical Office Building

Largo, Florida

Disclaimer

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Abstract

Largo Medical Office Building

Largo, Florida

The Largo Medical Office Building (LMOB) is an expansion of the Largo Medical Complex, designed to house a centralized patient check-in area and diagnostic center.

General Scope

Function: Office
Gross Area: 154,240 sq. ft.
As-Built Cost: $12.6 Million (not including equipment)
Dates of Construction: August 2008 — November 2009
Project Delivery Method: Design-Bid-Build
Owner and Developer: The Greenfield Group
Architect: Oliver, Glidden, Spina & Partners
Structural Consultant: McCarthy & Associates
MEP Consultant: Steve Feller, P.E. Inc.

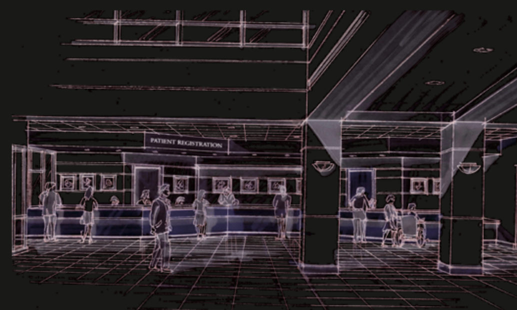


Figure 1.1, Sketch of Patient Check-in Area
Source: Oliver, Glidden, Spina & Partners

Structural Systems

Located in a hurricane zone, LMOB is designed to resist 130 mi/hr. winds. The facility utilizes reinforced concrete shear walls and a steel frame. Reinforced concrete shear walls serve to resist the lateral load and protect emergency egress.

In general, the structural floor system is primarily a 5” thick composite slab. Only the girders are compositely joined to the floor slab. To satisfy the 2-hour fire rating, defined by the 2004 Florida Building Code (FBC), the floor assembly received sprayed cementitous fireproofing,

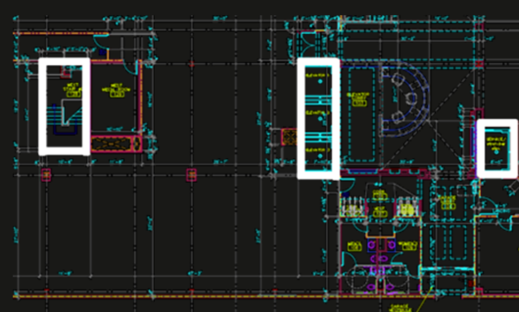


Figure 1.2, Shear Wall Locations
Source: Oliver, Glidden, Spina & Partners

MEP Systems

Primary Cooling: Direct Expansion (DX) with (2) Cooling Towers

Secondary Cooling: **Refrigerant based** Direct Expansion (DX) using Variable Frequency Motor

Diagnostic Equipment Cooling: Air Cooled Chiller

Heating: Resistant Heating Elements located at each floor

Electrical: 480/277V 3 phase - High Voltage
208/120V 3 phase - Low Voltage
GTD20A emergency power relay system

Lighting: Utilize LED and Fluorescent Lighting in conjunction with occupancy and photo-sensors

Figure 1.3, Interior Lighting
Source: Oliver, Glidden, Spina & Partners

Thaison Nguyen
Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2012/TZN106/Website%20Pages/Home/index.html

Executive Summary

The Largo Medical Office Building (LMOB) is a 154,240 ft^2 new medical office building which serves as an expansion of the Largo Medical Complex in Largo, FL. LMOB serves to replace the existing diagnostic center – which will likely be repurposed – and improved and centralized patient check-in. Built in the Fall of 2008 on a Design-Bid-Build contract, the facility incorporates several features not commonly found in other facilities built in Florida. For one, the gravity force resisting system uses structural steel, which is fairly unique for a region dominated by concrete. The lateral force resisting system however, is handled with reinforced concrete shear walls typically located around the emergency stairwells. LMOB's façade is composed primarily of reinforced masonry with a stucco finish. Since LMOB is located in an active hurricane zone, all window glazing is impact resistant.

This report primarily dives into redesigning LMOB's lateral force resisting system. Though the current lateral force resisting system adheres to strength and serviceability code requirements; the facility, in its present state, experiences significant torsional effects when exposed to wind and seismic loads. Should the facility be moved to a more seismically active region then the lateral force resisting elements will need to be redesigned to eliminate torsional irregularity and soft story irregularity. If the lateral force resisting elements are not redesigned then seismic induced damage will occur. One likely damage is the parking garage abutting to LMOB, which will become battered by the damaged and torsionally weak LMOB.

To solve torsion, two redesigns were generally studied and detailed. One lateral system involves adding additional lateral force resisting elements at the facility's perimeter, which became designated Design I. Majority of the original lateral force resisting elements in Design I require redesign arising from lateral load redistribution. As opposed to Design I, Design II eliminates all interior lateral force resisting elements and uses tilt-up walls to carry all the lateral forces to the ground. Surprisingly the controlling loads in Design II occur not during full occupancy but during the wall lifting process. The structural performance, like overall rigidity and resistance to torsion, are better for the redesigns. However, the redesigns are intrinsically complex to construct and carry a heavier financial burden – upwards to one million U.S. dollars (USD) more.

A façade redesign was also implemented to reduce weight, whilst maintaining moisture and thermal performance. The objectives were met, but attempts to reduce cost through using metal stud back-up wall were to no avail. As for acoustical attenuation, the redesign satisfies the recommended performance and had an acoustical performance that was generally similar to the original façade.

Acknowledgements

Before continuing any further, I would like to mention and extend my gratitude to the organizations and individuals for their support in completing this thesis project, this thesis report:

Hedrick Brothers Construction, for helping me search and find the building to do my thesis project. Greatly appreciated for their promptness, their willingness to extend a hand, their character going beyond the defined duty – breaking new ground; I'd like to specifically thank:

Dale R. Hedrick

Rick Ricalton

Oliver, Glidden, Spina & Partners; for providing the building, permission, and patience; especially:

Andrea Kokinakes

Eileen Trimble

My family, as well as the entire Architectural Engineering faculty and student body – including those who are no longer here; for their spirit, their company, their charisma; I'd like to call out:

Dr. Richard A. Behr

Dr. Andres Lepage

Cheuk Tsang

Corey Wilkinson

Patrick Zuza

Though these organizations and individuals may mean nothing to those reading this thesis report and history books would likely never mention their name; but they have gained my gratitude, my respect. All that I can say right now is that a day will come when I will pay my debt, a debt I owe to them.

Building Introduction

Largo Medical Office Building (LMOB) is an expansion of the Largo Medical Center complex. Designed in 2007 and completed in 2009, LMOB is managed and constructed by The Greenfield Group. Overall the project cost $12.6 million, not including the equipment. The design-bid-build facility is centrally positioned in the medical complex and is adjacent to the parking garage, as shown shaded red in Figure 1.1.

Located in Largo, Florida the six story facility was designed to house improved and centralized patient check-in area. The facility also houses office space for future tenants, as well as screening and diagnostic equipment. Office spaces for future tenants are not fitted out until the management agency signs a contract with the potential tenants.

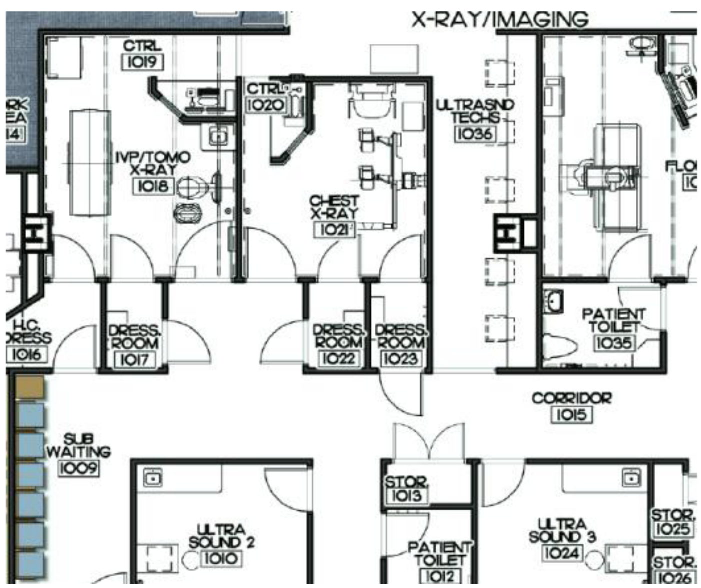


Figure 1.1, Building Location
Source: Google Maps

Figure 1.2, Partial Floor Plan
Source: Oliver, Glidden, Spina & Partners

Patient privacy is a major concern for facilities housing medical related activities. Oliver, Glidden, Spina & Partners answered this by clustering the screening and diagnostic spaces close to the dressing areas (Figure 1.2). The architect went a step further, to preserve privacy by compartmentalizing the building's interior.

The building's façade primarily consists of stucco finished CMU. All CMUs are grouted and reinforced, to resist hurricane force winds. Likewise, the façade's glazing is impact resistant. To enhance the architecture, LMOB uses an exterior insulation finish system (E.I.F.S.) to create architectural moldings. The other architectural feature of the building is the overhang over the building's north entrance. Both the stucco finished CMU and E.I.F.S. can be seen in Figure 1.3.
All three roof levels – main roof, east emergency stairwell roof, and the overhang – use one roof type, consisting of a 3-ply bituminous waterproofing applied over the insulated cast-in-place

concrete (Figure 1.3). To ensure adequate rainwater drainage, the insulated cast-in-place concrete is sloped ¼” for every 12” horizontal.

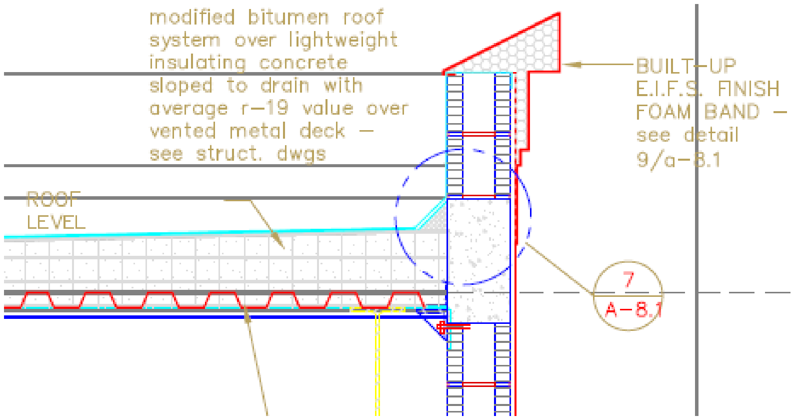


Figure 1.3, Wall and Roof Detail
Source: Oliver, Glidden, Spina & Partners

The insulated cast-in-place concrete was used in-lieu of rigid insulation with stone ballast. One reason is that the facility is in a hurricane zone, where loose material can potentially become airborne projectiles and cause damage when there is a hurricane. The insulated concrete has sufficient mass to resist becoming airborne in a hurricane.

Figure 1.4, Perspective View of Exterior
Source: Oliver, Glidden, Spina & Partners

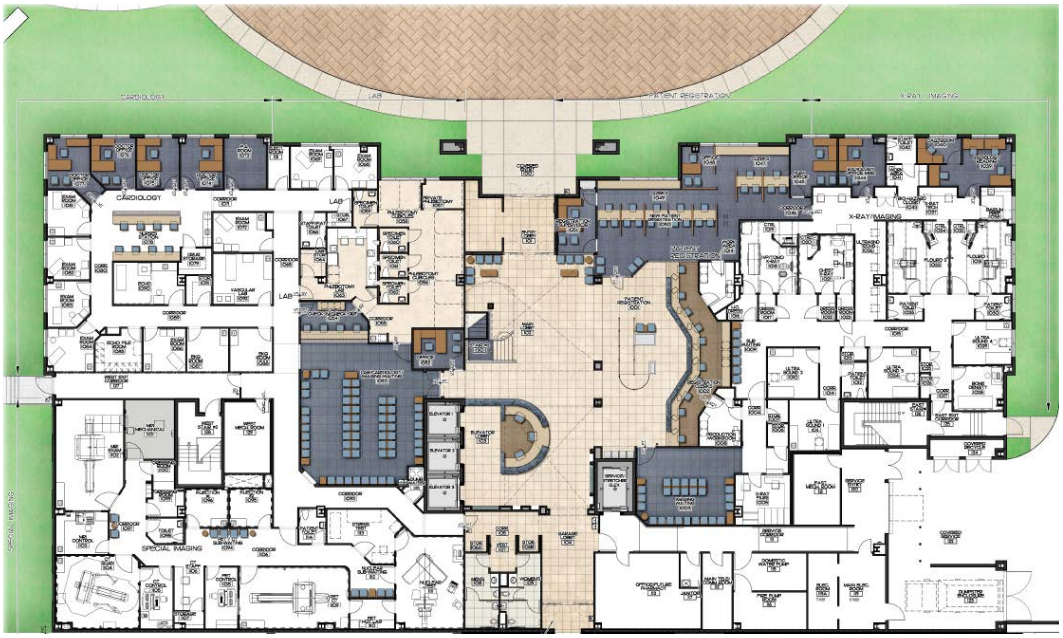


Figure 1.5, Illustrated Floor Plan
Source: Oliver, Glidden, Spina & Partners

LMOB is a steel framed facility with ordinary reinforced concrete shear walls to resist lateral loads. The structural consultant for LMOB is McCarthy & Associates. Shear walls are all located next to the elevators and emergency stairwells – to reduce impact on floor layout. All columns and shear walls rest on top of spread footings which are at least 27 in. below grade. As oppose to the primary gravity and main lateral force resisting system, the building's façade sit on top of strip footings. The internal bay sizes are generally smaller than the exterior bays. Increase exterior bay size is the result of architectural extrusions – shown above in Figure 1.4 and Figure 1.5 – in the facility's façade, primarily at the corners and entrances.

Existing Structural System

Design Codes

When designing the original LMOB structural engineering consulting firm, McCarthy and Associates, used the following codes and standards:

1. 2004 Florida Building Code (FBC)
   – Adoption of the 2003 International Building Code (IBC)
2. 13th Edition AISC Steel Construction Manual
3. Design Manual for Floor and Roof Decks by Steel Deck Institute (SDI)
4. ACI 318-05

Gravity Frame and Floor System

The steel frame is organized in the typical rectilinear pattern. Internal bay sizes are generally 30'-0" square, typical size for most facilities, but the exterior bays are 33'-0" square. Please see the appendix for typical plans and elevations. It was assumed that the columns, girders, and beams are fastened together by bearing bolts, as shown in Figure 1.6 – located below. A consequence of the assumption is that the steel frame only carries gravity loads.

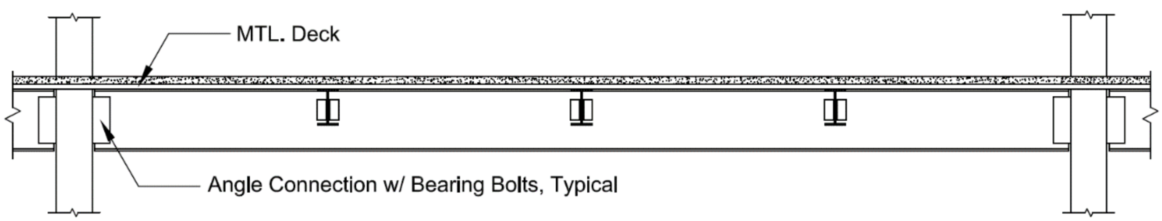


Figure 1.6, Typical Framing

Table 1.1, List of Structural Steel Used in LMOB	
Profile	Steel Type and Grade
W-Shapes	ASTM A992 Gr. 50
Angles	ASTM A36
Plates	ASTM A36
Reinforcing Bars	ASTM A615

As a note, many assumptions were made concerning the original structural system due to confidentiality on part of the owner and engineer of record.

Base on architectural plans and calculation spot checks building uses W12 columns throughout, W24 girders, and W16 beams. Table 1.1 shows the steel type and grades which were used in the original structure. Girders act compositely with the slab through shear studs, ¾" diameter. This composite action results in reduced structural floor depth. In order to reduce complexity the structural engineers ran most girders in the East/West (longitudinal) direction. Only unique conditions such as the overhang above the lobby entrance and loading area are girders are orientated differently.

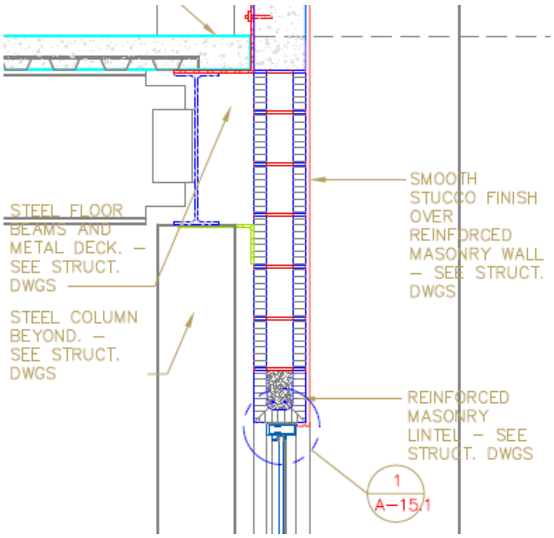


Figure 1.7, Typical Composite Slab Detail
Source: Oliver, Glidden, Spina & Partners

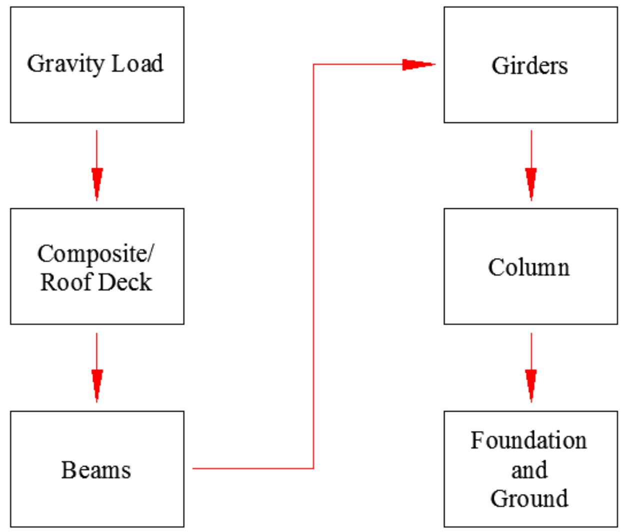


Figure 1.8, Gravity Load Distribution

The structural flooring system is primarily a 5" thick composite slab and spans 8'-3", except for the ground. Figure 1.7 shows the primary composite slab. In order to satisfy the 2-hour fire rating defined by the FBC, it is likely that the floor assembly received a sprayed cementitous fireproofing. The available architectural documents show an exposed 2" composite deck with 3" of normal weight (NW) topping. According to the 2008 Vulcraft Decking Manual, the shown system only has a 1.5-hour rating.

Gravity load distribution path through the gravity frame and floor system can be followed in Figure 1.8. As for the spot checks mentioned earlier, they can be found in the appendix.

Lateral Force Resisting System

Lateral load are handled by the building's ordinary reinforced shear walls. The shear walls help the facility resist wind from the North/South and East/West direction. All shear walls are 8" thick and continuously span from the ground floor level to the primary roof (86' above ground floor level). Figure 1.9 shows shear wall locations and the respective naming designation. Lateral load travels through the building starting at the building's façade, which then transfers to the floor diaphragm and collector elements. Then the lateral loads get transferred to the shear walls and finally to the ground.

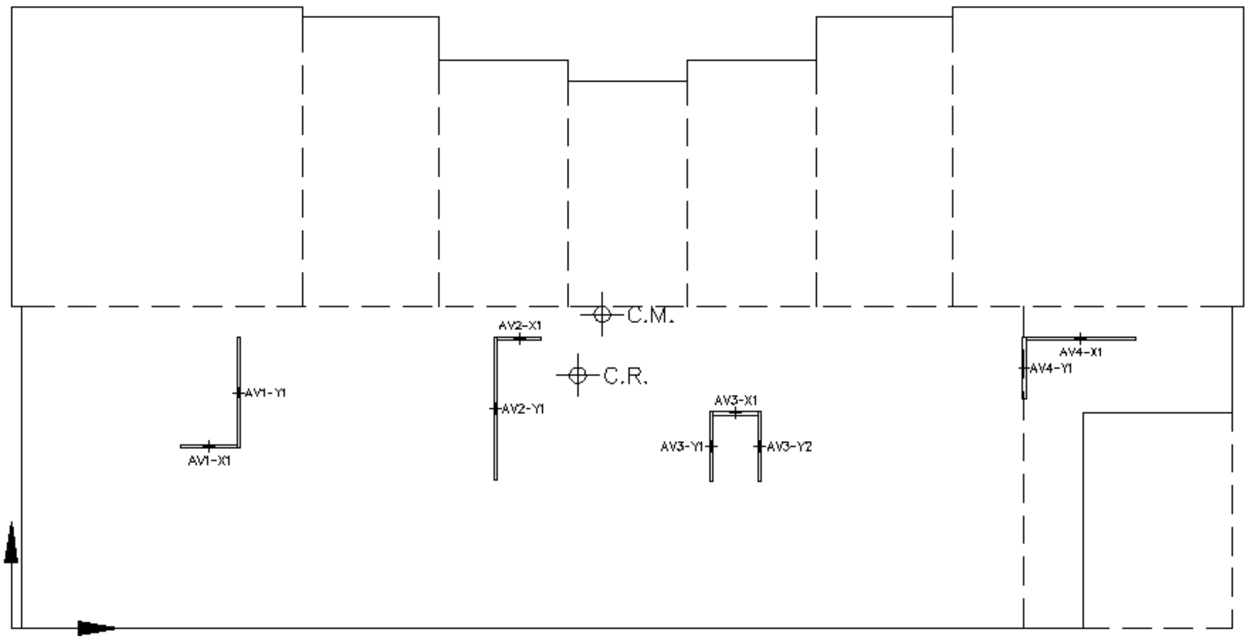


Figure 1.9, Shear Wall Locations

Rebar arrangement in the shear walls weren't provided, due to confidentiality issue mentioned earlier. The lack of information resulted in general design of the rebar in shear walls which may not coincide with the actual rebar configuration. General design of rebar in a shear wall for the original structure can be found in the appendix.

Large eccentricities between the center of mass and center of rigidity resulted in generally large torsion impact. Based on Table 12.3-2 in ASCE 7-05, the large torsion impact is a manifestation of torsional irregularity. In addition, increase floor-to-floor height of the first story creates a slight stiffness reduction. The result is a soft story irregularity at the first story and a slight shear reversal. Though it may appear that these irregularities are of little impact in a wind load dominated region, they are significant in affecting the building's maximum drift – especially if a neighboring building abuts to it and/or some lateral resisting elements are damaged. The irregularities are also significant should The Greenfield Group decide to expand operations into a more seismically active region and use a similar design then the building will have to be redesigned.

Loads on Building

Gravity Loads

Table 1.2, Weight of Building Materials		
Material	Weight	Reference
Normal-Weight (NW) Concrete	150 lb/ft^3	AISC 14th Edition – Table 17-13
Light-Weight (LW) Concrete	113 lb/ft^3	Arch. Graphics Standards 11 Edition
Vinyl Composition Tile (VCT)	1.33 lb/ft^2	Arch. Graphics Standards 11 Edition
Ceramic/Porcelain Tile	10 lb/ft^2	AISC 14th Edition – Table 17-13
3-Ply Roofing	1 lb/ft^2	AISC 14th Edition – Table 17-13
0.8” Laminated Glass	8.2 lb/ft^2	Assumed
MEP	15 lb/ft^2	Assumed

Table 1.3, Unfactored Dead Load	
Floor Level	Load (kip)
Ground	2425.2
1	3325.7
2	3289.7
3	3289.7
4	3289.7
5	3289.7
Roof	3248.9

Before beginning any design it is necessary to understand the various loads which act on the building. Table 1.2 contains the unit weight of the building materials used in the determination of the unfactored dead load at each floor level. To account for the unforeseen items, a collateral load of 5 lb/ft^2 was incorporated into the total unfactored dead load. Table 1.3 shows the determined total un-factored dead load by floor level, not including the self-weight of structural steel. Further calculation details concerning the unfactored dead load can be found in the appendix.

The total unfactored dead load, in Table 1.3, will change if the following assumptions aren’t respected:

1. Metal deck has equal rib volume
2. Glazing and concrete are the only façade materials
3. All floors except for the roof use the same type of concrete

Based on the 2009 IBC, LMOB is classified as a type B occupancy. The result of this classification is the use of office live loads. Another live load used to analyze the gravity system is emergency egress like stairwells and corridors. Below is Table 1.4 showing the live loads recommended by ASCE 7-05 and used to determine the total unfactored live load.

Table 1.4, Typical Live Loads	
Description	ASCE 7-05
Stairs	100 lb/ft^2
Lobby & First Floor Corridor	100 lb/ft^2
Corridors Above First Floor	80 lb/ft^2
Ordinary Flat Roofs	20 lb/ft^2
Partitions	15 lb/ft^2

The predominate code allowed for a reduction in the live load, however the option to use live load reductions was not implemented. One reason is that there is the likelihood that the busy hospital will expand its use of facility. Already the hospital occupies 39700 ft^2 of LMOB and has added a parking garage to accommodate additional patients. Another reason, it is likely that the facility will incorporate new equipment, un-foreseen by the designers, in the future.

Table 1.5, Unfactored Live Load	
Floor Level	Load (kip)
Ground	2313.6
1	2001.7
2	2103.9
3	2103.9
4	2103.9
5	2103.9
Roof	528.8

Table 1.5 is a tabulation of the total unfactored live loads acting on the gravity structural system. Similar to the dead loads, detailed calculations can be found in the appendix.

Moving on to rain and snow loads, the location of LMOB is the deciding factor in whether rain or snow loads controlled. Being that the facility is in Largo, Florida it generally doesn't snow. This is confirmed by Figure 7-1 in ASCE 7-05 which indicates that the ground snow load is zero. The result is rain loads control. Rain load was determined through the use of ASCE 7-05 and the International Plumbing Code (IPC). A ponding instability investigation was not required by ASCE 7-05, because the roof slope is a 1/4" rise for every 12" horizontal. Thus there was no study of ponding potential on the roof.

The hourly rain rate for Largo, Florida wasn't in the standards; the closest city's hourly rain rate was used. Tampa, Florida is the closest city to Largo, Florida. Calculations indicate that the rain load is 27.89 lb/ft^2. It was determined that the rain load is greater than the live roof load. Since the rain load is controlling it was used in lieu of the live roof load to check the gravity structural system.

Wind Load

Wind loads acting on LMOB are based on Method 2 in Chapter 6 of ASCE 7-05. When using the previously mentioned methods there are two classes of wind loads – those acting on the Main Wind Force Resisting System (MWFRS) and those acting on the Components & Cladding (CCL). Story forces and overturning moments were derived by calculating the wind pressures and loads.

Assumptions that were made to simplify method 2 are as follows:

1. Ignore the canopy
2. Due to multiple roof levels, that average roof elevation 95'-6" was utilized
3. Internal pressurization is unlikely due to use of impact resistant glazing
4. Type III for importance category

MWFRS wind loads in the North/South direction controls over the East/West direction. MWFRS Greater wind loads on the North/South building sides can be attributed to greater façade area. Detailed wind calculations and site characteristics are available for reference in the appendix. Shown below in Figure 1.10 to 1.13 are the MWFRS wind distribution and story shears acting in the cardinal directions.

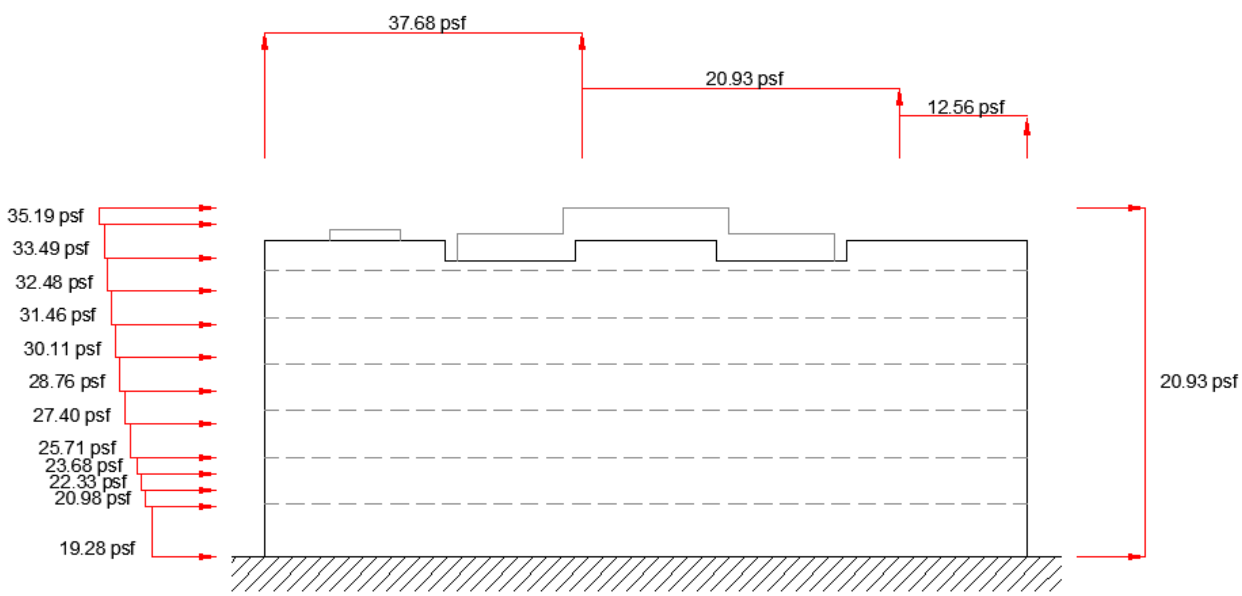


Figure 1.10, MWFRS East/West Wind Load Distribution

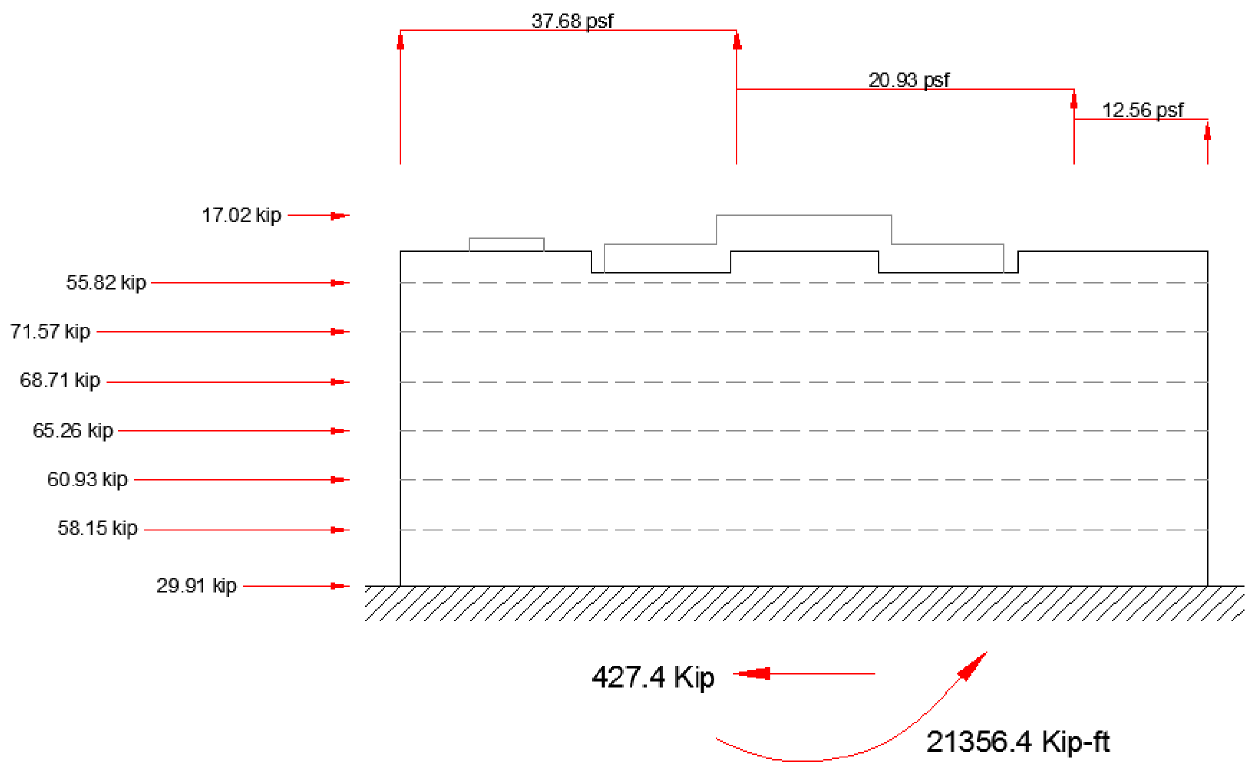


Figure 1.11, MWFRS Loads - East/West

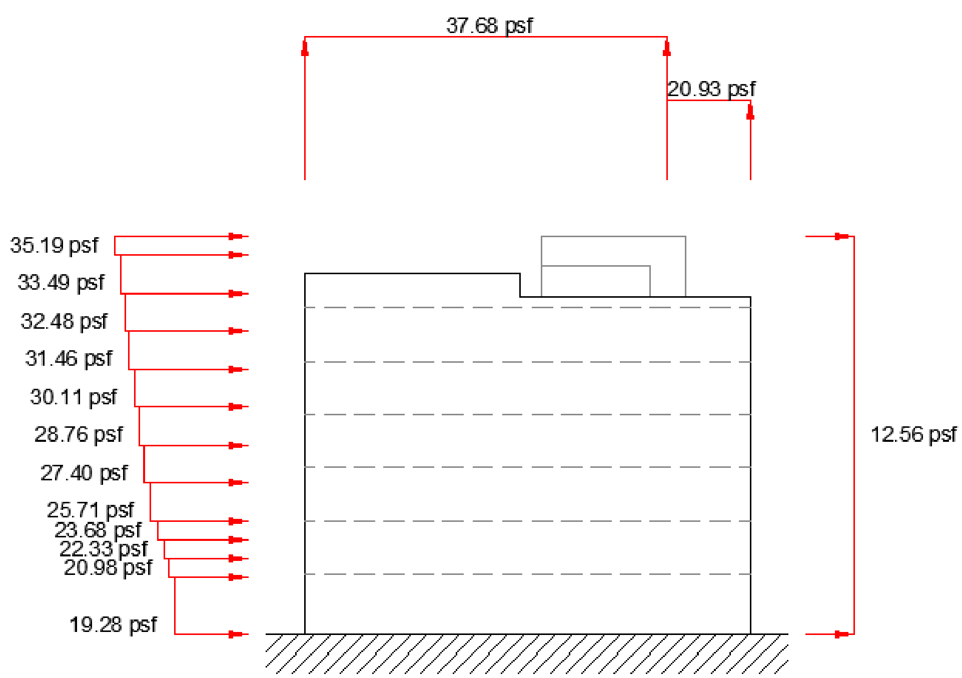


Figure 1.12, MWFRS North/South Wind Load Distribution

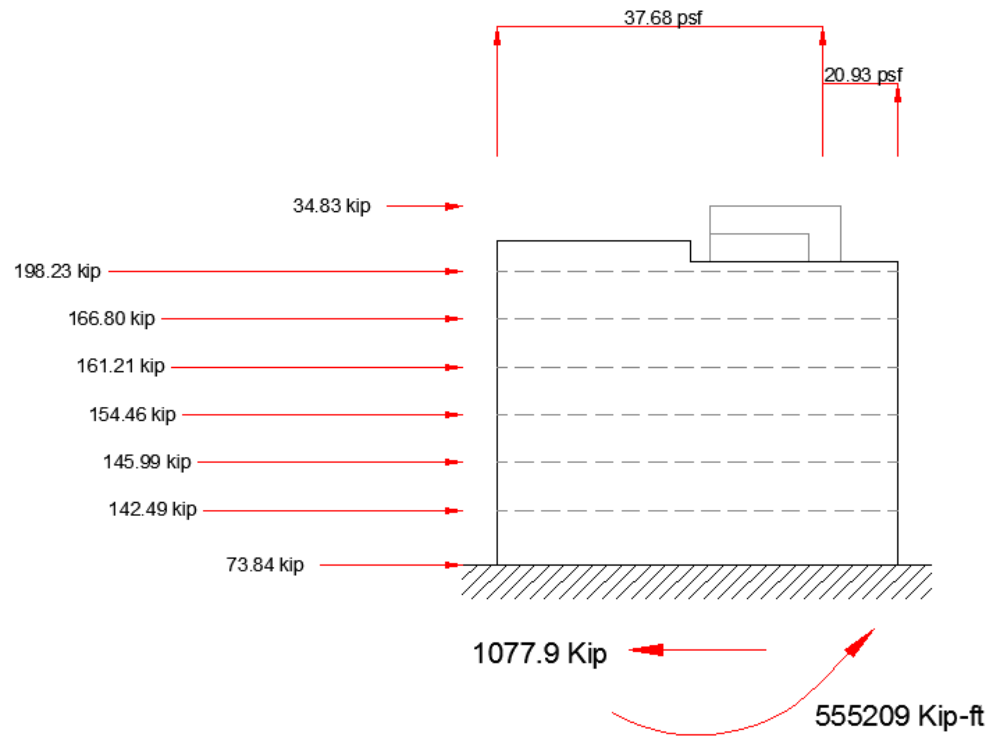


Figure 1.13, MWFRS Loads - North/South

Seismic Load

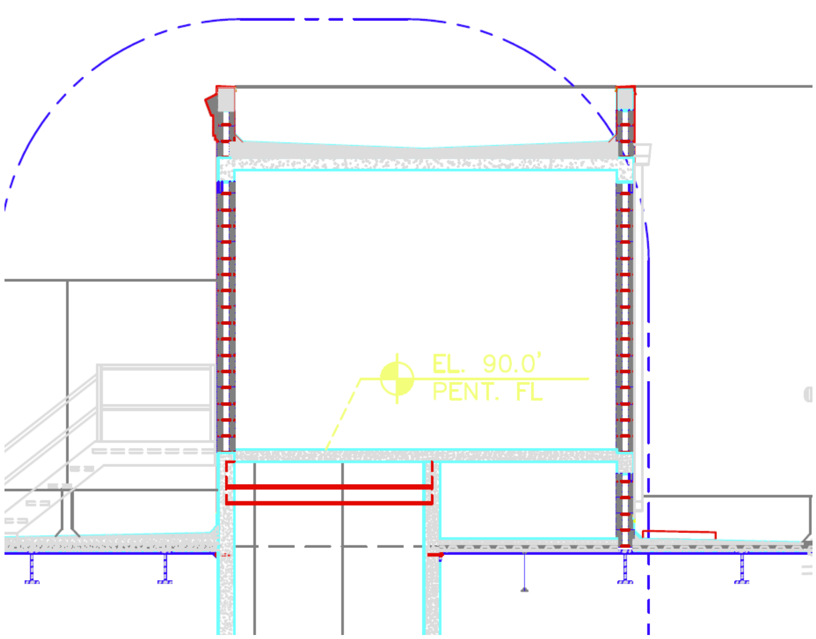


Figure 1.14, Non Seismic Design Top Roof
Source: Oliver, Glidden, Spina & Partners

Equivalent Lateral Force method was used to determine the seismic loads on LMOB. The seismic load, an inertia load, is caused by ground acceleration. Seismic load transfers from the floor diaphragms to the shear walls. The shear walls enclose the emergency stairwells and elevator core, an illustration of the shear wall locations are highlighted black in Figure 1.9. No seismic loads were transferred to the top roof, at 105’, due to the lack seismically designed masonry structure supporting the diaphragms (Figure 1.14).

Table 1.6, Effective Weight	
Floor Level	Level Effected Weight (kip)
Ground	0
1	3826.1
2	3891.6
3	3836.6
4	3770.4
5	3764.2
Roof	3381.1

When using ASCE 7-05 it was discovered that the facility doesn’t experience significant seismic forces and are only 1.0% of the effective building weight. The small seismic force means that the wind loads control in Largo, FL. Gravity loads determined previously were used to calculate the effective building weight – which can be referenced in greater detail in the appendix. Table 1.6, describes the effective building weight by floor level.

Scope of Study

Largo Medical Office Building (LMOB) satisfies strength and serviceability requirements. This was confirmed in Technical Reports I and III. As mentioned earlier, the center of rigidity (CR) and center of mass (CM) don't coincide. Eccentricity between the CR and CM is caused by concentrating the shear walls in the southern half of the building. In the current shear wall arrangement there is torsional irregularity.

Facilities in Largo, FL are governed by wind loads, as opposed to seismic loads. If the facility remains in a Florida, there is no need to rectify the seismic irregularities. However, under the current scenario, LMOB's owners intend to aggressively expand their operations beyond Florida to more seismically active regions of the U.S. in the future. With foresight the owners plan to minimize general logistics, maintenance, and repair costs through using similar building layout and systems. In order to use a similar layout, LMOB's structure will need to be revised to eliminate ASCE 7-05 code defined torsional irregularity and soft story irregularity. Both which create significant structural weakness when the building is exposed to significant seismic loads.

Two design solutions will be considered to eliminate torsional irregularity and soft story irregularity. These solutions focus on increasing resistance to torsion and reinforce the soft first story of LMOB. Success of the solutions will not only rest upon performance but also upon the structural solution's constructability.

The first design is a general revision of the current lateral structural system. In Technical Report III, it was discovered that LMOB experiences soft story and extreme torsional irregularity. As a result, the lateral force resisting elements will be strategically placed to minimize eccentricity between the CM and CR. All lateral force systems will be designed either by hand or with the help of ETABS.

A second design solution is the tilt-up exterior bearing wall system. The tilt-up walls will serve as a lateral load resisting system and be the same height as the original lateral load resisting system – 86'. 86' tall tilt-up walls will push close to the maximum feasible height for monolithically cast walls. Currently, the tallest panel feasibly cast monolithically and tilted into place is approximately 92' – for a commercial building in Hollywood, FL (TCA, 2014). The current limits to taller and heavier tilt-up walls are cost, lifting technology, and temporary bracing (Griffin, 2014). Internal lateral resisting elements will only be added, if it is determined that the tilt-up exterior walls are insufficient – however this is not expected. Due to the nature of tilt-up construction, the system's stability must be studied when under the various phases of construction. The purpose of the study is to ensure adequate temporary bracing and prevent failure during construction.

Though the existing building façade is generally code compliant and performs adequately, it is heavy. The façade's weight is detrimental if a similar facility is built in a more seismically active region due to increase strengthening of lateral force resisting elements – either through more expensive high strength materials or increase dimensions. Reducing the façade's weight is paramount along with preserving moisture resistance and acoustical performance, whilst reducing general construction cost, and improving relative ease of assembly.

In terms of the façade redesign, a light gauge cold formed steel (CFS) stud back-up wall will be used. What can be said is that the façade redesign strives to maintain – if not reduce – the general construction cost, and improve relative ease of assembly. Whether it has similar performance levels as the concrete masonry back-up wall remains to be determined.

Structural Redesign

Redesigns of LMOB's original lateral force resisting system were implemented in parallel. Parallel design is logical because both redesigns share the same center of mass and gravity loads, apart from the self-weight of the lateral force resisting systems. Generally the gravity structural system was disturbed minimally. The first step was to select the locations of the new lateral force resisting elements. In both redesigns, it was decided that lateral force resisting elements should be placed furthest from the center of mass – ideally at the building's perimeter – to efficiently resist torsional influences.

Once locations for lateral force resisting elements were selected, each new element in the redesigns went through stiffness modifications to reduce the eccentricity between the building's center of mass and center of rigidity. Stiffness modifications include: modifying the dimensions of the lateral force resisting elements and potentially increasing the concrete strength (f'_c). Only when the redesigns eliminated torsional irregularity, defined by ASCE 7-05, were the reinforcement designed and detailed. In both redesigns the controlling lateral load is wind. Increases in building mass – especially for the tilt-up walls – were not enough to make seismic the controlling lateral load. In the following sections, the design processes for each design will be discussed. The redesigns will be compared with the original structural system – serves as a baseline, to determine their competitiveness.

Design I

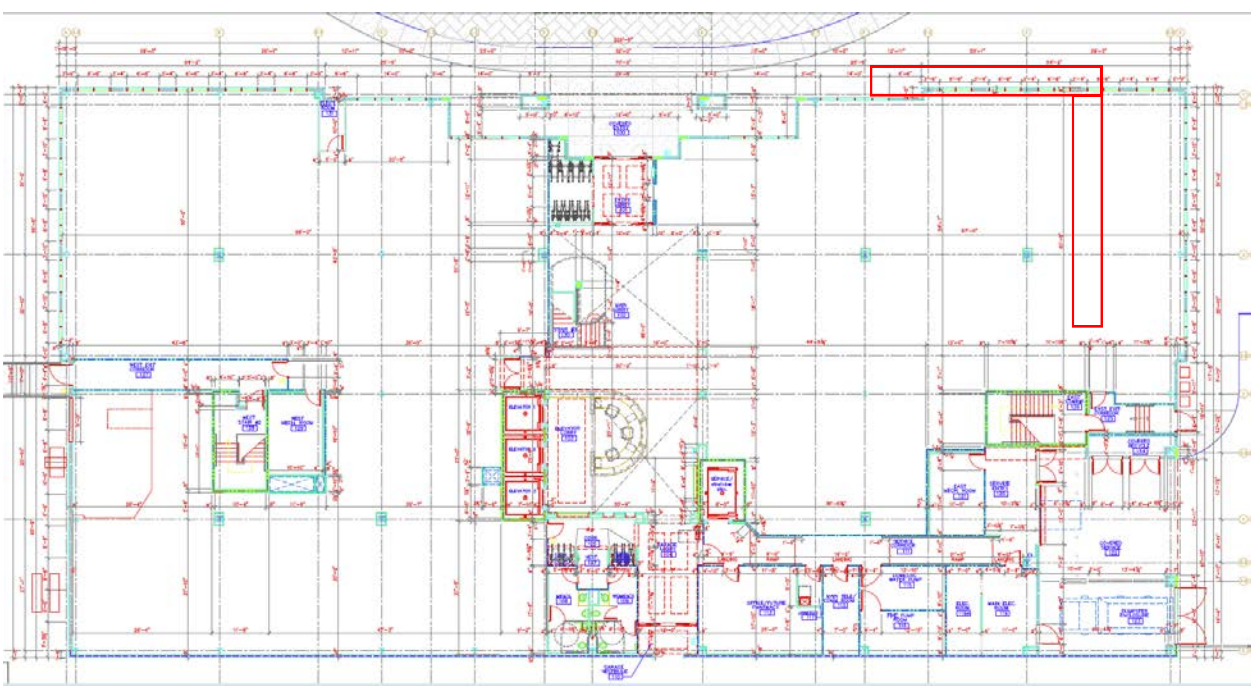


Figure 2.1, Potential Locations for Perimeter Lateral Force Resisting Elements
Source: Oliver, Glidden, Spina & Partners

The first redesign is a slight modification of the original, with additional lateral force resisting elements placed at the perimeter of LMOB. Perimeter locations where the addition lateral force resisting elements can be placed are shown in Figure 2.1. Eventually the decision was made to use the area between column lines F.4 and H, as well as the area bounded by 1 and 3.5 – both of these areas are designated AV5-X and AV5-Y respectively. Both locations will contain elements with a thickness of 8” to ensure that the formwork at the short sides are the same as those used on the current shear walls. Constructability is improved when components are similar.

Then the desired eccentricity between the center of mass and center of rigidity to eliminate code defined torsional irregularity was assumed. This assumption was 7.5% or less in both directions. With the assumption made, it was derived that elements in area AV5-X must have a stiffness of at least 32.3 force per in. As for element in area AV5-Y the required stiffness was 129.5 force per in. Using the cantilevered beam stiffness (k) formula $3EI/L^3$ and assuming that the total transformed elastic modulus – which includes steel reinforcement – is $1.5E_c$, it was determined that the spaces between the openings weren’t enough. To achieve the necessary stiffness each lateral force resisting element must span across the openings and engage the adjacent spaces between the openings. Calculations used to determine the required stiffness for the perimeter lateral force resisting elements are not in the appendix and are available only upon request, in an attempt to reduce paper usage.

Later it was determined that using 6000 lb/in^2 is far more economical than the lower strength concrete in the original design. The only downside when using 6000 lb/in^2 concrete is greater construction coordination, so that the 6000 lb/in^2 and lower strength concrete aren’t placed in the wrong lateral force resisting element. Figure 2.2 and Figure 2.3 shows the lateral force resisting elements on both the northern and eastern sides. For better visualization of the elements with 6000 lb/in^2 concrete Figure 2.4 – on the following page shows 6000 lb/in^2 concrete highlighted blue, while elements using the lower strength concrete are highlighted red.

Figure 2.2, Lateral Force Resisting Elements at East Side
Source: Oliver, Glidden, Spina & Partners

Figure 2.3, Lateral Force Resisting Elements at North Side
Source: Oliver, Glidden, Spina & Partners

Figure 2.4, 6000 lb/in2 (Grey) and Lower Strength Concrete (Red)
Source: Oliver, Glidden, Spina & Partners

In design checks by hand and ETABS it was verified that that the torsional irregularity had been eliminated with the addition of elements AV5-X1, AV5-Y1, and AV5-Y2. Both the center of mass and rigidity in the hand calculations and ETABS computer were nearly identical. Table 2.1, Table 2.2, and Table 2.3 contains the center of mass and rigidity derived by hand, as well as outputted by ETABS. This further permitted the use of ETABS output to design the individual lateral force resisting elements in RAM Elements.

Table 2.1, Formatted ETABS Center of Mass and Center of Rigidity Output

Story	Diaphragm	MassX	MassY	XCM	YCM	XCR	YCR
STORY6	D1	101.0603	101.0603	114.75	58.44	120.61	64.29
STORY5	D1	97.614	97.614	114.79	58.9	121.34	64.13
STORY4	D1	98.0577	98.0577	114.79	58.9	121.78	63.52
STORY3	D1	99.8325	99.8325	114.79	58.9	121.71	62.23
STORY2	D1	101.6073	101.6073	114.79	58.9	118.51	59.14
STORY1	D1	95.327	95.327	114.69	58.72	112.77	54.76

Table 2.2, Calculated Center of Mass

Floor Type	Component	Area (ft^2)	Center of Mass	
			x (ft)	y (ft)
A			110.07	59.34
	A1	11324.15	95.31	30.38
	AV1	-224.55	36.84	44.54
	AV2	-223.83	94.51	41.58
	AV3	-113.50	134.88	34.42
	AV4	-224.55	198.83	49.26
	A2	2362.09	208.07	30.38
	AV5	-1143.33	213.51	20.42
	A3	3069.82	27.09	89.09
	A4	1394.00	66.92	88.09
	A5	1115.96	91.63	84.09
	A6	949.17	114.76	82.01
	A7	1115.96	137.88	84.09
	A8	1394.00	162.58	88.09
	A9	3069.82	202.42	89.09
B			114.69	58.72
	B1	13701.04	114.76	30.38
	BV1	-224.55	36.84	44.54
	BV2	-223.83	94.51	41.58
	BV3	-503.6	119.39	41.21
	BV4	-5.75	128.09	34.92
	BV5	-113.50	134.88	34.42
	BV6	-224.55	198.83	49.26

	B2	3069.82	27.09	89.09
	B3	6623.78	114.76	88.09
	BV7	-757.99	114.76	76.48
	B4	3069.82	202.42	89.09
C			114.79	58.90
	C1	13701.04	114.76	30.38
	CV1	-224.55	36.84	44.54
	CV2	-223.83	94.51	41.58
	CV3	-113.50	134.88	34.42
	CV4	-224.55	198.83	49.26
	C2	3069.82	27.09	89.09
	C3	6623.78	114.76	88.09
	C4	3069.82	202.42	89.09

Table 2.3, Calculated Center of Rigidity

Lateral Resisting Element		Stiffness	Element Center of Rigidity		Global Center of Rigidity	
Designation	Resisting Direction		x (ft)	y (ft)	x (ft)	y (ft)
AV1-X1	X	15.18	36.84	34.33	117.18	63.61
AV1-Y1	Y	122.10	42.34	44.54		
AV2-Y1	Y	248.14	90.26	41.59		
AV2-X1	X	7.53	94.68	54.76		
AV3-Y1	Y	31.20	130.34	34.42		
AV3-X1	X	8.23	134.88	40.67		
AV3-Y2	Y	31.20	139.42	34.42		
AV4-Y1	Y	21.79	188.63	49.26		
AV4-X1	X	112.61	199.17	54.76		
AV5-Y1	Y	31.716	229.17			
AV5-Y2	Y	91.324	226.83			
AV5-X1	X	31.726		117.08		

Though the redesign eliminated code defined torsional irregularity, it redistributed the lateral forces among the existing lateral force resisting elements. When comparing the lateral forces acting on each lateral force resisting element before and the after redistribution, the general change isn't so significant, the majority of the existing lateral force resisting need not be redesigned. Lateral force tabulations, both before and after the redistribution can be compared in Table 2.4.

Table 2.4, Comparison of Base Shear of Lateral Force Resisting Elements (Values were derived from hand calculations and checked with ETABS)					
Element	V_{base} (Kip)		Element	V_{base} (Kip)	
	Original	Design I		Original	Design I
AV1-X1	76.5	62.0	AV3-Y2	121.7	102.0
AV1-Y1	325.0	229.1	AV4-Y1	84.0	89.5
AV2-Y1	304.4	335.4	AV4-X1	159.6	187.4
AV2-X1	63.9	43.7	AV5-X1	N/A	14.8
AV3-Y1	126.6	102.0	AV5-Y1	N/A	145.9
AV3-X1	121.7	33.4	AV5-Y2	N/A	23.8

Reinforced concrete code, ACI 318-11, was used to design and detail the reinforcement within the perimeter lateral force resisting elements. Continuing the theme of commonality and construction ease with the existing lateral force resisting elements, 60,000 lb/in^2 rebar was used.

In order to facilitate a durable and safe design, decisions were made, and are as follows:

1. Clear cover between the exterior concrete face and rebar was set to 2”
2. All flexural reinforcements are the same size across all lateral force resisting elements
3. All shear reinforcements are the same size across all lateral force resisting elements
4. All lateral force resisting elements are fixed at the base
5. During construction, lateral force elements are braced against wind until elements of the floor diaphragm are in place
6. No generally detrimental construction related defects
7. Lateral force resisting elements take no axial loads other than self-weight
8. Two layer of flexural rebar
9. ε_t = 0.005 for flexural reinforcement furthest from the neutral axis

LMOB is located no more than 3 miles from the Gulf of Mexico. The close proximity to a source of chlorides is significant because chlorides corrode the steel used in the reinforcement. It was this reason to increase the exterior concrete clear cover to 2” – in lieu of the typical 1-1/2”. Increasing the exterior concrete clear cover also reduces the detrimental effects of carbonation, which will be covered in greater detail in the façade breadth section. Structural steel columns – at the perimeter of LMOB – and intersect perimeter lateral force resisting elements are cast integrally with the perimeter lateral force resisting elements. The outcome is reduced impact on the architectural plan. The imbedded structural steel columns solely handle the gravity loads. Any potential interaction between the two systems, in sharing gravity and lateral loads are ignored.

All lateral force resisting elements incorporate two layers of flexural reinforcement. When coupled with hoops, this reinforcement cage confines the core concrete. Beneficial characteristics of the configuration are added resistance to damage and reduced hysteresis strength degradation. Shear reinforcement hoops continue until 4” from the foundations, even though ACI 318-11 states that these are not necessary at distances less than *d* from the face of support. Continuing the shear reinforcement hoops until they’re 4” from the foundations confines the concrete core at the lateral force resisting element’s base and avoids possible rebar buckling during the construction process. Top reinforcement is required, due to the likelihood that the wind load will reverse. The other reason is to strain the flexural reinforcement to 0.005, in order to use $\Phi = 0.9$.

As in the technical reports, the flexural reinforcement design was determined using the RAM Elements. One spot check on the design of element AV1-Y1 was done for Design I’s flexural reinforcement. The spot check revealed that designing lateral force resisting elements as accurate as designing them by hand. Design output by RAM Elements can be referenced in Appendix H.

Table 2.5, Hoop Design Criteria for Interior Lateral Force Resisting Elements

Element	Story	Pier	A_g (in^2)	0.1 F'_c A_g (Kip)	Hoop Criteria
AV1-X1	STORY6	P1X	992	396.8	F
	STORY5				F
	STORY4				F
	STORY3				C
	STORY2				C
	STORY1				C
AV1-Y1	STORY6	P1Y	2016	806.4	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				F
	STORY1				F
AV2-Y1	STORY6	P2Y	2592	1036.8	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				F
	STORY1				F
AV2-X1	STORY6	P2X	784	313.6	F
	STORY5				C
	STORY4				C
	STORY3				C
	STORY2				C
	STORY1				C
AV3-X1	STORY6	P3X	808	323.2	F

	STORY5				F
	STORY4				F
	STORY3				C
	STORY2				C
	STORY1				C
AV3-Y2	STORY6	P3Y2	1264	505.6	F
	STORY5				F
	STORY4				F
	STORY3				C
	STORY2				C
	STORY1				C
AV4-Y1	STORY6	P4Y	1120	448	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				C
	STORY1				C
AV4-X1	STORY6	P4X	1960	784	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				C
	STORY1				C

Table 2.6, Hoop Design Criteria for Perimeter Lateral Force Resisting Elements

Element	Story	Pier	A_g (in^2)	0.1 F'_c A_g (Kip)	Hoop Criteria
AV5-X1	STORY6	P5X1	465	279	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				F
	STORY1				F
	STORY6	P5X2	430	258	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				F
	STORY1				F
AV5-Y1	STORY6	P5Y1	465	279	F

	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				F
	STORY1				F
	STORY6	P5Y2	430	258	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				F
	STORY1				F
AV5-Y2	STORY6	P5Y3	645	387	F
	STORY5				F
	STORY4				F
	STORY3				C
	STORY2				C
	STORY1				C
	STORY6	P5Y4	855	513	F
	STORY5				F
	STORY4				F
	STORY3				F
	STORY2				C
	STORY1				C

Only the shear and hoop reinforcement were done outside of RAM Elements, through the assistance of Microsoft Excel. Piers in the perimeter lateral force resisting elements experience axial loads stemming from the wind loads, depending on magnitude of these axial loads – the hoops are either designed according to the compression or flexural member requirements in ACI 318-11. Table 2.5 and Table 2.6 shows the design criteria for the hoops – C represents compression criteria, while F is flexural criteria. It is evident from Table 2.5 and Table 2.6, that most lateral force resisting elements follows the flexural criteria. Design calculations for the hoops can be found in Appendix H.

Below – in Figure 2.5 to Figure 2.12 – are the reinforcement detailing for elements AV5-X1, AV5-Y1, and AV5-Y2.

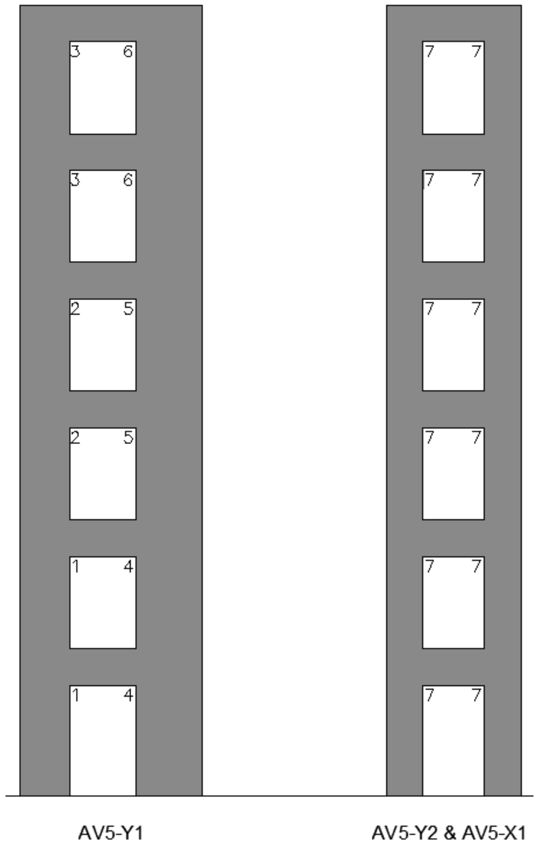
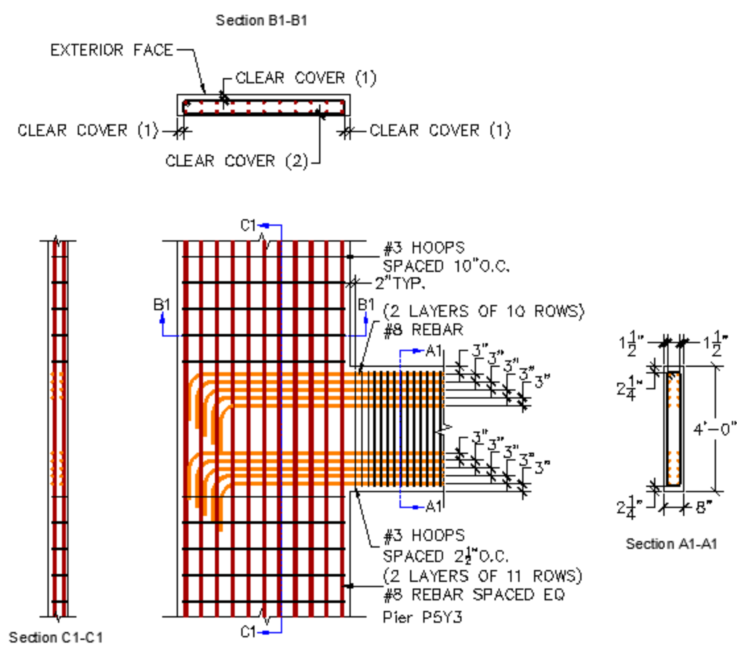
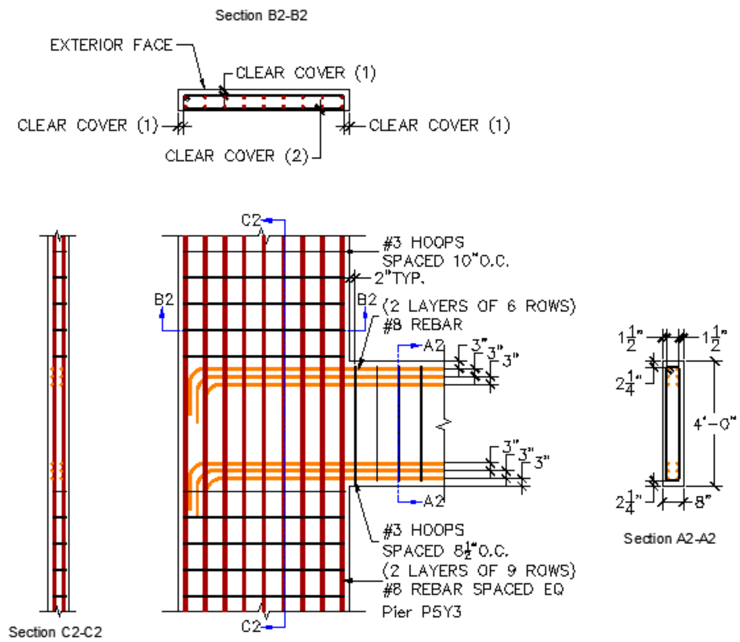


Figure 2.5, Reinforcement Designations



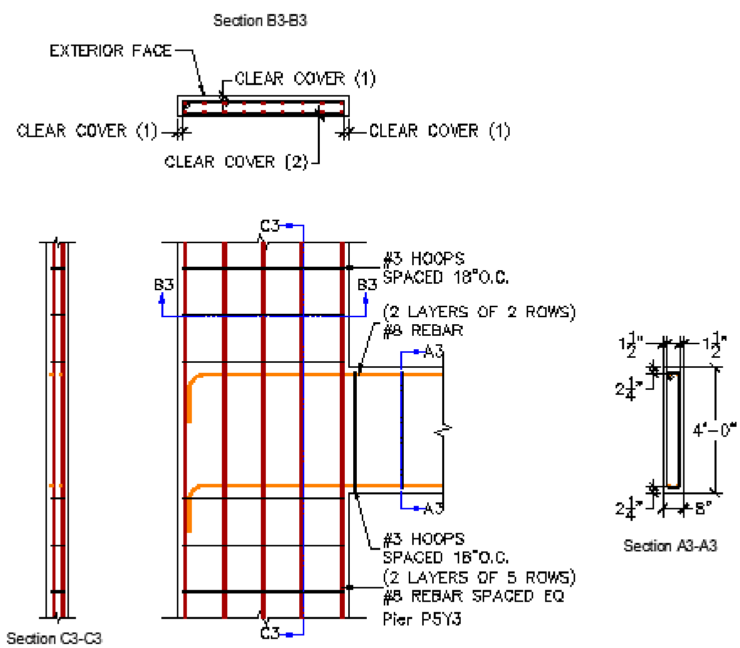
1

Figure 2.6, Reinforcement Detail 1



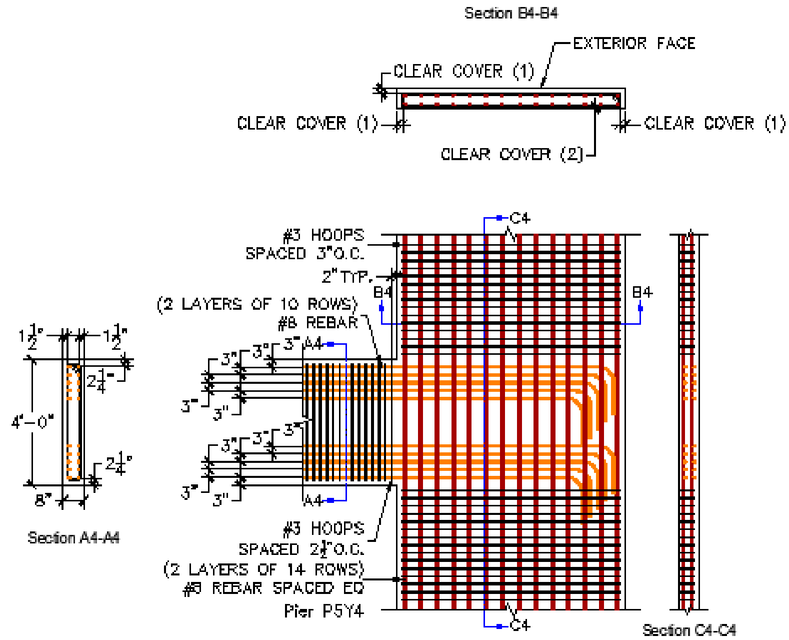
2

Figure 2.7, Reinforcement Detail 2



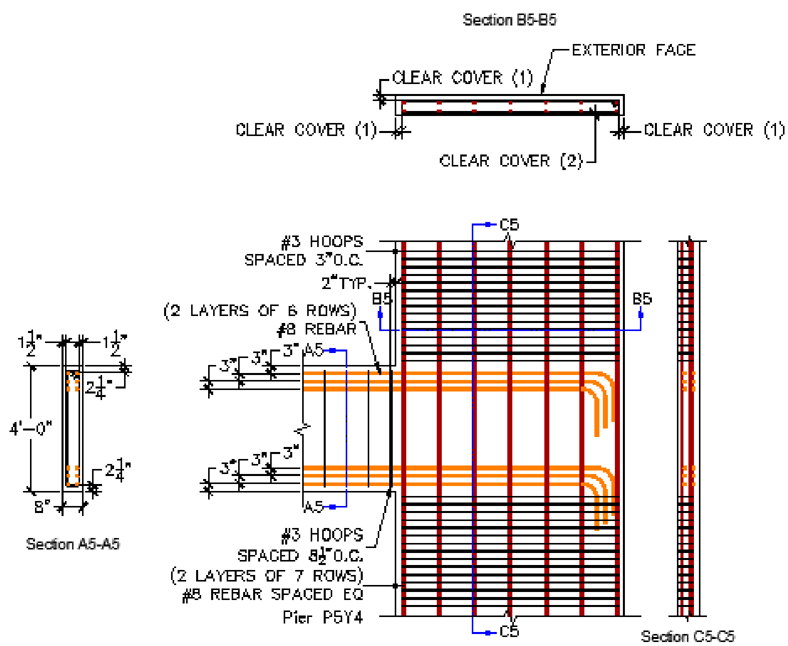
3

Figure 2.8, Reinforcement Detail 3



4

Figure 2.9, Reinforcement Detail 4



5

Figure 2.10, Reinforcement Detail 5

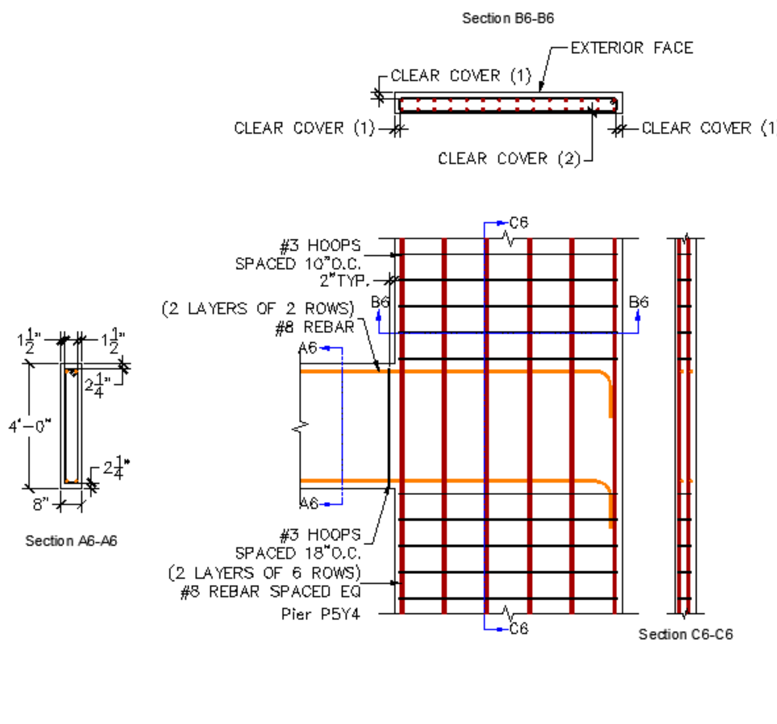


Figure 2.11, Reinforcement Detail 6

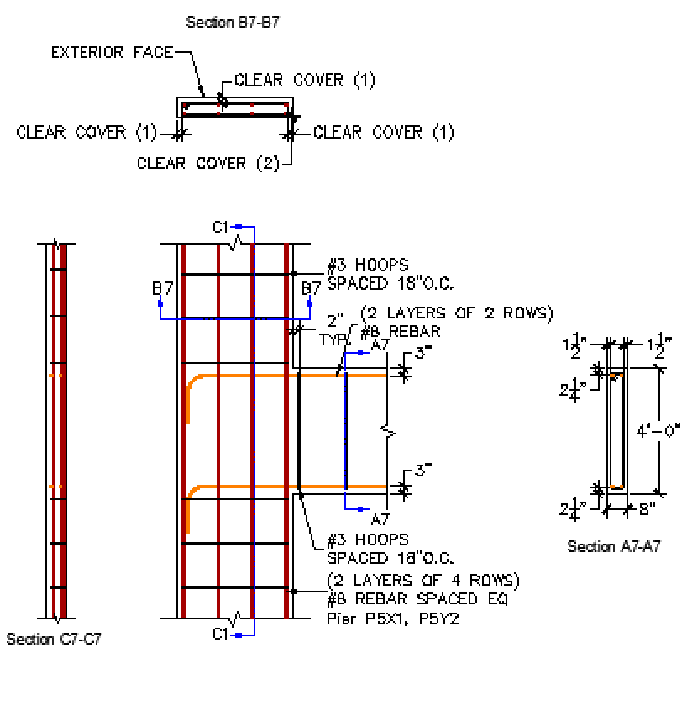


Figure 2.12, Reinforcement Detail 7

Design II

The second design revolves around tilt-up walls. Essentially, tilt-up is a system where cast on site concrete walls are lifted into position and secured. There is no limit on building square footage, when using tilt-up walls – as long as there is enough space on-site to cast the concrete walls. Building height however, is limited by the crane capacity and slenderness of the tilt-up wall members. To date the tallest and heaviest tilt-up wall panels lifted into place are 96' and 154 tons, respectively (TCA, 2013). Traditionally tilt-up walls were used for warehouses and industrial buildings, but recently have gain popularity in Florida as a cost effective option in commercial buildings – like offices. This is based on the top 10 Tilt-Up wall height and heaviest Tilt-Up wall panels lifted monolithically, the majority of whom are located in Florida (TCA, 2013).

In the case of LMOB, the entire lateral and perimeter gravity loads were handled by the tilt-up walls. Since the lateral force resisting tilt-up wall sections handled the entire lateral load, there is no need for interior lateral force resisting elements. This frees up the interior space arrangement, allowing more flexible room arrangements. Additionally, there is greater torsional resistance of perimeter lateral force resisting elements, in rectilinear buildings when compared to internally placed lateral force resisting elements, due to the greater moment arm between the building's center and the perimeter lateral force resisting elements. The efficient performance translates to reduced number of required lateral load resisting elements. In LMOB, only a few tilt-up wall sections were required to resist lateral loads and tackle code defined torsional irregularity, the result is that a majority of the tilt-up wall sections were designed as gravity load resisting members only. A secondary function of the tilt-up walls used is that they serve as back-up walls for the building's cladding.

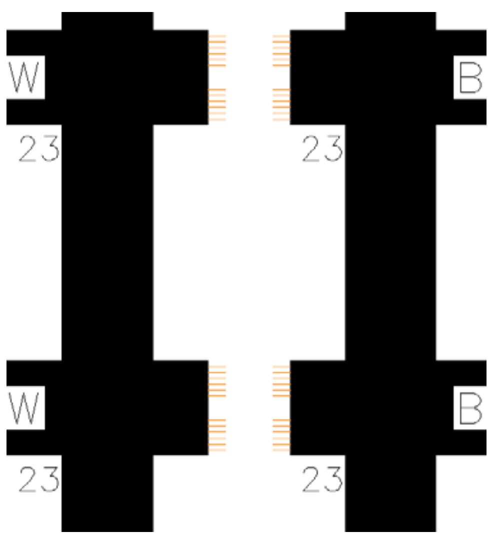


Figure 2.13, General Joint between Tilt-Up Wall Panels

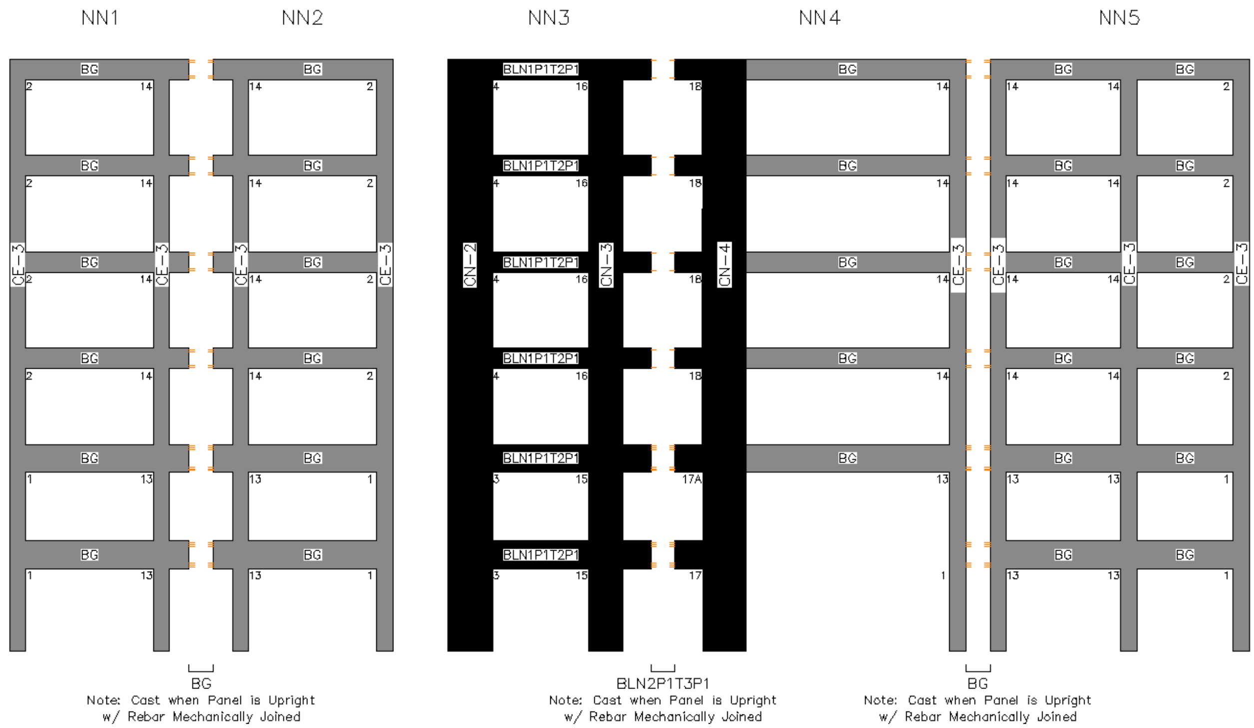


Figure 2.14, Joint Locations and Respective North Tilt-Up Wall Panels Designations

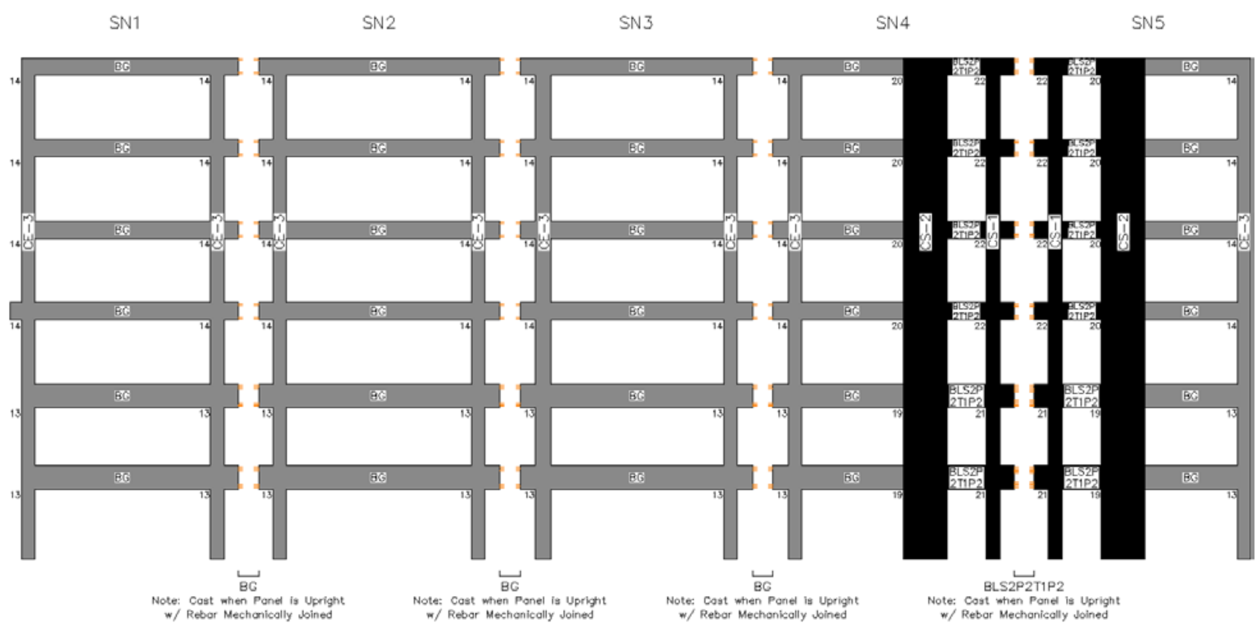


Figure 2.15, Joint Locations and Respective South Tilt-Up Wall Panels Designations

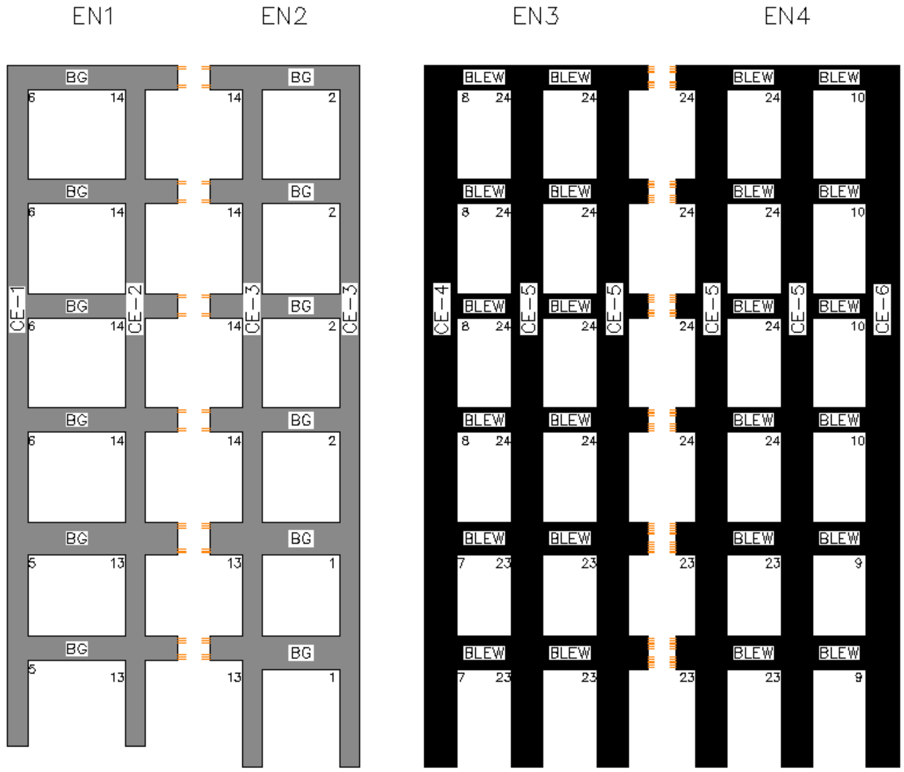


Figure 2.16, Joint Locations and Respective East Tilt-Up Wall Panels Designations

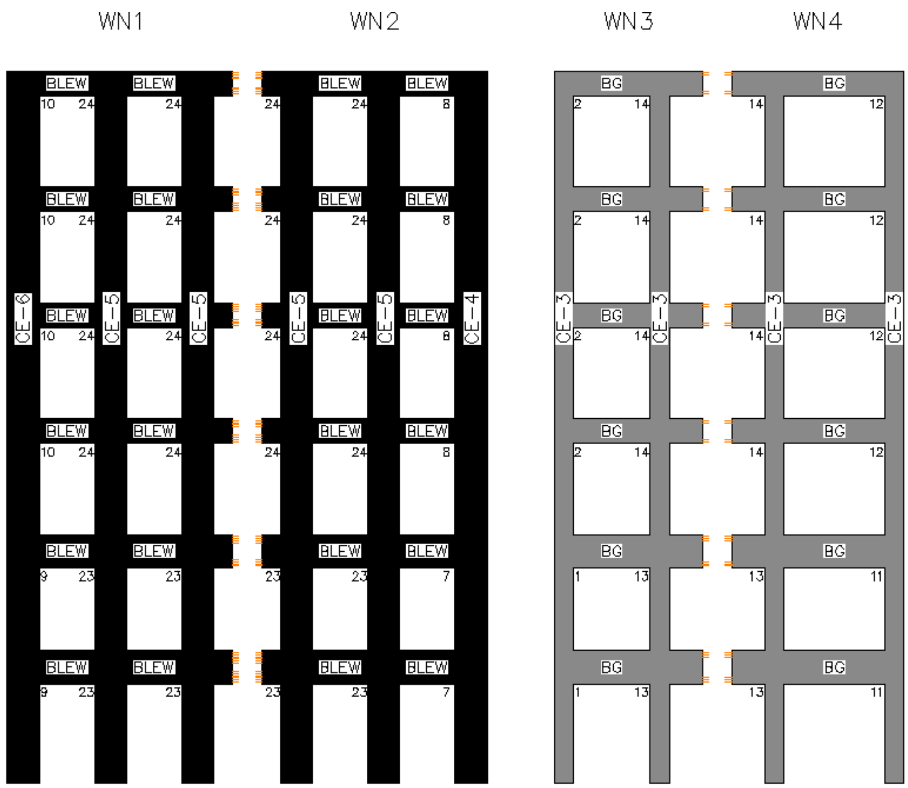


Figure 2.17, Joint Locations and Respective West Tilt-Up Wall Panels Designations

Major obstacles were encountered in designing a durable and constructible tilt-up structure. One of them is the need for greater protection of the reinforcement from carbonation and chloride attack, the latter is ever present in the Largo, FL – where LMOB is located. Tilt-up panel weight and dimensions was a lingering issue and resulted in casting some lateral force resisting elements in two or more parts. The multiple parts for the lateral force resisting elements were only joined once they were lifted. Joining the parts of the tilt-up elements starts with joining the rebar between the panels with mechanical splice connectors and then encasing them with cast-in-place concrete. An illustration of the joint between the tilt-up wall panels is and their locations shown in Figure 2.13 to Figure 2.18.

Other pervading obstacles include: temporary bracing and structural continuity. Structural continuity typically ensures that the structure is more resistant to damage, where the surrounding/neighboring undamaged elements can take to load of the damaged element. Structural continuity eventually manifested into general connection detailing. Let it be clear, that the general connection detailing is not a design but an idea to achieve structural continuity.

Lateral force resisting elements locations were selected based upon the desire to eliminate torsional irregularity. Symmetry played a large factor, where panels EN3 and EN4 (A2-Y1) – on the eastern façade – as well as panels WN1 and WN2 (A1-Y1) – on the western façade, were chosen to resist lateral loads in the north and south directions. Selection of lateral force resisting elements to resist lateral loads in the east and west directions was more difficult, since none of the panels on either north or south faces were the same. Eventually it was settled that panel NN3 and a portion of NN4 (A5-X1) – on the north façade – along with portions of panel SN4 and SN5 (A5-X1) – on the south façade – would resist lateral loads in the east and west directions. Shaded black in Figure 2.14 to Figure 2.17, are the lateral load resisting elements.

It was confirmed by both calculations and ETABS modeling that there was no torsional irregularity. Center of mass and rigidity in the calculations and ETABS computer were found to be within 5%. Table 2.7, Table 2.8, and Table 2.9 contains the center of mass and rigidity derived by calculations, as well as outputted by ETABS. The outcome further permitted the use of ETABS output to design the individual lateral force resisting elements in RAM Elements.

Table 2.7, Formatted ETABS Center of Mass and Center of Rigidity Output

Story	Diaphragm	MassX	MassY	XCM	YCM	XCR	YCR
STORY6	D1	101.5554	101.5554	114.77	58.42	116.8	59.52
STORY5	D1	98.4901	98.4901	114.83	58.89	117.01	59.08
STORY4	D1	98.9337	98.9337	114.83	58.89	117.23	58.76
STORY3	D1	100.7083	100.7083	114.83	58.89	117.4	58.6
STORY2	D1	102.4829	102.4829	114.83	58.89	117.47	58.77
STORY1	D1	97.5386	97.5386	114.74	58.71	116.62	59.1

Table 2.8, Calculated Center of Mass				
Floor Type	Component	Area (ft^2)	Center of Mass	
			x (ft)	y (ft)
A			110.07	59.34
	A1	11324.15	95.31	30.38
	AV1	-224.55	36.84	44.54
	AV2	-223.83	94.51	41.58
	AV3	-113.50	134.88	34.42
	AV4	-224.55	198.83	49.26
	A2	2362.09	208.07	30.38
	AV5	-1143.33	213.51	20.42
	A3	3069.82	27.09	89.09
	A4	1394.00	66.92	88.09
	A5	1115.96	91.63	84.09
	A6	949.17	114.76	82.01
	A7	1115.96	137.88	84.09
	A8	1394.00	162.58	88.09
	A9	3069.82	202.42	89.09
B			114.69	58.72
	B1	13701.04	114.76	30.38
	BV1	-224.55	36.84	44.54
	BV2	-223.83	94.51	41.58
	BV3	-503.6	119.39	41.21
	BV4	-5.75	128.09	34.92
	BV5	-113.50	134.88	34.42
	BV6	-224.55	198.83	49.26
	B2	3069.82	27.09	89.09
	B3	6623.78	114.76	88.09
	BV7	-757.99	114.76	76.48
	B4	3069.82	202.42	89.09
C			114.79	58.90
	C1	13701.04	114.76	30.38
	CV1	-224.55	36.84	44.54
	CV2	-223.83	94.51	41.58
	CV3	-113.50	134.88	34.42
	CV4	-224.55	198.83	49.26
	C2	3069.82	27.09	89.09
	C3	6623.78	114.76	88.09
	C4	3069.82	202.42	89.09

Table 2.9, Calculated Center of Rigidity						
Lateral Resisting Element		Stiffness, K (kip/in)	Element Center of Rigidity		Global Center of Rigidity	
Designation	Resisting Direction		x (ft)	y (ft)	x (ft)	y (ft)
A1-Y1	Y	291.715	0.42	88.75	114.80	61.30
A2-Y1	Y	291.715	229.08	88.75		
A5-X1	X	215.657	152.38	115.00		
A6-X1	X	190.186	179.00	0.42		

Unlike Design I, higher strength concrete was not required for the first story – to handle the soft story irregularity. Instead the lower 2' of the first story is filled with concrete and reinforced. This is only done for the lateral force resisting elements. The result is, equal column height for all the stories. ETABS modeling showed that there was no code defined soft story irregularity, after the modification.

Reinforced concrete code ACI 318-11 and published 2006 Tilt-Up Construction and Design Manual was used to design and detail the reinforcement within the perimeter lateral force resisting tilt-Up sections. Unlike Design I, certain tilt-up wall panels require the use of high strength reinforcing steel (75,000lb/in^2). The primary reason is not strength but reinforcement congestion – specifically for tilt-up panels SN1, SN2, and SN3. Everywhere else the reinforcement is 60,000 lb/in^2. The use of two different reinforcement grades is a construction coordination problem, to ensure that the high strength reinforcement isn't placed in the wrong panels. The result of wrong placement is panel failure during the tilt-up process. To get around this, the high strength reinforcing steel is delivered and assembled into rebar cages before the 60,000 lb/in^2 is delivered.

Design assumptions and decisions made in Design II are as follows:

1. Clear cover between the exterior concrete face and rebar was set to 2"
2. All lateral force resisting elements are modeled as pin at the base
3. All tilt-up panels are braced against wind until elements of the floor diaphragm are in place
4. No generally detrimental construction related defects
5. Two layers of flexural rebar to reduce hysteresis strength degradation
6. Continuing the shear reinforcement hoops until they're 4" from the foundations
7. ε_t = 0.005 for flexural reinforcement furthest from the neutral axis

Following the lead of Design I, all flexural reinforcement design was determined using the RAM Elements. To ensure modeling accuracy a spot check on the design of element CE-5 was done. The spot check revealed that designing lateral force resisting elements as accurate as designing them by hand. The spot check and output by RAM Elements can be found in the appendix.

It was determined that the bending, combined loading, and secondary effects during the lifting process controlled over the loads when the building is finished and occupied. The result is designing by hand – the flexural and out-plane shear – because RAM Elements don't consider the lifting process. Influence lines were used to determine the lifting points which would minimize the bending moments which the tilt-up panels will experience. Tilt-up walls were simplified as beams with a unit width of 1'-0". Figure 2.18 shows the lifting point configuration that'll minimize bending moments. The last step was designing the tilt-up wall panels for the controlling loads during the lifting process. Final reinforcement details can be found in Figure 2.19 to Figure 2.43, located below. For more information on tilt-up design for LMOB, see the appendix.

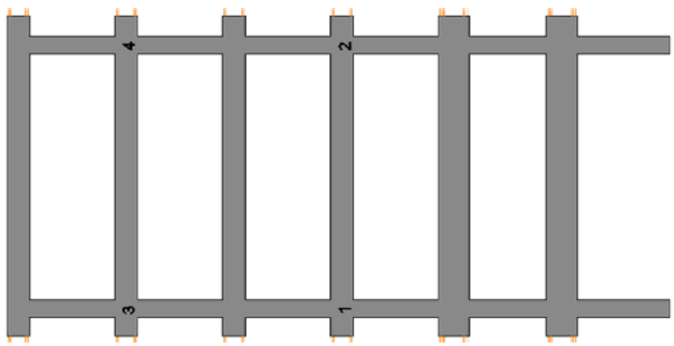
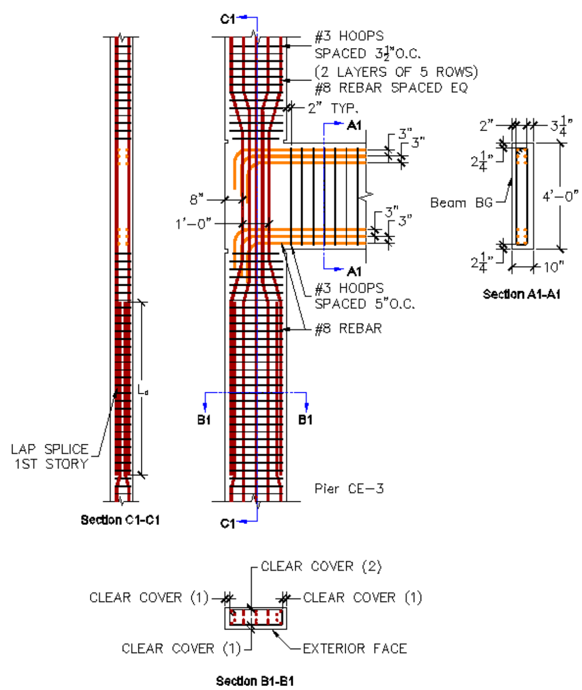


Figure 2.18, Panel Lifting Points



1

Figure 2.19, Reinforcement Detail 1

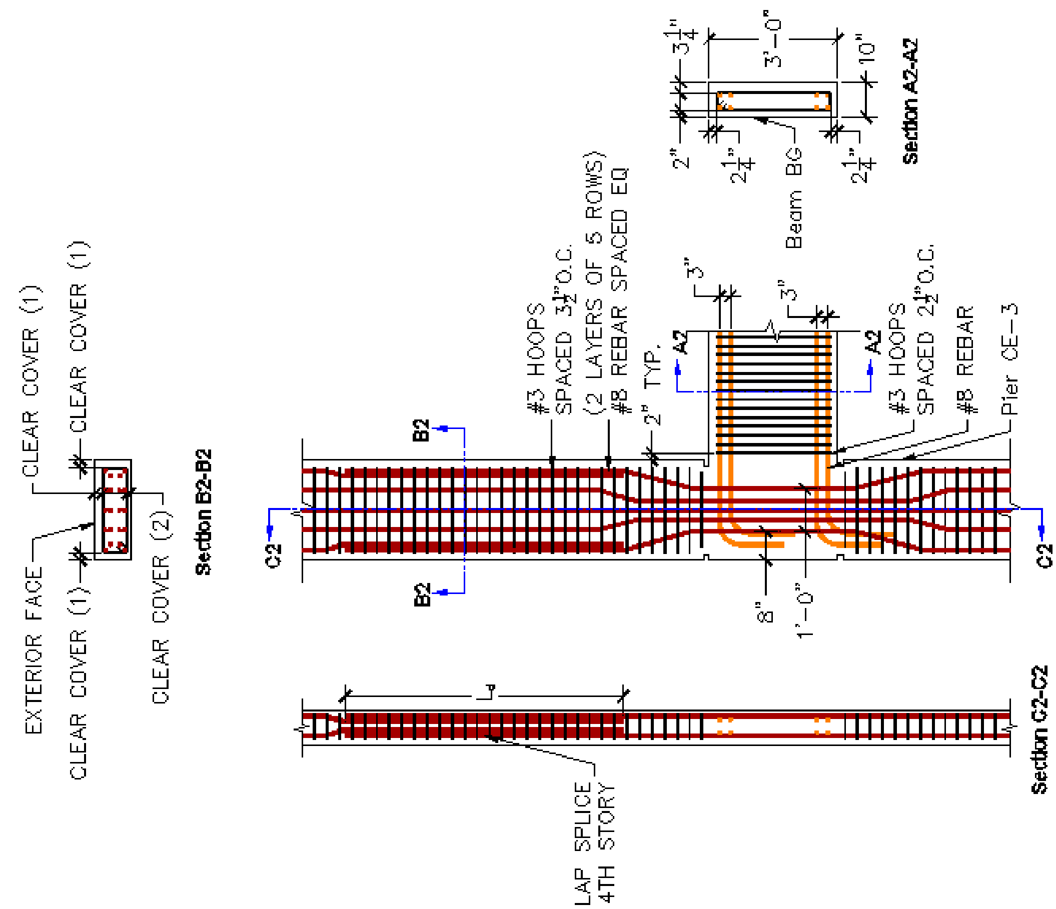


Figure 2.20, Reinforcement Detail 2

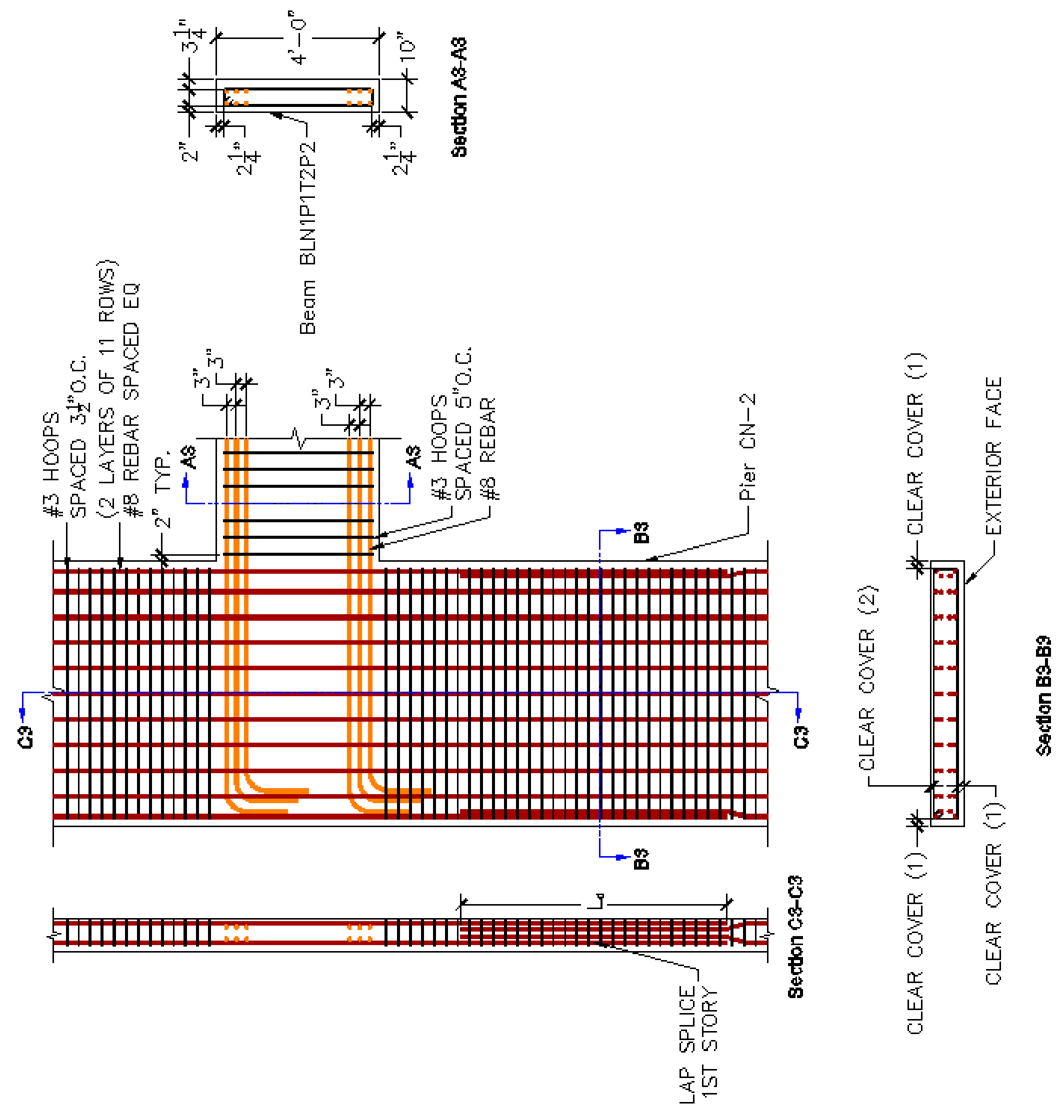


Figure 2.21, Reinforcement Detail 3

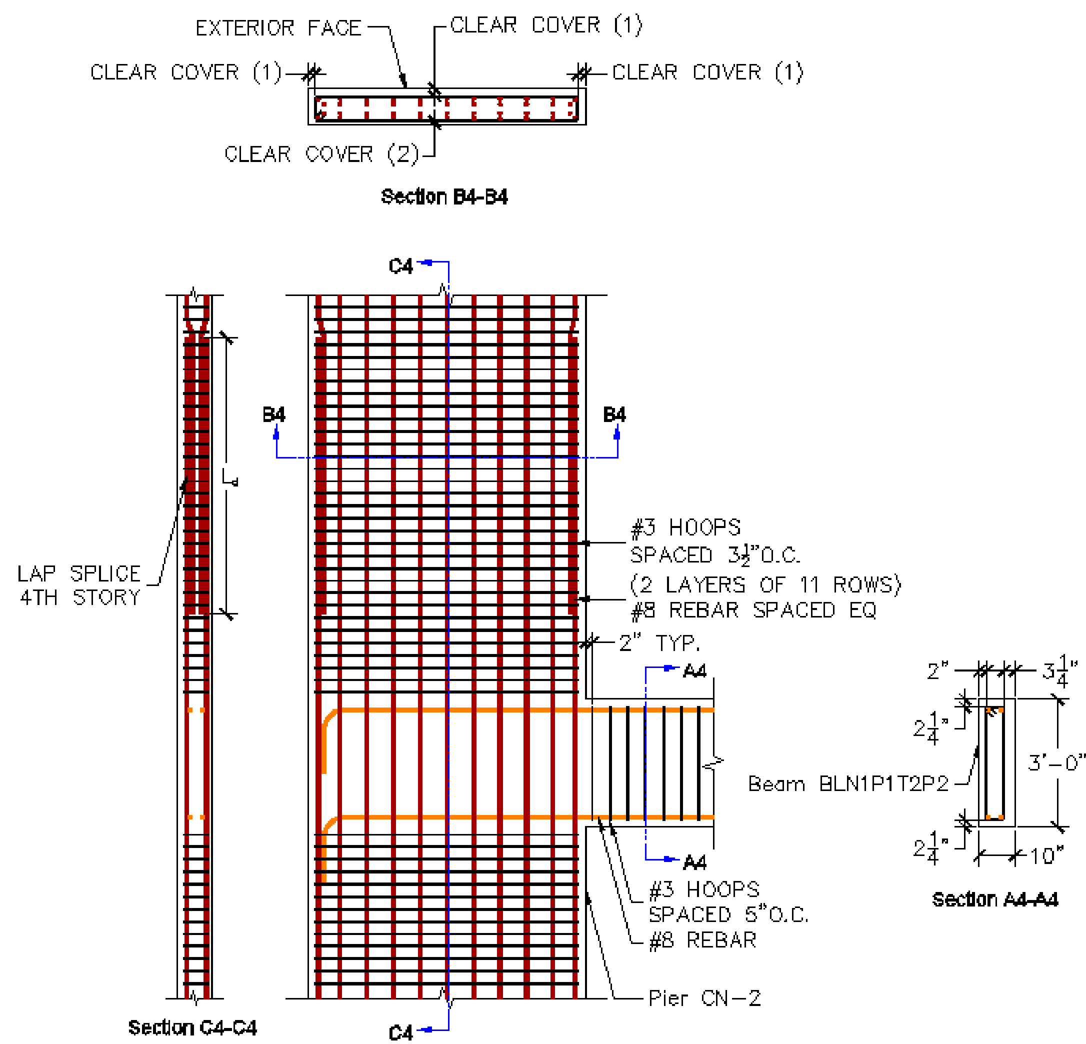


Figure 5.22, Reinforcement Detail 4

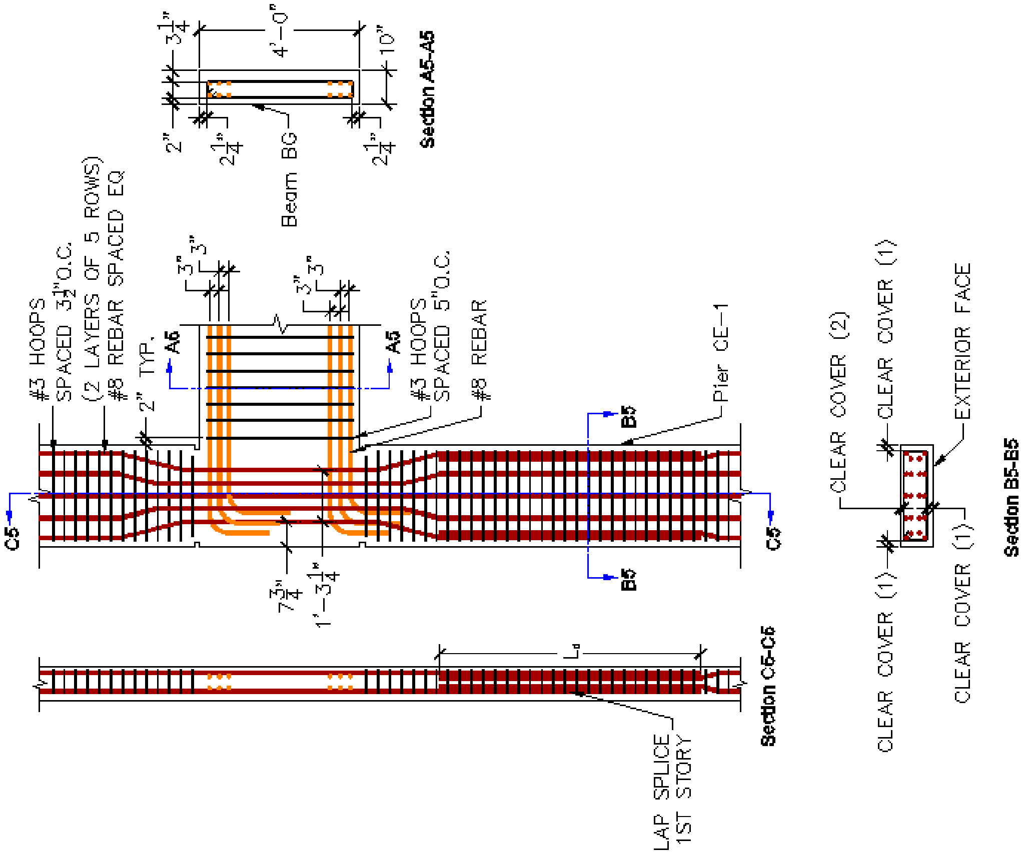


Figure 2.23. Reinforcement Detail 5

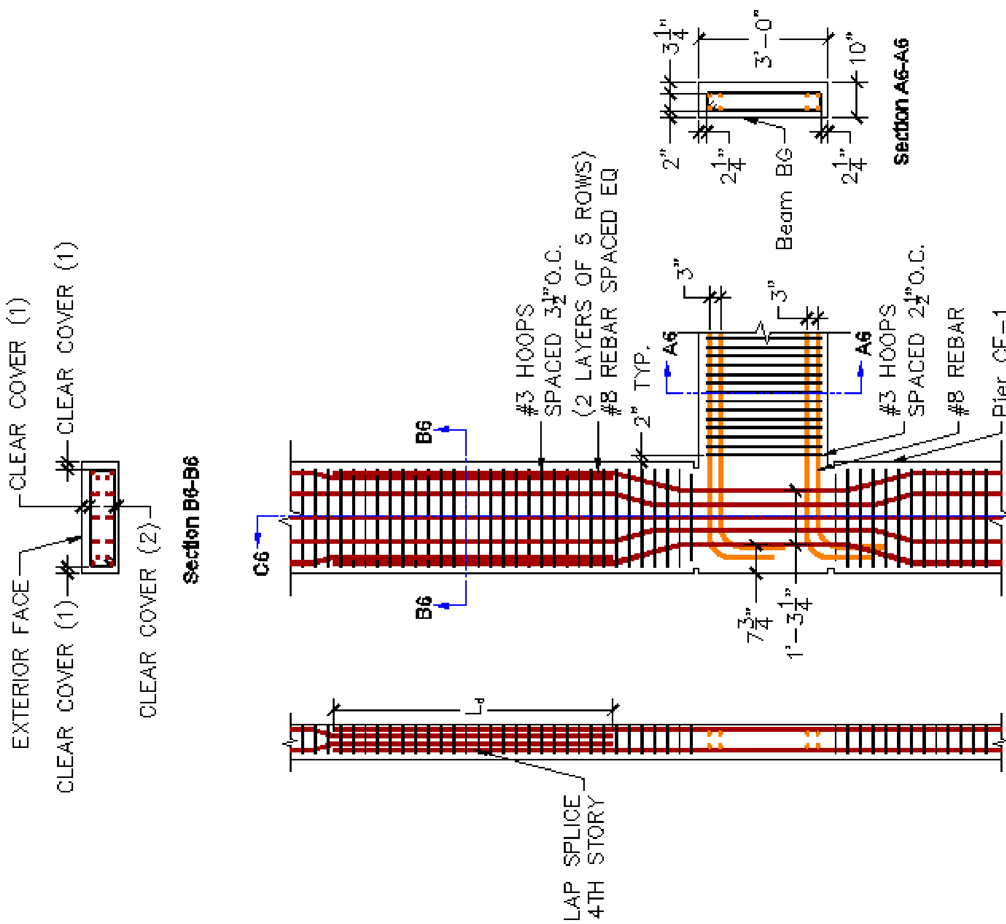


Figure 2.24, Reinforcement Detail 6

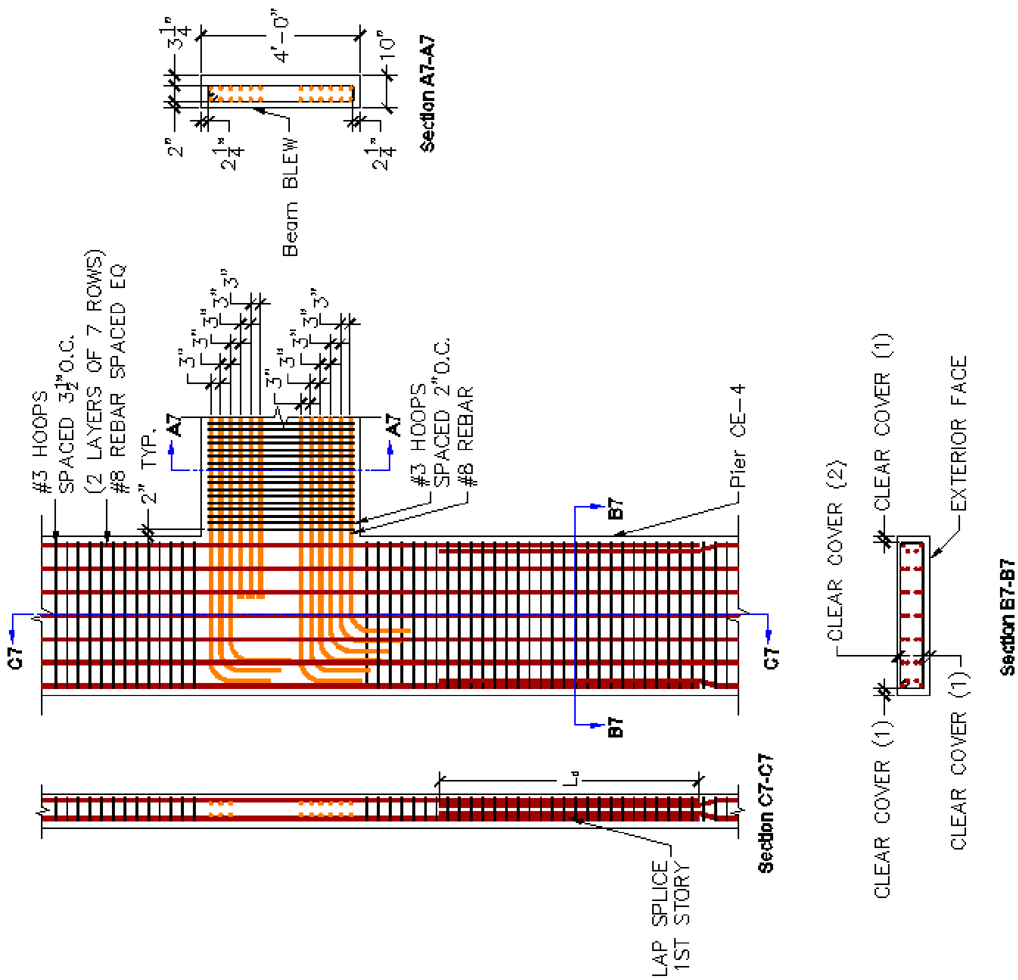


Figure 2.25, Reinforcement Detail 7

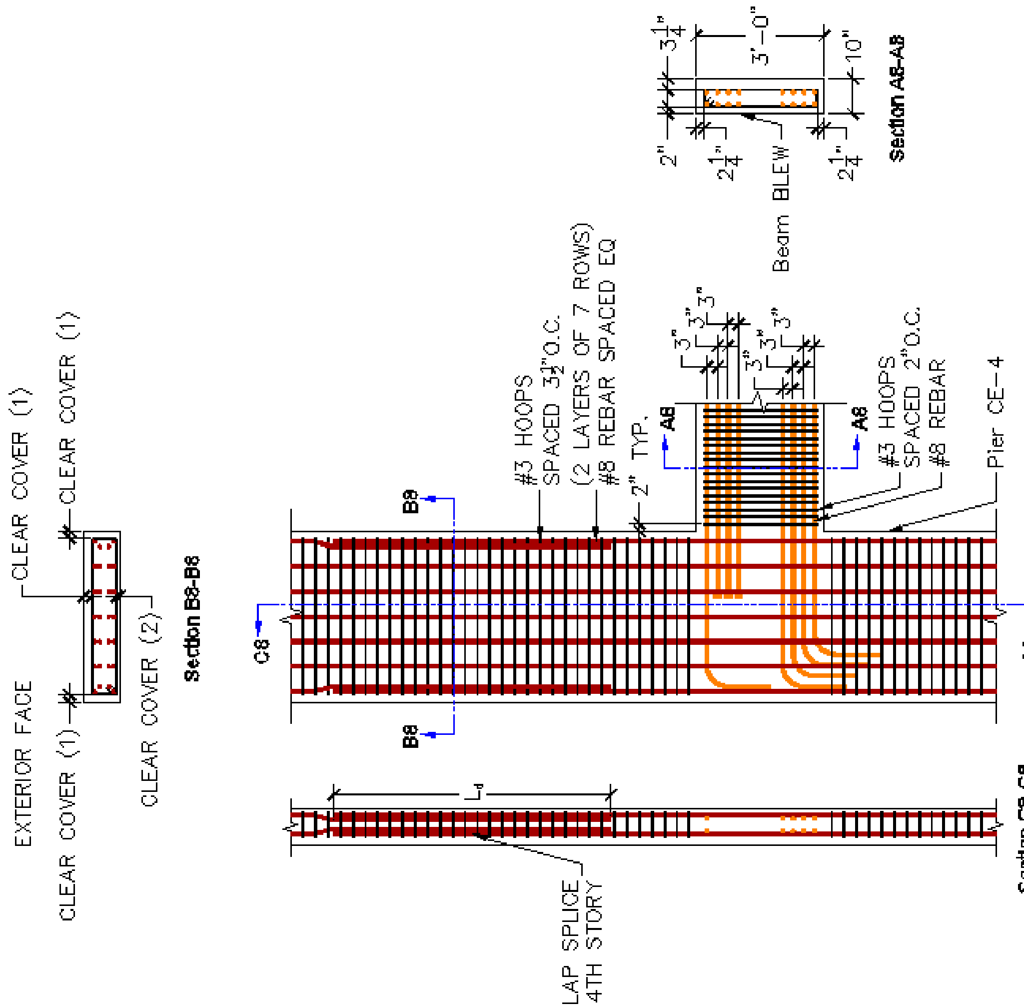


Figure 2.26, Reinforcement Detail 8

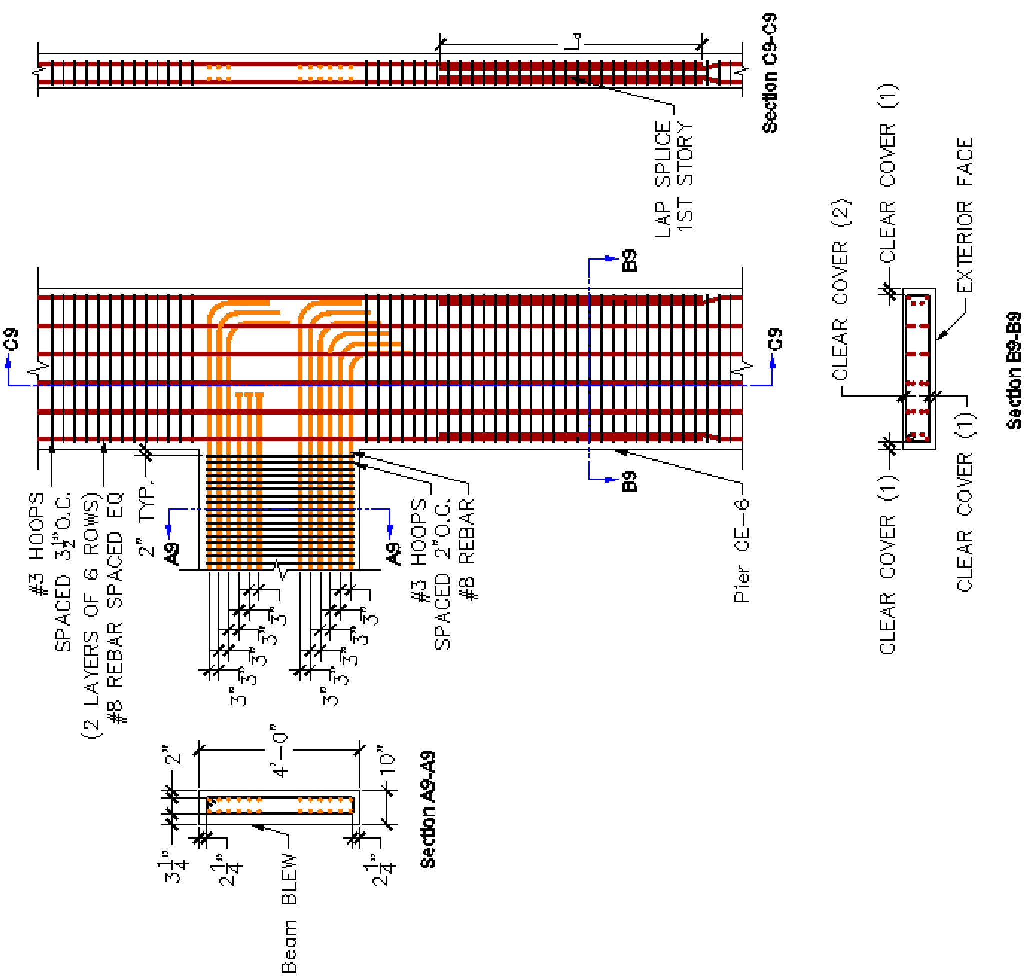


Figure 2.27, Reinforcement Detail 9

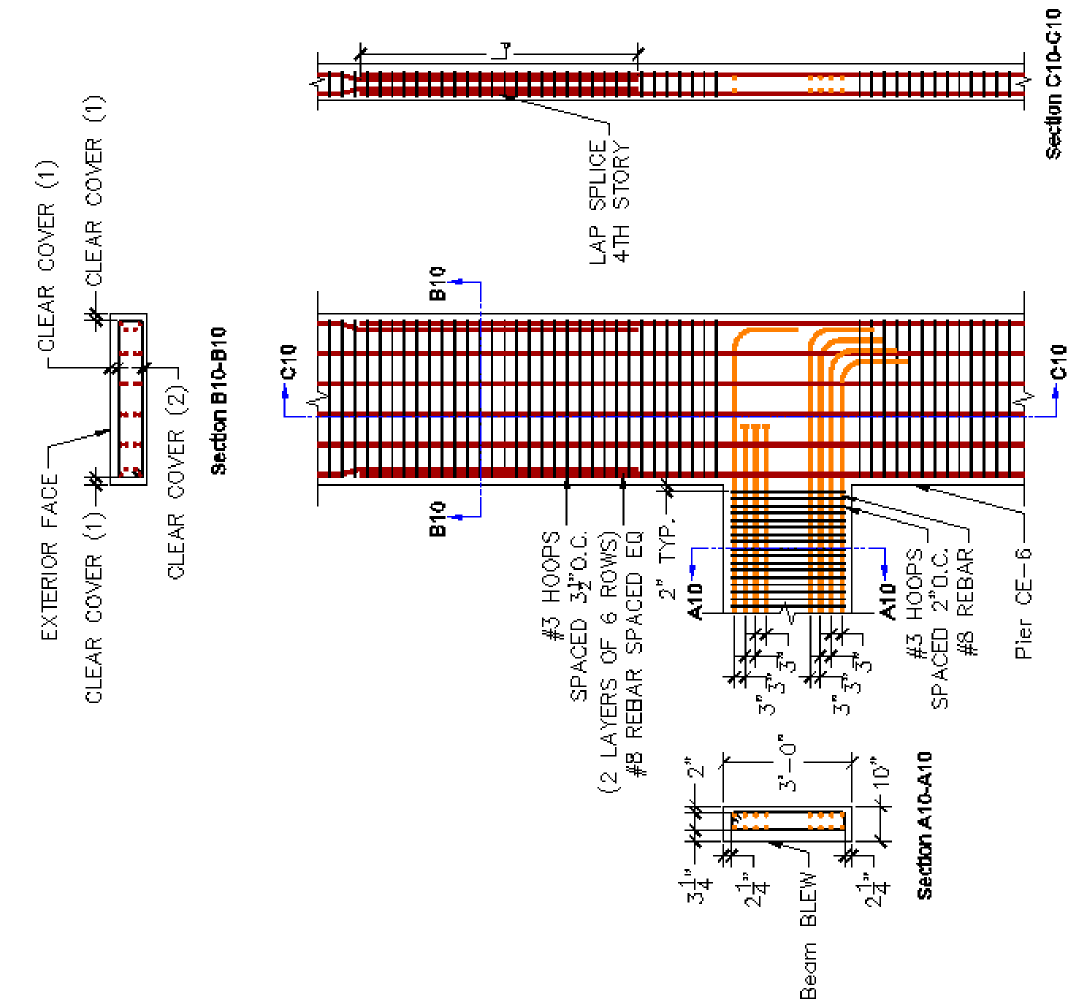


Figure 2.28, Reinforcement Detail 10

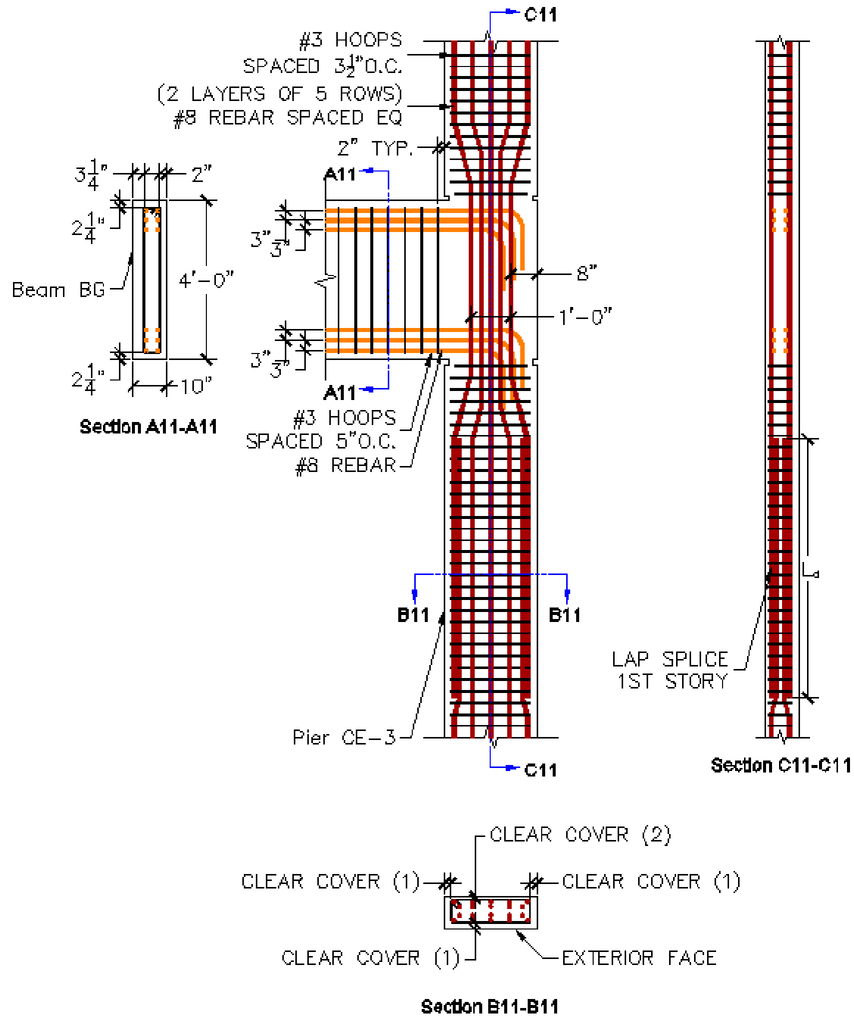


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Figure 2.29, Reinforcement Detail 11

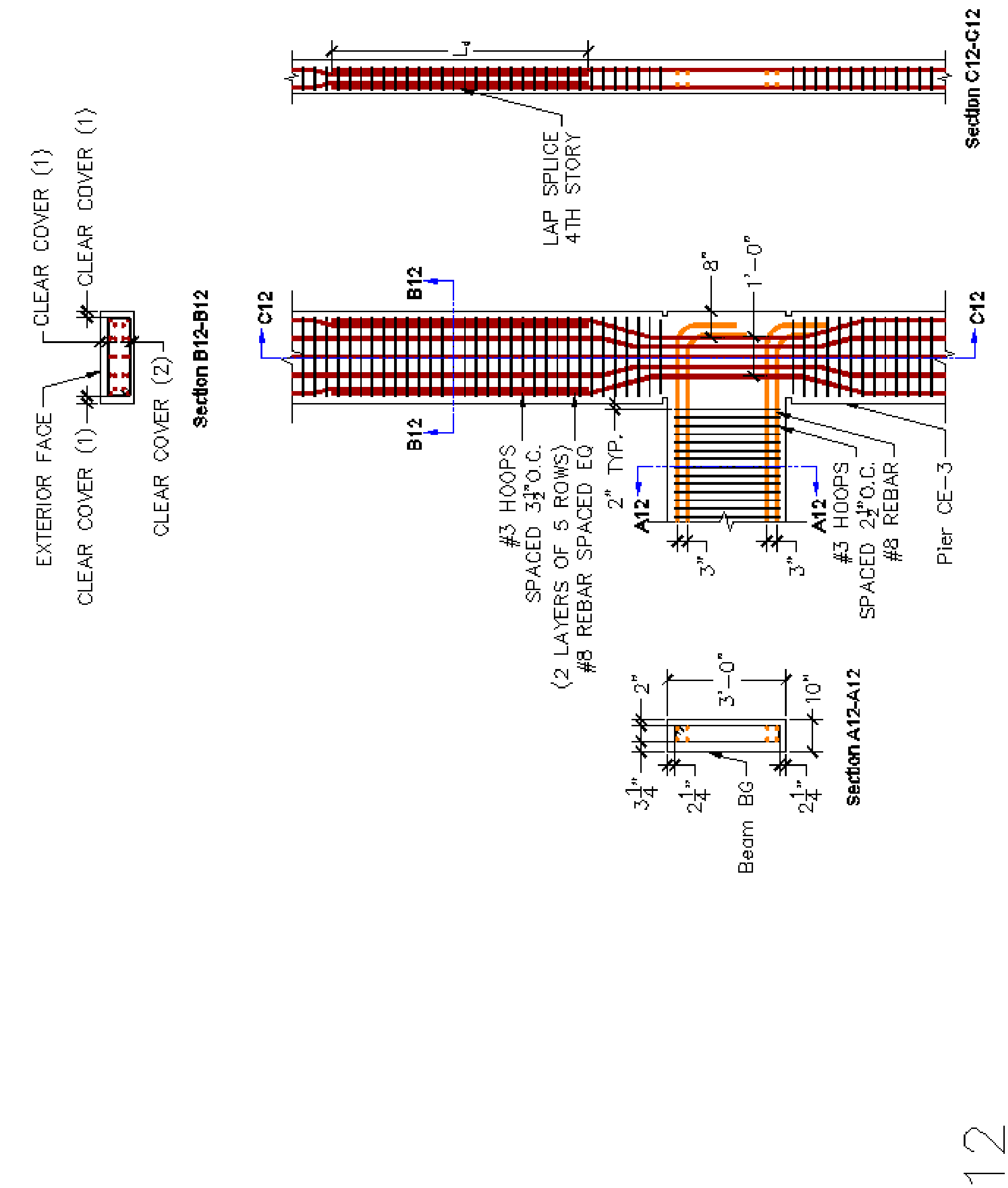


Figure 2.30, Reinforcement Detail 12

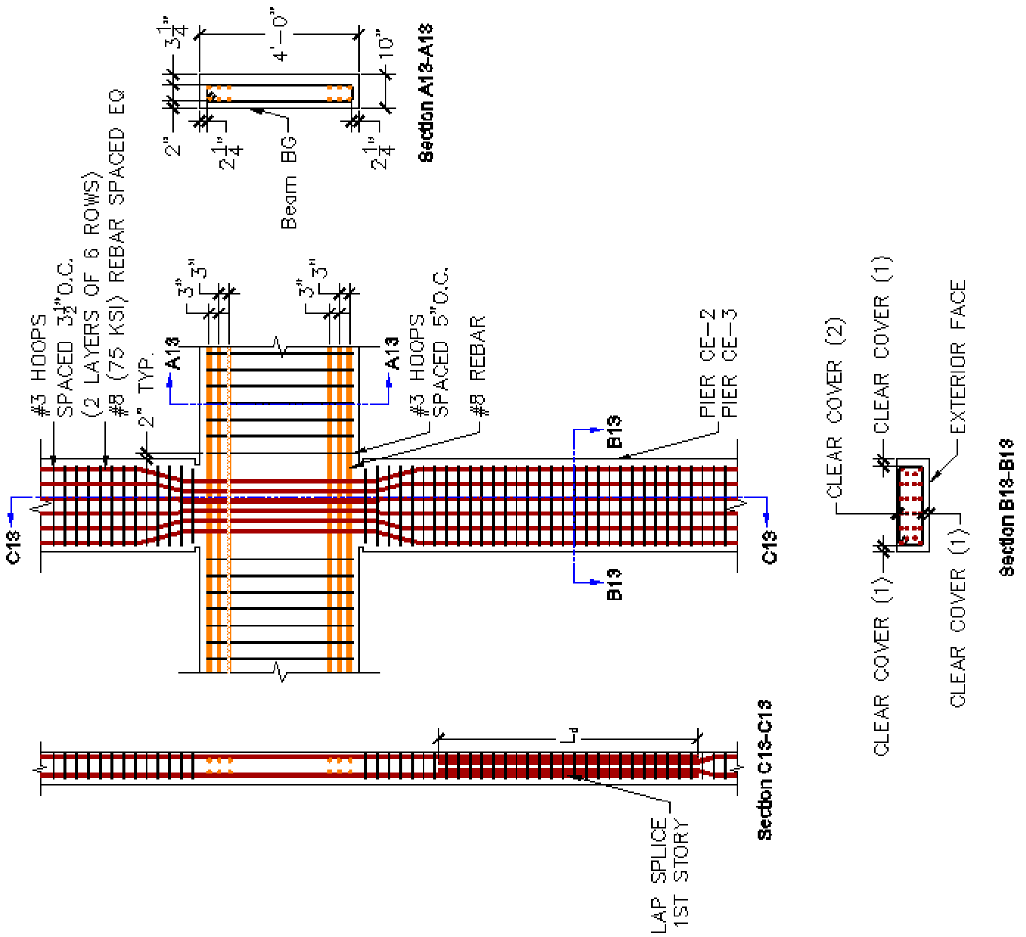


Figure 2.31, Reinforcement Detail 13

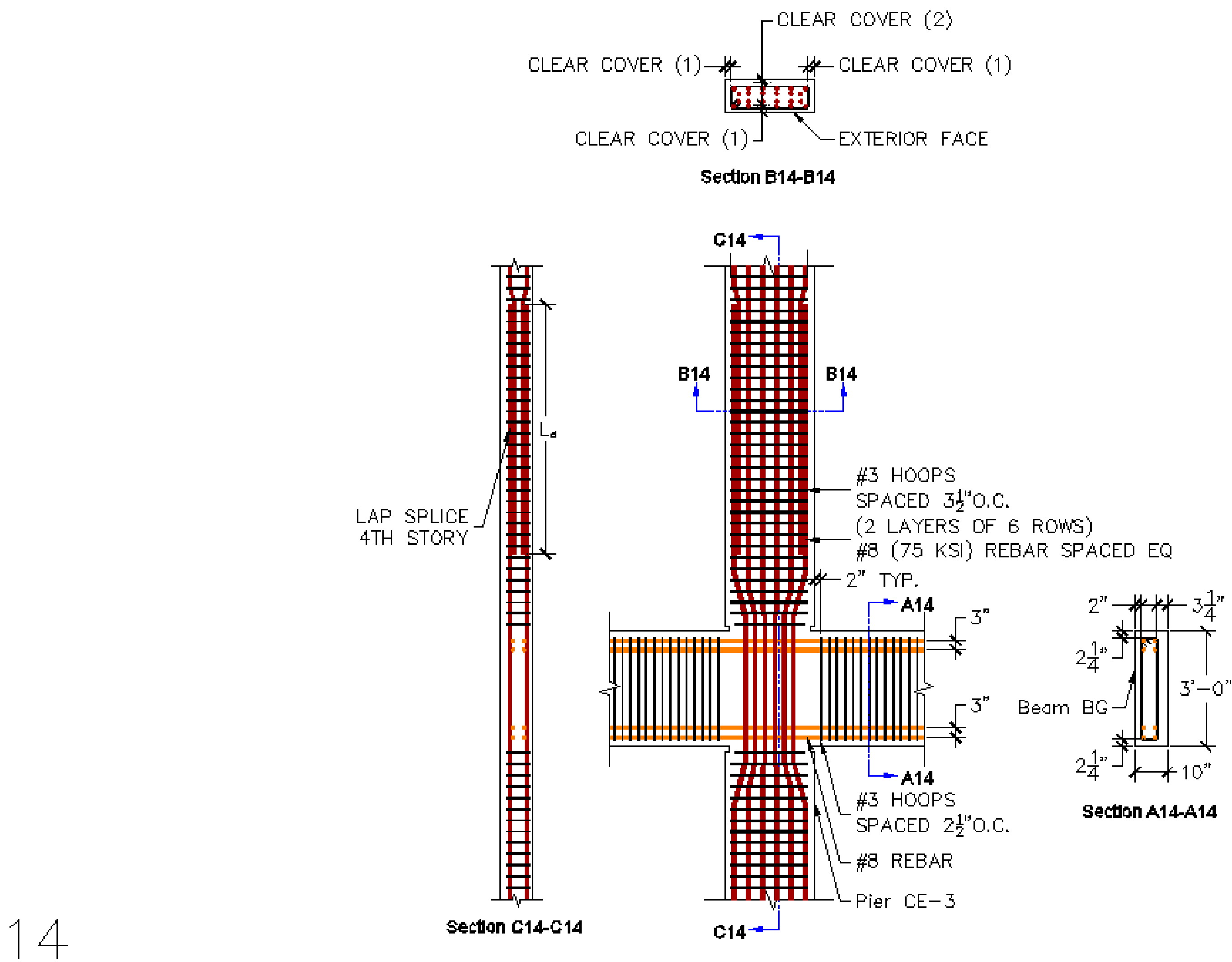


Figure 2.32, Reinforcement Detail 14

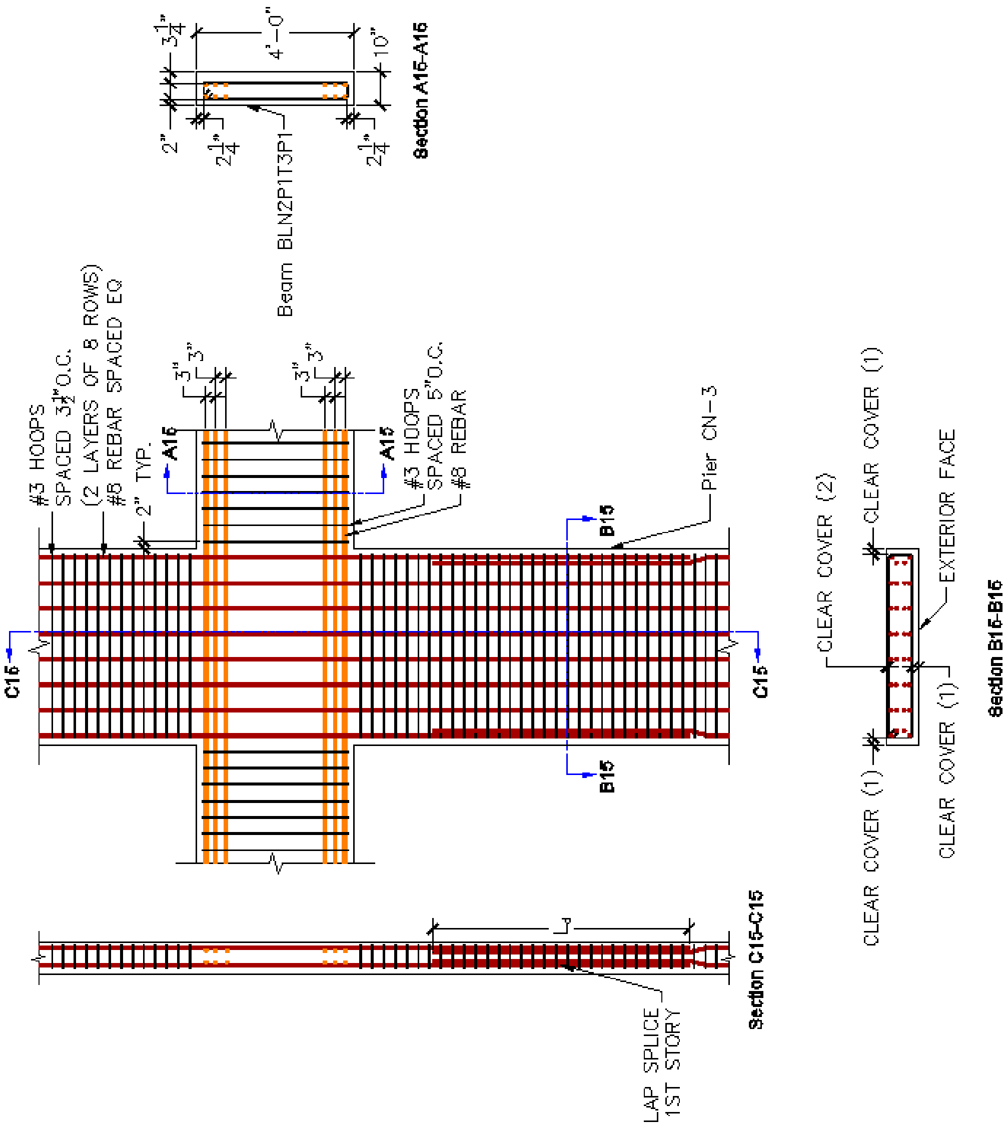


Figure 2.33, Reinforcement Detail 15

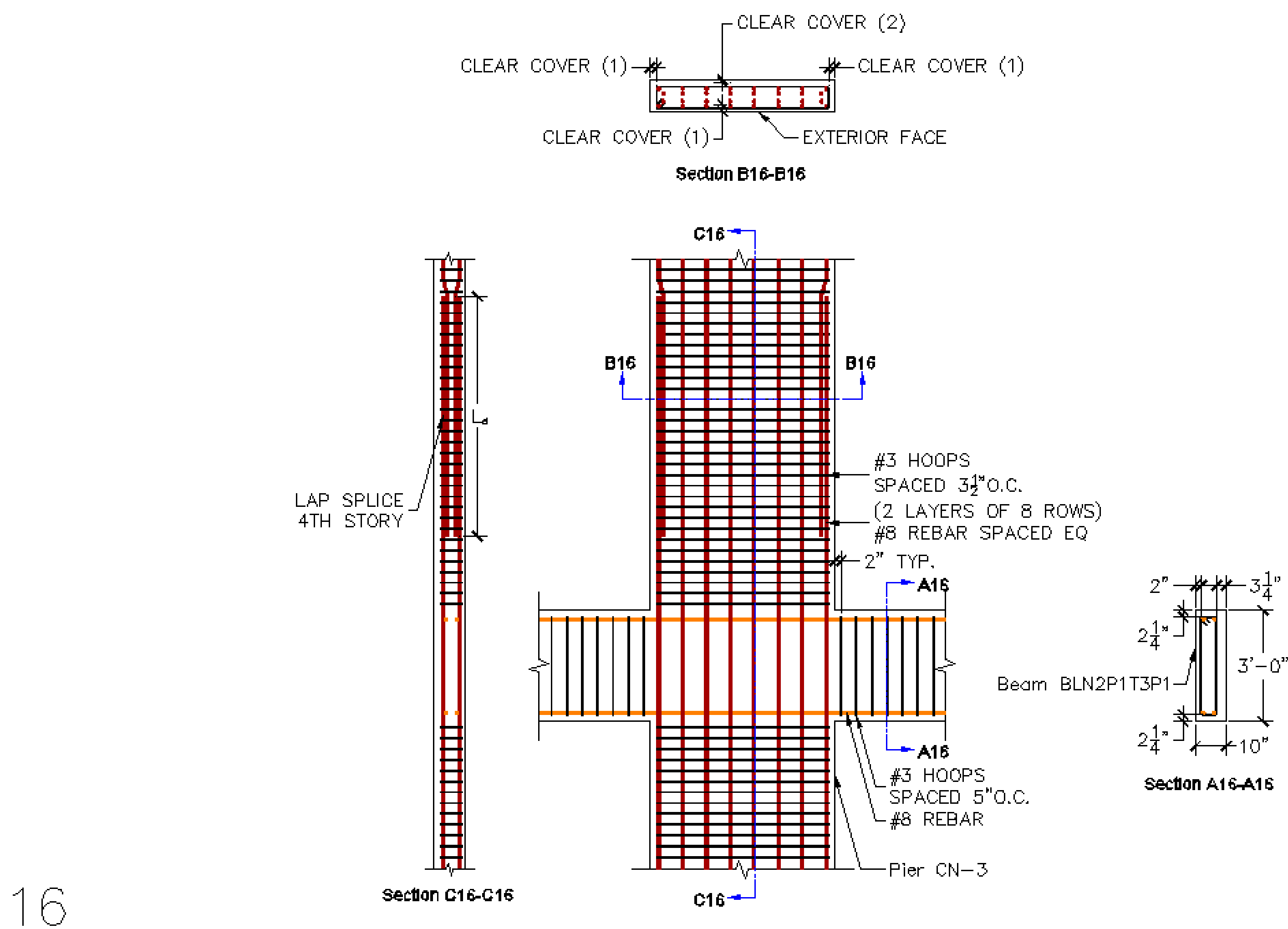


Figure 2.34, Reinforcement Detail 16

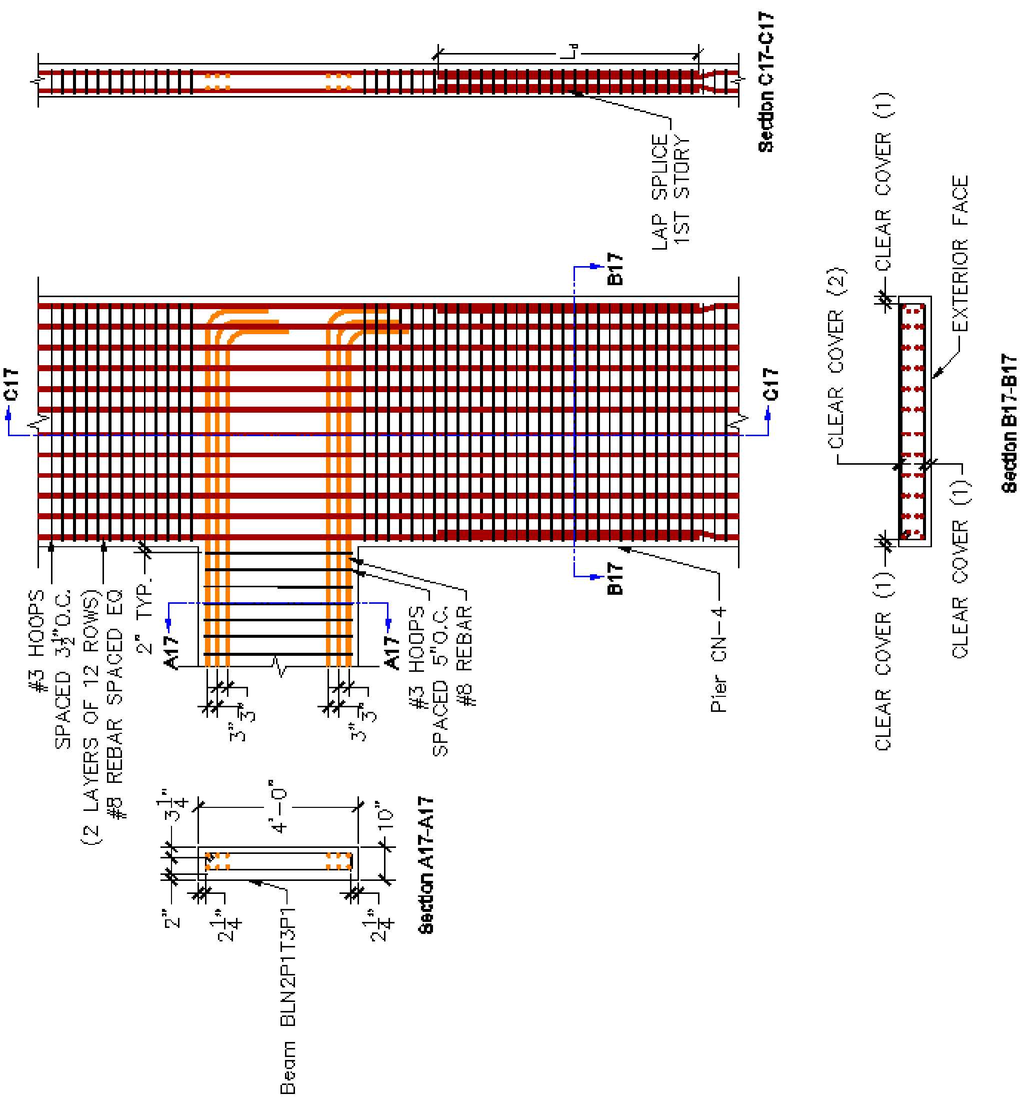


Figure 2.35, Reinforcement Detail 17

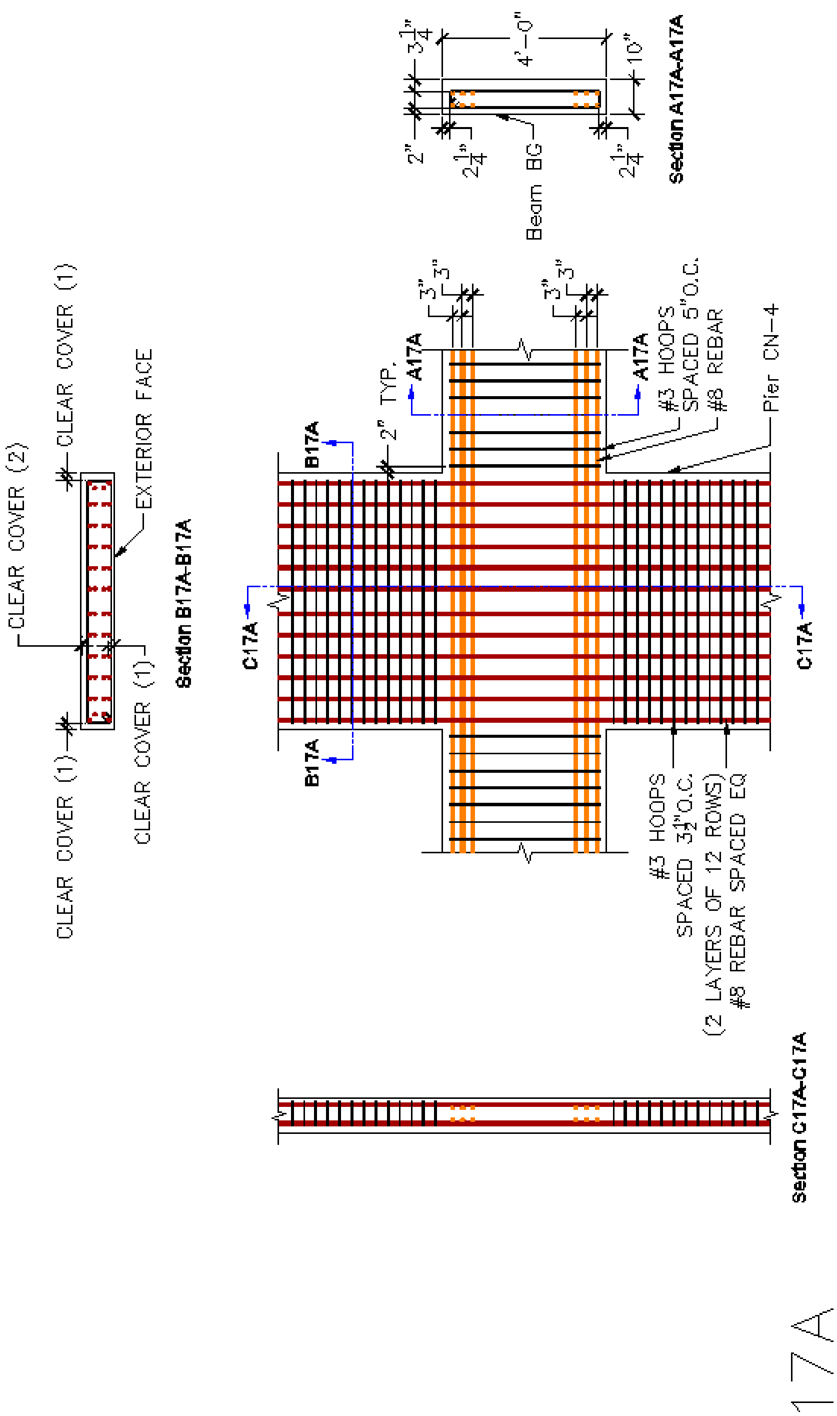


Figure 2.36, Reinforcement Detail 17A

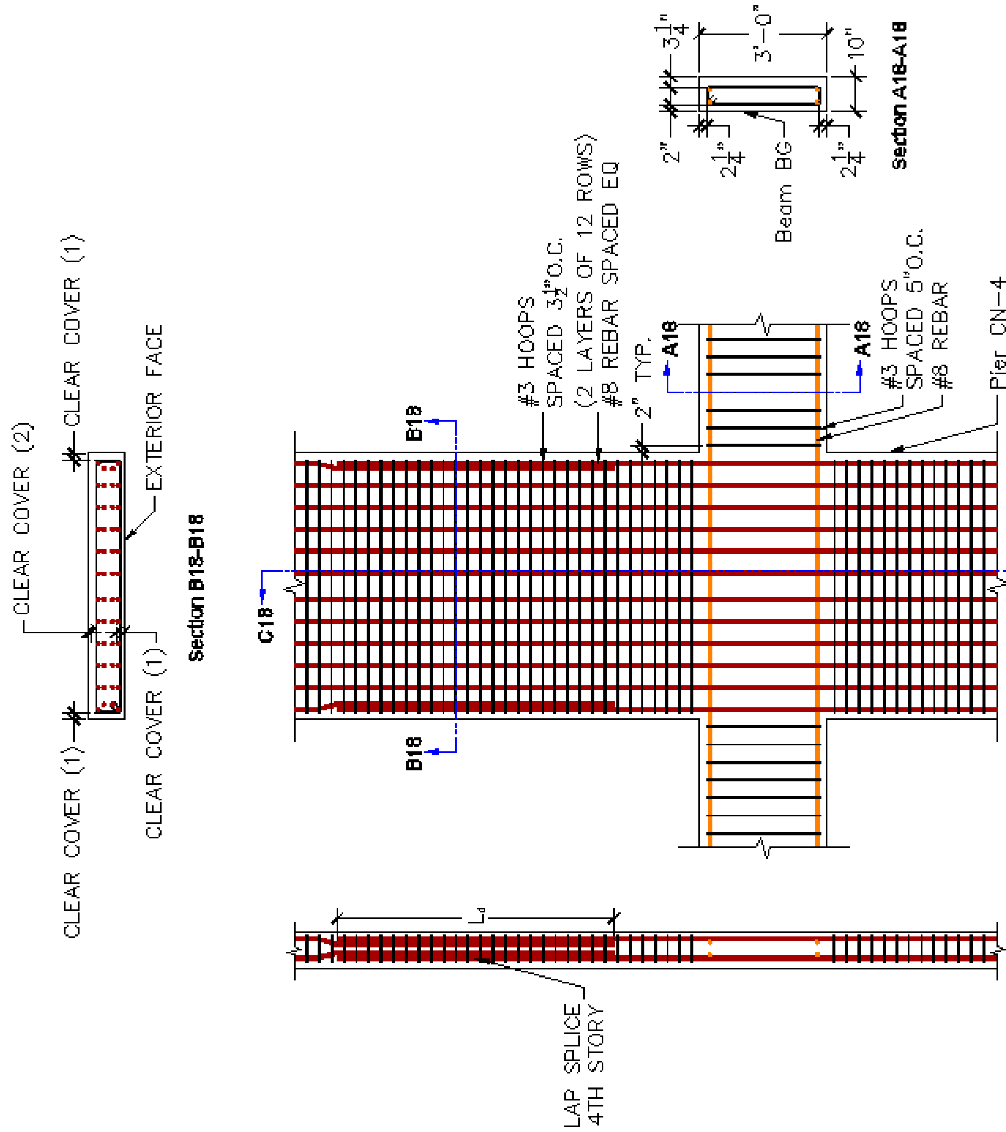


Figure 2.37, Reinforcement Detail 18

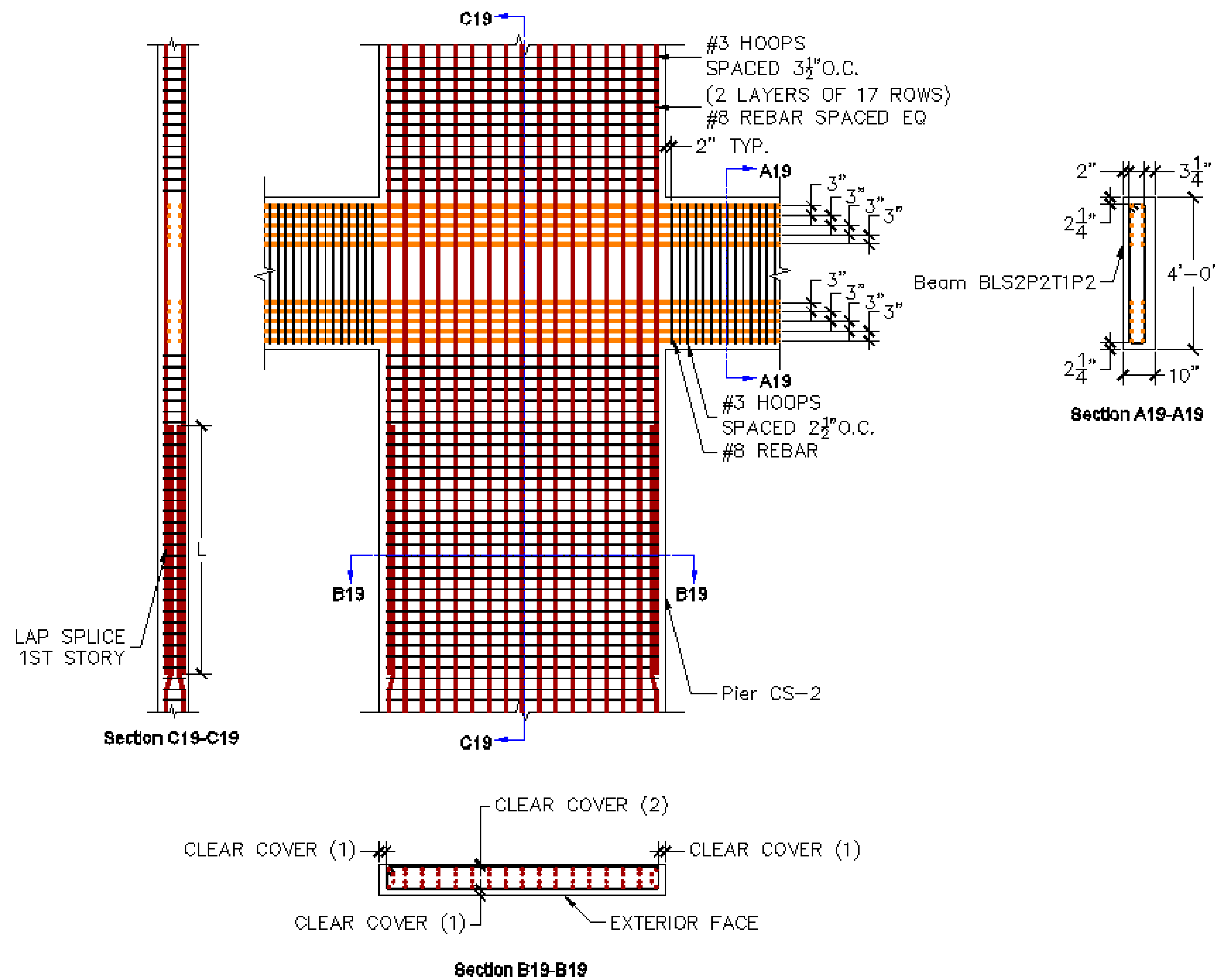


Figure 2.38, Reinforcement Detail 21

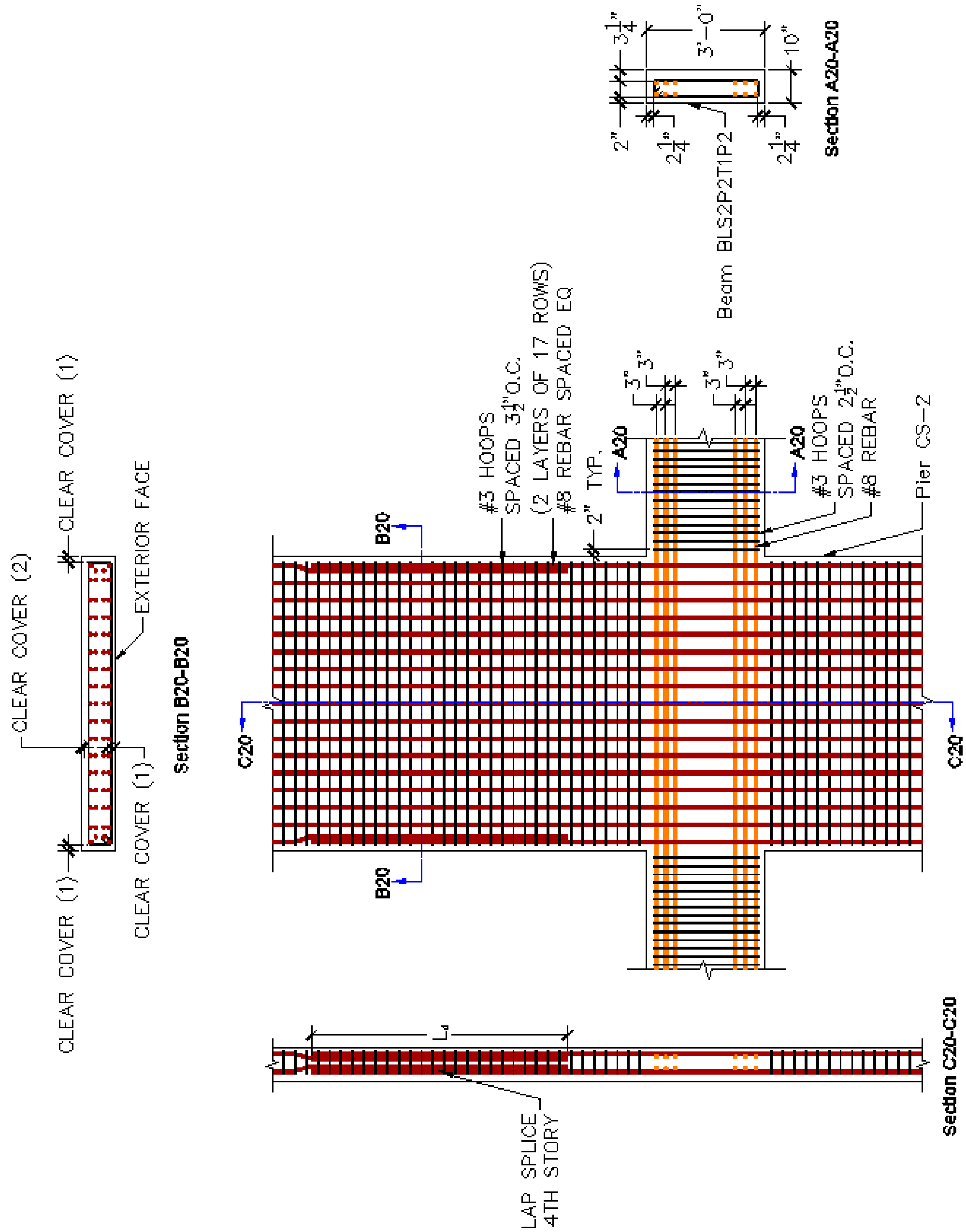


Figure 2.39, Reinforcement Detail 20

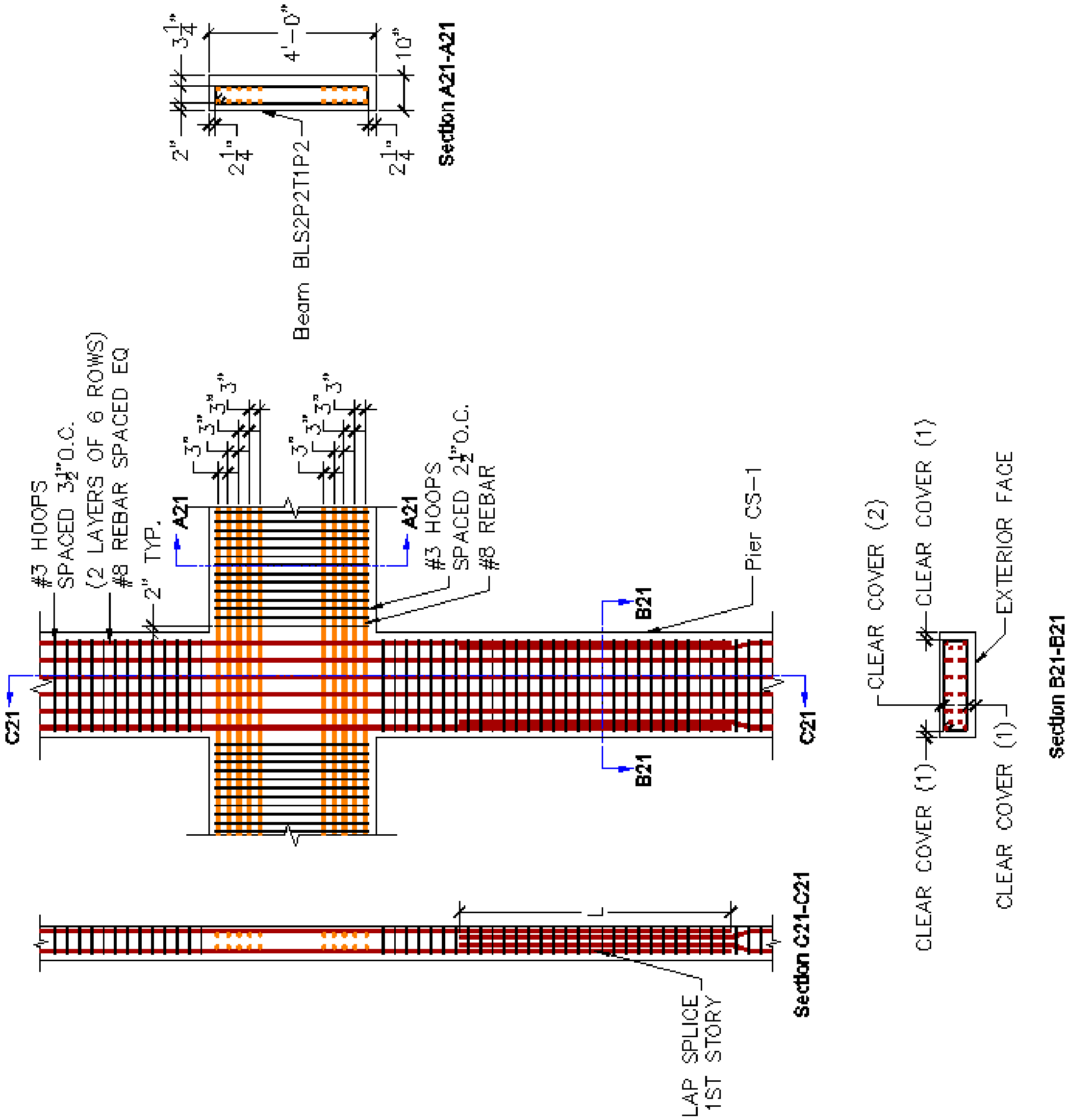


Figure 2.40, Reinforcement Detail 21

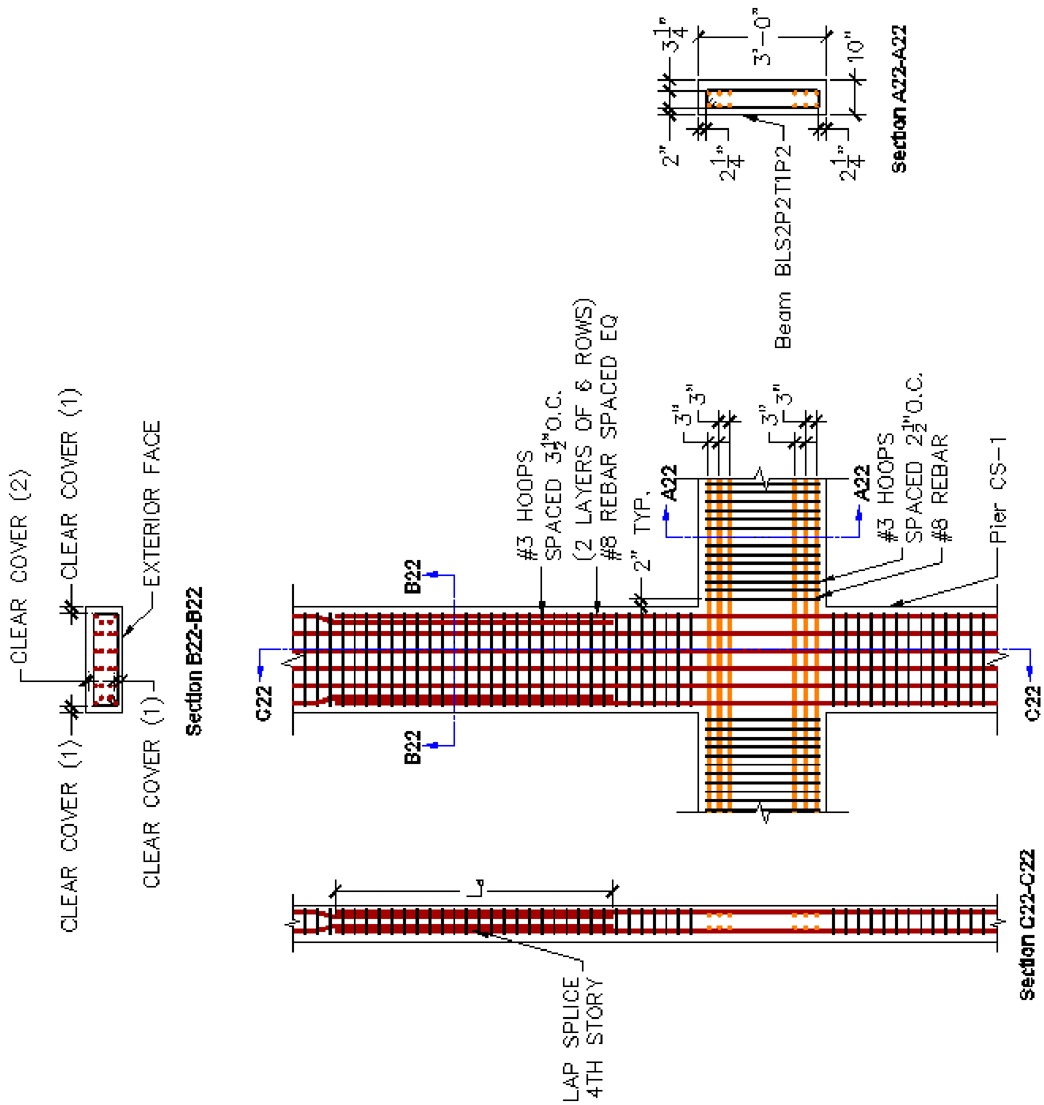


Figure 2.41, Reinforcement Detail 22

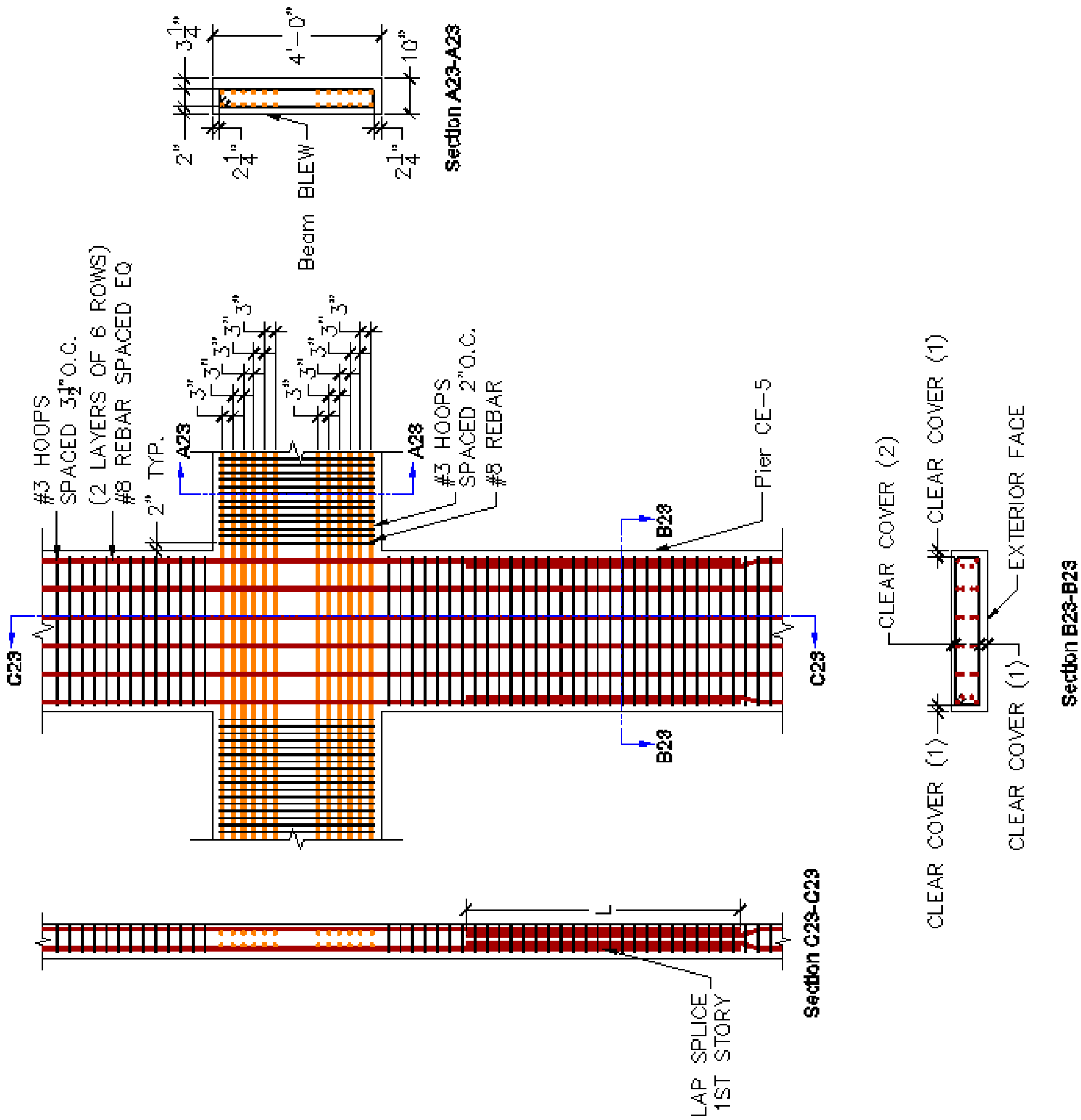


Figure 2.42, Reinforcement Detail 23

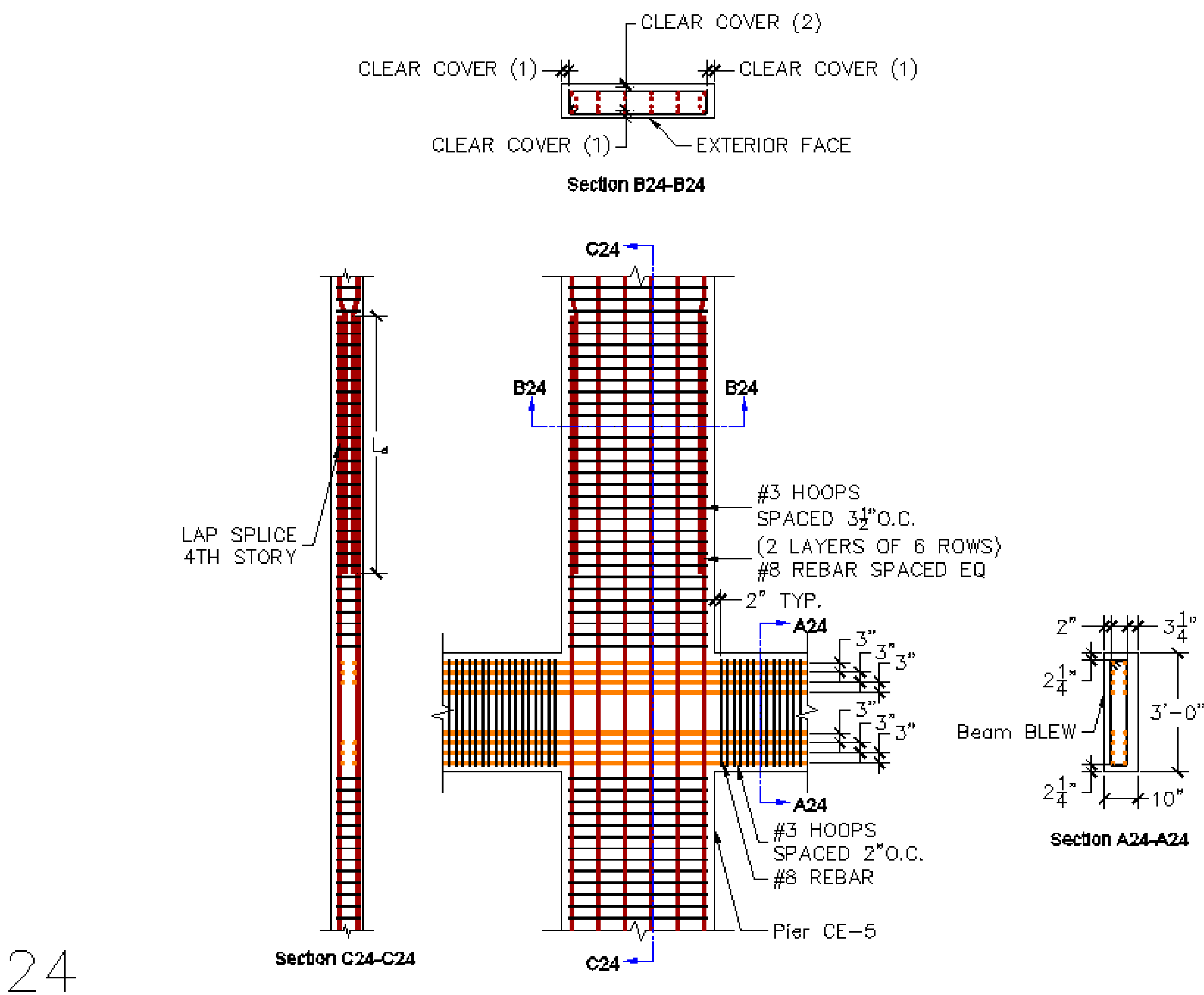


Figure 2.43, Reinforcement Detail 24

Structural Re-Design Evaluation

The redesigns were successful in taming torsion irregularity and soft story irregularity. Design II was found to be more effective against torsional effects and controlling drift along the building's short direction. On the other hand Design I is likely far easier to construct and coordinate, arising from no need for large concrete casting areas nor moving structural components over 100 tons around the site. The only possible constructability issue with Design I is the use of multiple concrete grades (strengths), especially those used in the first story – to tackle soft story irregularity. Performance wise, Design I is stiffer than Design II and the original design in the building's long direction. This is verified by the fundamental building period (original = 0.72 seconds, Design I = 0.62 seconds, Design II = 0.65 seconds) and building drift in the long direction.

Construction Management Breadth

Construction management is a broad topic of study. To maintain focus only two aspects were explored, they are: site logistics and direct construction costs. The two mentioned aspects serve as evaluation criterions for the structural solutions and façade wall redesign.

Literature Review and Benchmarks

Site properties, safety requirements, and environmental regulations define the progression and type of construction. These considerations will be explored in the literature review.

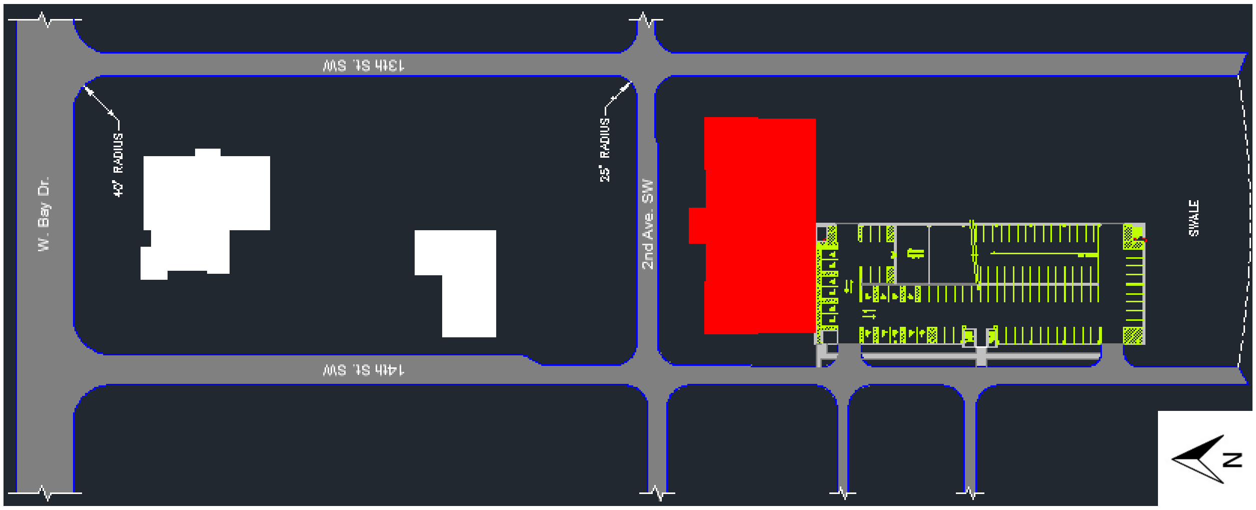


Figure 3.1, Major Roads and Facilities near LMOB

Table 3.1, Cargo Capacity and Turning Radius of Various Truck Types Source: *Texas Department of Transportation Roadway Design Manual*		
Truck Type	Maximum Cargo Length	Turning Radius for 90° Turn
Single Unit – 20’-0” Wheelbase	22’-0”	42’-0”
Semi-Truck – 23’-6” Wheelbase	30’-0”	40’-0”
Semi-Truck – 31’-4” Wheelbase	37’-4”	45’-0”
Semi-Truck – 42’-0” Wheelbase	42’-0”	45’-0”

The site which LMOB is built on is part of a medical complex and is adjacent to commercial businesses. Since adjacent businesses will continue to operate, construction traffic and activities were planned to have minimal impact on the roads. Figure 3.1 shows the facilities and roads flanking LMOB. Location of the construction site also impacts building component sizes and matter which the building components arrive to the site. Table 3.1 and Figure 3.1 shows the turning radius of various vehicles and available turning radiuses on site, respectively. Vehicles accessing 14th Street SW and 13th Street SW from West Bay Drive using a right hand turn cannot exceed 40 ft. However, semi-trucks with up to a 42’-0” wheelbase can access 14th Street SW and 13th Street

SW from West Bay Drive using a left hand turn. Tertiary roads have small turning radiuses at the intersections, this effectively rules out using 2nd Avenue SW as a location to offload material and equipment from single unit and semi-trucks.

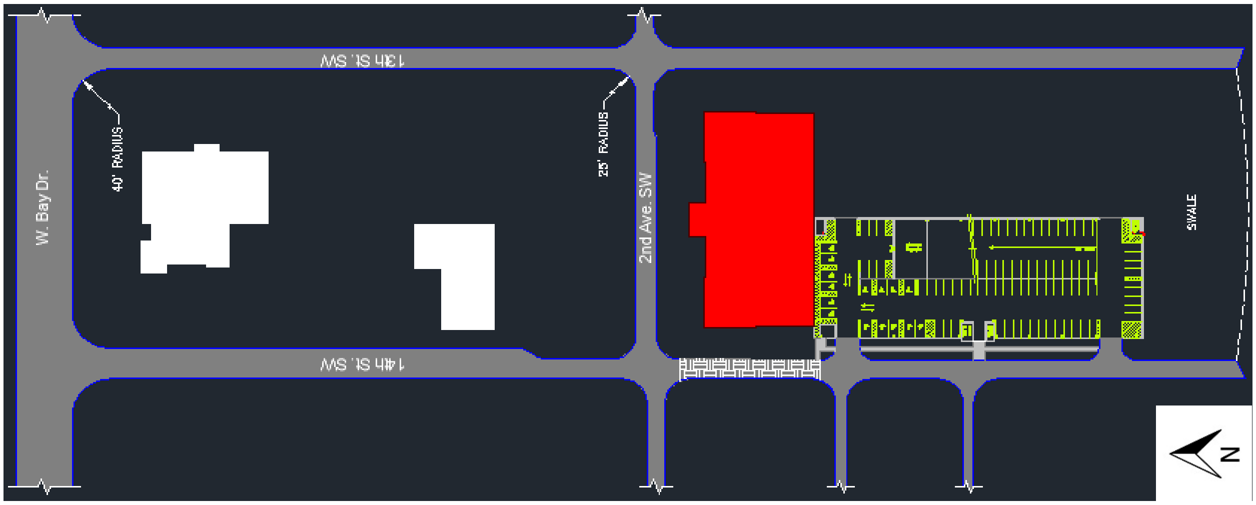


Figure 3.2, Option 1 for Offloading Area

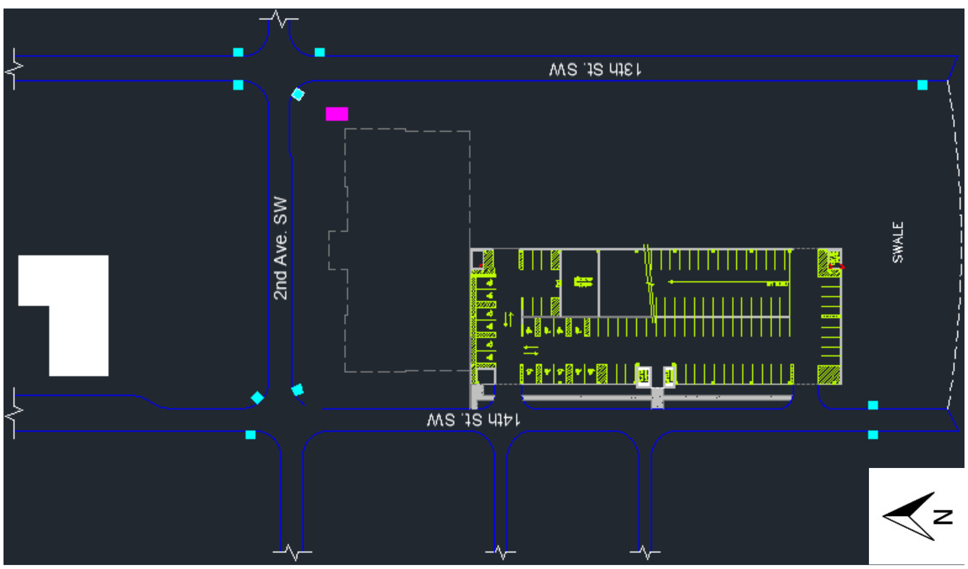


Figure 3.3, Locations of Storm Drains (Turquoise) and On-Site Utilities (Magenta)

Two options are left to offload material and equipment from single unit and semi-trucks, one is to close the hatched portion of 14th Street SW, as illustrated on Figure 3.2. Closing the highlighted portion of 14th Street SW will require a bypass through the adjacent parking lot and reduce patient accessibility to the parking deck. Reduced patient accessibility to the parking deck is not a concern because the parking deck will be renovated as LMOB is constructed. The second option is to prepare the area next to the shoulder of 13th Street SW. Option two requires additional time to properly outline, grade, and install drainage. Based on labor productivity values in R.S. Means

2013, it should take a minimum of one week to finish the tasked mentioned immediately above. Though extra time is required, there are benefits to the second option. One benefit is that the crane does not need to lift material and equipment over part of the parking structure. Lifting material and equipment over an adjacent facility requires the adjacent facility to be vacated, in order to prevent injury should the crane accidentally drops the load (OSHA 1926.704(e) and 1926.753(d)).

Further details of construction parking and detailed site logistics during each phase of construction will be covered later, now the site utilities and stormwater management will be addressed. Figure 3.3 illustrates the existing electric and water utilities on site, as well as the stormwater drains and drainage swale locations.

During construction, electrical and plumbing utilities will need to be made available to the construction crews. The first task is to extend the existing utilities to the north-east corner of the existing parking garage. Extending the existing utilities to the north-east corner of the existing parking garage is advantageous because it reduces material and labor costs by allowing both the parking garage renovation and LMOB construction to share a single electrical and plumbing feed. Another advantage is the close proximity between the extension and the future LMOB utility room. A second utility extension will be required for on-site construction management and will extend to the construction trailers. Unlike the permanent utility extension to the northeast corner of the existing parking garage, the extension to the construction trailers will be temporary.

Before construction begins the site will need to be fenced off and stormwater management systems will need to be installed. Fencing off the construction site will prevent non-construction entities from accessing the site and potentially injuring themselves. In addition, fencing off the construction site will reduce the possibility that construction equipment and materials are stolen or sabotaged by creating a secure area.

The importance of stormwater management lays in the need to reduce site erosion, stormwater sedimentation and pollution. Any violation or failure to comply is wholly responsible by the contractor. Enforcement is done through either a state environmental agency or the EPA, who will fine or close the site until stormwater management systems and strategies are in place (EPA Stormwater Management Guide). As defined by the *Clear Water Act* [Title 40 of the Code of Federal Regulations (CFR) 123.25(a)(9), 122.26(a), 122.26(b)(14)(x), and 122.26(b)(15)], stormwater management applies to site activities entailing clearing, grading, and excavating activities that disturbs more than one acre.

Stormwater runoff begins as rain or melting snow that does not percolate into the soil, instead it flows over land EPA Stormwater Management Guide). As stormwater runoff flows it picks up debris and pollutants in the way. The pollutants can range from trash and sediments to grease and other toxic chemicals.

The impact is monumental, whereby nearby waterways and habitats are harmed. This includes but not limited to maritime navigation impedance; cloudy water that prevents sunlight from reaching aquatic plants and clogs fish gills. In a year, runoff from a one acre construction site, without stormwater management systems, causes up to 45 tons of sediment and soil loss (EPA Stormwater Management Guide). As can be seen in Figure 3.4 runoff from construction sites, without stormwater management systems, is the largest land-based source of soil erosion. Impervious surfaces like roads increase the runoff quantity and velocity. Increasing the runoff quantity and velocity makes things worse with faster erosion rates and the potential for flooding.

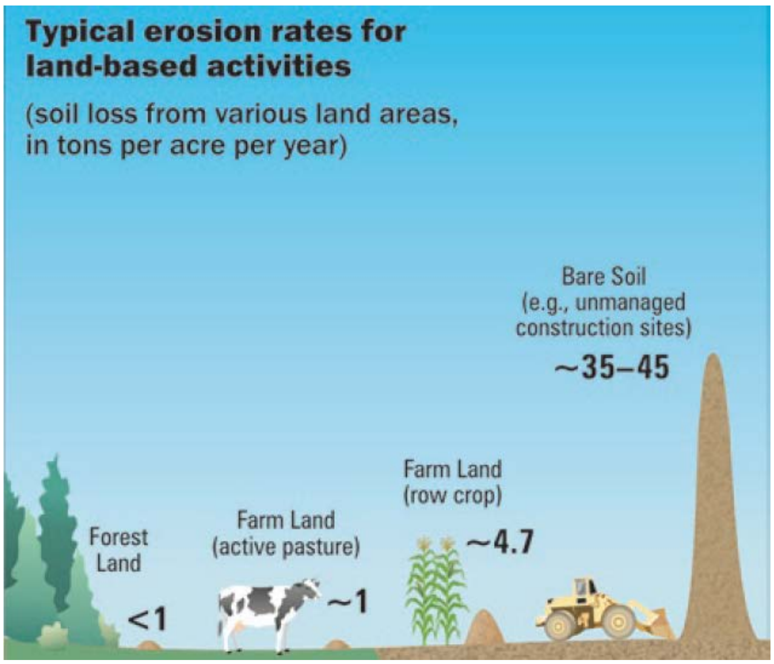


Figure 3.4, Typical Soil Loss/Erosion Rates of Selected Activities
Source: *EPA Stormwater Management Guide*

Table 3.2, Stormwater Management Site Considerations				
Site Considerations	Climate	Topography	Soil	Vegetation
Impact	- Seasons - Rain frequency, intensity, and duration	- Slope - Area exposed	- Compaction - Permeability - Structure of Soil	- Proximity of plants can helps absorb the rain's kinetic energy - Root system o Binds the soil together o Increases rain infiltration

Stormwater management is site specific, it means the climate, topography, soils, and vegetation have an impact. Table 3.2 shows the typical considerations concerning the climate, topography, soils, and vegetation.

Taming stormwater runoff is a two pronged approach. The first approach is structural based. Here physical barriers to erosion and sedimentation are built and used. The EPA recommends that any barrier should keep the soil in place, prevent it from moving. To do this there are four methods that reinforce each other.

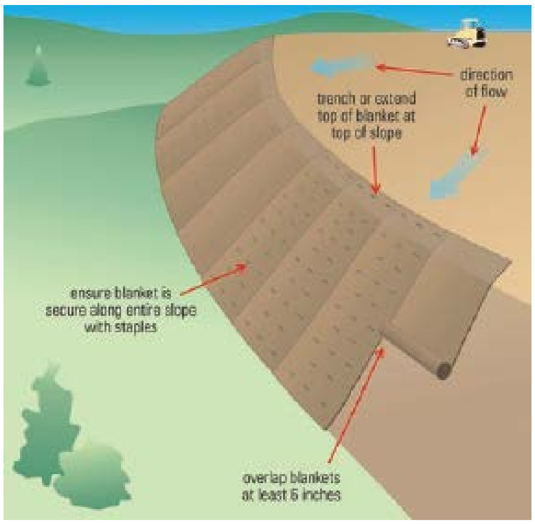


Figure 3.5, Erosion Control Blankets
Source: *EPA Stormwater Management Guide*

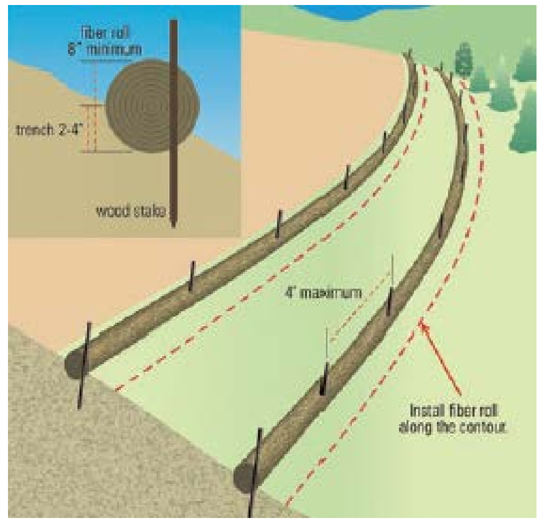
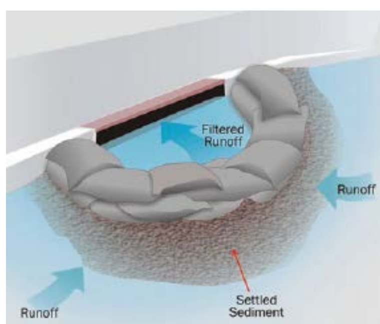


Figure 3.6, Erosion Control Fiber Rolls
Source: *EPA Stormwater Management Guide*



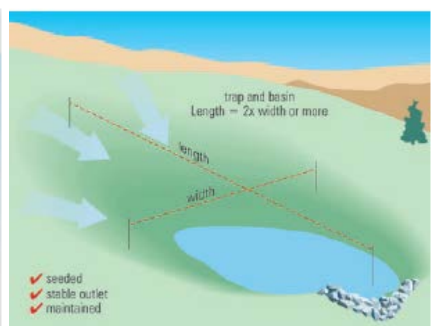


Figure 3.7, Silt Fence Figure 3.8, Drain Filter Figure 3.9, Runoff Basin
Source: *EPA Stormwater Management Guide*

First, stabilize the site quickly, especially exposed soils and slopes. Stabilizing the site can be done through control blankets, fiber rolls as shown in Figure 3.5 and Figure 3.6. Second, reduce impervious surfaces to promote rain infiltration into the ground. Next, the site perimeter must be controlled. Controlling the site perimeter is preventing runoff to contact disturbed areas of the construction site, filter any runoff originating from the site to capture sediment, or collect all runoff into a sediment basin; these site perimeter controls are illustrated in Figure 3.7, Figure 3.8, and Figure 3.9 respectively. Lastly, the most important is to minimize the area and duration of exposed soils.

Site Logistics

Construction Site Organization and Phasing

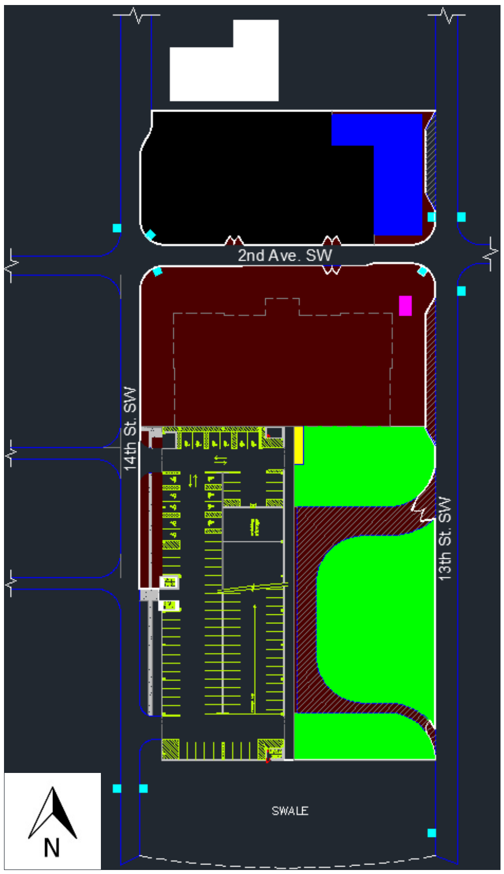


Figure 3.10, Phase I of Design I

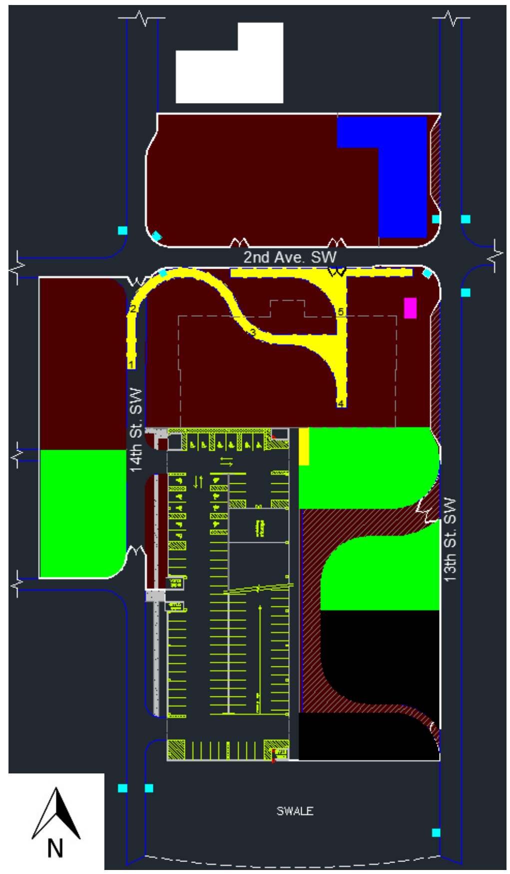


Figure 3.11, Phase I of Design II

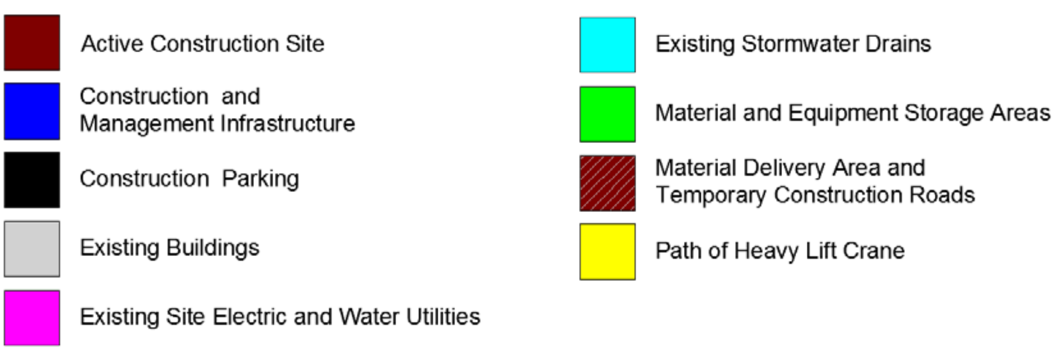


Figure 3.12, Color Key for Construction Phasing Diagrams

Site logistical details vary with changes in building design. The outcome is different construction phasing between Design I and Design II (Tilt-Up). Though there are differences between the site logistics of the two designs, similarities exist. For one, the site office is located at the corner of 2nd Ave. SW and 13th St. SW, as shown in Figure 3.10 and Figure 3.11. Refer to Figure 3.12, which is the color key, for construction phasing illustrations. One reason for placing the site office in the particular location is to reduce the distance between the construction site's utilities. There are no foreseeable problems in the construction trailer's placement because the site is an undeveloped lot 2nd Ave. SW and owned by the same owner – The Greenfield Group. The second reason is to increase the material and equipment storage area that is next to the parking garage. The purpose of the site office is to conduct on-site meetings and as centralized base for construction coordination.

Another similarity between the Design I and Design II site logistics is the position of the general purpose 5 ton crane. The general purpose 5 ton crane is positioned at the northeast corner of the parking garage to ensure that all positions within LMOBS footprint, material offloading areas, and a majority of the material storage areas can be reached without repositioning the crane.

Next, site logistics phases for Design I and Design II will be discussed in the following paragraphs. The site logistics phases for Design I is fairly constant and can be summed up by Figure 3.10. One reason is that the concrete shear walls are cast-in-place and cast upright, where the majority of the site area is used for the materials and equipment. In terms of the traffic flow through and adjacent to the site the goal is to ensure an uncongested and smooth flow. To achieve this, shoulders will be built along existing roads, as well as the placement of temporary construction roads. The shoulders on 13th St. SW serve to allow the delivery truck to pulls over and register their shipment with the superintendent in the construction trailer.

Once the shipment is registered the delivery truck progresses to the unloading area on-site. The unloading area is the wide stretch of temporary construction roads. Either the delivery can be offloaded to the on-site storage areas or directly placed into LMOB. Less wide temporary construction roads are primarily for directing delivery truck off the construction site. Delivery trucks get are directed towards the main roads by driving through the dirt road that is at the edge of the drainage swale and onto 14th St. SW to exit. Construction parking is located next to the construction trailer to minimize impact to the permanent parking lots adjacent to the surrounding businesses and increase the material and equipment storage areas next to the general purpose 5 ton crane.

Unlike the site logistics phases for Design I, Design II site logistics phases is far more complex and requires a longer construction time. Figure 3.13, Figure 3.14, and Figure 3.15 illustrates the change in site utilization as construction progresses. The complexity is primarily due in part to casting the structural concrete walls flat on the ground and tilting them upright. Casting locations for the structural tilt-up walls are shaded charcoal grey in Figure 3.16.

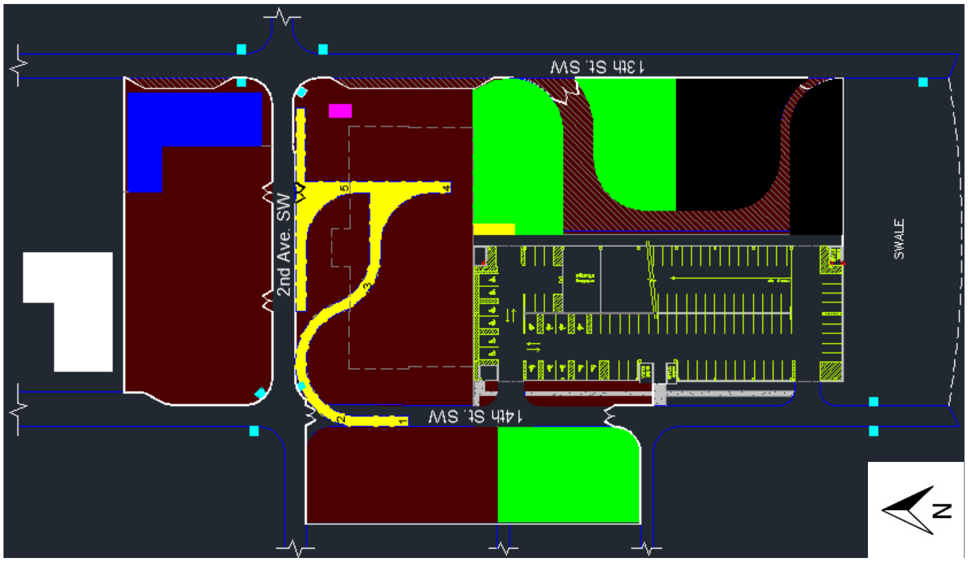


Figure 3.13, Phase II of Design II

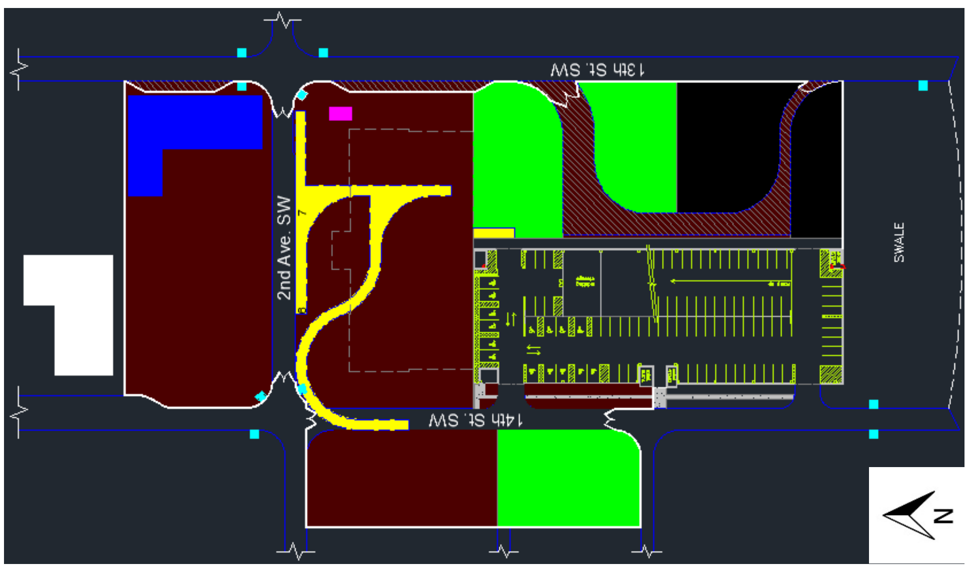


Figure 3.14, Phase III of Design II

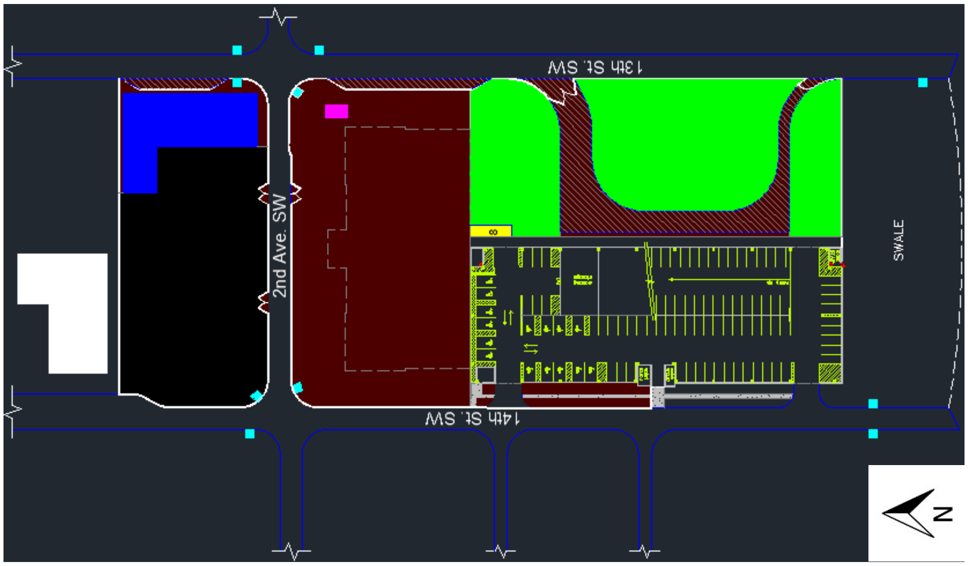


Figure 3.15, Phase IV of Design II

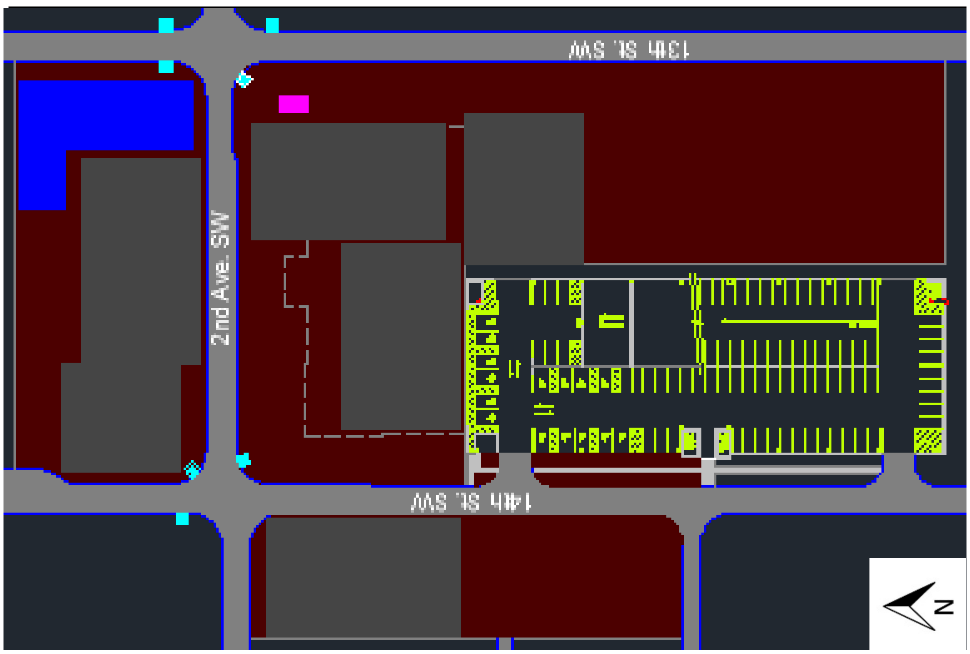


Figure 3.16, Casting Locations for Tilt-Up Walls

The first phase of construction consists of:

1. Preparing the site for construction
2. Take delivery and place construction trailers
3. Extend and install construction utilities
4. Construct formwork for the structural concrete tilt-up walls and foundation
5. Place concrete into formwork and let cure
6. Place temporary pre-cast concrete footings for temporary tilt-up supports

Once the concrete tilt-ups and footings are cured then the second phase begins. Phase two primarily consists of tilting up the structural concrete walls into place, erecting temporary supports, and casting the connections between the structural concrete tilt-up walls. The heavy lift crane will only be in use during the second phase. Each structural concrete tilt-up wall will be lifted in a certain sequence to reduce the number of time which the heavy lift crane changes location.

The order in which the structural concrete walls will be tilted up and temporary braced are listed below.

1. East structural concrete walls
2. Western half of the south structural concrete walls
3. Eastern half of the south structural concrete walls
4. West structural concrete walls
5. Western half of the north structural concrete walls
6. Eastern half of the north structural concrete walls

The path of the heavy lift crane is highlighted yellow in Figure 3.13 and Figure 3.14. As evident in Figure 3.14, 2nd Ave SW will be closed to regular traffic. Closing 2nd Ave SW allows the north structural concrete walls to be tilted into place with endangering non-construction traffic. For the week that 2nd Ave SW is closed non-construction traffic will be rerouted to use the dirt road at the edge of the swale.

In the final phase will entail a reduction the construction site and several relocations. Storage area to the west of 14th St. SW will be relocated to the parking lot in the second phase, while the parking lot in the second phase will be moved next to the construction trailers as seen in Figure 3.15. The heavy lift crane will be demobilized; in its place will be a general purpose 5 ton lift crane. The primary reasons are that the heavy lift crane is expensive to rent and the large capacity will not be utilized efficiently.

<u>Stormwater Management</u>

The stormwater management system for the LMOB construction site will consists of:

1. Closing off the site with a silt fence
2. Placing sand bags at the base of the silt fence
3. Place sandbag filter around stormwater drains as in Figure 3.8
4. Apply gravel layer on top of soil that is bare
5. Guide temporary construction run-off to the swale at the south end of the parking garage

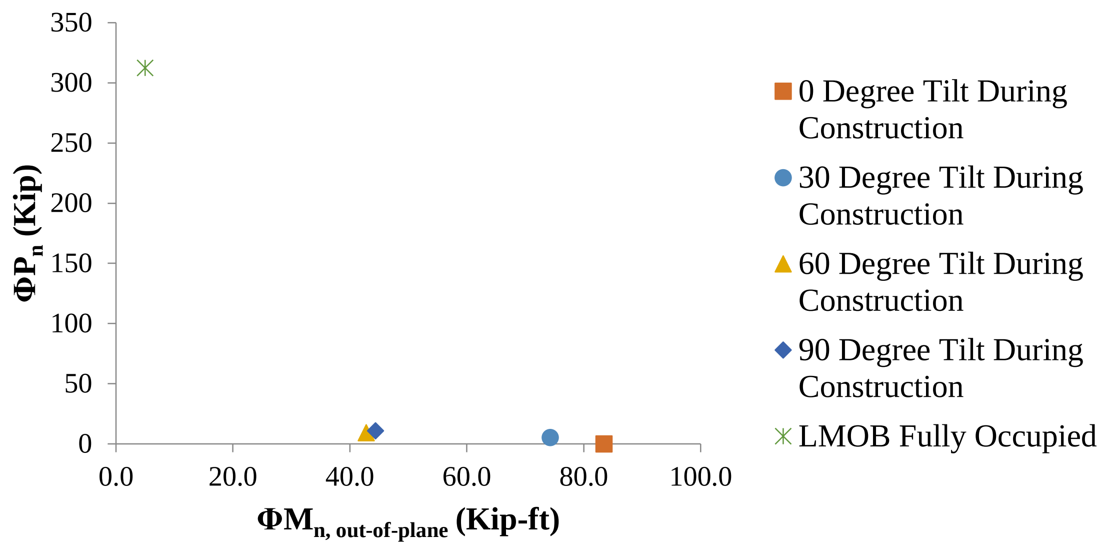
Supporting the physical methods described above, are non-structural methods. Non-structural methods involve people. Without people identifying potential erosion and sedimentation sources, and implementing structural methods; stormwater management will be worthless. Non-structural methods include personnel training and defining responsibilities, and construction routines that aids in stormwater management. Examples of construction routines that aid stormwater management are site clean-up, maintenance, and site inspection. For the LMOB construction site the non-structural methods that is recommended is regular inspections of the system by the project manager and superintendent, two required times for inspecting the stormwater management system are before and immediately after each rainstorm, as well as the methods mentioned in the preceding sentences.

Temporary Bracing for Design II

Temporarily bracing the structural tilt-up walls is critical to construction success. Properly installed temporary bracing serves to stabilize the incomplete structure against lateral loads and reduce second order effects through reducing the un-braced lengths. In doing so, the possibility for structural failure and collapse is less likely because final restraint and reinforcement provided by other structural components are not in place. The weakest stage of the structural system is generally when it is incomplete. One of the many tragic cases involving improperly braced pre-cast and tilt-up panels occurred on the 6th of March 1989 in Tampa, Florida. In this tragic accident, a maintenance worker was crushed – by a concrete panel over 36,000 pounds – when the inadequately fastened base connection gave of the braces failed due to a wind gust (OSHA, 2014). The result was a case that took over a year to resolve and significant compensation on the part of the general contractor.

The importance of temporary bracing the structural tilt-up walls facilitates reasonable brace point selection and determining necessary bracing strength under 100% of the wind load (most significant lateral load). Selection of the temporary brace members are based on axial, bending, and slenderness. Detailing the connections and temporary foundations is beyond the scope defined within the proposal and with that rational was not designed.

Figure 3.17, Structural Tilt-Up Wall Force Interaction



Cost and material use reductions are a central theme in construction, to do that the bracing points were selected to minimize moment and shear which the structural tilt-up experiences during the tilting process. The largest loads that a structural tilt-up wall will experience are during construction specifically in the time when the wall is tilted into place until the structural floor is installed (TCA, 2013). Structural tilt-up walls experience a generally full range of combined bending and axial loads (Figure 3.17). As a result, the bending and axial interaction was studied to ensure that the structural tilt-up walls do not fail. In addition to the temporary bracing strength capacities and the structural tilt-up wall strength capacity, construction ease is another predominant factor. Construction ease is achieved through in limiting the number of temporarily braced levels to two. Any more would get in the way.

Table 3.3, Maximum Factored Loads on Structural Tilt-Up Walls		
Loading Condition	Maximum Loads	
	Moment (Kip-ft)	Shear (Kip)
Two Level Brace Points	84.2	12.9
Wind MWFRS (Constr.)	40.5	4.8
Wind MWFRS	15.6	5.9

Influence lines were used to determine the bracing points that minimized moment and shear experienced in the structural tilt-up walls. The potential bracing points are all located at the floor level. Before using influence lines analysis, the structural tilt-up wall was idealized as a beam with a unit width, 1'. In the idealized beam, only continuous column components of the structural tilt-up wall experiences bending. The beams connecting columns are considered part of the dead weight. It was determined that two bracing levels, the third and fifth floor levels, produced the minimum moment and shear in the structural tilt-up walls. Panel brace points minimizing flexure experienced by the tilt-up walls during the lifting process can be found in Figure 3.18 respectively. For more details, concerning the determination of the brace points which least affected the tilt-up wall, see the appendix. Though the moment and shear in the structural tilt-up walls are minimized, the loads are still much greater than those stemming from the controlling lateral load, wind, as can be seen in Table 3.3.

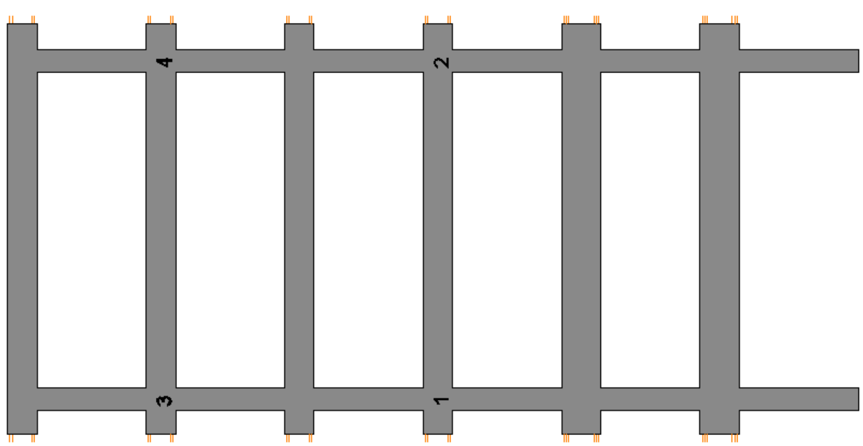


Figure 3.18, Brace Points on Tilt-Up Wall

Figures 3.19 to 3.22 illustrate the general tilting process for structural tilt-up walls. In order to reduce the number of connections to fasten and reduce worksite hazard, the temporary brace members are attached to the structural tilt-up walls before tilting. This first step is beneficial to worksite safety by reducing the time which the structural tilt-up walls are not fully stable, through reduced connections to fasten. The other major step is securing the structural tilt-up wall base, to prevent the wall from kicking-out. Bolted angles will restrain the structural tilt-up wall base. The structural tilt-up wall base will be grouted, once the structure is made plumb and straight. Grouting the structural tilt-up base will allow the bolted angle connection to develop and become more effective.

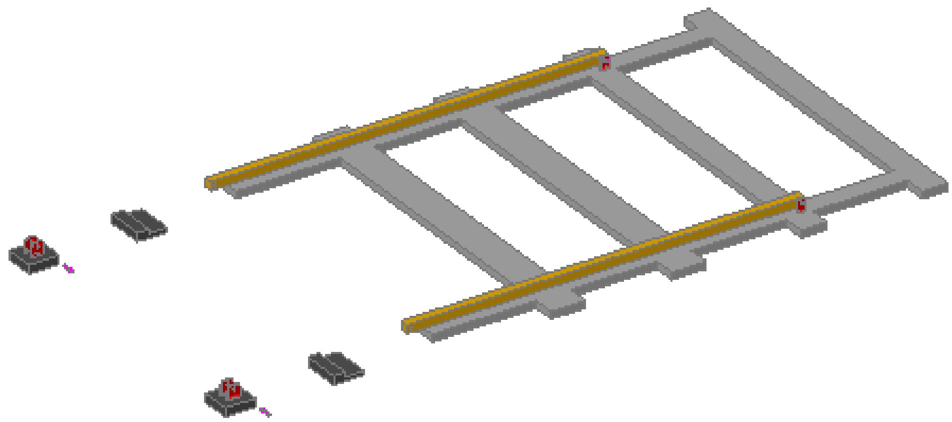


Figure 3.19, Step 1 of Lifting Tilt-Up

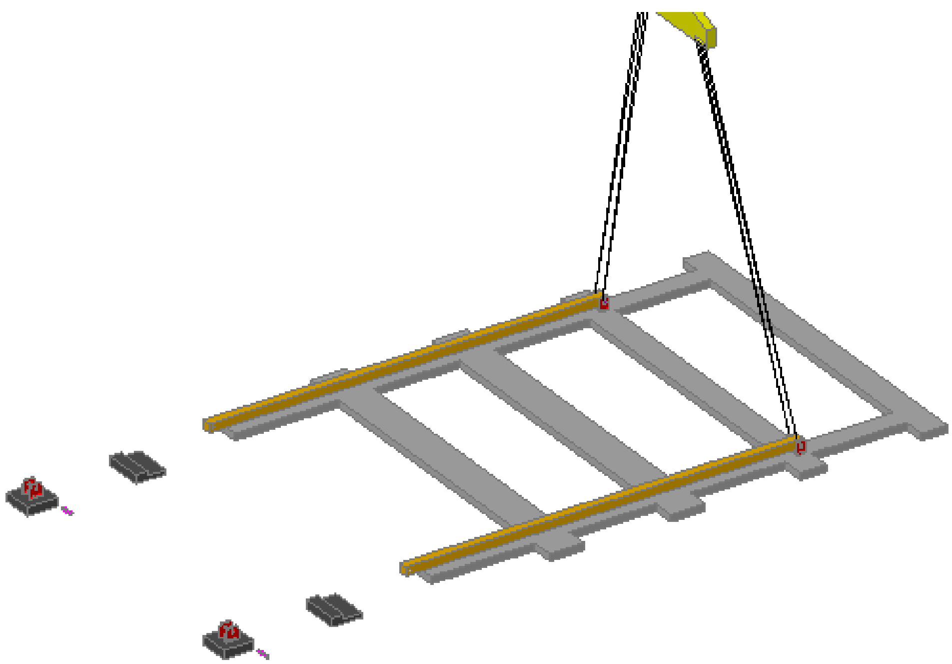


Figure 3.20, Step 2 of Lifting Tilt-Up

Figure 3.21, Step 3 of Lifting Tilt-Up

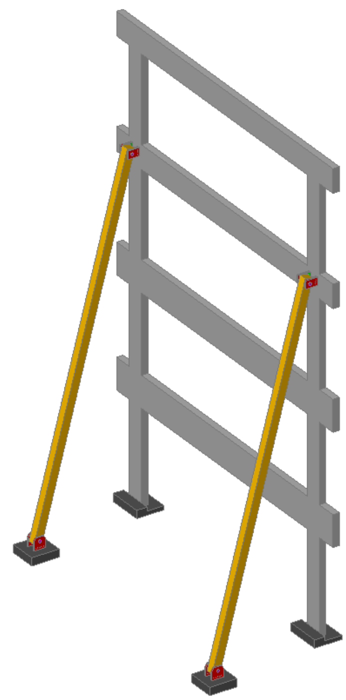


Figure 3.22, Step 4 of Lifting Tilt-Up

As a note, the temporary bracing members will only be removed once the structural tilt-up wall joints have cured, creating a continuous structural shell; and the structural steel columns and beams are in place

Table 3.4, Loads on Temporary Bracing at Each Level				
Panel(s)	Total Panel Area (ft^2)	Brace Level	Elevation (ft)	Factored Axial Load (Kip)
NN1/NN2	829.6	3	44	195.8
		5	72	24.8
NN3	1349.0	3	44	318.4
		5	72	40.3
NN4	1293.8	3	44	305.4
		5	72	38.7
NN5	1216.9	3	44	287.2
		5	72	36.4
SN1/SN2/ SN3	1127.7	3	44	266.2
		5	72	33.7
SN4/SN5	1994.0	3	44	470.7
		5	72	59.6
EN1/EN2	714.6	3	44	168.7
		5	72	21.4
	1319.3	3	44	311.4

EN3/EN4/ WN1/WN 2		5	72	39.4
WN3	667.2	3	44	157.5
		5	72	19.9
WN4	724.8	3	44	171.1
		5	72	21.7

Table 3.5, Initial Design Parameters Based on Factored Axial Loads		
Factored Axial Load (Kip)	Length (in)	I_{req} (in^4)
29.8	894	105.5
97.9	547	129.5
235.3	547	311.2

The temporary bracing members were designed to resist axial and second order effects arising from the full factored wind and dead loads. Table 3.4 shows the factored axial loads that the temporary bracing members must resist. Most loads on the temporary bracing members were determined through STAAD Pro. Table 3.5 only shows three axial load magnitudes, the actual axial loads experienced by the temporary bracing members are much greater. The reason to limit is that it is not economical, both in terms of logistics and cost, to select the optimal temp brace member for each panel.

Table 3.6, Axial and Bending Interaction				
Bracing Member	P_r (Kip)	M_r (Kip-ft)	P_r/P_c	$P_r/P_c + 8/9(M_r/M_c)$
HSS10x10x3/8	29.8	82.4	0.52	0.97
HSS10x10x3/8	97.9	40.6	0.64	0.86
HSS12x12x1/2	235.3	72.2	0.68	0.89

Structural steel member dimensional tables in AISC 14th Edition Steel Construction Manual assisted the hand calculations to size and select reasonably adequate temporary bracing members. Axial and bending caused by second order effects, along with the recommended temporary bracing member sizes are shown in Table 3.6. In total there are four brace points, two at each level. More details pertaining to the temporary bracing member sizing and selection can be found in the appendix.

Table 3.7, Temporary Bracing Schedule				
Panel	Brace Point	Brace Point Elevation (ft)	Bracing Member	
			Length (ft)	Size
NN1/NN2	1	44	46	HSS10x10x3/8
	2	44	46	HSS10x10x3/8
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
NN3	1	44	46	HSS12x12x1/2
	2	44	46	HSS12x12x1/2

	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
NN4	1	44	46	HSS12x12x1/2
	2	44	46	HSS12x12x1/2
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
NN5	1	44	46	HSS12x12x1/2
	2	44	46	HSS12x12x1/2
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
SN1/SN2/SN3	1	44	46	HSS12x12x1/2
	2	44	46	HSS12x12x1/2
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
SN4/SN5	1	44	46	HSS12x12x1/2
	2	44	46	HSS12x12x1/2
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
EN1/EN2	1	44	46	HSS10x10x3/8
	2	44	46	HSS10x10x3/8
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
EN3/EN4/ WN1/WN2	1	44	46	HSS12x12x1/2
	2	44	46	HSS12x12x1/2
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
WN3	1	44	46	HSS10x10x3/8
	2	44	46	HSS10x10x3/8
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8
WN4	1	44	46	HSS10x10x3/8
	2	44	46	HSS10x10x3/8
	3	72	75	HSS10x10x3/8
	4	72	75	HSS10x10x3/8

From Table 3.7 it is evident that the temporary bracing members are large. These temporary bracing members are large because the structural tilt-up walls themselves are large. The larger the structural tilt-up wall the larger the wind load, since the wind load is directly proportionate to the face area of the structural tilt-up wall. Also attributed to the structural tilt-up wall's dimensions is the height, 86', which is only 6' shy of the tallest structural tilt-up wall, used in the Lucky Street Garage in Hollywood, FL (TCA, 2013). The height of structural tilt-up walls meant that the temporary bracing members would be long, making it vulnerable to buckling and second order effects. In short, Design II pushed tilt-up wall construction and design to the current limit.

Direct Construction Costs and Conclusion

Akin to most projects, cost is a major determinant in whether a project progresses or not. The purpose of estimating the original and re-design structural costs, and alternate building envelopes is to determine the most reasonably advantageous system.

Assumptions governing the cost estimates for both Design I and Design II are as follows:

1. Open shop labor
2. Waste factor will be 5% unless noted
3. All structural steel has a density of 490 lb/ft^3
4. All anchor bolts are 24" long
5. Flashing around wall openings are 12" wide
6. Sales tax is 6%
7. Overhead and profit is 10%

Exhaustive cost estimation was not implemented for every item used in LMOB. Instead only the façade, structural, soil compaction, and necessary construction infrastructure were estimated in detail. Three estimates, incorporating the mentioned estimating categories, were implemented; specifically the original building, Design I, and Design II. Estimating the components in the original building allowed for later substitution of Design I and Design II component costs. Underestimation is detrimental to construction projects due to the need to negotiate with the owner to pay the additional cost, creating a less satisfied customer; or the contractor absorbs the additional cost and cut their profit. In order to hedge against underestimation, a 10% contingency incorporated into the cost estimate.

Adjustments factors are necessary to compensate the effect of waste, the effects of location on material and labor, time, as well as overhead and profit. To do this each estimate broke down the mentioned costs into three categories: material, labor, and equipment cost. The purpose behind the action is that the waste factor only applied to material cost, arising from potential material breakage and material used inefficiently. The second reason is that overhead and profit factor, 10%, was only applied to material and labor costs. To compensate for the incompatibility due to the effect of inflation in time, the estimated costs were modified by an adjustment factor. As directed by *R.S. Means* the inflation adjustment factor between 2008 costs and 2013 costs is a ration ratio of the location factor in 2008 and 2013. More details, such as the exact itemized cost breakdown and the entire estimate, please see the appendix.

Estimates of the original building, Design I, and Design II reveal the cost of each component in each design and how they add up to make Design II not cost effective. As evident from Table 3.8, the estimates revealed that the original design is still the most cost effective. The primary reason that Design I is more expensive than the original are the construction of temporary roads and unloading areas, not present in the original. Constructing temporary roads and unloading areas reduce construction traffic impact on the surrounding roads. Unlike Design I, Design II is costly due to a multitude of items.

Table 3.8, Total and Select Itemized Cost of Each Design				
Design Designation	Itemized Cost			Total Cost
	Necessary Infrastructure	Structural	Façade	
Original	$293,658	$3,710,785	$869,748	$12,600,000
Design I	$307,176	$3,776,745	$858,413	$12,668,143
Design II	$576,009	$3,546,273	$1,799,585	$13,647,676

Infrastructure necessary to construct LMOB is the largest culprit increasing Design II's cost. For one, the use of a heavy lift crane and the assumption that the contractor decides to buy temporary bracing instead of renting it, increase Design II's infrastructure cost. Temporary bracing accounts for approximately a fourth of the total infrastructure costs. There is no doubt that if the temporary bracing was rented, there would be a significant cost reduction.

The second major item causing Design II to be cost inefficient is the façade system chosen for the structure. The CFS framing is more expensive than the reinforced and grouted CMU façade of the original design and Design I. Originally the alternate façade system, metal studs sheath in fiber cement board, was chosen due to lightness and the ground assembly possibility. The potential benefit in ground assembly lies in quick assembly and scheduling flexibility, which the façade assembly can be done almost any construction phase before the applying interior and exterior finishes. Major assumptions and flaws in thinking erased the benefit of scheduling flexibility. It was discovered – late in the project – that assembling the metal studs took much longer than anticipated, thus requiring the task to be scheduled as early as possible to meet the desired completion date. Adding additional crews is possible to speed up the construction of the metal studs, however the shear number of workers on the site would get into each other's way and interfere with surrounding businesses. As a result, the alternate façade system offered neither speed nor construction flexibility.

Thus far it can be concluded that the redesigns are not as financially competitive, nor easy to construct as the original LMOB design. Design II is the least competitive, due to the complexity of site logistics – which require numerous reconfigurations and takes up significant space. It is likely that a Design II can be made more competitive by breaking the full height monolithically cast tilt-up walls into two vertically stacked panels. In doing so, a large capacity crane would not be required, vertical steel reinforcement would be reduced, and the temporary bracing members would be smaller – in the end the financial burden would be reduced. Though the constructability phase of this report is at an end, more studies should be done. These include, but are not limited to: determining the connection between the tilt-up panels and whether two vertically stacked panels make Design II more competitive.

Building Façade Breadth

The building envelope is an essential system that is often overlooked. This oversight recently resulted in building envelope accounting for the majority of building failures (Snoonian, 2000). In essence, a building's envelope protects the occupants and interior building systems from undesirable exterior environmental conditions. A few of the exterior environmental conditioned tamed by the building envelope include: high moisture levels in the air, significant temperature fluctuations, noise, rain, and airborne projectiles. This phase of the breadth focuses on using light gauge cold formed steel (CFS) stud back-up in lieu of concrete masonry. Structurally, only the stud and track were selected from determined design loads; other components and details like the connections were not explored. The rationale behind the redesign is reducing the façade wall weight. Reducing the wall weight has many benefits chief among which is reducing the seismic load – proportional to the building weight – and construction productivity. Though reducing the façade wall weight is paramount, it is not the only factor determining the redesign's success. Moisture resistance, thermal performance, acoustical performance, as well as general construction cost and assembly ease criterion were used to compare the original façade wall with the redesign.

Literature Review and Benchmarks

Uncontrolled moisture – whether in the form of vapor flow or wind driven rain – is detrimental to building operations and the occupants. To tame moisture, it must be understood how it crosses between barriers.

When moisture is in the form of vapor, it can move through wall assemblies in two ways: vapor diffusion and air transport of vapor. Vapor transportation through diffusion works through a difference in vapor pressure and/or temperature difference between the two environments which a barrier separates (Lstiburek, 2001). Vapor pressure is the concentration of moisture in air. Water vapor diffusion through barriers is governed by the *Second Law of Thermodynamics*.

The *Second Law of Thermodynamics* states that:

1. Water vapor moves from a location of higher vapor pressure to one of lower vapor pressure
2. Water vapor moves from hot to cold interfaces

The amount of vapor diffusion directly depends on the barrier face area. Controlling the amount of vapor diffusion through the wall assembly can be achieved through the use of vapor retarders. According to the *2012 International Building Code* (*IBC*), a vapor retarder is defined to have a permeability less than 1 *perm* under the dry cup testing method. The dry cup testing method measures a material's permeability by exposing one side of the material to 0% relative humidity (RH) and the other side to 50% RH (Lstiburek, 2000).

If diffusion through a barrier is primarily thermally driven then there is a potential for condensation to occur on the cold interface, especially when large thermal gradients exist. Condensation arises in the interface when the air temperature drops such that the air can no longer hold onto the moisture (Lstiburek, 2001). Typically condensation will form in the wall material exhibiting the largest insulation value. The material exhibiting the largest insulation value is also the location where the greatest temperature gradients exist in a wall. It is recommended that condensation occur in materials that resist moisture damage (Glantz, 2013). In addition, the condensation should be lead to the exterior or removed from the wall assembly by HVAC – to prevent health compromising diseases from taking root and thriving.

When compared to air transport of vapor, the vapor diffusion is relatively insignificant. This holds true unless the barrier is located in a hot/humid climate where the barrier is wetted by rain and experiences solar heat gain (Lstiburek, 2006). Air transport of vapor occurs when vapor moves from areas of higher air pressure to an area of lower air pressure. For vapor to be transported by air, the air must be moving. Air transport of vapor works independently from vapor diffusion. To tackle vapor movement arising from air transport, air movement must be stopped. Stopping air movement is can be achieved by using cladding and staggered joints. Staggering the joints of the wall's layers, prevents failure of one layer from letting air to freely move through the wall assembly unimpeded.

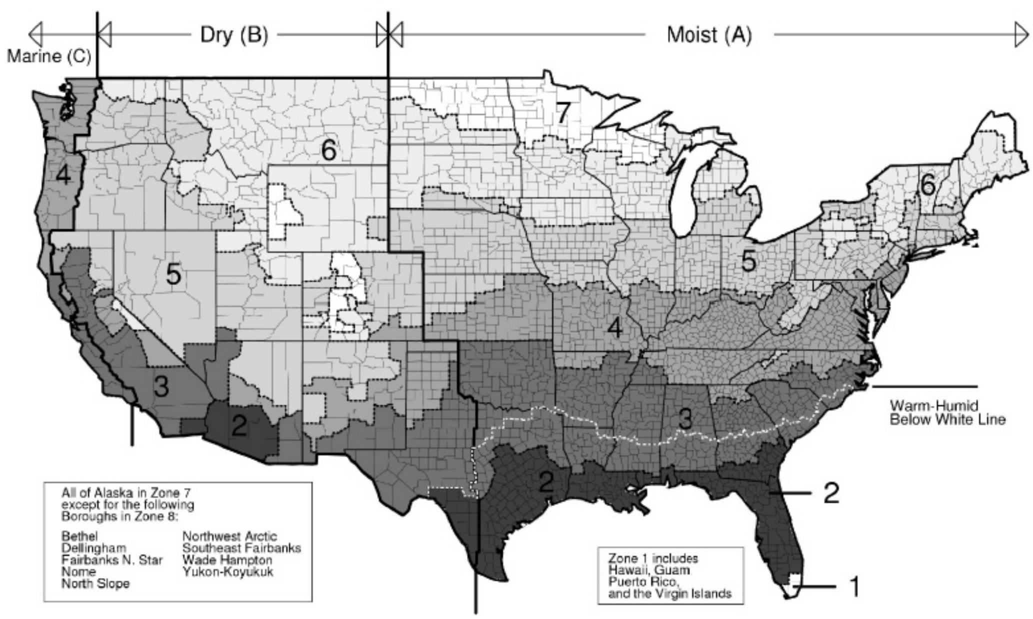


Figure 4.1, Continental U.S. Code Defined Climate Zones
Source: *2009 IECC §301.1*

Different locations have different climate conditions; the impact on building envelope design is significant. Figure 4.1 shows the various regions with similar and different climates in the continental U.S. The literature review for the façade wall redesign will not explore the aspects of the various climate zones, what will be said is that the climate zone for Largo, FL is 2. Code defined climate zones dictate the thermal insulation performance, from tables in *§502.2* of the *2009 IECC* – R-Value for metal framed wall is 13, but is only 5.7 for solid walls like concrete masonry. It should be noted that different design codes were used in designing the original façade wall and redesign. The result is different thermal performance requirements. The code used in designing the original façade wall didn't require any thermal resistance – R-value, zero. This is significant because it factors directly into the general cost comparison of the original façade wall and redesign.

Table 4.1, Metabolic Rate of Typical Human Activities Source: *2008 ASHRAE Std. 55*	
Activity	Metabolic Rate (Met)
Seated Quietly	1.0
Reading-Seated	1.0
Filing-Standing	1.4
Walking About	1.7
Lifting Packages	2.1

Table 4.2, Insulation Value of Typical Clothing Source: *2008 ASHRAE Std. 55*	
Clothing	Insulation Value (Clo)
Walking Shorts-Short Sleeve Shirt	0.36
Trousers-Short Sleeve Shirt	0.57
Trousers-Long Sleeve Shirt	0.61
Trousers-Long Sleeve Shirt w/ Coat	0.96
Trousers-Long Sleeve Shirt, Long Sleeve Sweater	1.01

Next, comfort level of a building's occupants is explored. Let it be clear that it is impossible make all occupants of a building comfortable. Instead occupant comfort is based on statistically satisfying 80% of the building's occupants. The method relies on the anticipated activity level and the clothing worn by the occupants. Located above are two tables, Table 4.1 and Table 4.2, listing the various activity levels and clothing levels. LMOB's occupants were classified into two categories, clinic personnel and patients.

Each occupant category entails specific activity levels and clothing levels. It was assumed that the clinic personnel are constantly walking about tending the patients and filing throughout the day while wearing trousers and long sleeve shirts. Patients on the other hand, are assumed to be either patiently waiting or reading and are more casual, wearing trousers and a short sleeve shirt. Before the determining the ideal interior temperatures, the interior humidity was established. The interior humidity for LMOB is set to be 50%, based on *ASHRAE Std. 170 Addendum D*. *ASHRAE Std. 170 Addendum D* recommends that the RH for a clinic or a hospital be less than 60% to eliminate mold and bacterial growth. With the help of *ASHRAE Std. 55 §5.2.1.1* and Figure 4.2, the recommended interior temperature range where approximately 80% of the clinic personnel and patients are comfortable is 72°F for the winter and 76°F for the summer.

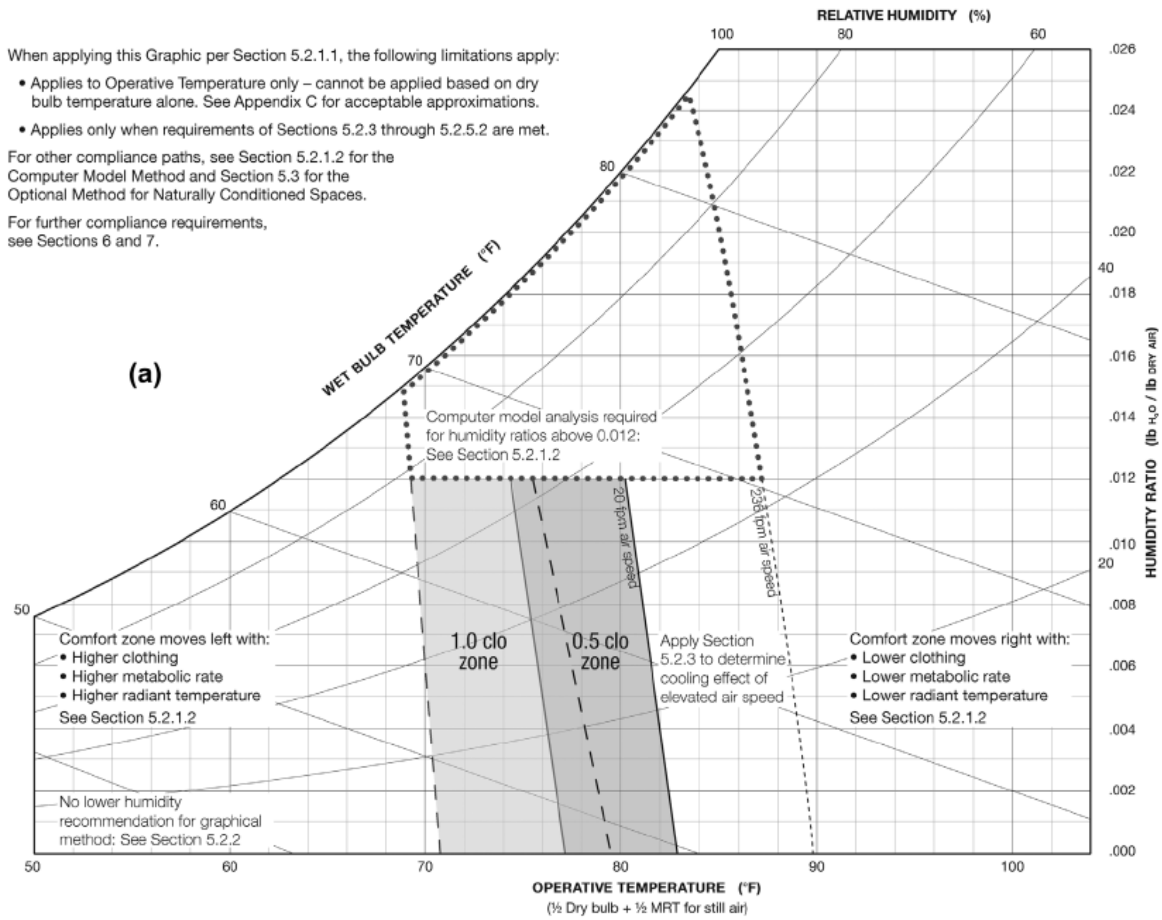


Figure 4.2, Graphical Representation of the Comfort Zone
Source: *2010 ASHRAE Std. 55*

Table 4.3, General Descriptions of Various STC Ratings Source: Harris, 1994	
STC	Description
25	Normal speech can be understood quite easily and distinctly through wall
30	Loud speech can be understood fairly well, normal speech heard but not understood
35	Loud speech audible but not intelligible
40	Onset of privacy
42	Loud speech audible as a murmur
45	Loud speech not audible
50	Very loud sounds such as musical instruments or stereo can be faintly heard

Airborne sound is another source of discomfort in an interior environment. The amount of airborne sound is limited by many building codes. These codes generally aim to limit intrusive exterior sound and maintain speech privacy. The *2009 IBC* defines that walls, partitions, and floor assemblies have a sound transmission class (STC) no less than 50, for airborne noise. STC is a single value that reflects an assembly's ability to dampen – transmission loss (TL) – the noise generated from various frequencies of human speech (Egan, 1988). Most human speech frequency ranges from 125 Hz to 4000 Hz (Egan, 1988). The greater the STC the greater the intimacy/privacy of the human speech. General privacy descriptions of STC ratings are shown in Table 4.3.

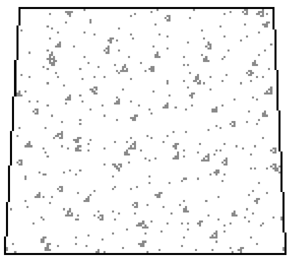


Figure 4.3, Mass Wall

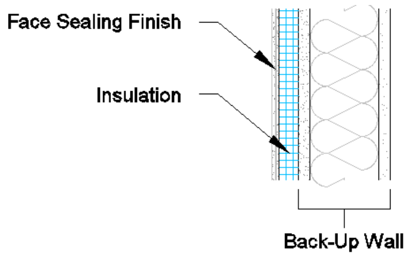


Figure 4.4, Sealed Wall

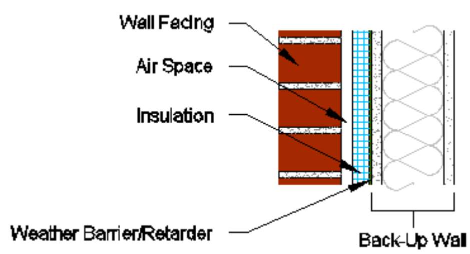


Figure 4.5, Cavity Wall

So far, the wall functions and ideal performance parameters have been discussed, what has not been covered are the different types of wall assemblies. Each type of wall assembly has its own benefits and disadvantages. Later redesign of the façade walls will be based on one of the types of wall assemblies.

Generally speaking, there exists three types of walls. Figures 4.3 to 4.5, shows the three general wall types. Each general wall type has benefits and disadvantages. The most basic wall system are mass barrier walls, these walls are usually thick and load bearing. The mass barrier wall's thickness acts as a reservoir to store infiltrated water (Dalrymple, 2012). Over time the wall dries out due to evaporation. The simplicity of the mass barrier wall is the main advantage, whereby little can go wrong. Other than that the mass barrier is a very heavy system.

The second general wall system are sealed walls, which work by preventing water infiltration through the wall assembly through impermeable coatings. On the surface, the rational in repelling all water at the exterior surface appears work. However it is not practical, relying on a perfect seal to prevent water intrusion is un-realistic because of imperfect installation, existence of expansion and control joints, and material degradation will allow water infiltration (Dalrymple, 2012). Since there are no backup waterproofing, once moisture penetrates the assembly it soaks moisture sensitive wall assemblies – batt and cellulose insulation, wood structure and sheathing, gypsum wall boards – causing damage.

Inadequate drying and drainage is the culprit for most building envelope failures. Adequate drying and drainage occurs whenever the wetting rate is equal or less than the drying rate (Lstiburek, 2003). The concept behind cavity walls is the acknowledgement that the water will eventually find a way into the wall assembly. To compensate for this, cavity walls use drainage planes, weep holes, and permeable materials to drain any water that penetrates the assembly. The wall system's Achilles heel is the complexity of constructing cavity walls.

Façade Wall Redesign and Analysis

Redesigning LMOB's façade wall and comparing it's competitiveness with the existing concrete masonry façade requires multiple tasks. The first task is to select the materials for the façade wall redesign. Next, the redesign will be designed according to the building science benchmarks. Only after the building science design is completed will the moisture performance be analyzed. Additional analysis will only commence once the CFS stud and track are selected. The additional analysis include: acoustical performance, construction cost, and general ease of assembly.

Understanding that the original façade wall of LMOB was designed to the previous iteration of the building code, a retrofit to meet the more stringent current building code will also be designed. The retrofit will permit a more direct and fair comparison to determine if the façade wall redesign is worth it. For some background, the previous iteration of the building code required no thermal resistance for mass walls – like the concrete masonry back-up wall used for LMOB's façade –, the current however requires a minimum R-value of 5.7.

<u>Material Selection</u>

Determining the materials for use in the redesign is a critical task. The design phase depends on the properties of the materials selected. The materials selected for the redesign are: structural sheathing, vapor retarder, thermal insulation, and CFS grades.

Table 4.4, Fiber Reinforced Cement Board Sheathing by Various Manufacturers

Board Thickness (in)	Max. Wind Speed (Mi/hr)	Max. Pressure (lb/ft^2)	Manufacturer	Product Name
0.38	120	N/A	U.S. Arch. Products	VERSAROC
	N/A	46	AmeriForm	ARMOROC
0.5	150	N/A	U.S. Arch. Products	VERSAROC
	N/A	68	AmeriForm	ARMOROC
	N/A	40	National Gypsum	PERMABASE

It was decided early that fiber reinforced cement board will be used as the structural sheathing. Fiber reinforced cement boards have been used as structural sheathing for structural insulated panels (SIP) and floors of prefabricated buildings, as well as exterior siding (Deluxe Building Systems, 2014). As exemplified by the various uses, fiber reinforced cement boards are durable and can be used for exterior applications. However, fiber reinforced cement board sheathing strength varies more widely than those of more commonly used plywood. Therefore, fiber reinforced cement board sheathing selection is generally based on strength properties compiled from various manufacturers. Table 4.4 shows the strength properties of fiber reinforced cement board sheathing from various manufacturers. Based on these strength properties, it can be concluded that the structural sheathing will be a minimum 1/2".

Table 4.5, Properties of Potential Vapor Retarding Materials
Source: DuPont

Material Type	Common Example	Max. Water Pressure (lb/ft^2)	Adhesion/Fastening Strength (lb/ft^2)	Max. Elongation	Weight (lb/ft^2)
Flashspun High-Density Polyethylene Fibers	Tyvek	15.0	> 33	420%	0.017
Asphalt-Impregnated Bldg. Paper	[redacted]	5.2	> 40	N/A	0.083
Spun Polypropylene Fibers	C2000	10.5	40 - 90	279%	0.055

A vapor retarding material was selected to control the quantity of moisture passing through the wall assembly towards the interior. Table 4.5, shows the three potential vapor retarding materials considered. Selection for redesign was based on maximum anticipated water pressure and ability to show significant distress before failure. The design water pressure on the vapor retarding material was based on a maximum rain accumulation of 1” before drainage. It turned out that all potential vapor retarding materials could resist the design water pressure – 5.2 lb/ft^2. Fluid applied flashspun high-density polyethylene fibers was selected for the redesign.

Table 4.6, Minimum Thickness of Various Thermal Insulation Materials
(Based on 2009 IECC Table 502.2(1))

Required R-Value (hr-ft^2-°F/BTU)		EPS		XPS	
Mass Wall	Metal Frame	Mass Wall	Metal Frame	Mass Wall	Metal Frame
5.7	13	0.78	1.79	0.69	1.57

In order to reduce condensation in the interior side of the wall assembly, it was decided to place a layer of thermal insulation on the exterior side of the wall assembly – right behind the stucco layer. By doing so, the various potential thermal insulation material is narrowed down. Mainly those which can resist moisture induced damage and prevent health hazards from thriving. Based on commercial availability, there were two viable choices – expanded polystyrene foam (EPS) and extruded polystyrene foam. As can be seen above, in Table 4.6, the minimum thickness for EPS is greater than XPS. The minimum thickness to achieve the required thermal resistance was based on the potential insulation accounting for no more than 60% of total wall assembly thermal resistance. Though XPS permits thinner wall assemblies, it was not selected for the façade wall redesign. The main reason is the higher cost of XPS thermal insulation and typical location of use. XPS thermal insulation is typically used for below grade and on the roof, walls however are the domain of EPS.

Figure 4.6, Wall Lath for Stucco
Source: This Old House, 2012

The stucco and lath selection is based on moisture performance. Two stucco materials were considered: Portland cement based and synthetic polymer based. Portland cement based stucco absorbs more moisture than the synthetic polymer based stucco. The benefit is that it is permeable, which results in whenever water that gets behind the stucco can easily get removed through evaporation. On the other hand synthetic polymer base stucco is an impermeable material (Lstiburek, 2006). Like any impermeable finish, any water that penetrates the coating or water on the rigid insulation surface that is not removed will get trapped. Another downside to using Portland cement based stucco is susceptibility to cracking (Lstiburek, 2006). However, the wall lath – an integral component of stucco finishes – can be used to reinforce the cement and limit crack formation as rebar in concrete. The desire dry penetrating moisture easily from the wall assembly meant that the Portland cement based instead of synthetic polymer base.

As for the wall lath materials, there are two main types. One is galvanized steel and the other is fiberglass. Galvanized steel lath is more commonly used, but it was decided that a PVC lath will be used – since LMOB is in a salty and moist environment. The abundance of salts and moisture will eventually corrode the galvanized steel lath. General PVC degradation arising from UV exposure is not a concern, as it will be protected by the stucco.

Table 4.7, CFS Mechanical Properties
Source: Clark Dietrich, 2014; ASTM, 2009

F_y (kip/in^2)	F_u (kip/in^2)	E (kip/in^2)	ν
33	45	29500	0.334
50	65		

Specific CFS material were not selected due to the multitude of choices – that varied with manufacturer. Instead common strength properties were gathered. Table 4.8 shows the strength properties. It is intended that the higher strength CFS will be used if it results in shallower and lighter members.

Expansion joint are essential because almost all materials expand and contract, either due to temperature or moisture changes. The duty of expansion joints is to prevent material failure such as undesirable cracks, and unnecessary water intrusion. All expansion joints were designed based on anticipated material expansion and contraction, along with sealant movement capacity. For thermal expansion and contraction determination, the maximum anticipated temperature change was used to prevent bucking the finish material. In Largo, Florida the maximum anticipated temperature change occurs during January.

Table 4.8, Properties of Various Sealant Types
Source: Cook, 1991

Sealant Type	Sealant Properties
Butyl	- Good adhesion, water resistance, and color stability - Minimal surface preparation - Cures slowly - High shrinkage and low shape recovery
Neoprene	- Good adhesion and water resistance - Compatible with bitumen and asphalt surfaces - Relatively inexpensive - Cures slowly - Typically available in dark colors only - Stains surrounding materials - High shrinkage
Solvent-Based Acrylics	- Good adhesion, UV resistance, and chemical resistance - Minimal surface preparation - Does not stain surrounding material - Cures slowly - Only for joints $\leq$ 3/4" wide - Low shape recovery - Poor water resistance
Urethanes	- Good tear resistance, UV resistance, chemical resistance, and shape recovery - 20 to 30 year mean life - Joints can be sized $\leq$ 6” wide - Surface preparation is required - Poor water immersion resistance
Silicones	- Good heat resistance , UV resistance, and shape recovery - 25% to 50% movement capacity - 20 to 30 year mean life - Does not stain surrounding material - Surface preparation is required

Dictating the expansion joint design are guidelines and assumptions, which specifically includes:

1. Vertical joints are spaced no more than 22' apart
2. Horizontal joints are positioned at each floor level
3. Expansion joints are hidden in the line details, shown in Figures 4.7 to 4.10 to limit aesthetic impact
4. Stucco properties are based on extreme expansion and contraction of concrete
5. All joint spaces are based on the largest cumulative panel length
6. Insulation expansion and contraction arising from moisture and thermal effects are small and negligible

Various sealants are available on the market as a result selecting them can be difficult. Sealant selection for the expansion joint was based on movement capacity, resistance to UV radiation, and durability. Table 4.8 lists the properties of various sealant types. From the selection criteria, silicon based sealants was chosen.

Table 4.9, Material Expansion Properties
Source: The Brick Industry Association, 2006

Material	Coefficients	
	Moisture Expansion	Temperature Expansion
Concrete	0.00045	0.0000055
Masonry	0.00045	0.0000045
Stucco	0.00070	0.0000055

Table 4.10, Recommended Expansion Joint Sizes

Joint Designation	Panel Length in the Expansion Direction (ft.)		Movement (in)	Joint Size (in)
	Panel 1	Panel 2		
Vertical	20.5	19.8	0.196	7/8
Horizontal	16.0	14.0	0.146	5/8

Figure 4.7, Designed Expansion Joints (Magenta) for North Façade

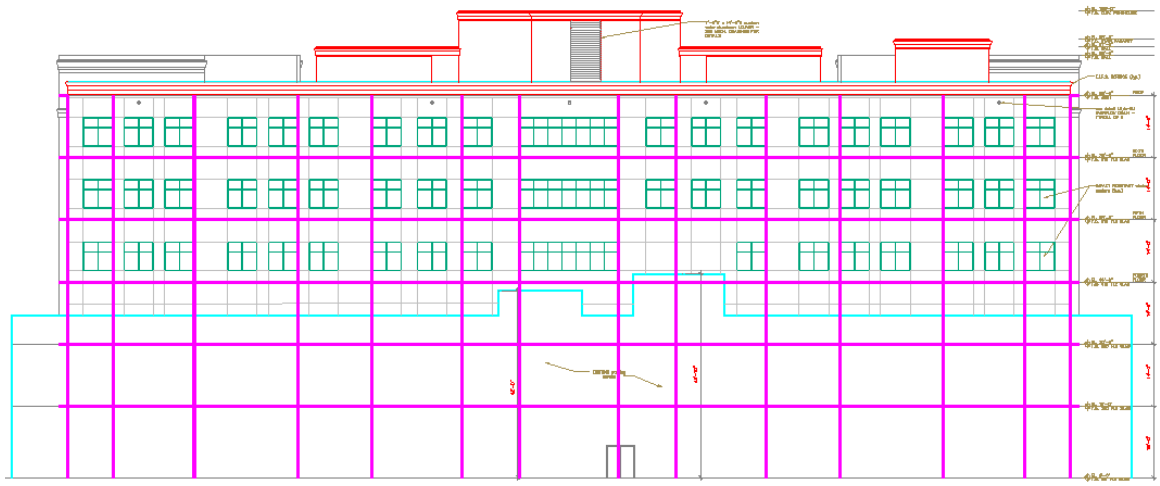


Figure 4.8, Designed Expansion Joints (Magenta) for South Facade

Figure 4.9, Designed Expansion Joints (Magenta) for East Facade

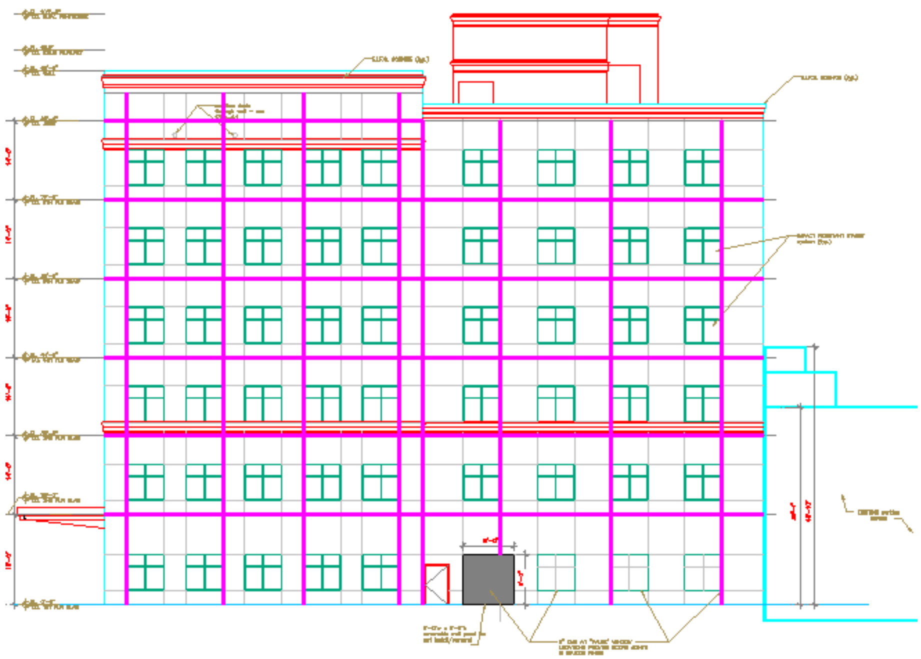


Figure 4.10, Designed Expansion Joints (Magenta) for West Facade

Using the material properties in Table 4.9 and conservatively allowing 25% joint movement in the silicon based sealant, was the recommended joint size was derived. Recommended joint sizes can be found on Table 4.10. Actual joint locations on the façade re-design are highlighted in magenta in Figures 4.7 to 4.10.

Building Science

Replacing the concrete masonry back-up wall with light gauge CFS wall system impacts the multiple performance aspects. Light gauge CFS is lighter and more thermally conductive than solid concrete masonry. The result is that the redesign cannot rely on shear mass and bulk to resist heat flow, moisture flow, and attenuate sound. This section will focus on addressing the thermal and moisture performance changes arising from the redesign, as well as retrofitting the original façade wall system to meet current code – for a more direct comparison.

The redesign is based on the cavity wall system – discussed in the literature review – and is illustrated in Figure 4.11. For convenience, the retrofit and original façade walls are illustrated in Figures 4.12 and 4.13. One reason for basing the redesign off of the cavity wall system is that perfect seals against moisture intrusion is not possible. The causes are – more often than not – improper installation, material degradation arising from lack of maintenance, as well as unreasonable high cost to ensure perfect seals. Cavity wall systems on the other hand acknowledge that water will eventually penetrate the wall assembly. Thereby compensating it by incorporating weep holes for drying out the wall and multiple layers – to retard water penetration. The second point is the lightweight of the entire system, when compared to mass walls.

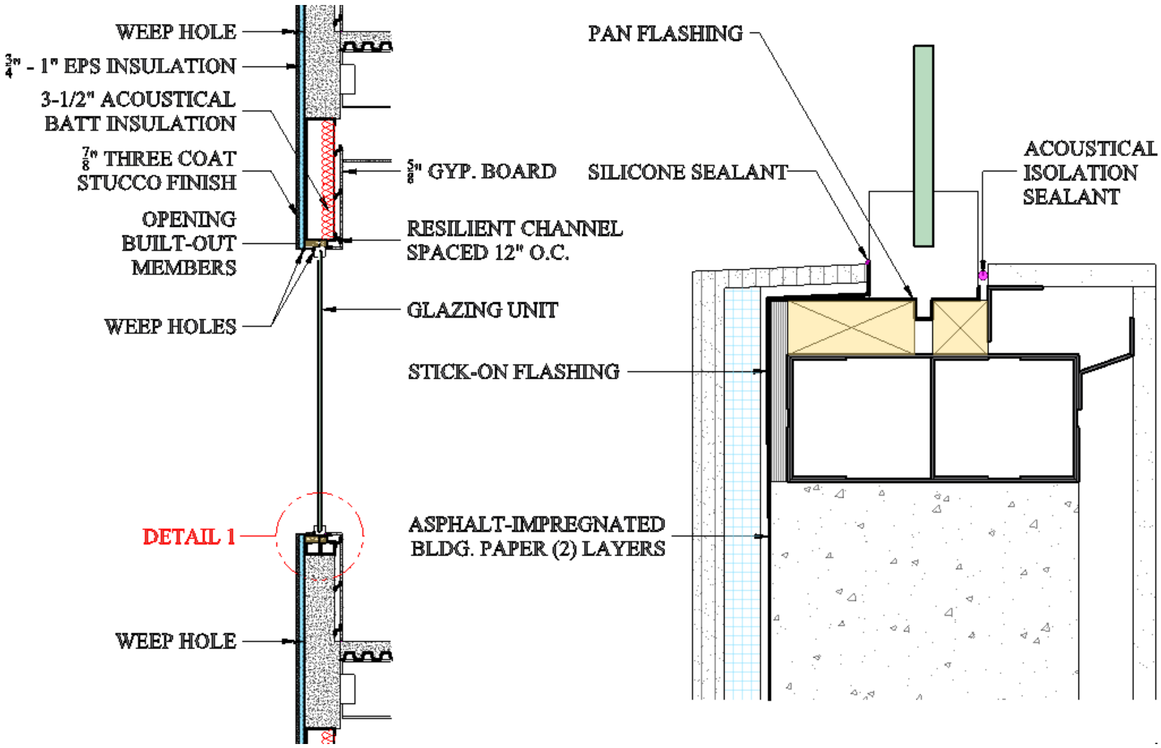


Figure 4.11, Façade Wall Redesign

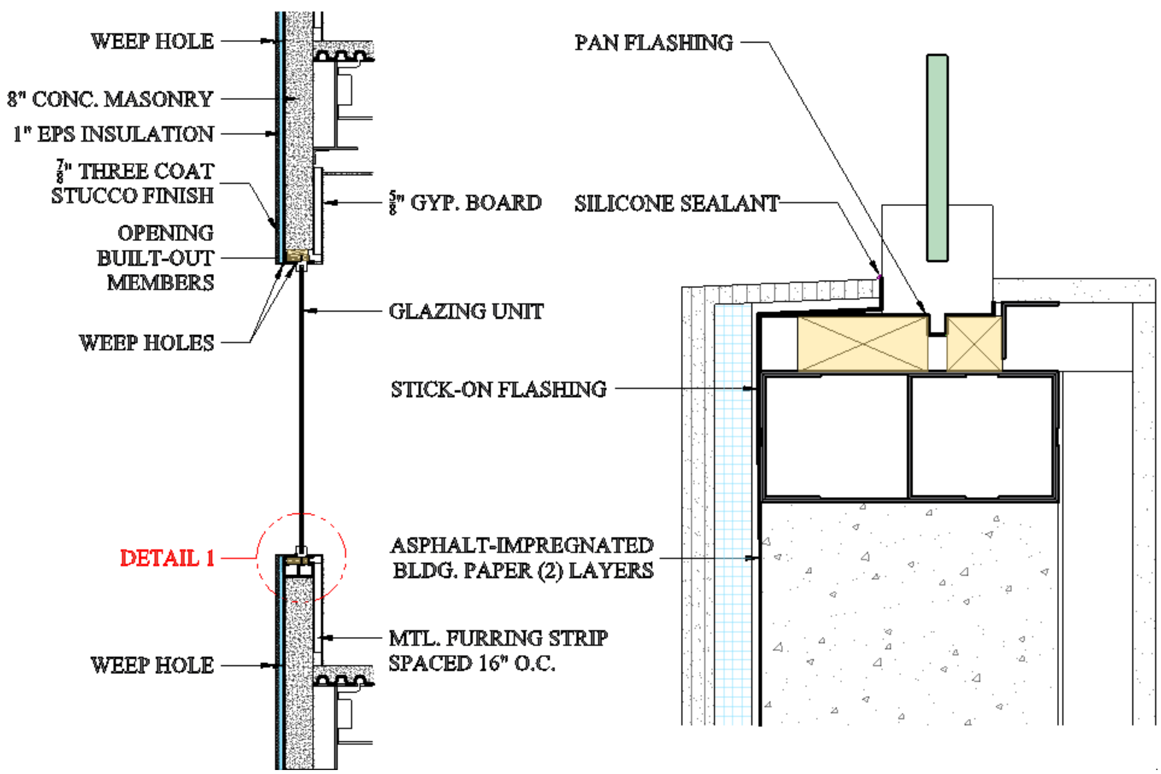


Figure 4.12, Façade Wall Retrofit

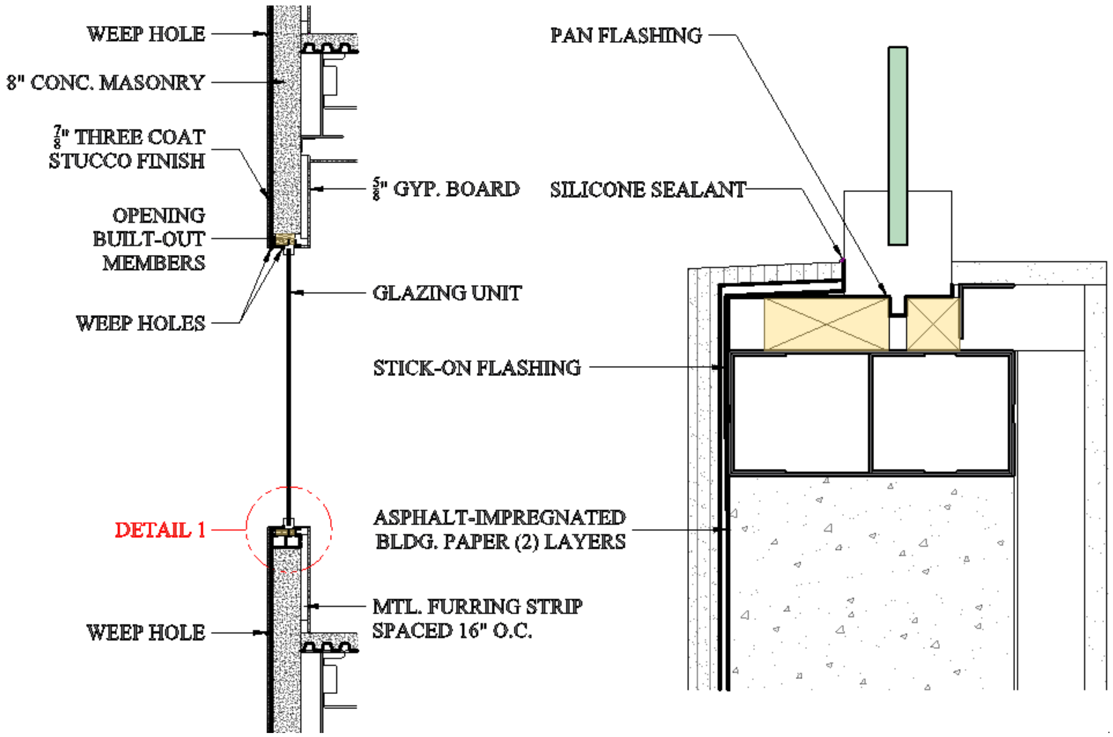


Figure 4.13, Original Façade Wall

Reasoning behind organizing the various layers in the redesign and retrofit wall assemblies will be discussed below. Starting from the exterior wall assembly layers then moving inwards, behind the stucco finish is EPS rigid insulation. Exterior rigid insulation was used in-lieu of thermal batt insulation – placed between the CFS members –, in order to reduce the amount of condensation in the interior side of the wall assembly. Condensation generally occurs at the interface where there is significant temperature and vapor pressure changes. Moving the majority of the thermal resistance and vapor flow resistance to the exterior, shifts the location of condensation to exterior. A condensation plane and vapor retarder is incorporated into the redesign. They are placed behind the EPS rigid insulation to facilitate drainage towards the exterior, thus permitting the wall assembly to dry.

The construction sequence used in applying flashing around windows and vapor retarders is important. An improper construction sequence will create laps that allow water to enter from the top edge of the flashings and vapor retarder. The only way to fix improper installation is to remove the originally installed water management system and install the new water management system properly. Proper flashing and vapor retarder installation ensure that each vapor retarder and flashing layer reinforces or backs-up other layers, also known as a shingle lap manner. As recommended by the *Canada Mortgage and Housing Corporation* (*CMHC*), a government housing agency, each lap is at least 4”. These are the reasons why this report defines the installation sequence for the flashing and waterproofing. Instead of explicitly defining the installation in words and having the contractor comprehend it, the flashing and waterproofing installation process is illustrated in Figures 4.14 to Figure 4.17.

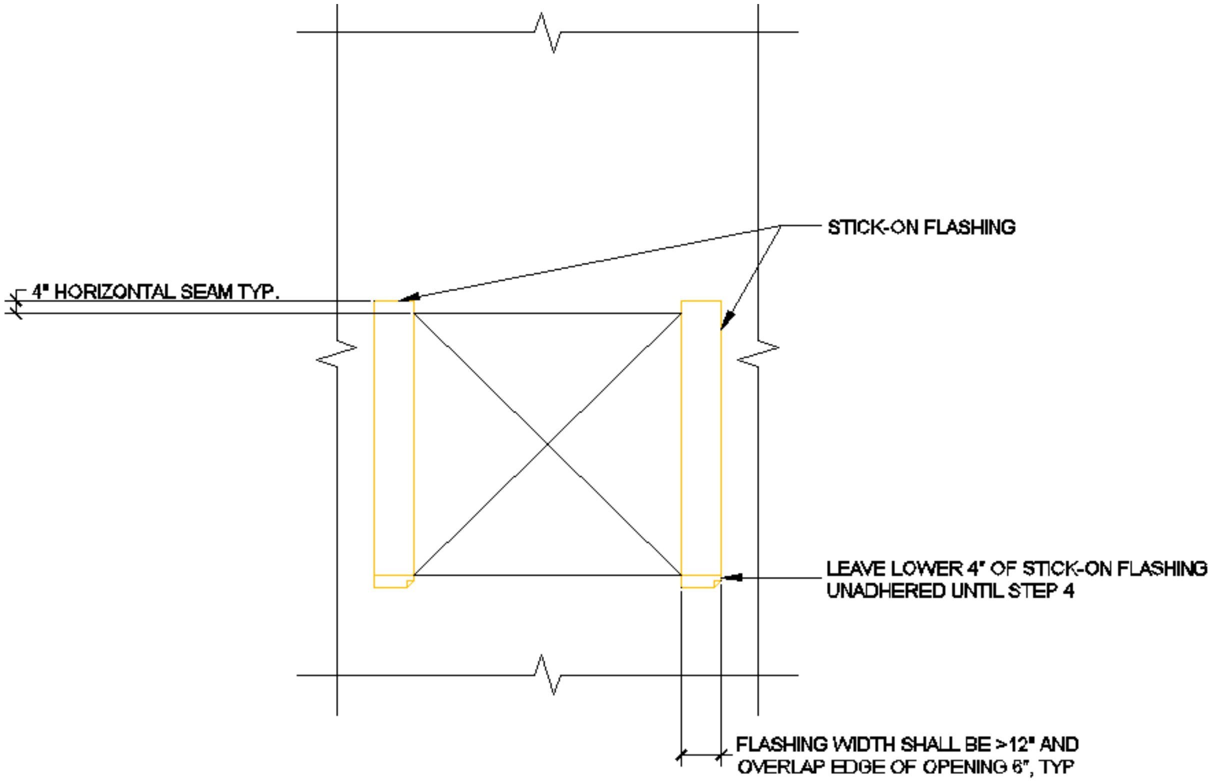


Figure 4.14, Step 1 of Applying Water Retarders

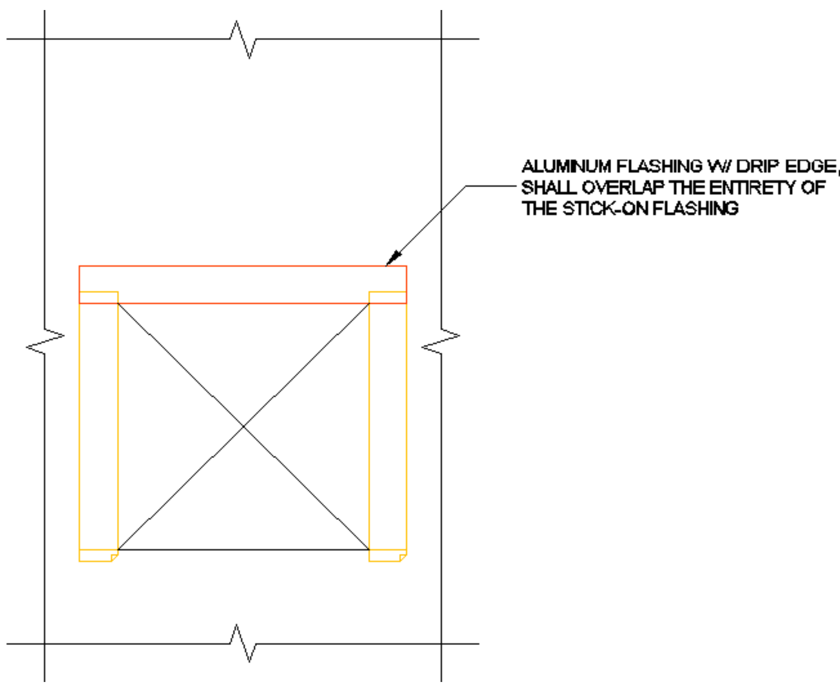


Figure 4.15, Step 2 of Applying Water Retarders

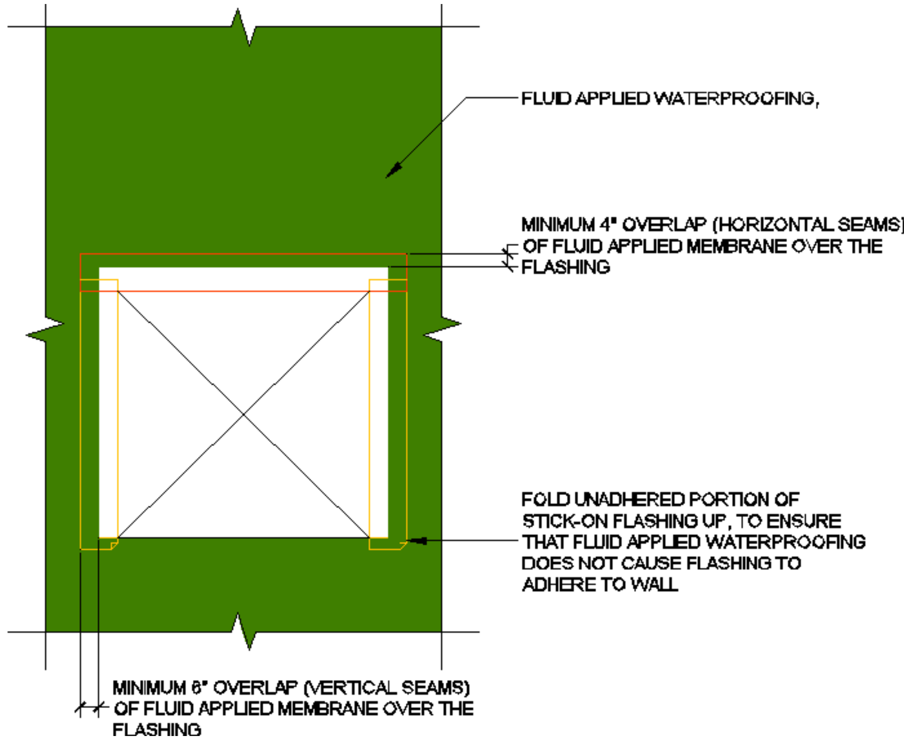


Figure 4.16, Step 3 of Applying Vapor Retarders

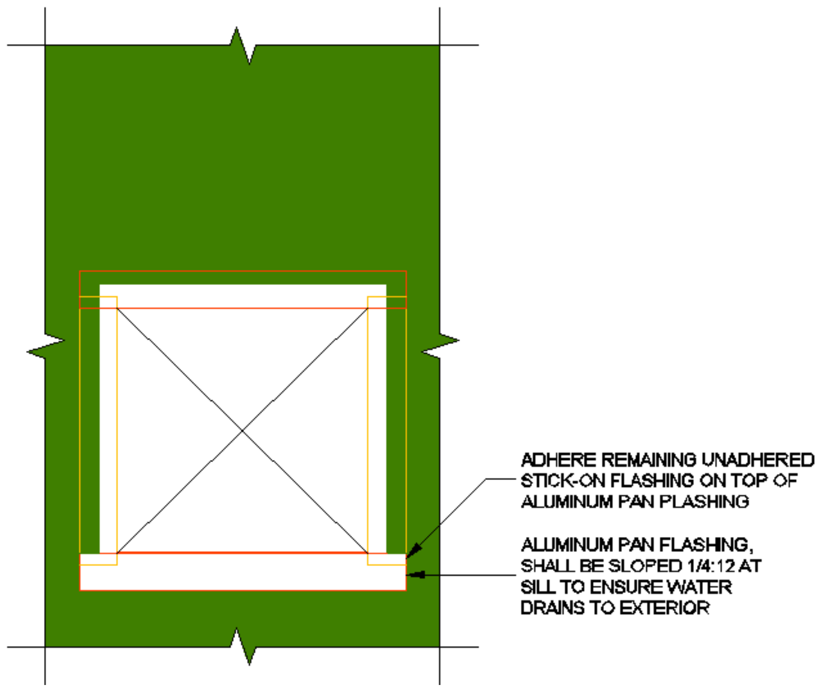


Figure 4.17, Step 4 of Applying Vapor Retarders

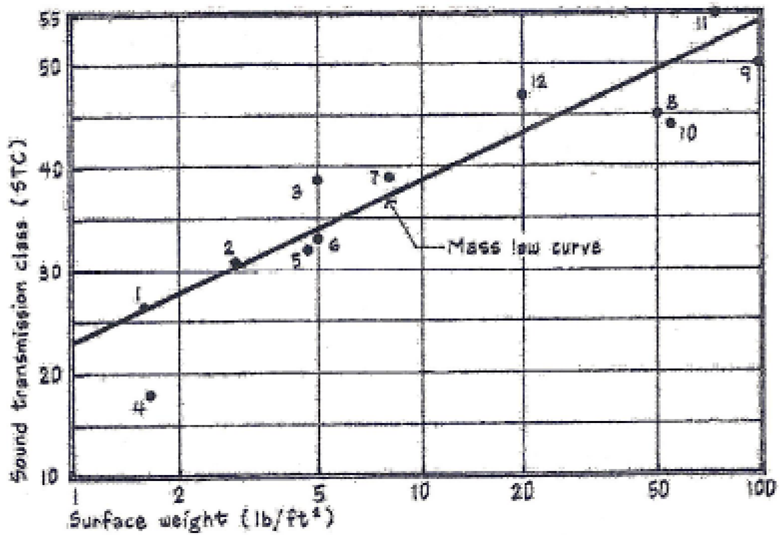


Figure 4.18, Correlation between Mass and Sound Attenuation
Source: terMeulen, 2011

Unlike the original façade wall system and the retrofit, the redesign incorporates sound attenuation material and profiles. As mentioned earlier, the reason is the lack of mass. The effects of mass on sound attenuation is known as the Mass Law. Figure 4.18, shows the direct correlation between material mass and sound attenuation – exemplified by surface weight and sound transmission class (STC), respectively. To achieve similar performance as the original façade wall system and satisfy code requirements; acoustical insulation is placed in the cavities between the CFS studs, along with resilient channels. Acoustical insulation between the CFS studs serve to dampen sound, while the resilient channels acoustically decouples the wall assembly (terMeulen, 2011). The resilient channels decouples the wall assembly, by isolating the gypsum wallboard panels from the CFS studs.

Additional parameters and assumptions defining the redesign and retrofit are listed below:

1. Weep holes permit negligible thermal exchange between the cavity and exterior
2. Weep holes are 3/8” diameter
3. Generally impermeable elements are damaged
4. When multiple materials exist at an interface, the average is used
5. Stucco is 7/8” thk. based on Portland Cement Association (PCA) recommendations
6. Fluid applied vapor retarder is 25 to 50 mils thk.
7. Steel stud flanges act as thermal bridges and are no more then 1-1/2” wide
8. Thermal resistance of the air film is neglected
9. Acoustical insulation has thermal resistance that is generally equivalent to fiberglass batt thermal insulation
10. Materials thicknesses were based on commercial availability and design recommendations

Once the original façade wall system was retrofitted and redesigned, they were compared – to determine if the redesign is reasonably feasible. Here only the thermal and moisture performance was analyzed for the comparison. Where, thermal performance is gauged to the intrinsic wall assembly R-value. Determining moisture performance based on occurrence of condensation. Condensation occurs when the relative humidity (RH) is greater than 100%. Additional comparison like acoustical performance and cost will be discussed later. See the appendix for details of the analysis and calculation method. Tables 4.11 to 4.15 details the intrinsic thermal resistance and moisture performance of the redesign, retrofit, and original façade wall systems.

Table 4.11, Thermal and Moisture Resistance of Redesign at Different Sections

Designation	Description	Total R-Value (h-ft^2-°F/Btu)	Total R_v-value
1	Through structural studs	4.2	25.7
2	Through air space between structural studs	15.4	

Table 4.12, Average Relative Humidity Across Retrofit Wall Assembly

Layer Interface	R_i/R	R_{vi}/R_v	Normal Conditions (%)				100% Exterior RH (%)			
			Winter		Summer		Winter		Summer	
			High	Low	High	Low	High	Low	High	Low
1			59.0	86.0	75.0	90.0	100.0	100.0	100.0	100.0
2	0.235	0.892	53.9	95.9	36.7	55.7	58.2	97.1	39.8	56.7
3	0.042	0.000	53.7	92.2	37.7	55.6	58.0	93.3	40.9	56.6
4	0.000	0.018	53.6	92.7	36.6	54.9	57.2	93.7	39.3	55.8
5	0.011	0.078	53.2	94.0	32.4	52.0	53.7	94.1	32.8	52.1
6	0.709	0.000	50.1	49.9	51.0	50.5	50.5	50.0	51.6	50.6
7	0.003	0.012	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0

Table 4.13, Thermal and Moisture Resistance of Retrofit and Original

Wall System	Total R-Value (h-ft^2-°F/Btu)	Total R_v-Value
Original	1.2	88.9
Retrofit	6.2	114.2

Table 4.14, Average Relative Humidity Across Original Wall Assembly

Layer Interface	R_i/R	R_{vi}/R_v	Normal Conditions (%)				100% Exterior RH (%)			
			Winter		Summer		Winter		Summer	
			High	Low	High	Low	High	Low	High	Low
1			59.0	86.0	75.0	90.0	100.0	100.0	100.0	100.0
2	0.082	0.043	58.4	81.0	76.6	88.1	97.3	93.3	101.7	97.6
3	0.000	0.000	58.4	81.0	76.6	88.1	97.3	93.3	101.7	97.6
4	0.000	0.000	58.4	81.0	76.6	88.0	97.3	93.4	101.7	97.6

5	0.442	0.953	52.1	75.9	36.9	51.1	52.2	75.9	37.1	51.2
6	0.002	0.000	52.1	75.7	37.0	51.1	52.2	75.8	37.1	51.2
7	0.474	0.004	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0

Table 4.15, Average Relative Humidity Across Retrofit Wall Assembly										
Layer Interface	R_i/R	R_{vi}/R_v	Normal Conditions (%)				100% Exterior RH (%)			
			Winter		Summer		Winter		Summer	
			High	Low	High	Low	High	Low	High	Low
1			59.0	86.0	75.0	90.0	100.0	100.0	100.0	100.0
2	0.016	0.034	58.8	86.0	74.1	88.7	98.3	99.3	98.5	98.3
3	0.710	0.222	54.4	48.9	98.3	77.9	83.0	54.2	127.6	85.1
4	0.098	0.000	53.9	44.9	104.8	77.6	82.3	49.7	135.9	84.8
5	0.000	0.000	53.9	44.9	104.8	77.6	82.3	49.7	135.9	84.8
6	0.085	0.741	50.4	54.0	47.3	50.3	50.5	54.1	47.4	50.3
7	0.000	0.000	50.4	54.0	47.3	50.3	50.5	54.0	47.4	50.3
8	0.091	0.003	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0

From the tables above, it can be determined that no condensation on the façade wall redesign, even when it is raining and the exterior becomes wet. Retrofit and original façade wall systems however, experienced some condensation. The condensation quantity is insignificant, meaning that it can drain from the wall assemblies adequately.

Table 4.16, Required Weep Hole Capacity					
Wall System	Layer Interface	Max Wall Area Served		Volumetric Flow Rate ($\lvert\Delta P/(\Sigma Rv,n \times \rho)\rvert$)	
		m^2	ft^2	m^3/24hr	in^3/hr
Original	2	64	689	0.000432	1.097
Retrofit	3	64	689	0.000372	0.946
	4	64	689	0.063706	162.007

Table 4.17, Estimated Exit Flow Rate for 3/8" Weep Hole						
Head Height		Max Wall Area Served		Exit Flow Rate ($(2\rho gh/m)^{1/2}$)		Drainage Time (s)
in	mm	m^2	ft^2	m/s	ft/s	
0.1875	4.7625	64	689	1.2	4.0	0.02

Tables 4.16 and 4.17 show the required and estimated drainage capacity of the 3/8" weep holes. While determining the actual drainage capacity, it was assumed that 50% of the weep hole is effective. The assumption was made to simulate imperfect construction, as well as build-up of minerals and dust. Using the Conservation of Energy, it was determined that moisture in the wall assemblies drains to the exterior quickly. The actual drainage rate will likely be greater from the estimate because the head height used in the estimate is conservative.

CFS Stud and Track

Replacing the concrete masonry with light gauge CFS members require structural redesign. Light gauge CFS members were designed according to AISI 100. In terms of the design load, the redesign wall system is not a part of the main wind force resisting system (MWFRS) – therefore it experiences out-of-plane wind loads and effects of self-weight. Seismic loads are not part of the design load because it is less than the more dominant wind loads. The entire façade wall redesign is based on the worst case scenario: the corner zones and a deflection of no more than L/360.

In order to simplify design and analysis of the CFS stud and track members, assumptions were made; and are as follows:

1. CFS façade walls act as simply supported beams when exposed to out-of-plane lateral loads
2. CFS façade walls carry no in-plane lateral loads
3. Structural sheathing and gypsum wallboards brace the CFS studs in the weak axis
4. Windows have equivalent weight to the wall sections which they replace
5. CFS stud spacing is 16” O.C.
6. All holes made in the compression members adhere to AISI 100 §B2.2
7. CFS track is connected to the stud in such a way that the track fails only by shear

Table 4.18, Unfactored Loads Acting on Exterior Walls

Floor Level	Gravity Load (lb/ft)			Out-of-Plane Lateral Load (lb/ft^2)		
	Dead	Live	Snow	12" O.C.	16" O.C.	24" O.C.
All	159.7	0.0	0.0	42.0		

Table 4.19, Controlling Load Combination Check

1.4D			1.2D + 1.6W + 0.5(L$_r$ ‖ S ‖ R)				
Vertical (lb/ft)		Lateral (lb)	Vertical (lb/ft)		Lateral (lb/ft)		
Gravity	Lateral		Gravity	Lateral	12" O.C.	16" O.C.	24" O.C.
223.6	0.0	0.0	191.7	0.0	67.2	89.6	33.6

Table 4.20, Recommended Stud Members

Member Designation	Member Size			Location of Applicability
	b (in)	h (in)	Thk (in)	
800S137-43	1.375	8	0.0451	Typical wall studs spaced 16" O.C.
(2)1200S162-54	1.625	12	0.0566	King Stud for opening(s) < 16' wide
(3)1200S162-54	1.625	12	0.0566	King Stud for opening(s) < 26' wide

Table 4.21, Recommended Track Members

Member Designation	Member Size			Applicability
	b (in)	h (in)	Thk (in)	

800T150-33	1.5	8	0.0346	Use w/ 8" deep studs
1200T150-54	1.5	12	0.0566	Use w/ 12" deep studs

LRFD method was used to derive the design loads for the CFS members. Above Tables 4.18 and 4.19 shows the unfactored design loads and controlling load combinations. Three studs were selected, based on required strengths and maximum deflection. Under most conditions, the maximum deflection controls and local failure of the member's elements – like flanges and webs – control over the overall global strength properties. Tables 4.20 and 4.21 details the CFS stud and track members selected for the redesign façade wall system. Studs next to openings experience the greatest design loads, therefore built-up members with multiple studs connected together were used.

Table 4.22, Potential Dimensional and Strength Limits for Dimensional Lumber								
CFS Member Designation	$M_{u,wood}$ (lb-ft)				S_{req} (in^3)		I_{req} (in^4)	
	1.4D		Other		Other		Other	
	AW2	AW3	AW2	AW3	AW2	AW3	AW2	AW3
600S137-54	0.0	0.0	7071.2	11271.2	6.2	9.9	498.5	747.7
600S162-43			7498.6	11698.6	6.6	10.3	498.5	747.7
800S137-43			7013.2	11213.2	6.7	10.7	540.0	810.0

Using a single CFS stud, next to the openings was not possible, because the required depth and thickness is not readily available. The relative bulkiness of the built-up members cannot be reduced with the addition of dimensional lumber. As dimensional lumber lacked strength and elastic modulus necessary for a less bulky assembly. Table 4.22, shows the unreasonable required section modulus and moment of inertia if dimensional lumber is used along with one CFS stud.

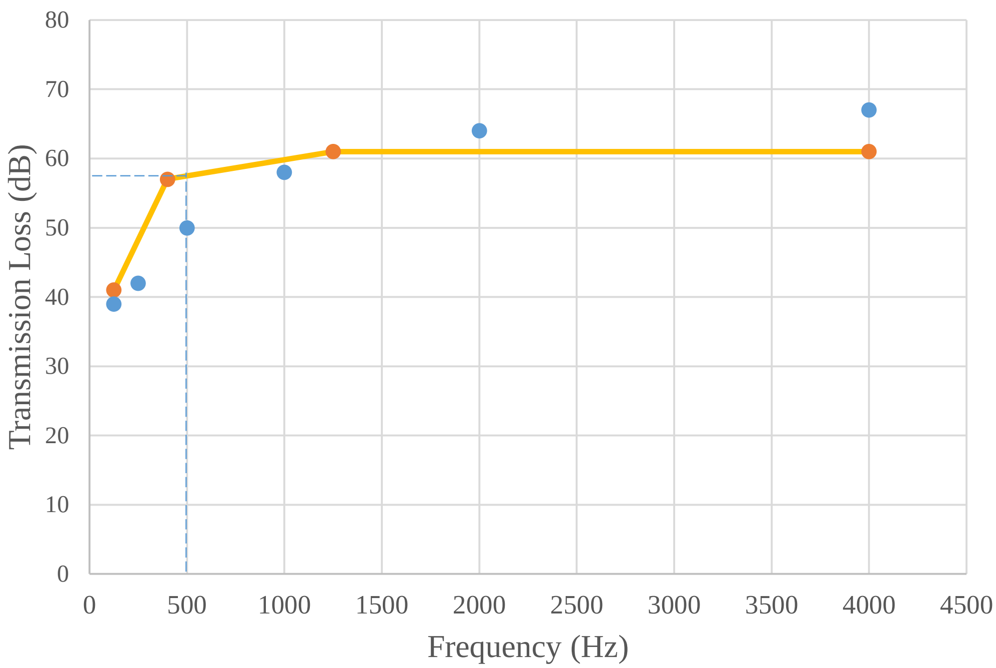
Performance Analysis and Comparison

Earlier the thermal and moisture performance of the redesign, retrofit, and original façade wall systems were analyzed. Based on the two mentioned criterions, the redesign performed the best – greater general intrinsic thermal resistance and no condensation occurrence. However, it is not enough to flat out select the redesign; because the code required thermal resistance for metal framed wall is greater than those of mass walls, and the amount of condensation in the retrofit and original façade wall systems is so small that it is insignificant. It will depend on other criterions to determine if the redesign is reasonably feasible. These criterions include: acoustical attenuation, construction cost, and ease of assembly. Ease of assembly is based on number of crews necessary to put the façade wall systems together in similar timeframe, as well as the assembly's weight.

Table 4.23, General Acoustical Properties of the Original Wall System Source: Architectural Acoustics by M. David Egan; pp 53, 205, 211	
Material	Transmission Loss (dB)

	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Rigid Insulation	39	42	50	58	64	67
Concrete						
Air Space w/ Z-Shape Furring						
Gypsum Board						

Figure 4.19, STC of Original Wall System.



Sound attenuation is an important aspect of wall assemblies. It is a requirement in the 2009 IBC, serving to maintain speech privacy. To determine the STC rating for, the transmission loss (TL) through the wall components must be determined. TL for frequencies between 125 Hz and 4000 Hz, is then plotted (Egan, 1988). The STC contour is then superimposed onto the mentioned graph, as shown in Figures 4.19. The difference between the STC contour and the plotted TL must not be greater 8 – for each frequency –, nor should the summation of the difference be greater than 32 (Egan, 1988). It is at this point that the STC rating can be determined, the STC rating is the TL on the STC contour at 500 Hz.

Table 4.24, Estimated STC of Wall	
Wall Type	STC
Façade Wall Redesign	54
Original and Retrofit Wall Design	57

For the acoustical analysis only the retrofit and original façade wall system could be determined in the described method. The redesign façade wall's STC rating, on the other hand, was determined by implementing a search for a similar wall system. A different method was used for the redesign because of the lack of acoustical data for the fiber reinforced cement board and acoustical insulation batt. The similar wall system incorporated gypsum wallboard sheathing in-lieu of fiber cement board (Owens Corning, 2004). Understanding that the fiber reinforced cement board has greater mass than the gypsum wallboard, it is expected that the redesign have a slightly better STC rating. Table 4.24 shows the STC rating of the three wall systems. What could be said immediately is that the redesign, retrofit, and original façade wall systems satisfy the 50 STC rating. In addition, the retrofit and original façade wall systems attenuate sound better than the redesign. The better acoustical attenuation arises from greater mass of the retrofit and original façade wall system, when compared to the redesign.

Table 4.25, Total Cost of Wall Systems (USD)		
Redesign	Retrofit	Original
1,799,585	858,413	869,748

Table 4.26, Number of Laborers to Complete the Façade Wall Systems in 160 Days		
Redesign	Retrofit	Original
111	83	80

Next, the cost associated with constructing the wall systems and their respective ease of assembly was analyzed. The unit cost for each façade material was acquired from *R.S. Means 2013*. The appendix shows greater detail involved in deriving the estimated total cost of the wall systems, which includes: quantity take-off, cost calculations, and respective assumptions From Table 4.25, the façade wall redesign is significantly more expensive to construct – over two times. High construction cost of the redesign primarily stems from the material and labor associated with the fiber reinforced cement board and CFS members.

Moving on, the façade wall systems' ease of assembly will be discussed. Determining which façade wall system is easier to construct can be done in various ways. The most comprehensive method includes implementing a study with a group of laborers. In this method the laborers would be required to build the façade wall systems, after that they will complete a survey. From the survey responses the systems' ease of assembly would be determined. The described method is exhaustive and requires approval – arising from the use of human subjects. Therefore, a numerical method was used. Ease of constructability was based on the number laborers needed to complete constructing the system in 160 days, and the systems' unit weight. It was assumed that only one task can be implemented at a time, only once completed can the next task be implemented. Daily output of the laborer(s) was taken from *R.S. Means 2013*. The number of laborers required to complete the façade wall redesign in 160 days is the greatest. The result is that it is more difficult to construct – low level of productivity. Laborers required for the other two systems can be found in Table 4.26.

What can be concluded is that the redesign is not a reasonable solution with the defined parameters. In most numerically significant evaluation criterions – acoustical performance, cost, and constructability – the redesign has not achieved superior performance to the retrofit and original façade wall system. On the surface, the only bright spot is the redesign's weight. When factoring in the knowledge that LMOB will be the building template for the owners – to expand to other regions in the continental U.S. – the weight advantage of the redesign becomes insignificant. The reason is that other regions do not have as great a wind load as those in Florida. As a result, the concrete masonry's cells would not need to be completely grouted and reinforced, thereby reducing the unit weight. The lesson learned here is lightweight assemblies do not necessarily translate to better constructability.

The façade redesign study in this report has reached the end of its defined scope, but it is in no way complete. Additional studies should be done to analyze the implementation of prefabrication to reduce cost of CFS stud walls and in-the-field constructability. Also other wall systems should be studied – like use of SIPs with fiber reinforced cement boards, lightweight concrete masonry – to determine if the original façade wall remains a reasonable choice.

Conclusion

The original LMOB suffered from torsional and soft story irregularity; to solve it two redesigns were completed. Each redesign has advantages and disadvantages, each measured against the original – the benchmark. One thing that the redesigns share is improved building rigidity as exemplified by the smaller fundamental building period (original = 0.72 seconds, Design I = 0.62 seconds, Design II = 0.65 seconds). This report not only delved into the structural redesign; but also their impacts on construction logistics and cost. An additional system was studied, but to a lesser extent, is replacing the masonry back-up wall with one made of CFS studs.

Both structural redesigns were designed according to defined loads in ASCE 7-05, structural concrete design criteria ACI 318-11 and TCA's 2006 Tilt-Up Construction and Design Manual. LMOB was classified as an important structure equivalent to a hospital, due to the potential of the facility becoming converted to a hospital like facility. Already the hospital has rented a few floor levels in LMOB. Despite the weight increases in both redesigns, seismic loads don't control over wind loads. In terms of serviceability, the maximum allowable drift limit $H_{story}/400$ was respected.

Construction scheduling and cost were based on R.S. Means' daily crew output and unit cost. Design II's structural complexity – having to do with the lifting process and assuming that the contractor buys temporary bracing members – meant that its structure is slightly more expensive than those corresponding to Design I and the original. The need of a large site area required to cast the tilt-up walls and temporarily closing public roads made Design II less competitive with Design I and the original design. Design I is the most reasonable alternative to the original design, while costing no more than 100,000 U.S. Dollars (USD) greater than the original. As long as LMOB's owners limit their operations in a low seismic region – like Florida – the original design is the most cost effective and reasonable solution. This changes entirely if the owner decide to expand operations to more seismically active regions, where by Design I is recommended, even though Design II's roof drift is much less.

Wrapping up the thesis project was the façade improvement. Moisture and thermal performance was slightly better than the original system. However the attempt to replace the reinforced masonry back-up wall with a metal stud back-up wall caused major façade cost increases and prolonged the construction duration.

Each redesign was achieved through the combined use of hand calculations and computer software. Computer software was used to ease hand calculations and expedite the redesign process; and are as follows: Microsoft Excel, ETABS, spBeam, and RAM Elements.

Works Cited

American Concrete Institute (ACI). "Durability - Carbonation." *American Concrete Institute (ACI)*. American Concrete Institute (ACI), June 2006. <http://www.concrete.org/Tools/FrequentlyAskedQuestions.aspx>.

- - -. "Guide to Durable Concrete." *American Concrete Institute (ACI)*. American Concrete Institute (ACI), June 2008. <http://www.maestriaenconstrucciones.com.ec/archivos/guide-to-durable-concrete.pdf>.

ASHRAE. *Standard 170 Addendum D*. 2008 ed. Atlanta: ASHRAE Standards Committee, 2010.

Belson Outdoors. "Concrete Safety Bollard." *Belson Outdoors*. Belson Outdoors, 2014. <http://www.belson.com/Concrete-Safety-Bollards>.

The Brick Industry Association. "Volume Changes - Analysis and Effects of Movement." *GoBrick*. The Brick Industry Association, Oct. 2006. <http://www.gobrick.com/portals/25/docs/technical%20notes/tn18.pdf>.

Building in Alaska. "Permeability of Common Building Material to Water Vapor." *University of Alaska Fairbanks*. University of Alaska Fairbanks, June 2011. <http://www.uaf.edu/files/ces/publications-db/catalog/eeh/EEM-00259.pdf>.

Bureau of Labor Statistics. "Top 5 U.S. States Employing Concrete/Cement Contractors." *Bureau of Labor Statistics*. U.S. Department of Labor, May 2013. <http://www.bls.gov/oes/current/oes472051.htm>.

- - -. "Top 5 U.S. States Employing Structural Steel Contractors." *Bureau of Labor Statistics*. U.S. Department of Labor, May 2013. <http://www.bls.gov/oes/current/oes472221.htm>.

Canada Mortgage and Housing Corporation, trans. "Building Technology - Flashings." *Canada Mortgage and Housing Corporation*. Canada Mortgage and Housing Corporation, 1998. <http://www.civil.uwaterloo.ca/beg/ArchTech/CMHC_flashing_BPG%20Compact.pdf>.

Clark Dietrich. "Structural Studs." *Clark Dietrich*. Clark Dietrich, Nov. 2011. <http://www.clarkdietrich.com/products/shared/398/structural-studs>.

Cook, John, and Julian Panek. *Construction Sealants and Adhesives*. Wiley, 1991.

Dalrymple, Ryan K. "Waterproofing 101." Willard Building, University Park. Oct. 2012.

DuPont. "Fluid Applied WB System." *DuPont*. DuPont, 2014. Jan. 2014. <http://www.dupont.com/products-and-services/construction-materials/building-envelopesystems /brands/air-barrier-systems/products/tyvek-fluid-applied-wb.html>.

Egan, David M. *Architectural Acoustics*. New York: McGraw-Hill, 1988.

Harris, Cyril M. *Noise Control in Buildings: A Practical Guide for Architects and Engineers*. New York: McGraw-Hill, 1994.

Koiny, Jan, Jeffrey E. Christian, and Andre O. Desjarlais. "The Performance Check Between Whole Building Thermal Performance Criteria and Exterior Wall Measured Clear Wall R-value, Thermal Bridging, Thermal Mass, and Airtightness." *SciTech Connect*. U.S. Department of Energy, 1998. May 2013. <http://www.osti.gov/bridge/servlets/purl/672157-jv1Yj4/webviewable/672157.pdf>.

Lepage, Andres. "Panel Zones." Engineering Unit B, University Park. 15 Mar. 2012.

- - -. "SAP2000 Frame Element." Engineering Unit B, University Park. 1 Mar. 2012.

- - -. "Simplifying Structural Modeling Assumptions." Engineering Unit B, University Park. 17 Jan. 2011.

Lstiburek, Joseph. "Air Barriers vs. Vapor Barriers." *Building Science*. Building Science, 2000. <http://www.buildingscience.com/documents/reports/rr-0004-air-barriers-vs-vapor-barriers>.

- - -. "EIFS - Problems and Solutions." *Building Science*. Building Science, July 2007. <http://www.buildingscience.com/documents/digests/bsd-146-eifs-problems-and solutions/?searchterm=stucco>.

- - -. "Face Sealed vs. Drainable EIFS." *Building Science*. Building Science, Mar. 2004. <http://www.buildingscience.com/documents/reports/rr-0406-face-sealed-drainableeifs>.

- - -. "Moisture, Building Enclosures, and Mold." *Building Science*. Building Science, Dec. 2001. <http://www.buildingscience.com/documents/published-articles/pa-moisture-building-enclosuresmold/view?searchterm=moisture,%20building%20enclosures,%20and%20mold>.

- - -. "Water-Managed Wall Systems." *Building Science*. Building Science, Mar. 2003. <http://www.buildingscience.com/documents/digests/bsd-146-eifs-problems-and-solutions /?searchterm=stucco>.

OSHA. "OSHA Accident Report." *OSHA*. U.S. Department of Labor, 2014. <https://www.osha.gov/pls/imis/accidentsearch.accident_detail?id=601997>.

Pactiv, ed. "C2000." *GreenGuard*. Pactiv, 2011. <http://www.trustgreenguard.com/commercial.aspx#building-wrap.c2000>.

Portland Cement Association (PCA). "Concrete Durability." *Portland Cement Association (PCA)*. Portland Cement Association (PCA), 2013. <http://www.cement.org/for-concrete-books-learning/concretetechnology/durability/ corrosion-of-embedded-materials>

Snoonian, Deborah. "Sleuthing Out Building Failures." *Architectural Record*. McGraw Hill, Aug. 2000. <http://archrecord.construction.com/resources/conteduc/archives/research/8_00_1.asp>.

Solnosky, Ryan L. "Intro Progressive Collapse Analysis & Design." Willard Building, University Park. 30 Nov. 2012.

- - -. "Structural and Facade Design." Willard Building, University Park. 15 Oct. 2012.

Texas Department of Transportation (TxDot). "Roadway Design Manual." *Texas Department of Transportation (TxDot)*. Texas Department of Transportation (TxDot), Dec. 2013. <http://onlinemanuals.txdot.gov/txdotmanuals/rdw/rdw.pdf>.

This Old House. "Attach the Lath." *This Old House*. This Old House, 2012. <http://www.thisoldhouse.com/toh/how-to/step/0,,20208184_20474853,00.html>.

Tilt-Up Concrete Association (TCA). *Tilt-Up Construction and Engineering Manual*. 6th ed. Mount Vernon: Tilt-Up Concrete Association (TCA), 2006.

- - -. "Top Ten Lists." *Tilt-Up Concrete Association (TCA)*. Tilt-Up Concrete Association (TCA), 2013. <http://www.tilt-up.org/topten/index.php>.

U.S. Army Alaska Public Affairs Office. "Force Protection Design Criteria." *U.S. Army Alaska*. U.S. Army Alaska Public Affairs Office, 2014. <http://www.usarak.army.mil/conservation/Aviation/FWA_InstallationDesignGuidelines16_Section12.pdf>.

Appendix A: Floor Plans & Elevation

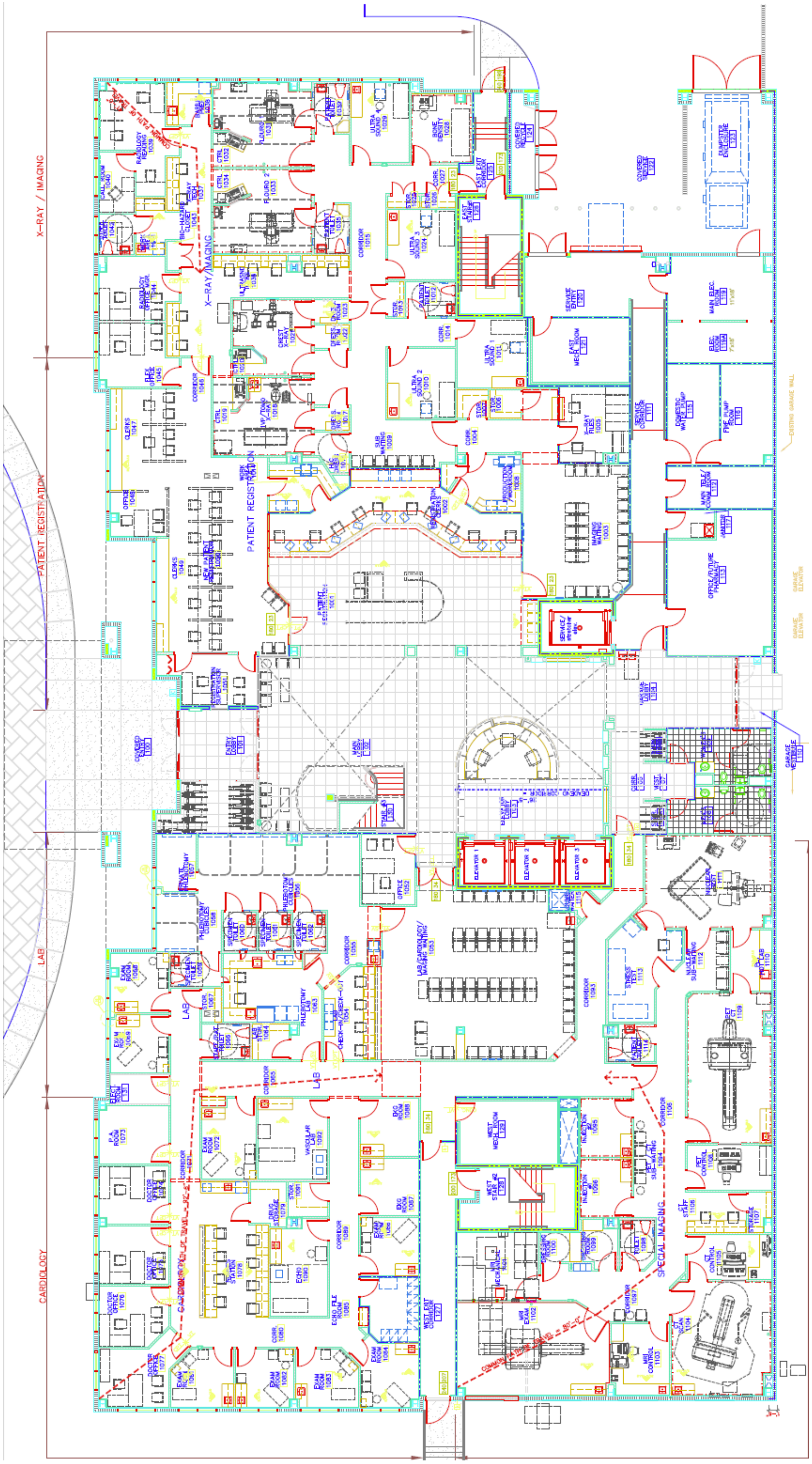


Figure AA.1, First Floor Plan w/ Tenant Build-Out
Source: Oliver, Glidden, Spina & Partners

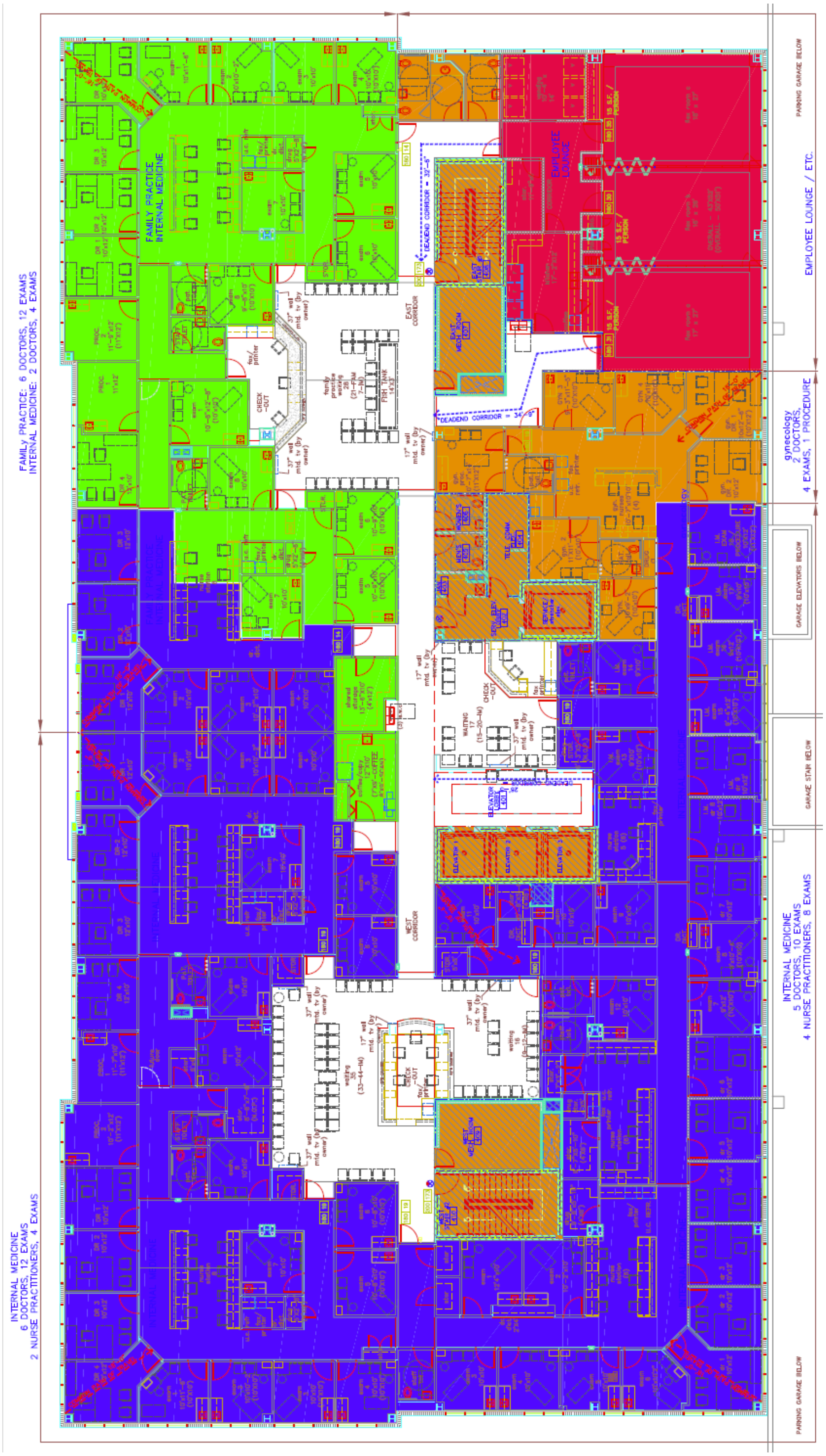


Figure AA.2, Typical Upper Floors
Source: Oliver, Glidden, Spina & Partners

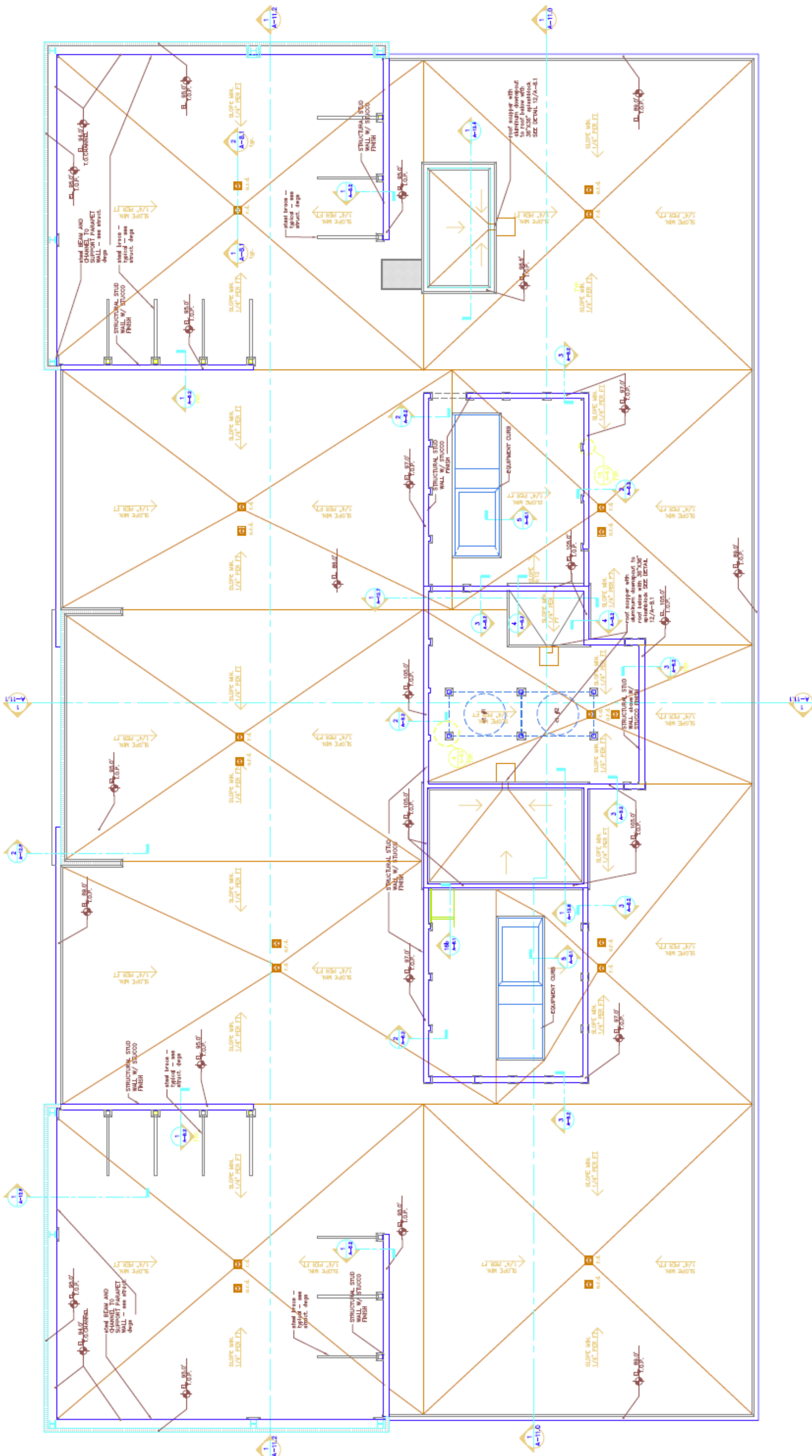


Figure AA.3, Roof Drains
Source: Oliver, Glidden, Spina & Partners

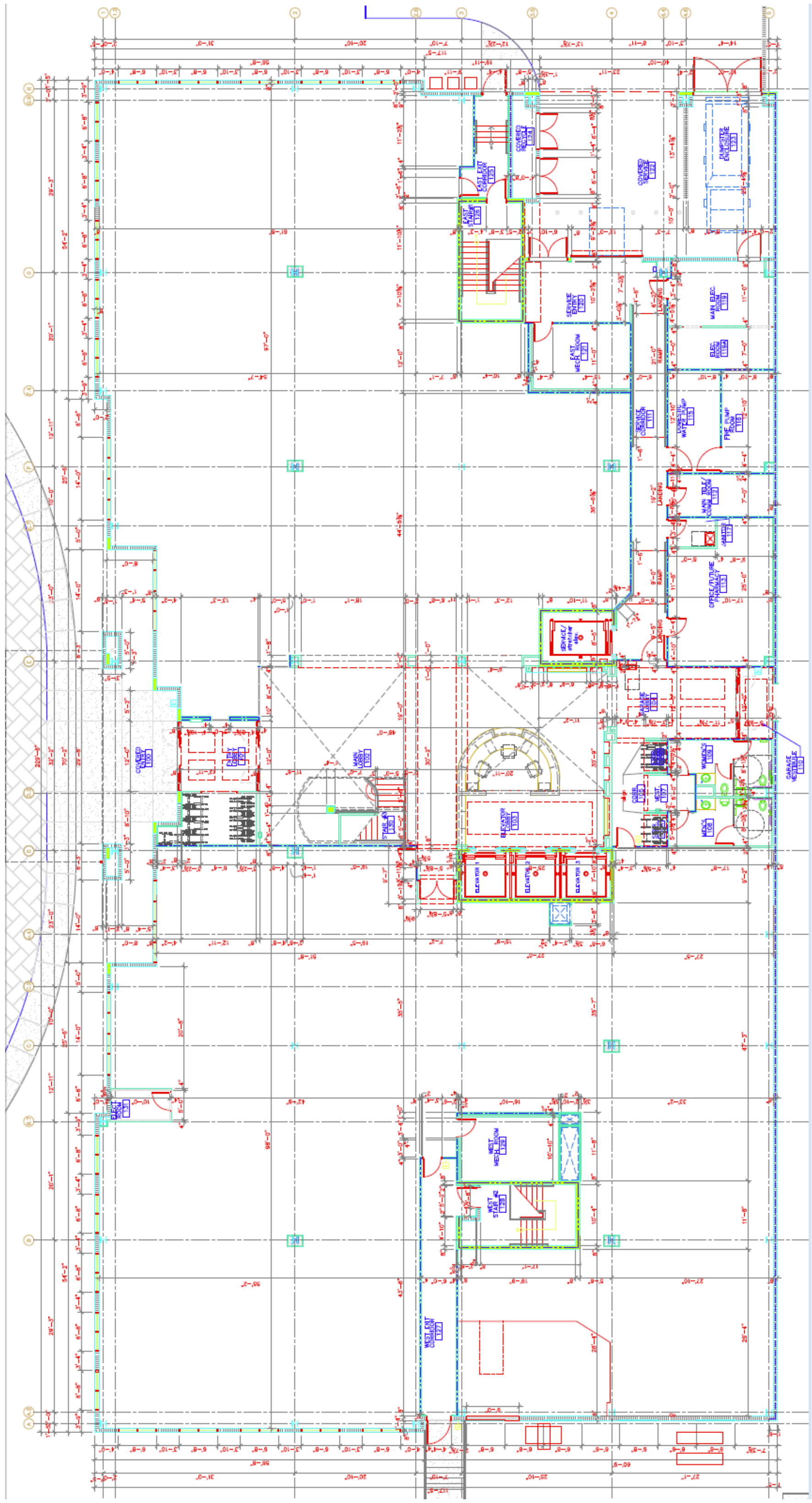


Figure AA.4, Typical Column Layout
Source: Oliver, Glidden, Spina & Partners

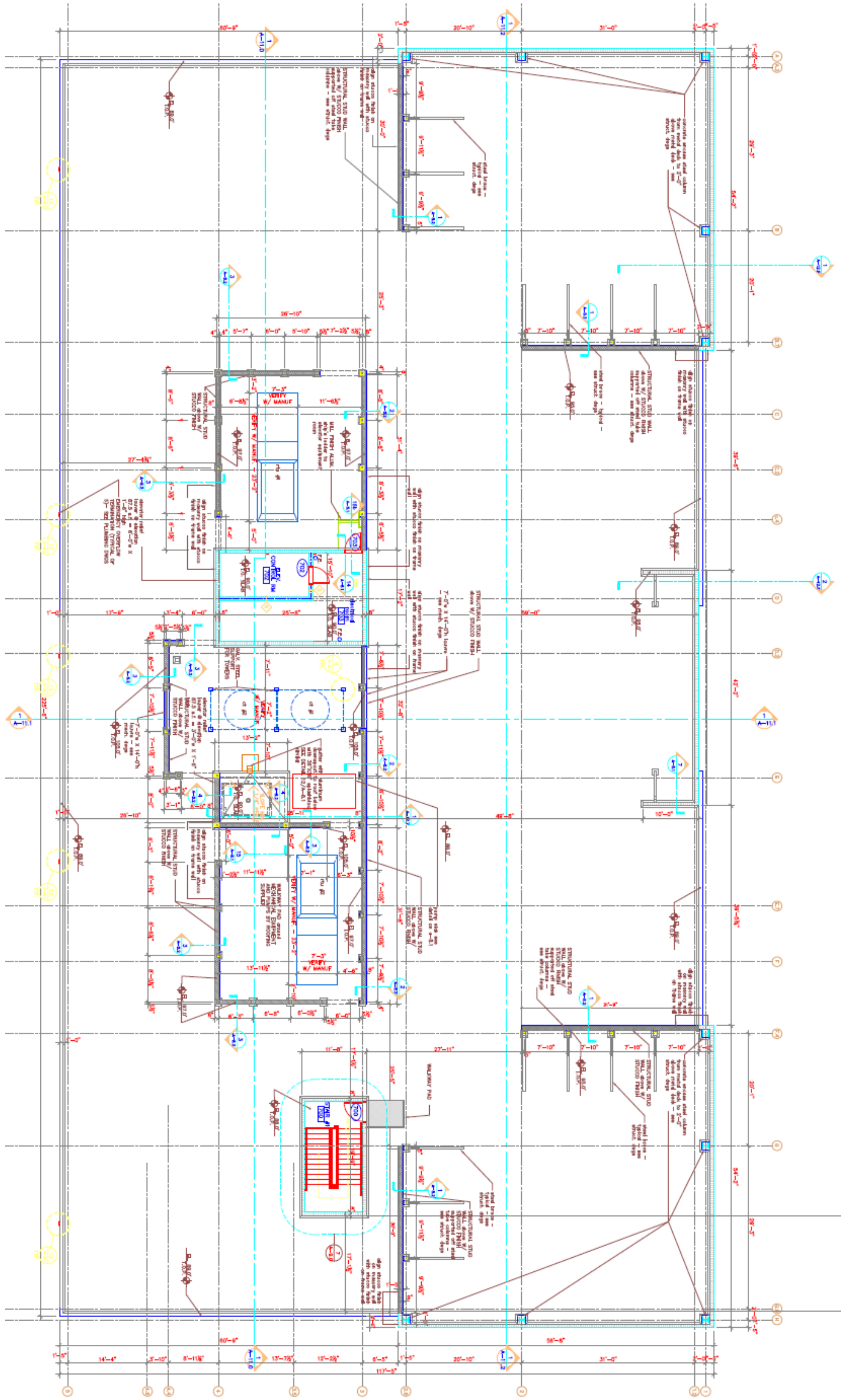


Figure AA.5, Roof Plan
Source: Oliver, Glidden, Spina & Partners

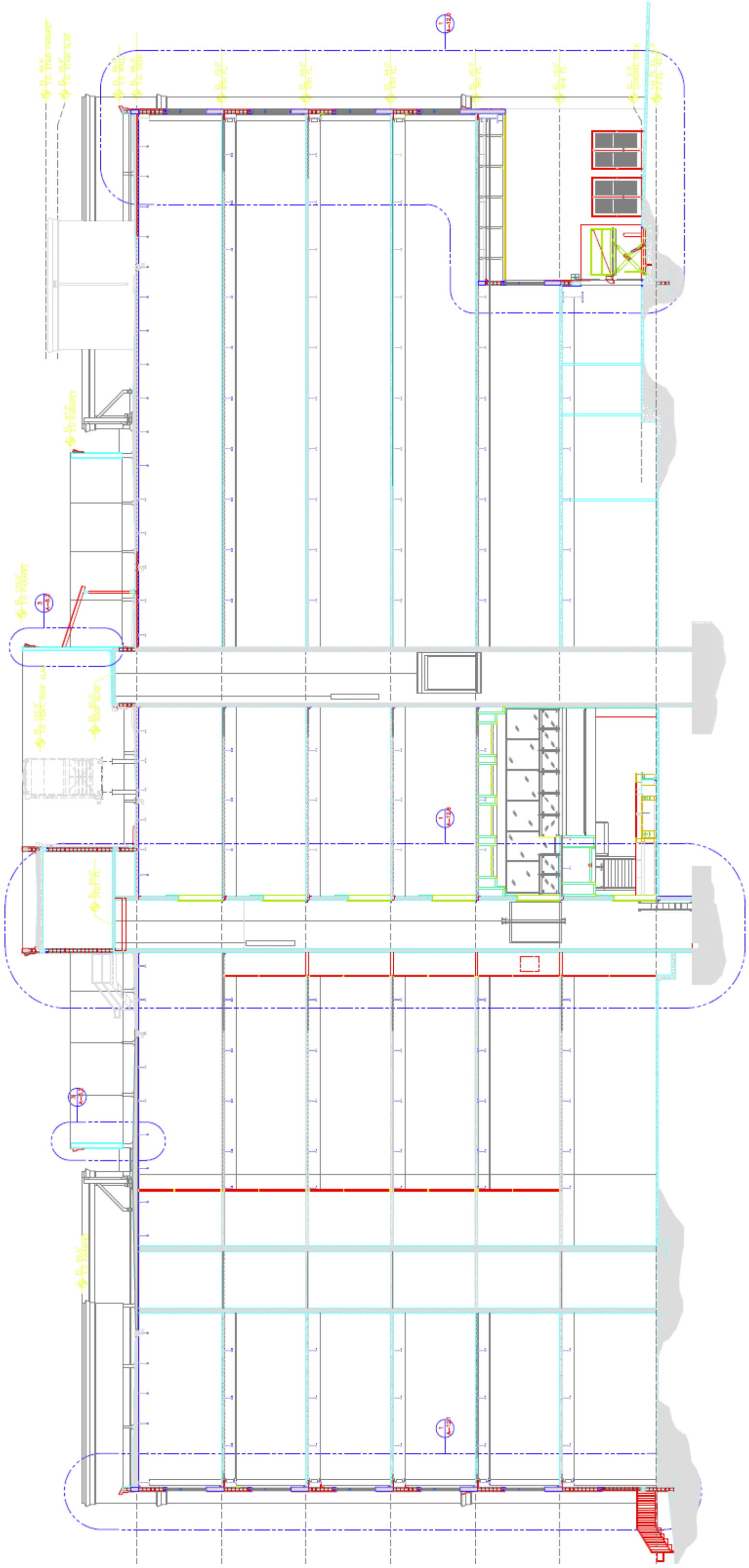


Figure AA.6, Longitudinal Building Section
Source: Oliver, Glidden, Spina & Partners

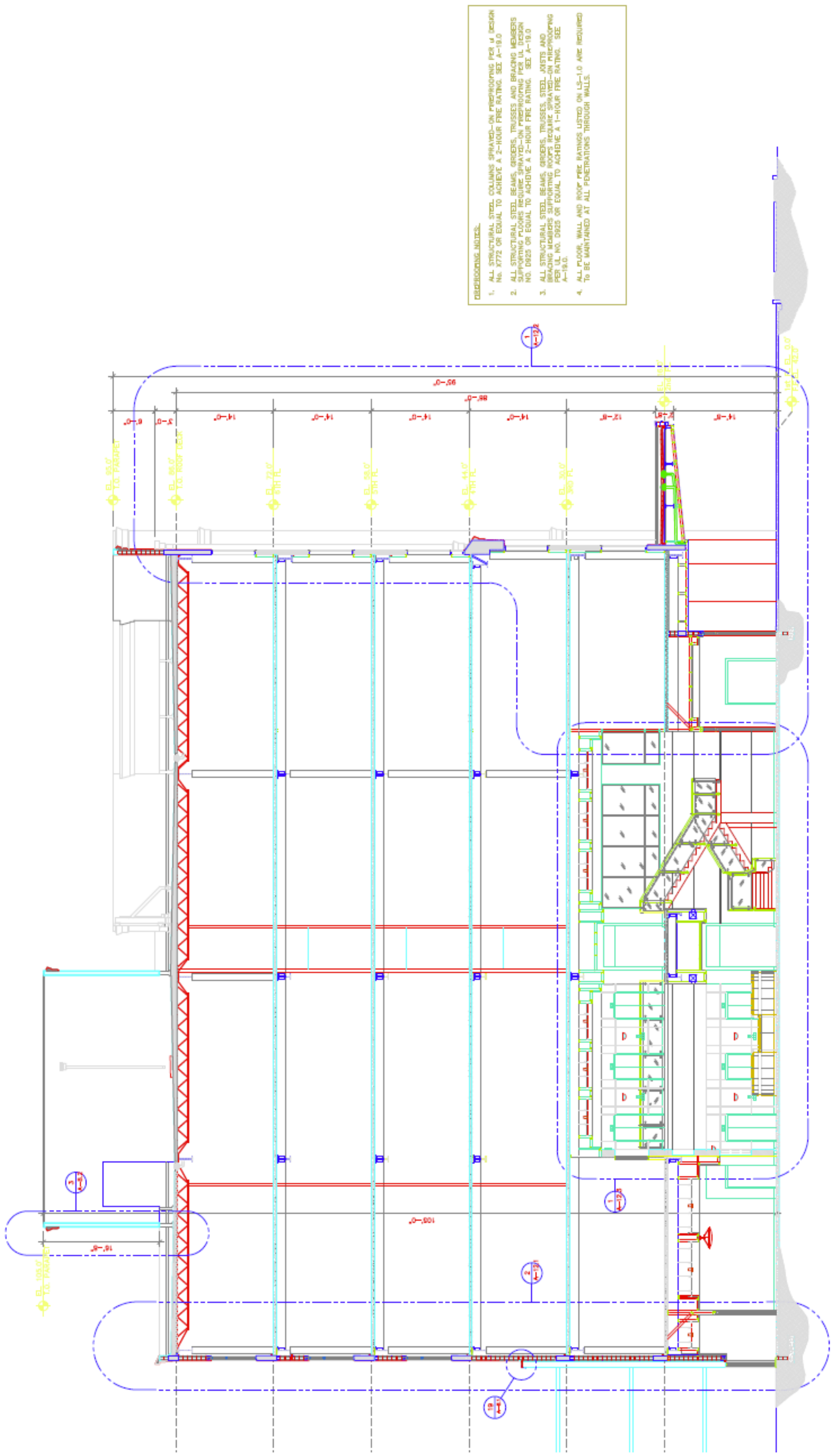


Figure AA.7, Building Section
Source: Oliver, Glidden, Spina & Partners

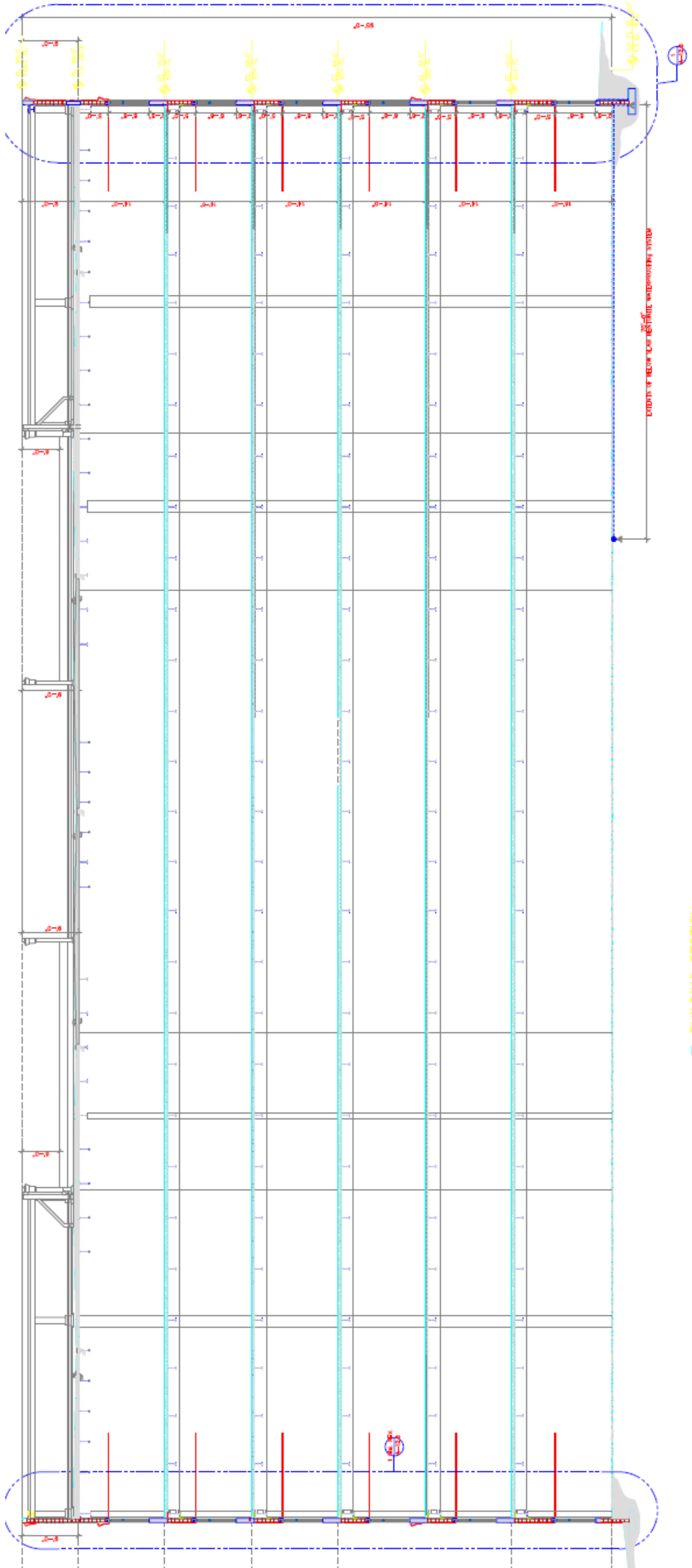


Figure AA.8, Building Section Showing Gravity System
Source: Oliver, Glidden, Spina & Partners

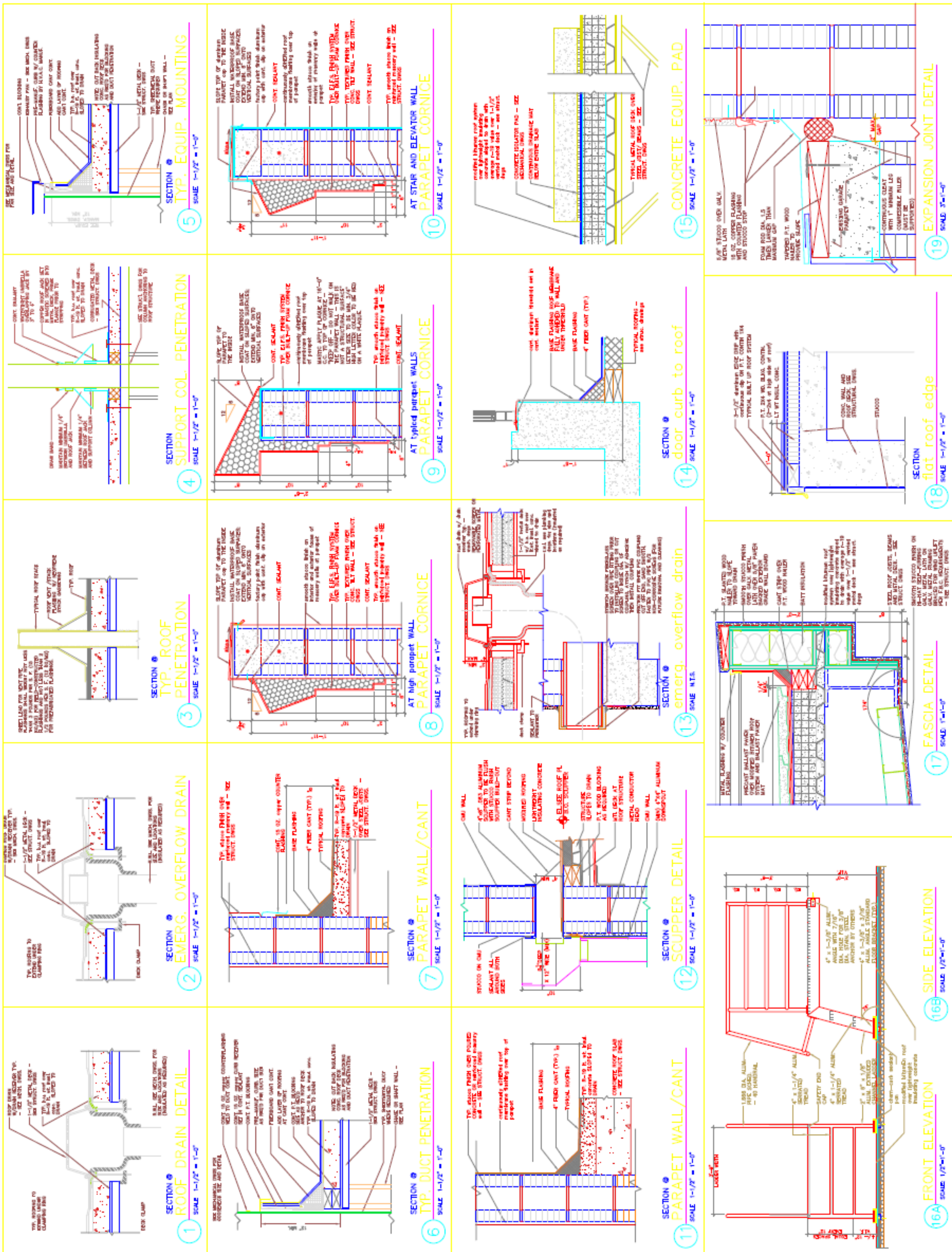


Figure AA.9, Roof Details (Part 1)
Source: Oliver, Glidden, Spina & Partners

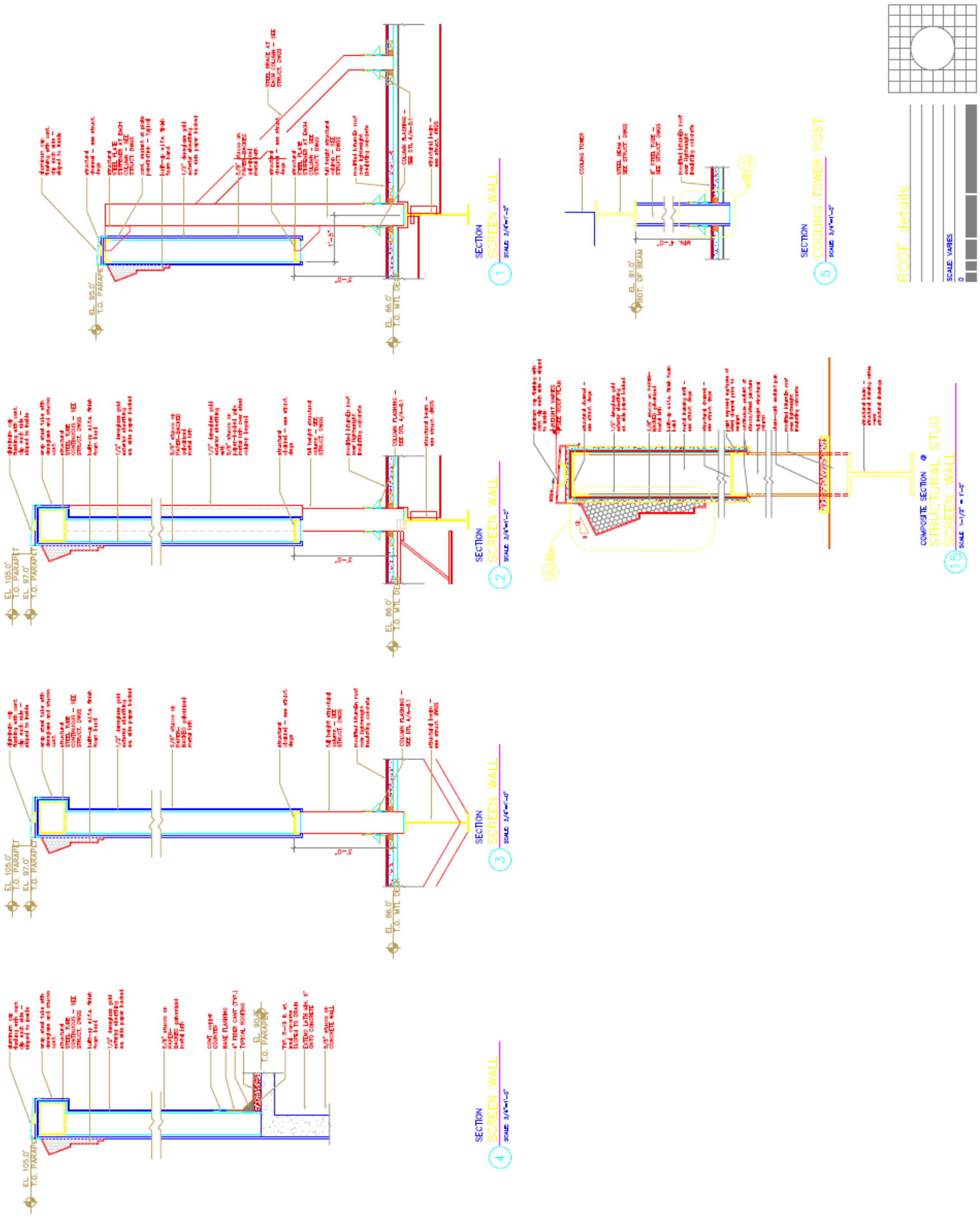


Figure AA.10, Parapet and EIFS Details
Source: Oliver, Glidden, Spina & Partners

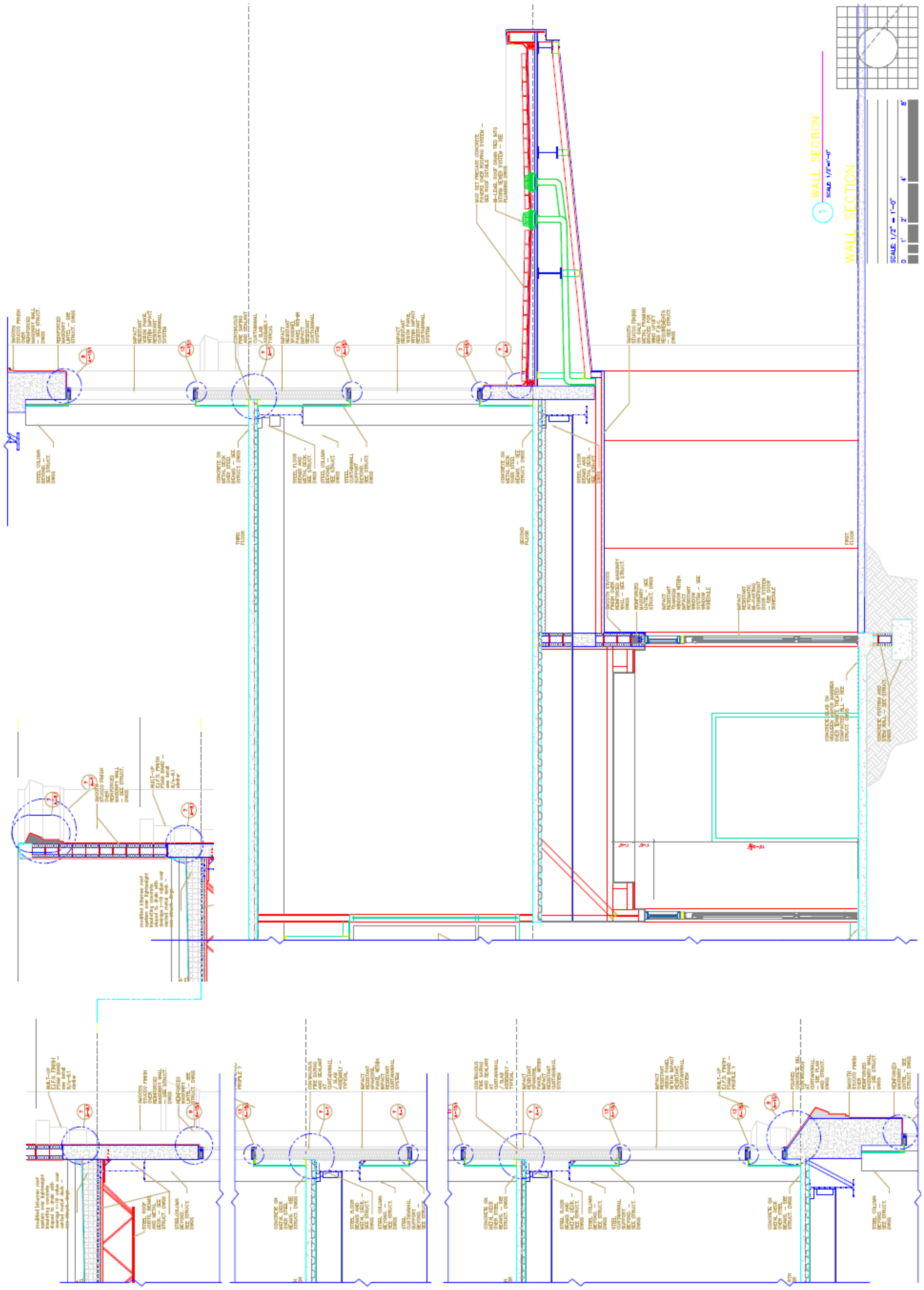


Figure AA.11, Overhang Details
Source: Oliver, Glidden, Spina & Partners

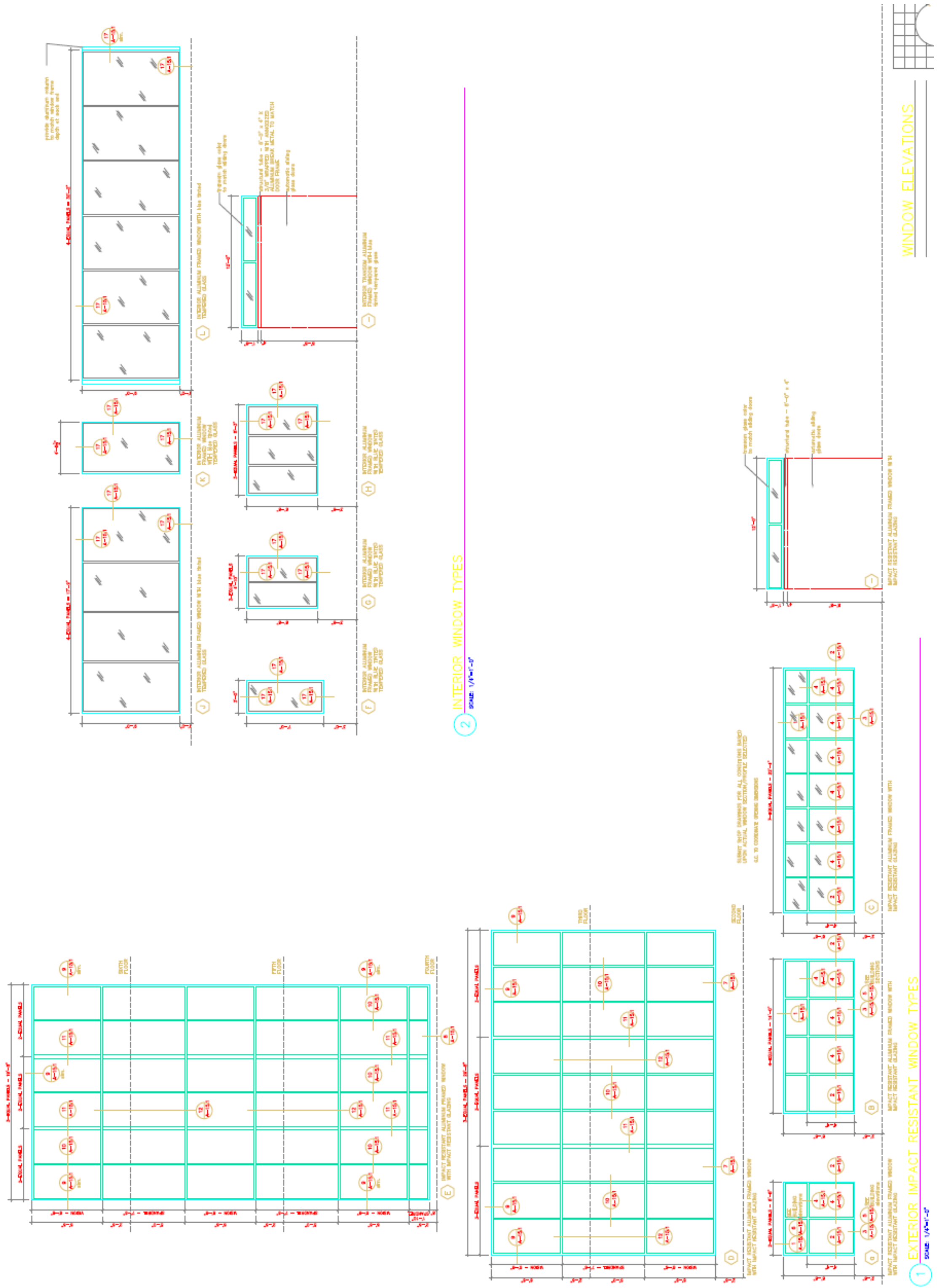


Figure AA.12, Glazing Details (Part 1)
Source: Oliver, Glidden, Spina & Partners

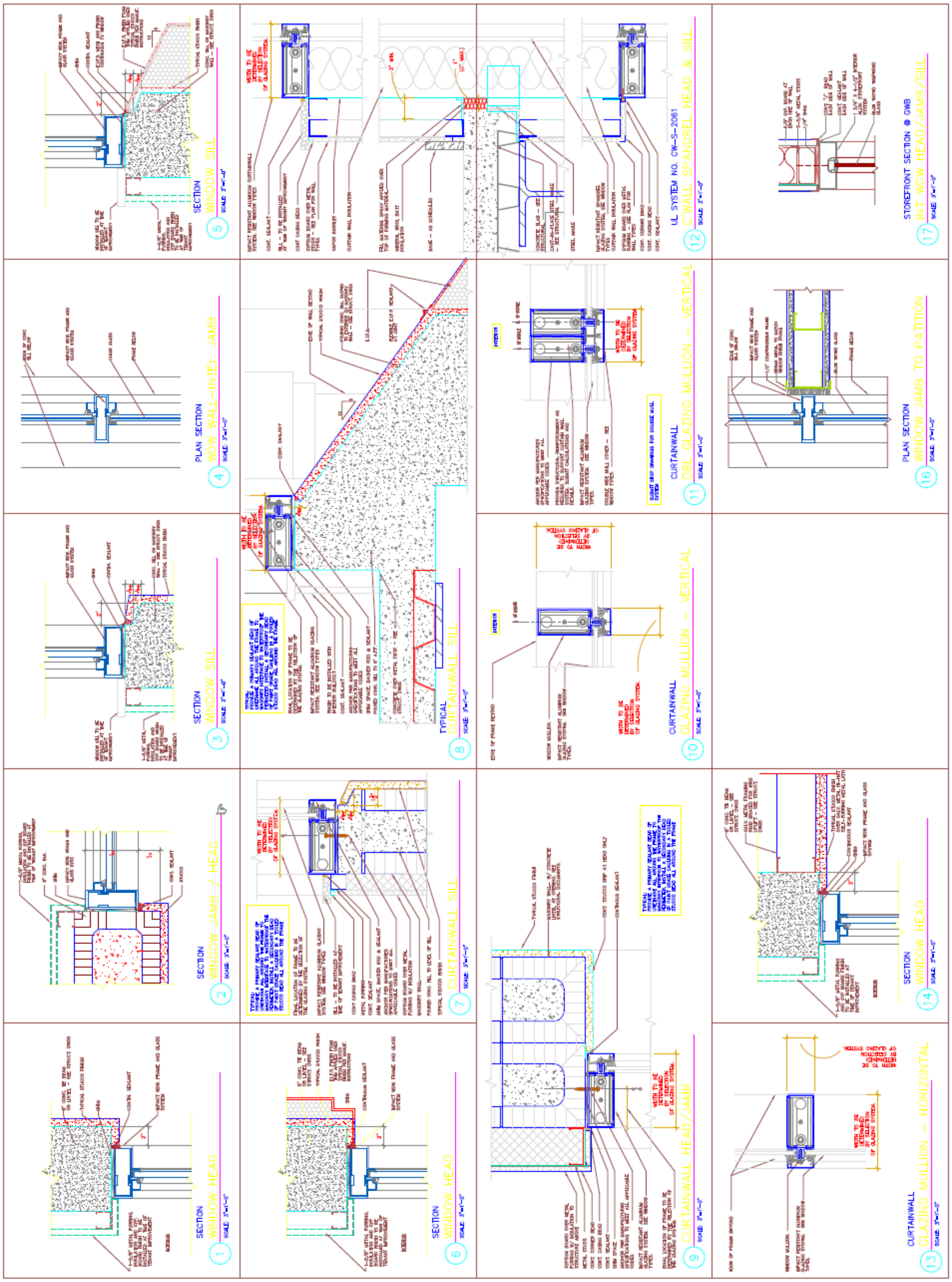


Figure AA.13, Glazing Details (Part 2)
Source: Oliver, Glidden, Spina & Partners

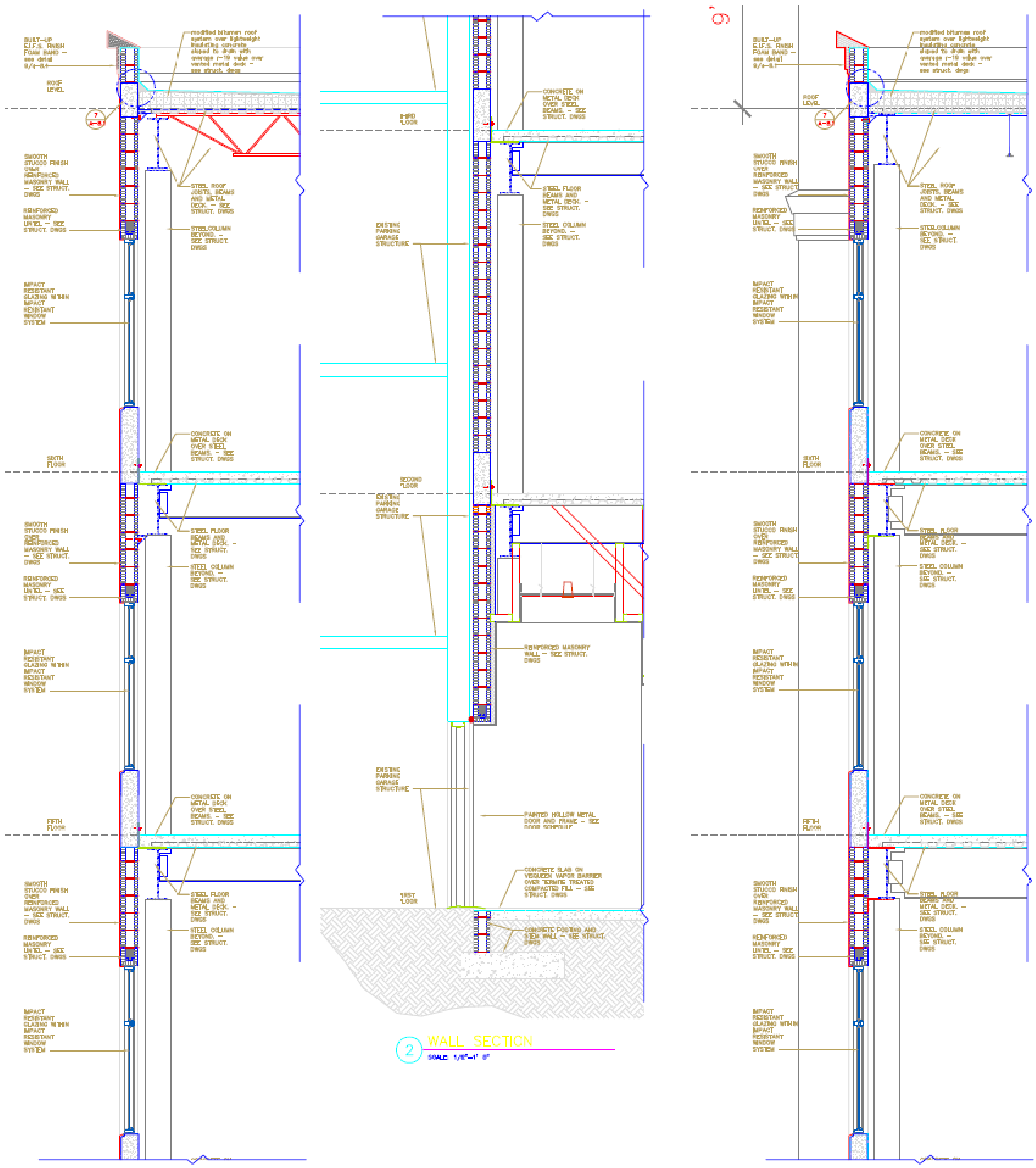


Figure AA.14, Façade Details
Source: Oliver, Glidden, Spina & Partners

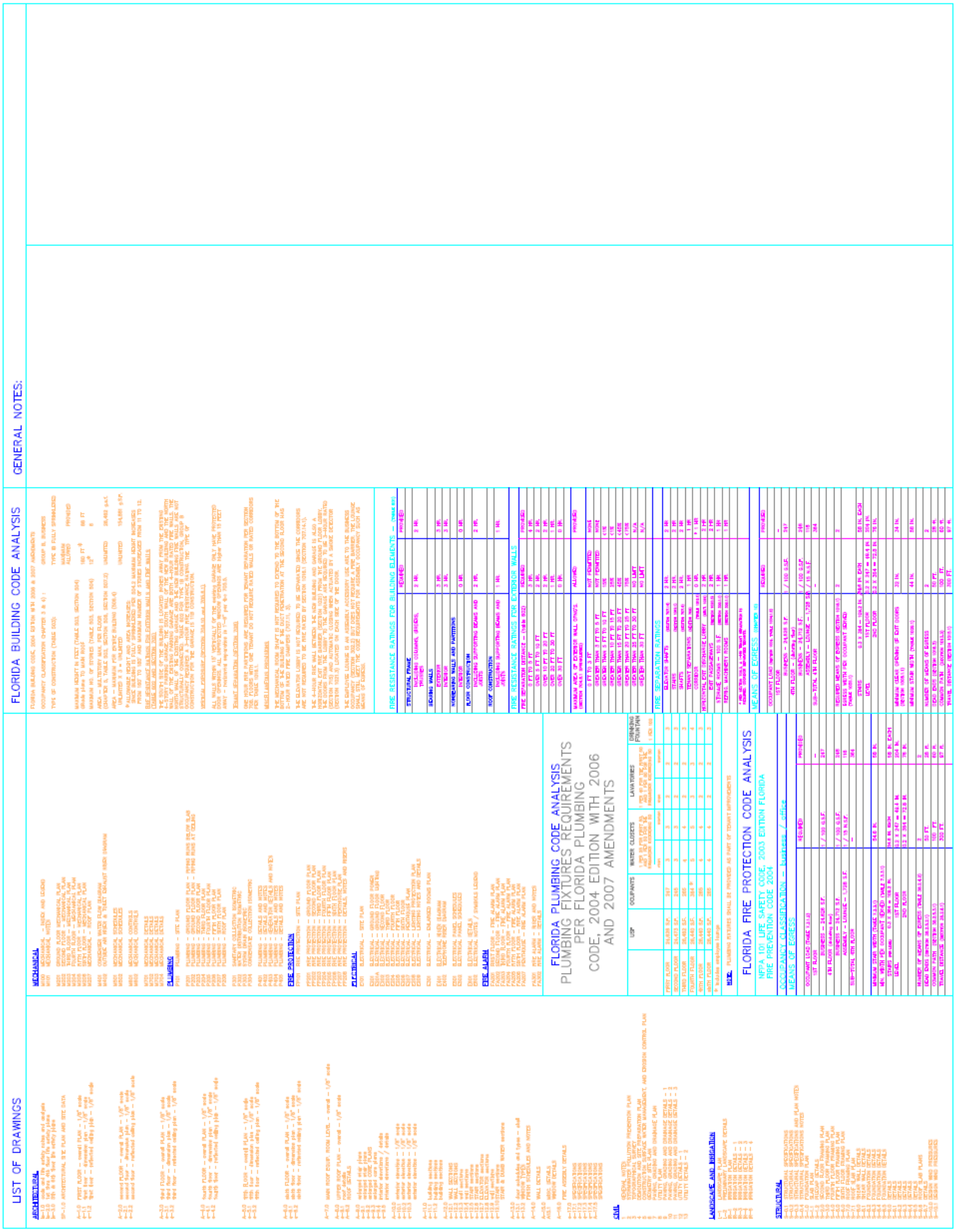


Figure AA.15, Partial Design Spec (Part 1)
Source: Oliver, Glidden, Spina & Partners

Figure AA.16, Partial Design Spec (Part 2)
Source: Oliver, Glidden, Spina & Partners

Figure AA.17, Partial Design Spec (Part 3)
Source: Oliver, Glidden, Spina & Partners

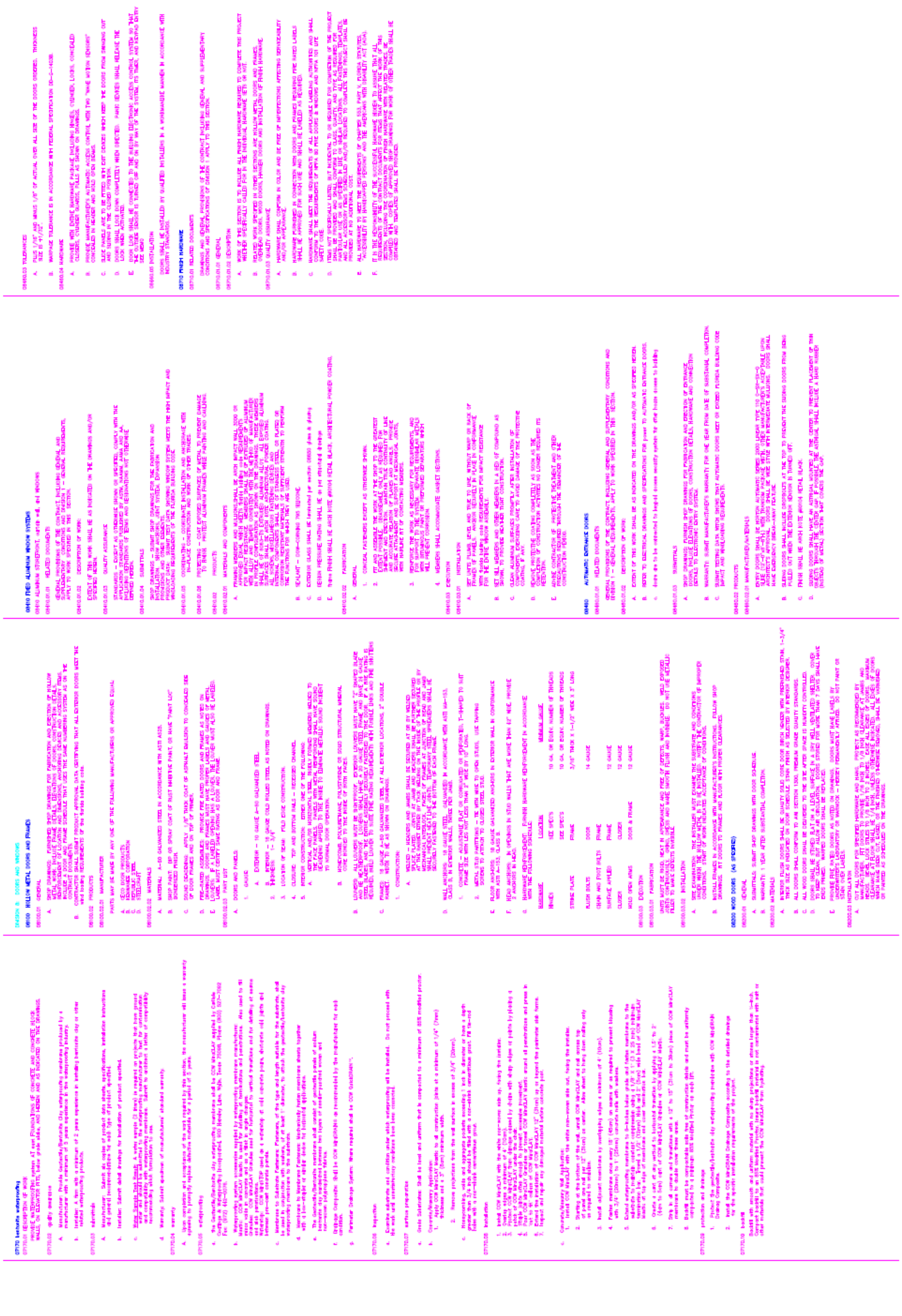


Figure AA.18, Partial Design Spec (Part 4)
Source: Oliver, Glidden, Spina & Partners

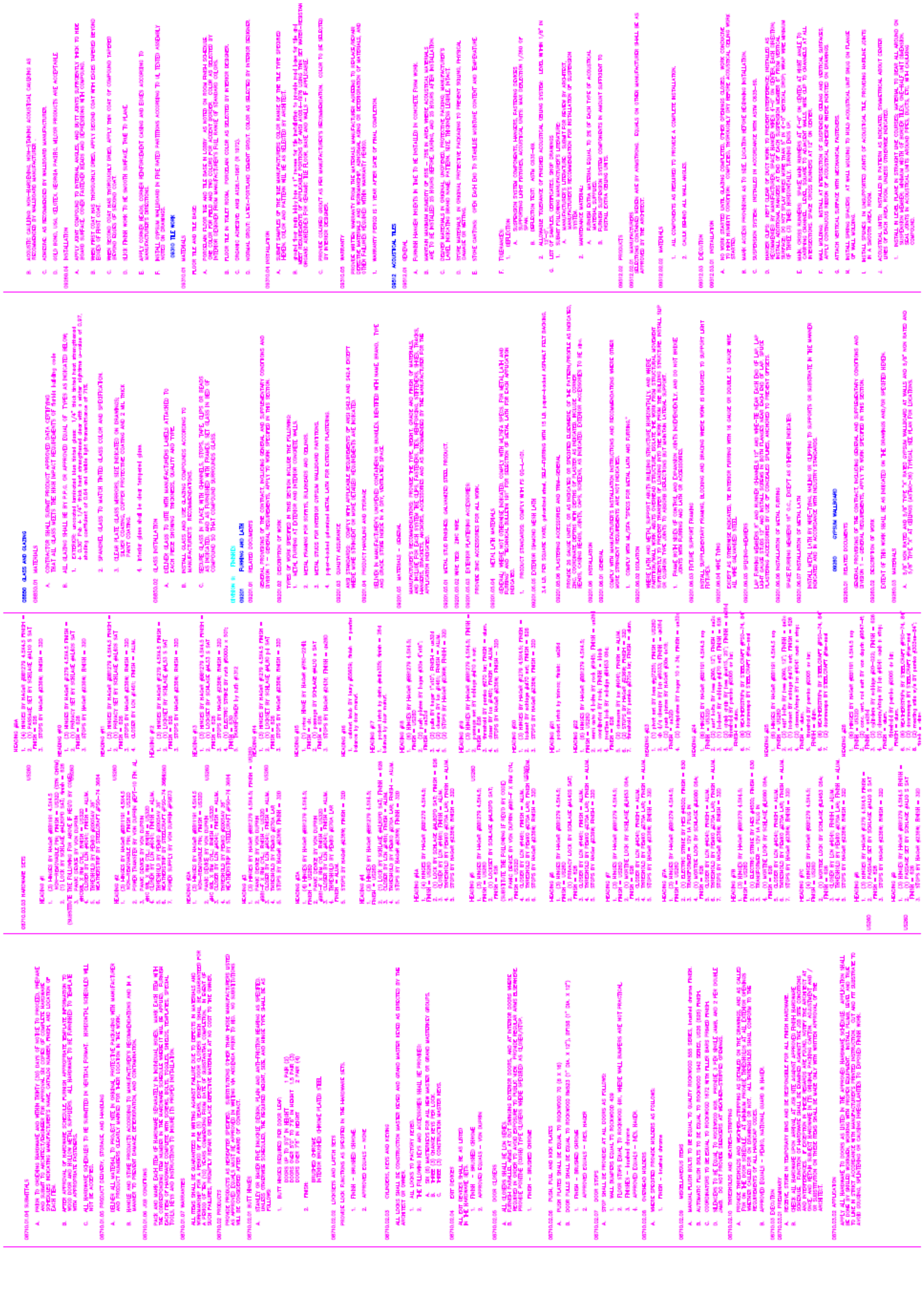


Figure AA.19, Partial Design Spec (Part 5)
Source: Oliver, Glidden, Spina & Partners

Figure AA.20, Partial Design Spec (Part 6)
Source: Oliver, Glidden, Spina & Partners

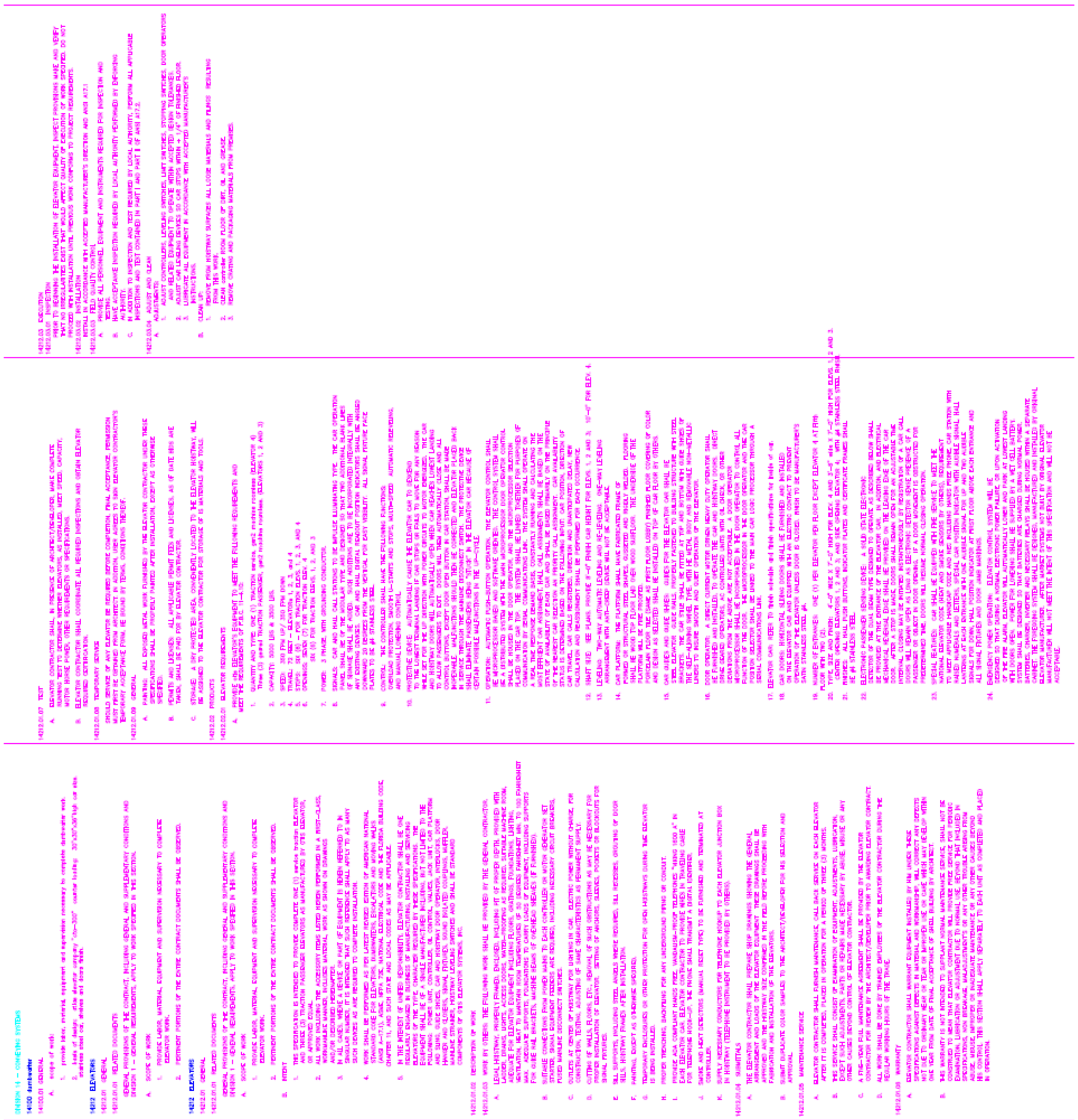


Figure AA.21, Partial Design Spec (Part 7)
Source: Oliver, Glidden, Spina & Partners

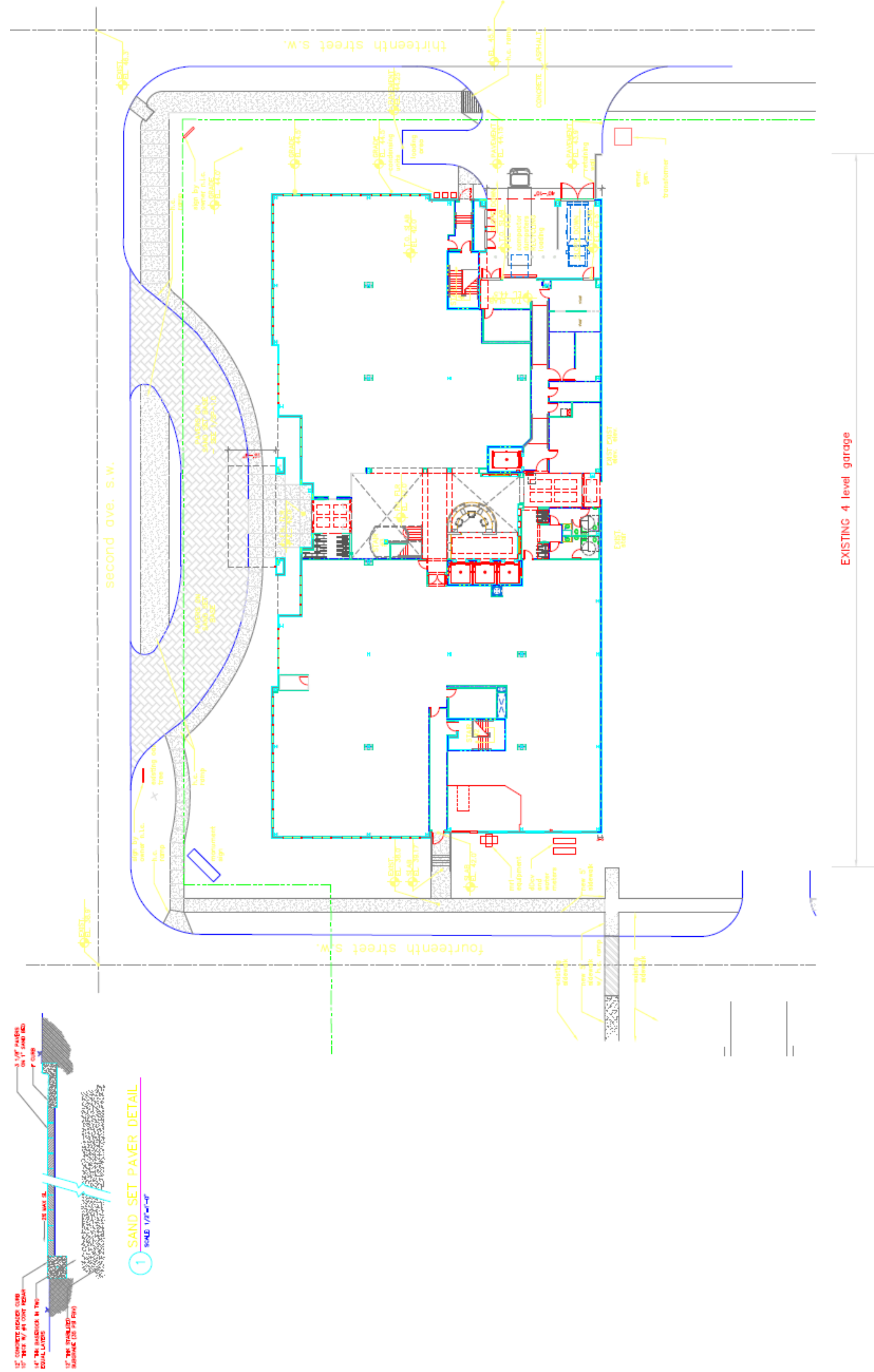


Figure AA.22, Site Plan
Source: Oliver, Glidden, Spina & Partners

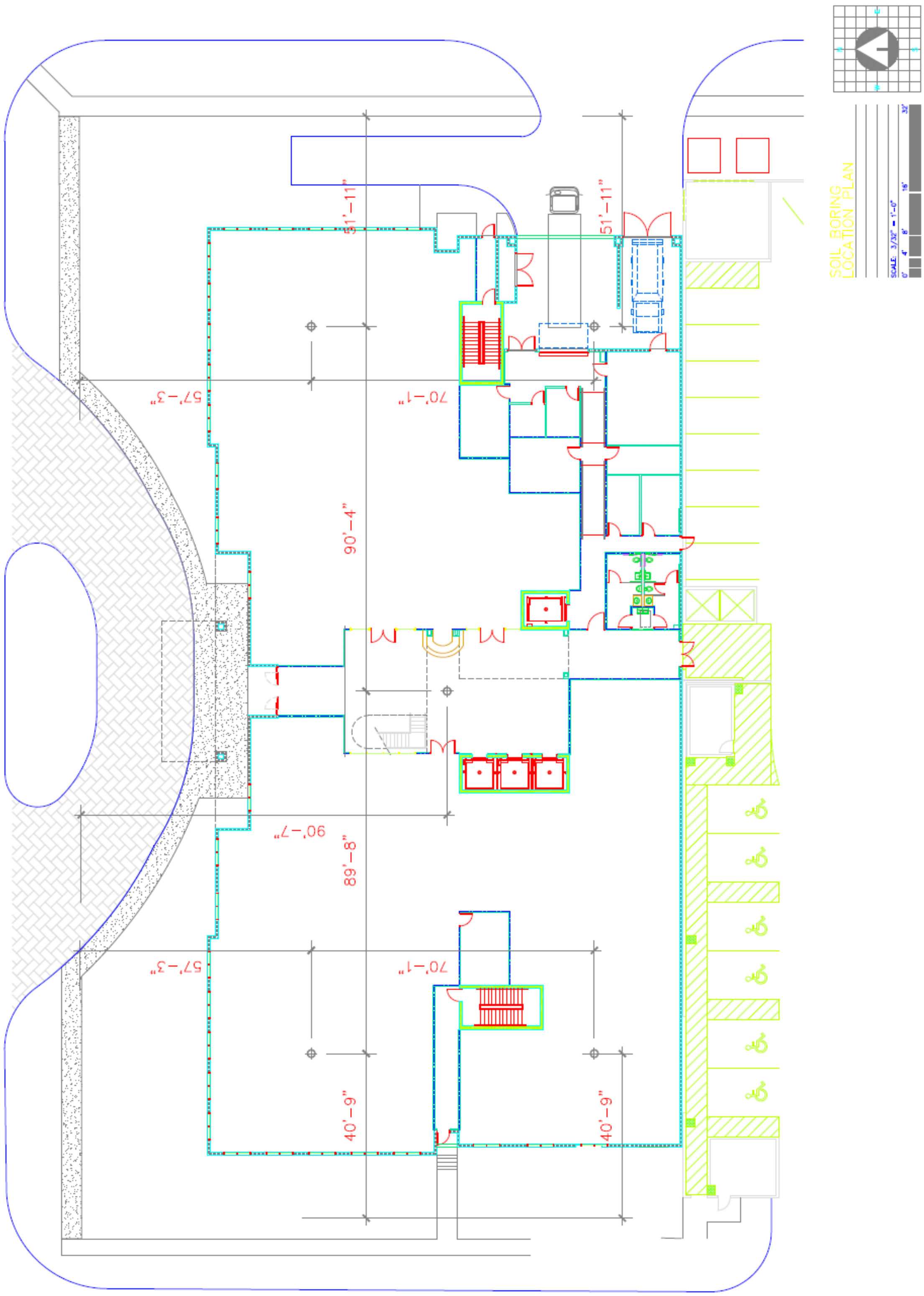


Figure AA.23, Site Boring Locations
Source: Oliver, Glidden, Spina & Partners

Appendix B: Load Determination Dead, Live, Rain

Floor Level	A_{gross} (ft^2)	$A_{fl\ opening}$[1] (ft^2)	A_{stairs} (ft^2)
0	24153.00	293.00	724.00
1	26440.00	1571.00	609.00
2	26440.00	293.00	609.00
3	26440.00	293.00	609.00
4	26440.00	293.00	609.00
5	26440.00	293.00	609.00
Roof[2]	26440.00	N/A	204.00

[1] Does not include stairwell openings

[2] Stairs extending to roof top has a roof

Story	A_{facade} (ft^2)	$A_{glazing}$ (ft^2)
1	11093.33	1588.00
2	9706.67	1920.20
3	9706.67	1846.20
4	9706.67	2681.60
5	9706.67	2780.40
6	9706.67	2780.40
Roof[3]	5079.00	N/A

[3] Roof has partitions enclosing mechanical equipment and stairwell.

** 5 lb/ft^2 dead load collateral

Material	Weight		Notes
NW. CONC	150	lb/ft^3	AISC 14 Ed. Table 17-13
LW. CONC	113	lb/ft^3	Arch. Graphics Standards 11 Ed.
VCT	1.33	lb/ft^3	Arch Graphics Standards 11 Ed.
Ceramic / Porcelain Tile	10	lb/ft^2	AISC 14 Ed. Table 17-13
3 Ply Roofing	1	lb/ft^2	AISC 14 Ed. Table 17-13
Laminated Glass – 0.8"	8.2	lb/ft^2	
MEP	15	lb/ft^2	
Partitions	15	lb/ft^2	ASCE 7-05 4.2.2

a) Floor / Deck Thickness

1) Level: 0

$t_{floor} = 4''$, solid reinf. conc.

2) Level: 1 → 5

$d_{deck} = 2''$, assume metal deck has equal size corregations

$t_{floor} = 5''$

$t_{floor,eq} = t_{floor} - d_{deck}/2 = 4''$, use to determine conc. weight

→

3) Level: Roof

$d_{deck} = 1.5''$, assume metal deck has equal site corregations

$t_{floor} = 10\,^1/_8{}'' \rightarrow 3\,^{11}/_{16}{}''$
$t_{floor,avg} = (10\,^1/_8 + 3\,^{11}/_{16})/2$
$t_{floor,avg} \approx 7''$

$t_{floor,eq} = t_{floor,avg} - d_{deck}/2 \approx 6.25''$, use to determine conc. weight

b) Floor level Dead Weight w/o structural steel, Metal Deck, Flooring, Facade

1) Level: 0

$DL = 0.150(t_{floor})(A_{gross}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$
$DL = 0.150(4/12)(24153) + 0.015(24153 - 293 - 724) + 0.005(24153)$
$DL = 1675.5$ kip

2) Level: 1

$DL = 0.150(t_{floor,eq})(A_{gross} - A_{fl\,opening}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$
$DL = 0.150(4/12)(26440 - 1571) + 0.015(26440 - 1571 - 609) + 0.005(26440)$
$DL = 1739.6$ kip

3) Level: 2→5

$DL = 0.150(t_{floor,eq})(A_{gross} - A_{fl\,opening}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$
$DL = 0.150(4/12)(26440 - 293) + 0.015(26440 - 293 - 609) + 0.005(26440)$
$DL = 1822.6$ kip/floor level

4) Level: Roof

$DL = 0.113(t_{floor,eq})(A_{gross}) + 0.015(A_{gross} \times 0.20) + 0.001(A_{gross}) + 0.005(A_{gross})$
$DL = 0.113(6.25/12)(26440) + 0.015(26440)(0.20) + 0.001(26440) + 0.005(26440)$
$DL = 1794.1$ kip

c) Dead Weight of Flooring

Floor Level	0		1		2 or 3 or 4 or 5	
Flooring	VCT	Ceramic	VCT	Ceramic	VCT	Ceramic
Area (ft^2)	1410	2841	531	653	531	339

*Other areas have exposed conc.

1) Level: 0

$DL = 1.33(1410) + 10(2841) = 30.3$ Kip

2) Level: 1

$DL = 1.33(531) + 10(653) = 7.2$ Kip

3) Level: 2 → 5

$DL = 1.3(531) + 10(339) = 4.1$ Kip / floor level

d) Dead Weight of Facade Envelope (by story)

1) Story: 1

$DL = 0.150(A_{facade} - A_{glazing}) + 0.0082(A_{glazing})$
$DL = 0.150(11093.33 - 1588.00) + 0.0082(1588.00)$
$DL = 1438.8$ Kip

2) Story: 2

$DL = 0.150(9706.67 - 1920.20) + 0.0082(1920.20)$
$DL = 1183.7$ Kip

3) Story: 3

$DL = 0.150(9706.67 - 1846.20) + 0.0082(1846.20)$
$DL = 1194.2$ kip

4) Story: 4

$DL = 0.150(9706.67 - 2681.60) + 0.0082(2681.60)$
$DL = 1073.7$ Kip

→

5) Story : 5

$DL = 0.150(9706.67 - 2780.40) + 0.0082(2780.40)$
$DL = 1061.7$ Kip

6) Story : 6

$DL = 0.150(9706.67 - 2783.40) + 0.0082(2783.40)$
$DL = 1061.3$ Kip

7) Story : Roof

$DL = 0.150(5079.00)$
$DL = 761.85$ Kip

e) Live Load w/o Live Load Reduction

Room Type	Load (lb/ft^2)	Notes
Stairs	100	ASCE 7-05 Table 4-1
Lobby & First Floor Corridor	100	↓
Corridor Above First Floor	80	
Ordinary Flat Roofs	20	

* Partitions : 15 lb/ft^2 , per ASCE 7-05 4.2.2

1) Level : 0

$LL = 0.100(A_{gross} - A_{el\ opening} - A_{stairs}) + 0.100(A_{stairs})$
$LL = 0.100(24153 - 293 - 724) + 0.100(724)$
$LL = 2313.6$ Kip

2) Level : 1

$LL = 0.080(26440 - 1571.00 - 609.00) + 0.100(609.00)$
$LL = 2001.7$ Kip

3) Level : 2 → 5

$LL = 0.080(26440 - 293.00 - 609.00) + 0.100(609.00)$
$LL = 2103.9$ Kip

→

f) Rain Load

Rainfall Rate(I): 4.5" per hour (100 year return period); per International Plumbing Code 2009 Appendix B, ASCE 7-05 C8.5

$(A) = 52 * 60.17 = 3128.7$, per ASCE 7-05 C8.5
$(Q) = 0.0104(A)(I) = 146.42$, per ASCE 7-05 C8.3

$d_s = 2\frac{5}{8} + 4(\frac{1}{4}) = 3.63"$
$d_h = 1 + \left[\frac{(Q-80)}{(170-80)}\right] = 1.738"$, interpolation of ASCE 7-05 Table C8-1

$R = 5.2(d_s + d_h)$
$R = 5.2(3.63 + 1.738)$
$R = 27.89\ lb/ft^2 > (\text{Roof live load} = 20\ lb/ft^2)$

→

Appendix C: Gravity and Lateral Spot Check

Thaison Nguyen | | Gravity Spot Check | 1/5

Member Type	Typical Span (ft)	Typical Spacing (ft)	Location
Beam	33	8.25	B1 → B2
Girder	33	33	B2 → C2
Joist	28.67	5.5	B1 → B2

a) Roof and Floor Deck, Joists

Load Combination: $1.2D + 1.6L + 0.5(L_r \text{ or } R \text{ or } S)$

	Roof Deck[1]	Floor Deck[1]	Joist
l_{span} (ft)	5.5	8.25	28.67
Spacing (ft)	N/A	N/A	5.5

[1] Assume 3 span decks

1) Roof Deck

* Assume 2 hr fire rating.

Total Load (TL) = DL + LL + R

TL = 79.9 + W_{DECK} + 27.98

TL = 107.9 lb/ft² + W_{DECK}

$DL = 0.113(6.25/12) + 0.015 + 0.001 + 0.005 + W_{DECK}$

DL = 0.0799 kip/ft² + W_{DECK}

DL = 79.9 lb/ft² + W_{DECK}

Check 1.5B24 (using Vulcraft 2008 Manual)

Max SDI Span = 5'-10" > 5'-6" ✓, Good.

Max Allowable Load = 128 lb/ft²

TL = 107.9 + 1.46

TL = 109.4 lb/ft² < 128 lb/ft² ✓, Good

* Since roof live load = 20 lb/ft² is smaller than Rain load (27.98 lb/ft²) and unlikelines of work performed on roof during rain → Use Rain load

Load Causing $l/180$ = 4/3 * 90

Load Causing $l/180$ = 120 lb/ft² > 109.4 lb/ft² ✓, Good.

* Serviceability Criteria
$\Delta \le l/180$, Supporting Non-Plaster Ceiling

* Un-protected deck is rated up to 2 hrs ✓, Good.

May use 1.5B24

2) Floor Deck

* Assume 2hr fire rating
* Assume floor deck is composite type

LL = 100 lb/ft², areas close to stairs

Check 2VLI22 using Vulcraft 2008 Manual

Weight of deck = 1.62 lb/ft²

Max SDI span = 8'-11" > 8'-3" ✓, Good

Max Superimposed Live Load = 153 lb/ft² > 100 lb/ft² ✓, Good

→

• Use Cementitious or sprayed fiber fire protection to achieve 2 hr rating

May use 2VLI22 w/ either cementitous or spray fiber protection.

3) Joists

$W_u = 1.2DL + 0.5R$
$W_u = [1.2(71.5 + W_{Joist}) + 0.5(27.89)]\ 5.5$
$W_u = [99.8\ lb/ft^2 + 1.2(W_{Joist})]\ 5.5$
$W_u = 548.9\ lb/ft + 6.6\ W_{Joist}$

$DL = 0.150(4/12) + 0.015 + 0.005 + W_{deck} + W_{Joist}$
$DL = 70\ lb/ft + 1.46 + W_{Joist}$
$DL = 71.5\ lb/ft + W_{Joist}$

* Since roof live load = 20 lb/ft² is smaller than Rain load (27.89 lb/ft²) and unlikeliness of work performed on roof during rain ⇒ use Rain load

Check 22K6 using SJI Economy Table
* Assume 2 hr fire rating
$W_u = 548.9 + 6.6(9.2)$, $W_{Joist} = 9.2\ lb/ft$
$W_u = 609.6\ lb/ft$

* Servicability Criteria
$\Delta \le \ell/180$, supporting Non Plaster ceiling

$W_{u,capacity} = (29 - 28.67)(640 - 597) + 597$
$W_{u,capacity} = 611.2\ lb/ft > 609.6\ lb/ft$ ✓, Good

$LL_{capacity} = [(29 - 28.67)(328 - 295) + 295]\ 360/180$
$LL_{capacity} = 611.8\ lb/ft > 27.89(5.5)$

$611.8\ lb/ft > 153.4\ lb/ft$ ✓, Good

* Use spray applied fire resistive materials (ex. Cementitious or fiber) to achieve 2 hr. rating, per SJI

May use 22K6 w/ spray applied fire resistive materials

b) Beam, Girders

Load Combination: $1.2D + 1.6L + 0.5(L_r \text{ or } R \text{ or } S)$
* Assume beams and girders are pinned connected, A992 Gr 50

1) Beam

$W_u = [1.2(DL) + 1.6(LL)] * \text{Spacing of bm}$
$W_u = [1.2(71.6) + 1.6(80)] * 8.25 + 1.2(W_{bm})$
$W_u = 1765\ lb/ft + 1.2\ W_{bm}$

$DL = 0.150(4/12) + 0.015 + 0.005 + W_{bm} + W_{Deck}$
$DL = 71.6\ lb/ft^2 + W_{bm}$
$LL = 80\ lb/ft^2$

$M_u = W_u \ell^2/8$
$M_u = (1765 + 1.2\ W_{bm})(33^2)/8$
$M_u = 240251 + 163.4\ W_{bm}$

$V_u = W_u \ell/2$
$V_u = (1765 + 1.2\ W_{bm})(33/2)$

→

Thaison Nguyen | Gravity Spot Check | 3/5

$V_u = 291.23 + 19.8\, w_{bm}$

Check W14x74 using AISC 14 Ed. Table 3-10, 3-6

$M_u = 240261 + 163.4(74)$

$M_u = 252.4$ Kip·ft

$\phi M_n = 272.0$ Kip·ft > 252.4 Kip·ft ✓, Good.

$V_u = 291.23 + 19.8\,(74)$

$V_u = 30.6$ Kip

$\phi V_n = 192$ Kip > 30.6 Kip ✓, Good.

May use W14x74

$\Delta_{TL} \le \ell/240$

$\Delta_{TL} \le 33(12)/240$

$\Delta_{TL} \le 1.65''$

$w_T = (DL+LL)$

$w_T = 8.25\,(71.6+80) + w_{bm}$

$w_T = 1250.7 + w_{bm}$

$\Delta_{TL} = \dfrac{5\,(1250.7+74)(33^4)}{384(29\times10^6)(795)}$

$\Delta_{TL} = 1.53'' < 1.65''$ ✓, Good

2) Girder

* Assume girders use shear studs to have composite action.
* For ease in constructability assume all beams are W16x89

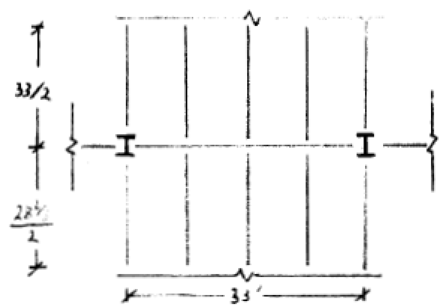
$\ell_{brace} = 0$

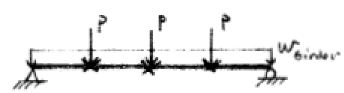
$M_u = \frac{33}{4}(P_u)(1.5) + 1.2\,w_{Girder}\left(\frac{33^2}{8}\right)$

$M_u = \frac{33}{4}(98.4)(1.5) + 1.2\,w_{Girder}\left(\frac{33^2}{8}\right)$

$M_u = 1217.7 + 204.2\, w_{Girder}$

↑ in Kip





$P_D = [0.150(4/12) + 0.015 + 0.005 + 1.42](8.25)(33+28\tfrac{2}{3})/2 + 0.089(33+28\tfrac{2}{3})/2$

$P_D = 52.1 + 2.7$

$P_D = 54.8$ kip, unfactored Dead Load

$P_L = 0.080(8.25)(33+28\tfrac{2}{3})/2$

$P_L = 20.4$ kip, unfactored Live Load

$P_u = 1.2\,P_D + 1.6\,P_L$

$P_u = 1.2(54.8) + 1.6(20.4)$

$P_u = 98.4$ kip

$W_u = 1.2\, W_{Girder}$

$b_{eff} = \min \begin{cases} 2 \cdot \ell/8 \\ (\text{Spacing 1} + \text{Spacing 2})\,1/2 \end{cases}$

$b_{eff} = \min \begin{cases} 8.25' \\ 30.8' \end{cases} = 8.25'$

$A_s f_y = 0.85 f'_c\, b_{eff}\, a$, assume $f'_c = 4000$ psi

$a = \dfrac{50 A_s}{0.85(4)(8.25)(12)}$

$a = 0.149 A_s$, if neutral axis (Plastic) is in conc.

→

Check W24x76 using AISC 14Ed. Table 3-19, Table 1-1

* Assume perfect shear transfer

$A_s = 22.4\ in^2$

$a = 0.149(22.4)$

$a = 3.34'' > 3''$ (Solid part of floor slab), PNA is in flange of STL. member

$A_s f_y = 0.85 f'_c b_{eff} \bar{x}_{solid} + 2 F_y b_f x$

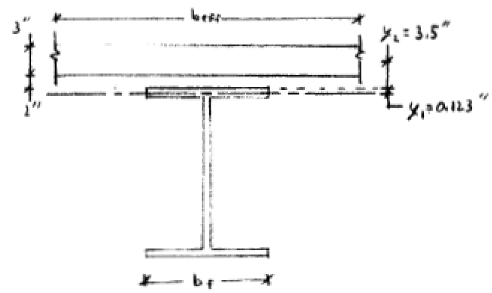
$\frac{A_s f_y - 0.85 f'_c b_{eff} \bar{x}_{solid}}{2 f_y b_f} = x$

$x = \frac{22.4(50) - 0.85(4)(8.25 \times 12)(3)}{2(50)(8.99)}$

$x = 110.2/899$

$x = 0.123''$

$y_1 = x$



$M_u = 1217.7 + 204.2(76/1000)$

$M_u = 1233.2$ kip-ft

$I_{LB} = \frac{(0.17 - 0.123)(4770 - 4580)}{0.17} + 4580$

$I_{LB} = 4632.5\ in^4$

$\phi M_n = \frac{(0.17 - 0.123)}{0.17} \times (1300 - 1260) + 1260$, interpolation of Table 3-19

$\phi M_n = 1271.1$ kip-ft > 1233.2 kip-ft ✓, Good.

$\Delta_{LL} \le \ell/360$, final live Load

$\Delta_{LL} = \frac{5(P_L/33)(33^4)(1728)}{384(29000)(4632.5)}$

$\Delta_{LL} = 0.123'' < 1.1''$ ✓, Good.

$\Delta_{LLD} \le \ell/360$

$\Delta_{LLD} \le 33(12)/360$

$\Delta_{LLD} \le 1.1''$

$P_{constr} = [0.150(4/12) + 0.005 + 1.52](8.25)(33 + 28\tfrac{2}{3})/2 + 0.089(33 + 28\tfrac{2}{3})/2$

$P_{constr} = 48.3 + 2.7 = 51.0$ Kip

$W_{Girder} = 0.076$ Kip/ft

May use W24x76 w/ Shear Studs (Composite Action, Partial)

$\Delta_{LLD} = \frac{5(0.076)(33^4)(1728)}{384(29000)(2100)} + \frac{5(51/33)(33^4)(1728)}{384(29000)(2100)}$

$\Delta_{LLD} = 0.033 + 0.677$

$\Delta_{LLD} = 0.71''$, during construction

$0.71'' < 1.1''$ ✓, no shoring req.

→

C) Column

Location: B-2

*Assume pinned base

$K = 1$

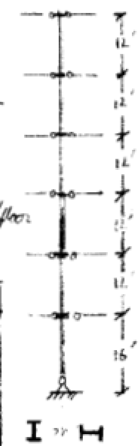
$P_L = 0.080(A_{trib})(5 \text{ floors})$

$P_L = 0.080(990.5)(5)$

$P_L = 396.2$ Kip, live load w/o reduction

$A_{trib} = \frac{(31.25+33)}{2} * \frac{(33+28^2/_3)}{2}$

$A_{trib} = 990.5\ ft^2/floor$



$P_R = 0.02780(990.5)(1)$

$P_R = 27.6$ Kip, rain load

Dead Load Components[2]	Weight	Notes
LW CONC.	113 lb/ft³	Arch Graphics Standard 11 Ed.
Roof Deck	1.46 lb/ft²	Vulcraft 2008 Deck Manual, 1.5B24
Joist	9.2 lb/ft	Vulcraft 2008 Joist Manual, 22K6
3 Ply-Roofing	1 lb/ft²	AISC 14 Ed Table 17-13
NW CONC	150 lb/ft³	AISC 14 Ed Table 17-13
Floor Deck	1.62 lb/ft²	Vulcraft 2008 Deck Manual, 2VLI 22
Beam	74 lb/ft	AISC 14 Ed, W14×74
Girder	76 lb/ft	AISC 14 Ed, W24×76
MEP	15 lb/ft²	

[2] 5 lb/ft² per floor level, dead load collateral to be included

$$P_D = 113(7/12)(990.5) + 1.46(990.5) + 9.2(33+28^2/_3)(0.5)(5.5) + 1(990.5)$$
$$+ [150(4/12)(990.5) + 1.62(990.5) + 74(33+28^2/_3)(0.5)(3.5) + 76(33+31.25)(0.5)]5$$
$$+ 15(990.5)(6) + 5(990.5)(6)$$

$P_D = 69.3 + 61.6(5) + 14.9(6) + 5.0(6)$

$P_D = 496.7$ Kip, dead load

$P_{TL} = 1.2P_D + 1.6P_L + 0.5P_R$

$P_{TL} = 1243.8$ Kip

$K\ell_x = 1(16) = 16'$

$K\ell_y = 1(16) = 16'$, weak axis bending controls.

<u>Check W14×120 using Table 4-1 in AISC 14Ed</u>

$\phi P_n = 1310$ kip > 1243.8 kip ✓, Good

May use W14×120 for Column B-2

Thaison Nguyen | | Lateral spot check / Design | 1/4

AMPAD

Controlling Lateral Load : Wind
Load Combination: $1.2D + L + 0.5L_r + 1.6W$

*** Design lateral force resisting member w/ maximum overturning and shear

Member w/ Max Overturning and Shear : AV1-Y1

$M_{u,max} = 1.6(16608.2)$, using value calculated in Irregularity Analysis - Irregularity and Wind
$M_{u,max} = 26573.1$ Kip*ft
$V_{u,max} = 1.6(325)$, using value calculated in Irregularity analysis - Irregularity and Wind
$V_{u,max} = 520$ Kip

A. Classify Shear Wall design

$L_t/W_L = 86/21.08$
$L_t/W_L = 4.08 > 3$; Slender /Flexural behavior per Reinforced Conc. Mech. & Design by James K. Wright , James G. MacGregor §18-5 pp.937
$L_t/T_b = 86/(8/12)$
$L_t/T_b = 129$

B. Design (Flexural)

*** Assume #8 flexural reinforcement , #4 shear reinforcement

$f'_c = 3000$ psi
$f_y = 60$ ksi
$b = 8''$
$A_s = 0.79(2) = 1.58$ in^2, (2) #8 per row

$a = \frac{A_{s,tot} f_y}{0.85 f'_c b}$
$a = \frac{nA_s(60)}{0.85(3)(8)}$
$a = 2.94 n A_s$

*** Assume $\phi = 0.9$, where $\varepsilon_s \geq 0.005$
*** Initially assume no top reinf.

$M_u = \phi A_{s,tot} f_y (d - a/2)$
$\frac{M_u}{\phi f_y} = n A_s (d - a/2)$
$5905.1 = nA_s\{[251.21 - 0.5nS_c + 0.5S_c] - 1.47nA_s\}$
$5905.1 = nA_s\{[251.21 + 0.5S_c] - [0.5S_c + 1.47A_s]n\}$
$5905.1 = 1.58n\{[251.21 + 0.5S_c] - [0.5S_c + 1.47(1.58)]n\}$
$5905.1 = (396.9 + 0.79S_c)n - (0.79S_c + 3.67)n^2$

$d = 21.08(12) - 0.75 - 0.5 - 0.5 - \frac{(n-1)}{2} S_c$
S_c = Space btw bars (o.c.)
n = # of Rows
$d = 251.21 - \frac{(n-1)}{2} S_c$

$a_1 x^2 + b_1 x + c_1 = 0$
$a_1 = -0.79S_c - 3.67$
$b_1 = 396.9 + 0.79S_c$
$c_1 = -5905.1$

$n_{max} = \frac{21.08(6) - 1.75}{S_c} + 1$
$n_{max} = \frac{124.73}{S_c} + 1$

→

Thaison Nguyen | | Lateral Spot Check / Design | 2/4

$\underline{S_c = 2''}$

$a_1 = -0.79(6) - 3.67$
$a_1 = -8.41$
$b_1 = 396.9 + 0.79(6)$
$b_1 = 401.64$
$n = 20.2$
$n \approx 21$ rows $\leq n_{max}$ ✓

$n_{max} = 63$ rows

$a = 2.94(21)(1.58)$
$a = 97.5''$

$d = 251.21 - \frac{(21-1)(2)}{2}$
$d = 231.21''$

$\underline{S_c = 3.5''}$

$a_1 = -6.44$
$b_1 = 399.25$
$n = 24.3$
$n \approx 25$ rows $\leq n_{max}$

$n_{max} = 35$ rows

$a = 2.94(25)(1.58)$
$a = 116.1''$

$\rho_{min,\ thermal/cracks\ control} = 0.0018$

$\rho_{s,min} = \max\begin{cases} 3\sqrt{f'_c}/f_y \\ 200/f_y \end{cases} = 0.0033$, ACI 318-11 §10.5.1 flexure members.

Max rebar spacing shall be 18'', per ACI 318-11 §7.12.2.2
$\rightarrow \rho_{3min} = 2(0.79)/18 = 0.087$, controlling min. reinf.

n_{add} = additional rows of (2) #8 to satisfy controlling min. reinf.
$n_{add} = [251.21 - 2(25-1)(3.5) - 2(1.75)]/18 - 1$
$n_{add} = 79.71/18 - 1$
$n_{add} = 4$, assumed to not contribute to strength.

$d = 251.21 - \frac{(24)(3.5)}{2}$
$d = 209.21''$
$c = a/0.85$
$\varepsilon_{s,extreme} = \frac{0.003}{136.59}(251.21 - 136.59)$
$\varepsilon_{s,extreme} = 0.00252 < 0.005$, can't use $\phi = 0.9$

$\underline{\text{Determining } A'_{s,min} \text{ to ach. } \phi = 0.9}$

$0.85 f'_c a b + A'_{s,min} \varepsilon'_s E_s = A_s f_y$
$0.85 f'_c \beta_1 b c^2 + n' A_s (0.003)(c-d') E_s = A_s f_y$
$0.85^2(3)(8)(94.2)^2 + n'(1.58)(0.003)(94.2 - 1.75n')E_s = 39.5(60)$
$153868.9 + 12948.7n' - 240.6n'^2 = 2370$
$-240.6n'^2 + 12948.7n' + 151498.9 = 0$
$n' \approx 25$ rows

$A'_{s,min} = 25(1.58)$
$A'_{s,min} = 39.5\ in^2$

$\varepsilon_s = \frac{0.003}{c}(d - c)$
$0.005c = 0.003(d_{max} - c)$
$c = \frac{3}{8} d_{max}$
$c = 0.375(251.21)$
$c = 94.2''$

$d' = \frac{(n'-1)S_c}{2} + 1.75$
$d' = 0.5S_c n' - 0.5S_c + 1.75$
$d' = 1.75n'$

→

Thaison Nguyen		Lateral Spot Check/Design	3/4

Use 25 rows of (2)#8 at each end of the flexural wall, 3.5" O.C.

*See drawing on following page to see arrangement (Rebar)

$\varepsilon_s' = \frac{0.003}{94.2}(94.2 - 1.75 \times 25)$

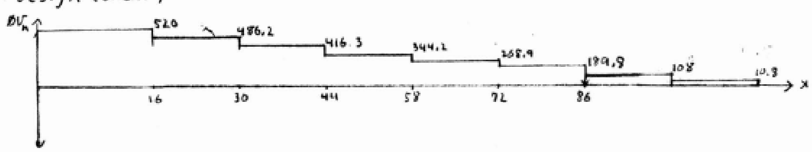
$\varepsilon_s' : 0.0016 < 0.002$, top reinf doesn't yield.

$\varepsilon_s = \frac{0.003}{94.2}(209.21 - 94.2)$

$\varepsilon_s = 0.0037 > 0.0021$, yields.

$\varepsilon_{s,extreme} = 0.005$

C. Design (Shear)



*** Assume shear reinforcements are (2)#4

$d = 209.21" = 17.43'$

$b_w = 8"$

$M_{u,ed} = 486.2$ kip

$V_c = 2\sqrt{3000}\,(8)(209.21)$

$V_c = 183.3$ kip

$V_s = \frac{486.2}{0.75} - 183.3$

$V_s = 465$ kip

$V_s \leq 8\sqrt{3000}\,(8)(209.21)$

$V_s \leq 733.3$ kip

$V_{s,spacing} = 733.3/2$

$V_{s,spacing} = 366.7 < 465$

$S_{max} = \min \begin{cases} d/4 \\ 12" \end{cases} = 12"$

$A_v = 0.2(2) = 0.4\ in^2$

$S = A_v(f_y)\,d/V_s$

$S = 0.4(60)(209.21)/(465)$

$S \approx 10"$

→

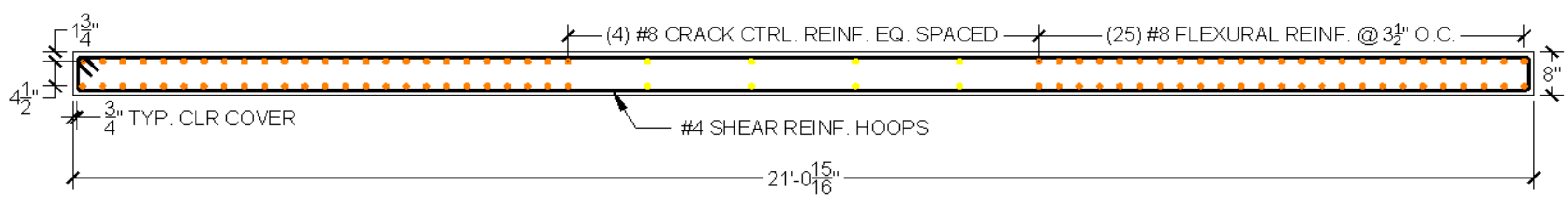


Figure AC.1, Reinforcement

Thaison Nguyen | Lateral Spot Check / Design | 4/4

AMPAD

1) Determine when $s = 12''$

$V_s = A_v(f_y)d/s$

$V_{tot,u} = (V_s + V_c)0.75$
$V_{tot,u} = [0.4(60)(209.21)/12 + 183.3]0.75$
$V_{tot,u} = 451.29$ kip , where s can equal 12''

*** Use $s = 12''$ when $x < d$, for thermal and crack control.

*See drawings on following page see shear reinf. arrangement.

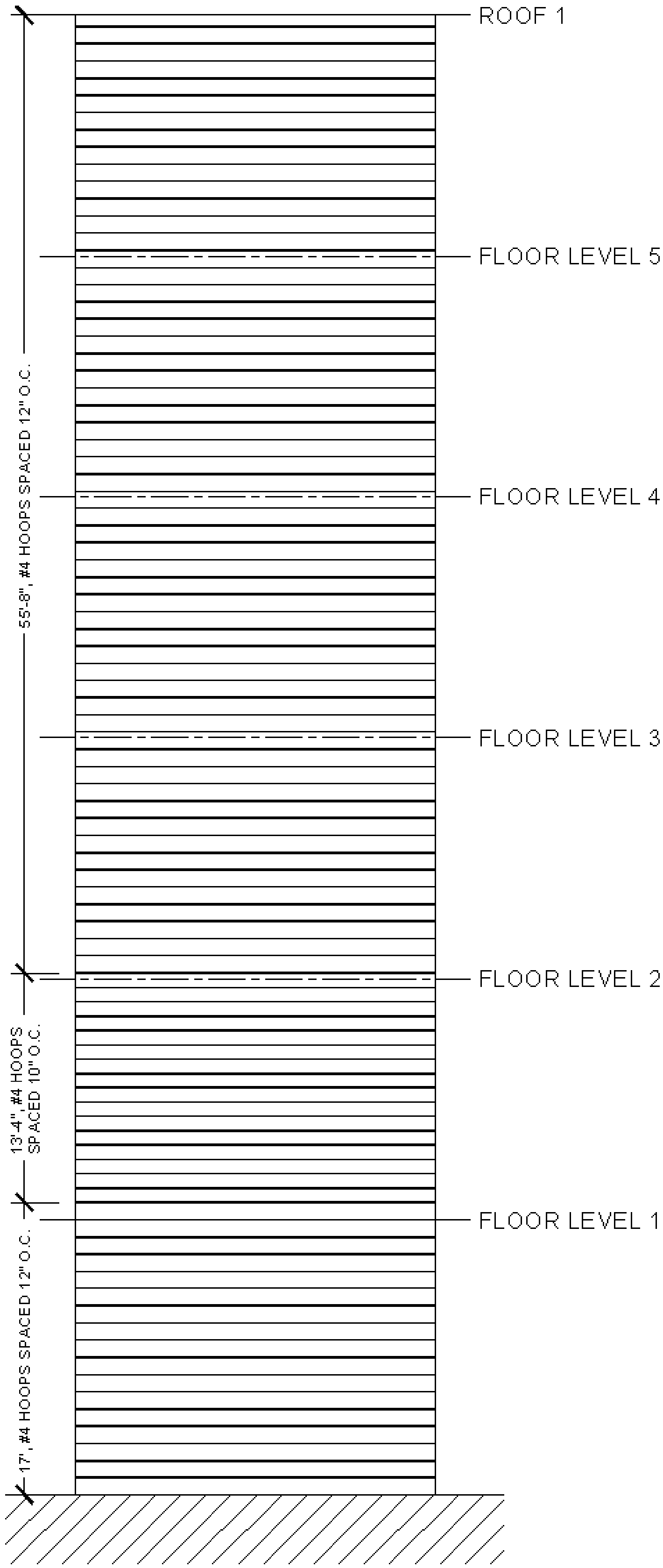


Figure AC.2, Shear Reinforcement Spacing

Appendix D: Wind Load Calculations

Thaison Nguyen | Wind Load | 1/2

Importance Category : III , ASCE 7-05 Table 1-1
Importance Factor (I) : 1.15 , ASCE 7-05 Table 6-1
Exposure Category : B , ASCE 7-05 §6.5.6.3
Mean Height : 95.5'

Building Face	North	South	East	West	Roof
A_{facade} (ft^2)	21412.9	21412.9	10957.9	10957.9	26440

*** When comparing lateral force resisting shear wall and floor diaphragm rigidity, it is assume that the floor diaphram will crack first

V = 130 mi/hr , ASCE 7-05 Figure 6-1

Component	MWFRS	CCL [1]	Notes
K_d	0.85	0.85	ASCE 7-05 Table 6-4

[1] Components and Cladding

Height (ft)	K_z		Notes
	Case I : CCL	Case II : MWFRS	
≤15	0.7	0.57	ASCE 7-05 Table 6-3
20	0.7	0.62	↓
25	0.7	0.66	↓
30	0.7	0.7	↓
40	0.76	0.76	↓
50	0.81	0.81	↓
60	0.85	0.85	↓
70	0.89	0.89	↓
80	0.93	0.93	↓
90	0.96	0.96	↓
100	0.99	0.99	↓
120	1.04	1.04	↓

$K_{zt} = 1$, no ridges or escarpments at site

$GC_{pi} = \pm 0.18$, ASCE 7-05 Figure 6-5

$a = \max \begin{cases} 0.1 * \text{Least Horizontal Dimension} \\ 3' \end{cases}$, ASCE 7-05 Figure 6-17

$a = \max \begin{cases} 0.1(117.42) \\ 3' \end{cases}$

$a = 11.74'$

Wind Perpendicular To:	North/South Wall	East/West Wall
B (ft)	229.5	117.42

→

	Windward	Leeward		Side	Roof			Notes
L/B		1.95	0.51					ASCE7-05 Fig.6-6
Distance from Windward Edge					[0,95'-6"]	[95'-6", 191'-0")	[191'-0", ∞)	ASCE7-05 Fig.6-6, §6.5.11.3
C_p, MWFRS	0.8	0.3	0.5	0.7	0.9, 0.18	0.5, 0.18	0.3, 0.18	↓

Wall Perpendicular to: North and South Facing Walls										Notes
Wall	Windward		Leeward		Side		Roof			
Zone	4	5	4	5	4	5	1	2	3	
Area (ft^2)	19170	2243	19170	2243	8715	2243	18845	5941	1654.4	
GC_p, CCL	0.6	0.6	0.7	1	0.7	1	0.9	1.6	2.3	ASCE 7-05 Fig 6-17

Wall Perpendicular to: East and West Facing Walls										Notes
Wall	Windward		Leeward		Side		Roof			
Zone	4	5	4	5	4	5	1	2	3	
Area (ft^2)	8715	2243	8715	2243	19170	2243	15545	5941	1654.4	
GC_p, CCL	0.6	0.6	0.7	1	0.7	1	0.9	1.6	2.3	ASCE 7-05 Fig 6-17

Building Natural Frequency (n_1) ~ 100 / Mean Height , ASCE C6.5.8 , Eq C6-17
Building Natural Frequency (n_1) ~ 1.047
$T_{n,\text{Lower bound}}$ = 75/(height = 86) = 0.87 sec , per ASCE 7-05 Eq C6-18
$T_{n,avg}$ = 100/(height = 86) = 1.16 sec , per ASCE 7-05 Eq C6-17

$G_f = 0.85$, conservative rigid diaphragms

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$, see excel table following this page

$q_h = 0.00256\,(0.99)(1)(0.85)(130^2)(1.15)$; leeward and side walls
$q_h = 41.9\ lb/ft^2$

$P_{MWFRS} = q G_f C_p - q_i (GC_{pi})$, $q_i = q_h$ for conservative internal pressurization.

$P_{CCL} = q(GC_p) - q_i (GC_{pi})$, $q_i = q_h$ for conservative internal pressurization.

* See excel table following this page for MWFRS and CCL Wind loads.

V_{base} = Σ Wind Load at floor diaphragm
$V_{base,\ wind \perp North/South\ Wall}$ = 916.2 Kip
$V_{base,\ wind \perp East/West\ Wall}$ = 363.3 Kip

$M_{Tot\ Overturn,\ wind \perp North/South\ Wall}$ = 47192.8 Kip-ft
$M_{Tot\ overturn,\ wind \perp East/West\ Wall}$ = 18152.9 Kip-ft

* Load distribution is on excel sheet following this page

→

Design Wind Pressures (lb/ft²)											
Height (ft)	Velocity Pressure q_z (lb/ft²)		MWFRS								
			qG_fC_p								$q_i(GC_{pi})$, Conservative
	CCL	MWFRS	Windward	Leeward		Side	Roof				
				B/L = 1.95	B/L = 0.51		Cp = 0.3	Cp = 0.5	Cp =0.9	Cp = 0.18	
≤ 15	29.6	24.1	16.39	10.68	17.79	24.91	10.68	17.79	32.03	6.41	7.54
20	29.6	26.2	17.83								
25	29.6	27.9	18.98								
30	29.6	29.6	20.13								
40	32.1	32.1	21.86								
50	34.3	34.3	23.29								
60	35.9	35.9	24.44								
70	37.6	37.6	25.59								
80	39.3	39.3	26.74								
90	40.6	40.6	27.61								
100	41.9	41.9	28.47								
120	44.0	44.0	29.91								

External Wind Forces								
Floor Level	Elevation (ft)	Mid Elevation (ft)	Wind Load on Floor Diaphram (kip)		Story Shear (kip)		Story Overturning Moment (kip-ft)	
			Wind Perpendicular to North/ South Wall	Wind Perpendicular to East/ West Wall	Wind Perpendicular to North/ South Wall	Wind Perpendicular to East/ West Wall	Wind Perpendicular to North/ South Wall	Wind Perpendicular to East/ West Wall
0	0.0	8	62.76	25.43	916.18	363.28	0.00	0.00
1	16.0	23	121.12	49.43	853.41	337.85	1937.85	790.87
2	30.0	37	124.09	51.79	732.30	288.42	3722.82	1553.67
3	44.0	51	131.29	55.47	608.20	236.63	5776.66	2440.65
4	58.0	65	137.03	58.41	476.92	181.16	7947.72	3387.61
5	72.0	79	141.78	60.84	339.89	122.76	10208.27	4380.35
Roof 1	86.0	95.5	168.50	47.45	198.10	61.92	14490.78	4080.71
Top	105.0		29.61	14.47	29.61	14.47	3108.66	1519.07

Design Wind Pressures (lb/ft²)													
Height (ft)	Velocity Pressure q_z (lb/ft²)		CCL										
			$q(GC_p)$, Wind Perpendicular to North/South Wall										$q_i(GC_{pi})$, Conservative
	CCL	MWFRS	Windward		Leeward		Side		Roof				
			Zone 4	Zone 5	Zone 4	Zone 5	Zone 4	Zone 5	Zone 1	Zone 2	Zone 3		
$\leq$ 15	29.6	24.1	17.76	17.76	29.31	41.87	29.31	41.87	37.68	66.99	96.30	7.54	
20	29.6	26.2	17.76	17.76									
25	29.6	27.9	17.76	17.76									
30	29.6	29.6	17.76	17.76									
40	32.1	32.1	19.28	19.28									
50	34.3	34.3	20.55	20.55									
60	35.9	35.9	21.57	21.57									
70	37.6	37.6	22.58	22.58									
80	39.3	39.3	23.60	23.60									
90	40.6	40.6	24.36	24.36									
100	41.9	41.9	25.12	25.12									
120	44.0	44.0	26.39	26.39									

Design Wind Pressures (lb/ft²)													
Height (ft)	Velocity Pressure q_z (lb/ft²)		CCL										
			$q(GC_p)$, Wind Perpendicular to East/West Wall										$q_i(GC_{pi})$, Conservative
	CCL	MWFRS	Windward		Leeward		Side		Roof				
			Zone 4	Zone 5	Zone 4	Zone 5	Zone 4	Zone 5	Zone 1	Zone 2	Zone 3		
$\leq$ 15	29.6	24.1	17.76	17.76	29.31	41.87	29.31	41.87	37.68	66.99	96.30	7.54	
20	29.6	26.2	17.76	17.76									
25	29.6	27.9	17.76	17.76									
30	29.6	29.6	17.76	17.76									
40	32.1	32.1	19.28	19.28									
50	34.3	34.3	20.55	20.55									
60	35.9	35.9	21.57	21.57									
70	37.6	37.6	22.58	22.58									
80	39.3	39.3	23.60	23.60									
90	40.6	40.6	24.36	24.36									
100	41.9	41.9	25.12	25.12									
120	44.0	44.0	26.39	26.39									

Appendix E: Seismic Load Calculations

Thaison Nguyen | | Seismic Loads | 1/4

Importance Category : III , ASCE 7-05 Table 1-1
Importance Factor: 1.25 , ASCE 7-05 Table 11.5-1
Site Class : D , ASCE 7-05 §11.4.2, 20.3.3 ; Table 20.3.1

*** Assume ordinary reinforced concrete shear walls ⟶ Lateral System

a) Effective Building Weight ($w_x = DL + 0.25LL$)

1) Level : 1

$DL = DL_{slab} + DL_{deck} + DL_{Bm} + DL_{Girder} + DL_{flooring} + DL_{envelope}$

$DL = 1675.5 + \frac{1.62}{1000}(A_{gross} - A_{flopening} - A_{stair}) + 217.6 + 74.8 + 7.2 + \left(\frac{1438.8 + 1183.7}{2}\right)$

$DL = 1675.5 + 39.3 + 217.6 + 74.8 + 7.2 + 1311.25$
$DL = 3325.7$ kip

$LL = 2001.7$ kip , value for Load Determination - DEAD, LIVE, RAIN section in the Appendix.

$W_x = 3325.7 + 0.25(2001.7)$
$W_x = 3826.1$ kip

$DL_{Bm} = \frac{W_{Bm}}{Spacing} * (A_{gross} - A_{flopening} - A_{stair})$

$W_{Bm} = 74$ lb/ft , W14×74 from spot check

$DL_{Bm} = \frac{74}{8.25}(26440 - 1571 - 609)$

$DL_{Bm} = 8.97(24260)$
$DL_{Bm} = 217.6$ kip

$DL_{Girder} = l_{Girder} W_{Girder}$
$W_{Girder} = 76$ lb/ft , W24×76 from spot check

$DL_{Girder} = \{[31.25(2) + 33(4) + 32\tfrac{1}{6}]3 + [29.25(2) + 33 + (33-9) + 32\tfrac{1}{6} + (33-8.5) + 33(4)]\} * 76$

$DL_{Girder} = [680 + 304.2] * 76$
$DL_{Girder} = 74.8$ kip

2) Level : 2→5

$DL = \Big\{1822.6 + \frac{1.62}{1000}(A_{gross} - A_{flopening} - A_{stair}) + 8.97(A_{gross} - A_{flopening} - A_{stair}) + \frac{76}{1000}(680 + 304.2 + 29.25 + 32\tfrac{1}{6}) + 4.1\Big\}4 + \left[\frac{1}{2}(1183.7) + 1194.2 + 1073.7 + 1061.7 + \frac{1}{2}(1761.3)\right]$

$A_{gross} - A_{flopening} - A_{stair} = 26440 - 293 - 609 = 25538\ ft^2$

$DL = 8706.5 + 4452.3$
$DL = 13158.8$ kip

$LL = 2103.9(4)$, value from Load Determination - DEAD, LIVE, RAIN section in the Appendix.
$LL = 8415.6$ kip

→

Floor Level	$DL_{envelope}$ (Kip)	DL (Kip)	W_x
2	1188.95	3365.6	3891.6
3	1133.95	3310.6	3836.6
4	1067.70	3244.4	3770.4
5	1061.50	3238.2	3764.2

3) Level : Roof

$$DL = 1794.1 + \frac{1.46(26440)}{1000} + \frac{9.2(26440)}{5.5(1000)} + \frac{76}{1000}(680 + 304.2 + 29.25 + 32\tfrac{1}{6}) + \left[\frac{1}{2}(1061.3) + 761.85\right]$$

$$LL = \frac{20}{1000} * 26440$$

$LL = 528.8$ Kip

$DL = 1956.4 + 1292.5$

$DL = 3248.9$ kip

W_x : 3381.1 Kip

4) Total Effective Weight

$W_{x,tot} = 3826.1 + 3891.6 + 3836.6 + 3770.4 + 3764.2 + 3381.1$

$W_{x,tot} = 22470$ Kip

b) Equivalent Lateral Load

1) V_{base}

$S_s = \frac{6.3}{100}$, ASCE 7-05 Fig. 22-1

$S_s = 0.063$

$S_1 = \frac{2.2}{1000}$, ASCE 7-05 Fig. 22.2

$S_1 = 0.022$

$F_a = 1.6$, ASCE 7-05 Table 11.4-1

$F_v = 2.4$, ASCE 7-05 Table 11.4-2

$S_{MS} = S_s F_a$

$S_{MS} = 0.063(1.6)$

$S_{MS} = 0.101$

$S_{m1} = S_1 F_v$

$S_{m1} = 0.022(2.4)$

$S_{m1} = 0.053$

→

Appendix F: Irregularity Analysis

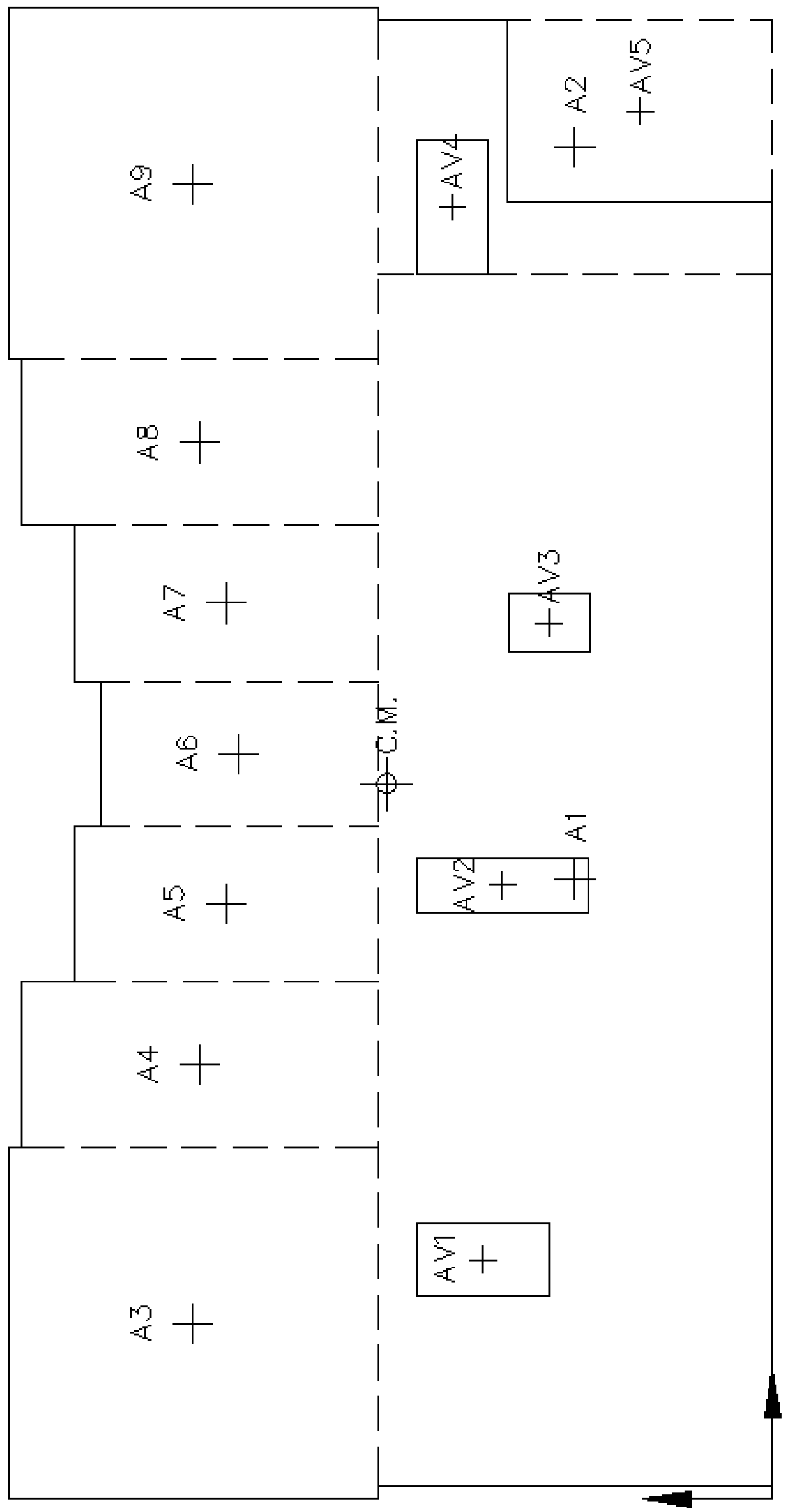


Figure AF.1, Floor Type A Area Divisions and Designations

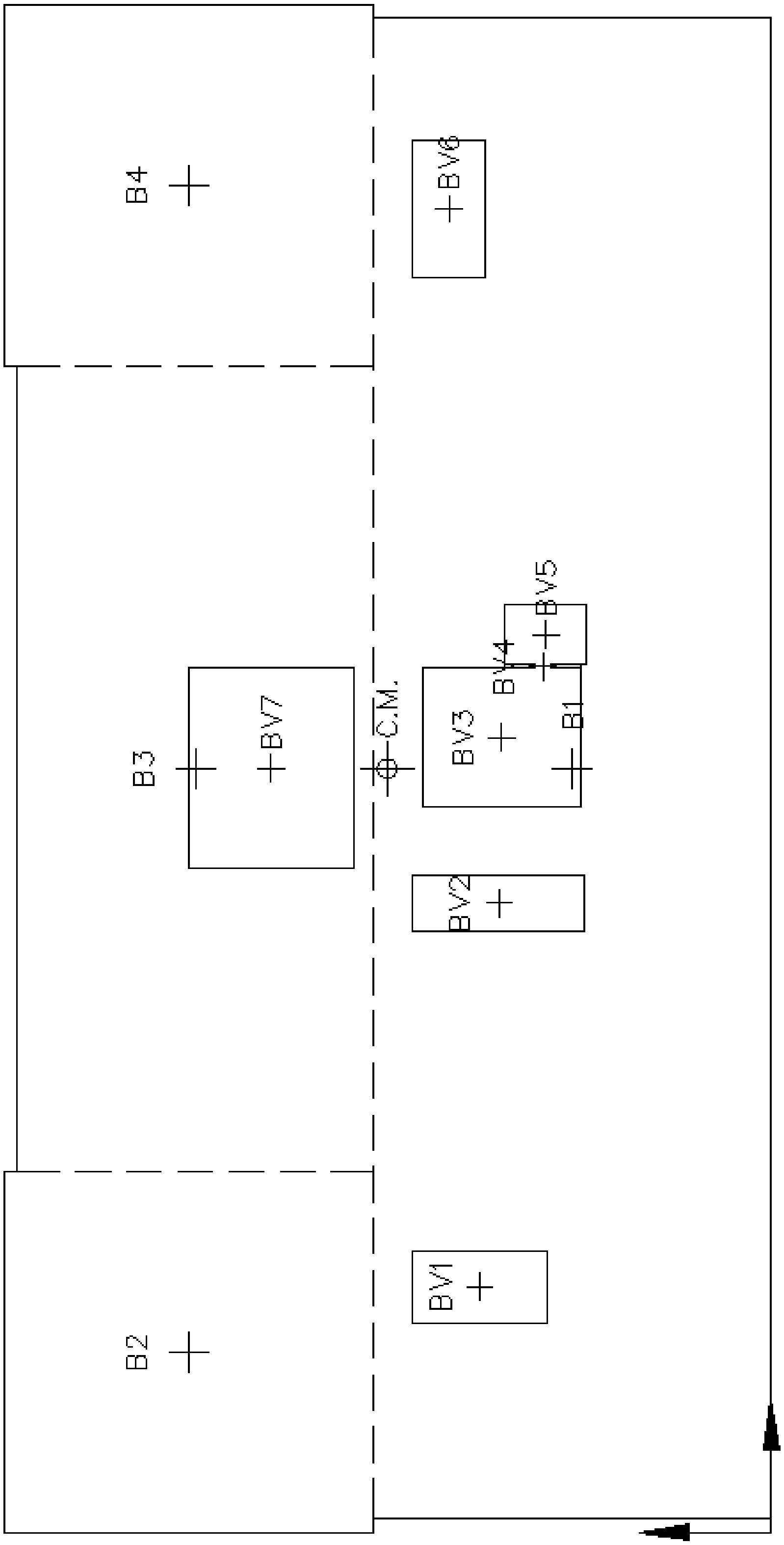


Figure AF.2, Floor Type B Area Divisions and Designations

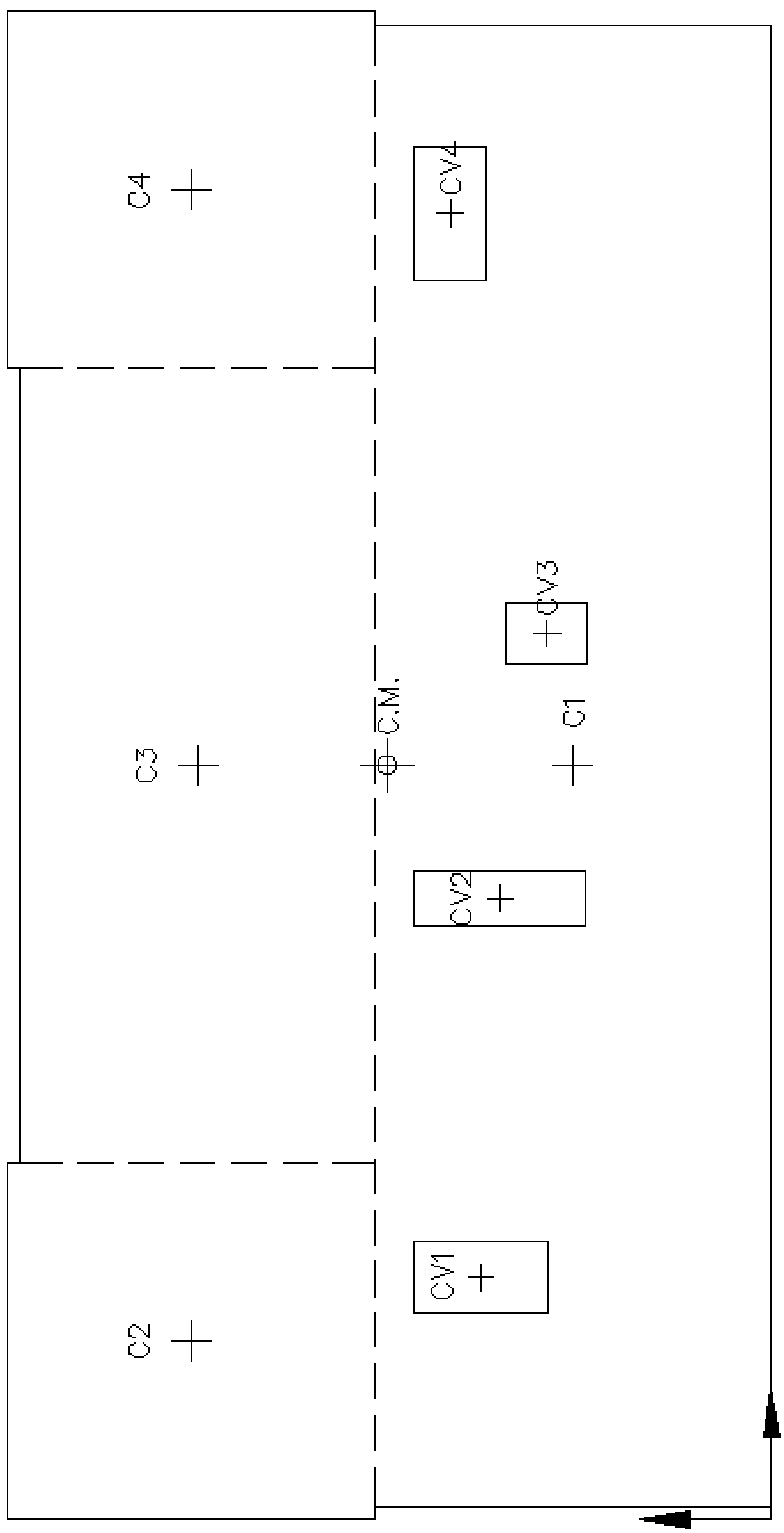


Figure AF.3, Floor Type C Area Divisions and Designations

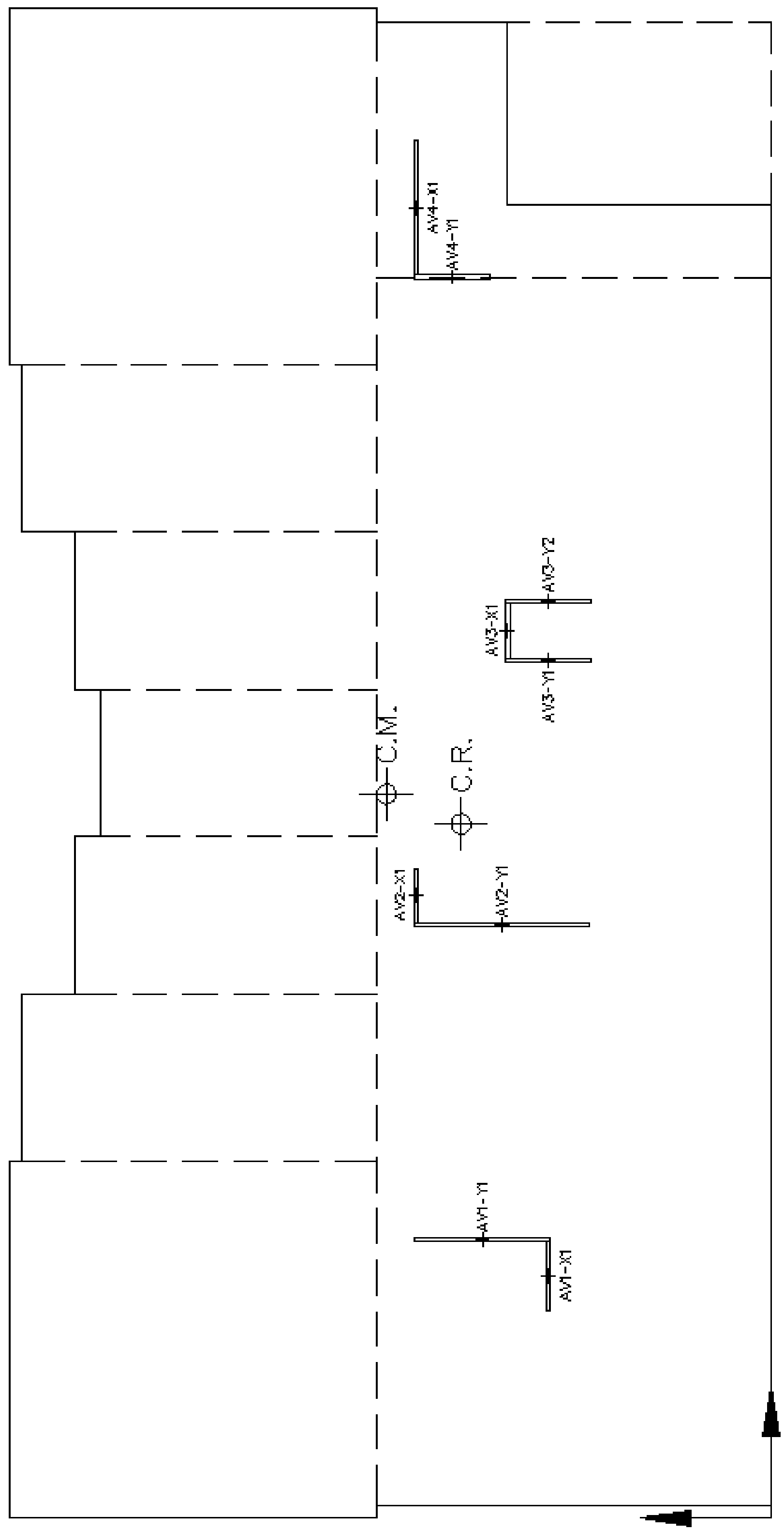


Figure AF.4, Shear Wall Designations

*** Lateral Load Resisting Structure considered to be rigid (Concrete Shear Walls)

A. Center of Mass and Rigidity

*** Assume all lateral resisting elements have a stiffness proportional to respective length.

Floor Type	Floor Level
A	0
B	1
C	2,3,4,5, Roof 1

Floor Type	Component	Area (ft^2)	Center of Mass	
			x (ft)	y (ft)
A				
	A1	11324.15	95.31	30.38
	AV1	-224.55	36.84	44.54
	AV2	-223.83	94.51	41.58
	AV3	-113.50	134.88	34.42
	AV4	-224.55	198.83	49.26
	A2	2362.09	206.07	30.38
	AV5	-1143.33	213.51	20.42
	A3	3069.82	27.09	89.09
	A4	1394.00	66.92	88.09
	A5	1115.96	91.63	84.09
	A6	949.17	114.76	82.01
	A7	1115.96	137.88	84.09
	A8	1394.00	162.58	88.09
	A9	3069.82	202.42	89.09
B				
	B1	13701.04	114.76	30.38
	BV1	-224.55	36.84	44.54
	BV2	-223.83	94.51	41.58
	BV3	-503.60	119.39	41.21
	BV4	-5.75	128.09	34.92
	BV5	-113.50	134.88	34.42
	BV6	-224.55	198.83	49.26
	B2	3069.82	27.09	89.09
	B3	6623.78	114.76	82.09
	BV7	-757.99	114.76	76.48
	B4	3069.82	202.42	89.09
C				
	C1	13701.04	114.76	30.38
	CV1	-224.55	36.84	44.54
	CV2	-223.83	94.51	41.58
	CV3	-113.50	134.88	34.42
	CV4	-224.55	198.83	49.26

→

	C2	3069.82	27.09	89.09
	C3	6623.78	114.76	88.09
	C4	3069.82	202.42	89.09

Floor Type	Global Center of Mass	
	X (ft) [1]	y (ft) [2]
A	110.07	59.34
B	114.69	58.72
C	114.79	58.90

[1] $x_{cm} = \frac{\Sigma(x_i A_i)}{\Sigma(A_i)}$

[2] $y_{cm} = \frac{\Sigma(y_i A_i)}{\Sigma(A_i)}$

Lateral Resisting Elements		Length	Element Center of Rigidity		Global Center of Rigidity	
Designation	Resisting Direction	(ft)	X (ft)	y (ft)	X_r (ft) [3]	y_r (ft) [4]
AV1-X1	x	10.33	36.84	34.33		
AV1-Y1	y	21.08	42.34	44.54		
AV2-Y1	y	27.00	90.26	41.59		
AV2-X1	x	8.17	94.68	54.76		
AV3-Y1	y	13.17	130.34	34.42	105.51	47.79
AV3-X1	x	8.41	134.88	40.67		
AV3-Y2	y	13.17	139.42	34.42		
AV4-Y1	y	16.67	188.63	49.26		
AV4-X1	x	20.41	193.17	54.76		

[3] $x_r = \frac{\Sigma(x_i L_y)}{\Sigma(L_y)}$

[4] $y_r = \frac{\Sigma(y_i L_x)}{\Sigma(L_x)}$

Floor Type	Eccentricity	
	$\|x\|$ [5]	$\|y\|$ [6]
A	4.56	11.55
B	9.18	10.93
C	9.28	11.10

[5] $|x| = |x_{cm} - x_r|$

[6] $|y| = |y_{cm} - y_r|$

→

Lateral Resisting Element	d_i [7]	
	x	y
AV1-X1	-68.67	-13.46
AV1-Y1	-63.17	-3.25
AV2-Y1	-15.25	-6.20
AV2-X1	-10.83	6.96
AV3-Y1	24.83	-13.38
AV3-X1	24.37	-7.13
AV3-Y2	33.91	-13.38
AV4-Y1	83.12	1.46
AV4-X1	93.66	6.96

[7] d_i = Element Center of Rigidity − Global Center of Rigidity

$$\Sigma(K_{x,i}\, d_{x,i}^2 + K_{y,i}\, d_{y,i}^2) = 197931 \quad , \quad K = L$$

Lateral Resisting Elements	$K_i \lvert d_i \rvert d_x / [\Sigma(K_{x,i} d_{x,i}^2 + K_{y,i} d_{y,i}^2)]$					
	Floor Type: A		Floor Type: B		Floor Type: C	
	x	y	x	y	x	y
AV1-X1	0.0032	0.0081	0.0065	0.0077	0.0065	0.0078
AV1-X1	0.0031	0.078	0.062	0.074	0.062	0.075
AV2-Y1	0.0095	0.024	0.019	0.023	0.019	0.023
AV2-X1	0.0013	0.0033	0.0026	0.0031	0.0027	0.0032
AV3-Y1	0.0075	0.0191	0.0152	0.0181	0.0153	0.0183
AV3-X1	0.0014	0.0035	0.0028	0.0033	0.0028	0.0034
AV3-Y2	0.0010	0.026	0.021	0.025	0.021	0.025
AV4-Y1	0.022	0.057	0.045	0.054	0.045	0.054
AV4-X1	0.0033	0.0083	0.0066	0.0078	0.0067	0.0080

d_x = eccentricity between Center of Mass and Center of Rigidity.

B. Irregularity and Wind

$e_{acc,long} = 0.15(229.51) = 34.43'$, per ASCE7-05; Figure 6-9 Case II
$e_{acc,short} = 0.15(117.42) = 17.61'$

Floor Level	Wind ⊥ to Long side			Wind ⊥ to Short side		
	Case I	Case II, III	Case IV	Case I	Case II, III	Case IV
0	62.76	47.07	35.30	25.43	19.07	14.30
1	121.12	90.84	68.13	49.43	37.07	27.80
2	124.09	93.07	69.80	51.79	38.84	29.13
3	131.29	98.47	73.85	55.47	41.60	31.20
4	137.03	102.77	77.08	58.41	43.81	32.86
5	141.78	106.34	79.75	60.84	45.63	34.22
Roof 1	168.50	126.38	94.78	47.45	35.59	26.69
Top	29.61	22.21	16.66	14.47	10.85	8.14

→

Thaison Nguyen		Irregularity Analysis	4/7

Floor Level	$M_{acc,torsion}$, Kip-ft [8]			
	Wind ⊥ to Long Side		Wind ⊥ to Short Side	
	Case II	Case IV	Case II	Case IV
0	1215.32	911.49	251.95	188.96
1	2345.43	1759.07	489.73	367.29
2	2402.94	1802.21	513.11	384.83
3	2542.37	1906.77	549.57	412.18
4	2653.52	1990.14	578.70	434.02
5	2745.50	2059.12	602.77	452.08
Roof 1	3262.92	2447.19	490.11	352.58
Top	573.38	430.04	143.36	107.52

[8] $|M_{acc,torsion}| = |\text{Wind Load} * e_{acc}|$

* See excel on following page for the load on each lateral resisting element per case.

Maximum Wind Base Shear is in element AV1-Y1
Maximum Wind Base Shear = 325 Kips, element AV1-Y1
Maximum Overturning moment on Element = 16608 kip*ft

1) Story Drift

*** Assume concrete remains elastic, for drift calculations.
*** Deflection calculations don't consider creep or other long term effects
*** Assume only 25% of I_g is effective

Lateral Resisting Member	$P_{max-w,level}$ (kip)							
	0 [9]	1	2	3	4	5	Roof 1	Top
AV1-X1	5.35	10.41	10.90	11.68	12.30	12.81	9.99	3.05
AV1-Y1	21.15	43.53	44.76	47.36	49.43	51.15	60.79	9.10
AV2-Y1	20.28	40.31	41.32	43.72	45.63	47.21	56.11	9.86
AV2-X1	4.47	8.69	9.10	9.75	10.27	10.69	8.34	2.54
AV3-Y1	9.13	16.69	17.08	18.07	18.86	19.51	23.19	4.08
AV3-X1	4.43	8.62	9.03	9.67	10.19	10.61	8.27	2.52
AV3-Y2	8.95	16.02	16.38	17.33	18.09	18.72	22.25	3.91
AV4-Y1	7.10	10.92	11.18	11.82	12.34	12.77	15.18	2.67
AV4-X1	11.18	21.71	22.75	24.37	25.66	26.73	20.84	6.36

[9] Lateral load goes to ground.

$\Delta_{drift} = \Delta_{i+1} - \Delta_i$
$\Delta_{drift} = (P_{i+1} l_{i+1}^3 - P_i l_i^3)/C$
$\Delta_{drift} = l^3 (P_{i+1} - P_i)/C$, when $l_{i+1} = l_i$

$\Delta = P l^3 / C$
C = Constant = $12EI$, fixed ends.
$E = 57000\sqrt{f'_c}$

→

Wind Perpendicular to Long Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	15.37	19.69	0.00	9.60	0.00	9.60	8.51	0.00
1	0.00	29.66	37.99	0.00	18.53	0.00	18.53	16.42	0.00
2	0.00	30.39	38.92	0.00	18.98	0.00	18.98	16.82	0.00
3	0.00	32.15	41.18	0.00	20.08	0.00	20.08	17.79	0.00
4	0.00	33.55	42.98	0.00	20.96	0.00	20.96	18.57	0.00
5	0.00	34.72	44.47	0.00	21.69	0.00	21.69	19.22	0.00
Roof 1	0.00	41.26	52.85	0.00	25.77	0.00	25.77	22.84	0.00
Top	0.00	7.25	9.29	0.00	4.53	0.00	4.53	4.01	0.00

Wind Perpendicular to Short Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	5.55	0.00	0.00	4.39	0.00	4.52	0.00	0.00	10.97
1	10.79	0.00	0.00	8.53	0.00	8.79	0.00	0.00	21.32
2	11.31	0.00	0.00	8.94	0.00	9.21	0.00	0.00	22.34
3	12.11	0.00	0.00	9.57	0.00	9.86	0.00	0.00	23.93
4	12.75	0.00	0.00	10.08	0.00	10.38	0.00	0.00	25.19
5	13.28	0.00	0.00	10.50	0.00	10.81	0.00	0.00	26.24
Roof 1	10.36	0.00	0.00	8.19	0.00	8.43	0.00	0.00	20.47
Top	3.16	0.00	0.00	2.50	0.00	2.57	0.00	0.00	6.24

Wind Perpendicular to Long Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.20	1.93	0.60	0.08	-0.47	-0.09	-0.65	-1.40	0.21
1	-0.78	7.48	2.31	0.32	-1.84	-0.34	-2.51	-5.45	0.80
2	-0.81	7.75	2.40	0.33	-1.90	-0.35	-2.60	-5.64	0.83
3	-0.86	8.20	2.53	0.35	-2.01	-0.37	-2.75	-5.97	0.88
4	-0.89	8.56	2.65	0.37	-2.10	-0.39	-2.87	-6.23	0.91
5	-0.92	8.85	2.74	0.38	-2.17	-0.40	-2.97	-6.45	0.94
Roof 1	-1.10	10.52	3.25	0.45	-2.58	-0.47	-3.53	-7.66	1.12
Top	-0.19	1.85	0.57	0.08	-0.45	-0.08	-0.62	-1.35	0.20

Wind Perpendicular to Short Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.21	1.98	0.61	0.08	-0.49	-0.09	-0.66	-1.44	0.21
1	-0.38	3.63	1.12	0.16	-0.89	-0.16	-1.22	-2.65	0.39
2	-0.40	3.87	1.20	0.17	-0.95	-0.17	-1.30	-2.82	0.41
3	-0.43	4.14	1.28	0.18	-1.02	-0.19	-1.39	-3.02	0.44
4	-0.46	4.36	1.35	0.19	-1.07	-0.20	-1.46	-3.18	0.47
5	-0.47	4.54	1.41	0.19	-1.12	-0.20	-1.52	-3.31	0.49
Roof 1	-0.37	3.54	1.10	0.15	-0.87	-0.16	-1.19	-2.58	0.38
Top	-0.11	1.08	0.33	0.05	-0.27	-0.05	-0.36	-0.79	0.12

Wind Perpendicular to Long Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.20	17.29	20.28	0.08	9.13	-0.09	8.95	7.10	0.21
1	-0.78	37.14	40.31	0.32	16.69	-0.34	16.02	10.97	0.80
2	-0.81	38.13	41.32	0.33	17.08	-0.35	16.38	11.18	0.83
3	-0.86	40.35	43.72	0.35	18.07	-0.37	17.33	11.82	0.88
4	-0.89	42.11	45.63	0.37	18.86	-0.39	18.09	12.34	0.91
5	-0.92	43.57	47.21	0.38	19.51	-0.40	18.72	12.77	0.94
Roof 1	-1.10	51.78	56.11	0.45	23.19	-0.47	22.25	15.18	1.12
Top	-0.19	9.10	9.86	0.08	4.08	-0.08	3.91	2.67	0.20

Wind Perpendicular to Short Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	5.35	1.98	0.61	4.47	-0.49	4.43	-0.66	-1.44	11.18
1	10.41	3.63	1.12	8.69	-0.89	8.62	-1.22	-2.65	21.71
2	10.90	3.87	1.20	9.10	-0.95	9.03	-1.30	-2.82	22.75
3	11.68	4.14	1.28	9.75	-1.02	9.67	-1.39	-3.02	24.37
4	12.30	4.36	1.35	10.27	-1.07	10.19	-1.46	-3.18	25.66
5	12.81	4.54	1.41	10.69	-1.12	10.61	-1.52	-3.31	26.73
Roof 1	9.99	3.54	1.10	8.34	-0.87	8.27	-1.19	-2.58	20.84
Top	3.05	1.08	0.33	2.54	-0.27	2.52	-0.36	-0.79	6.36

Table AF.1, Wind Case I

Floor Level	Wind Perpendicular to Long Side — Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	11.53	14.76	0.00	7.20	0.00	7.20	6.38	0.00
1	0.00	22.24	28.49	0.00	13.90	0.00	13.90	12.31	0.00
2	0.00	22.79	29.19	0.00	14.24	0.00	14.24	12.61	0.00
3	0.00	24.11	30.89	0.00	15.06	0.00	15.06	13.35	0.00
4	0.00	25.17	32.24	0.00	15.72	0.00	15.72	13.93	0.00
5	0.00	26.04	33.35	0.00	16.27	0.00	16.27	14.41	0.00
Roof 1	0.00	30.95	39.64	0.00	19.33	0.00	19.33	17.13	0.00
Top	0.00	5.44	6.97	0.00	3.40	0.00	3.40	3.01	0.00

Floor Level	Wind Perpendicular to Short Side — Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	4.16	0.00	0.00	3.29	0.00	3.39	0.00	0.00	8.23
1	8.10	0.00	0.00	6.40	0.00	6.59	0.00	0.00	15.99
2	8.48	0.00	0.00	6.70	0.00	6.90	0.00	0.00	16.75
3	9.08	0.00	0.00	7.18	0.00	7.39	0.00	0.00	17.94
4	9.57	0.00	0.00	7.56	0.00	7.79	0.00	0.00	18.90
5	9.96	0.00	0.00	7.87	0.00	8.11	0.00	0.00	19.68
Roof 1	7.77	0.00	0.00	6.14	0.00	6.33	0.00	0.00	15.35
Top	2.37	0.00	0.00	1.87	0.00	1.93	0.00	0.00	4.68

Floor Level	Wind Perpendicular to Long Side — Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.15	1.44	0.45	0.06	-0.35	-0.07	-0.48	-1.05	0.15
1	-0.59	5.61	1.74	0.24	-1.38	-0.25	-1.88	-4.09	0.60
2	-0.61	5.81	1.80	0.25	-1.43	-0.26	-1.95	-4.23	0.62
3	-0.64	6.15	1.90	0.26	-1.51	-0.28	-2.06	-4.48	0.66
4	-0.67	6.42	1.98	0.27	-1.58	-0.29	-2.15	-4.67	0.68
5	-0.69	6.64	2.05	0.28	-1.63	-0.30	-2.23	-4.83	0.71
Roof 1	-0.82	7.89	2.44	0.34	-1.94	-0.36	-2.65	-5.75	0.84
Top	-0.14	1.39	0.43	0.06	-0.34	-0.06	-0.46	-1.01	0.15

Floor Level	Wind Perpendicular to Short Side — Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.15	1.48	0.46	0.06	-0.36	-0.07	-0.50	-1.08	0.16
1	-0.28	2.73	0.84	0.12	-0.67	-0.12	-0.91	-1.99	0.29
2	-0.30	2.90	0.90	0.12	-0.71	-0.13	-0.97	-2.11	0.31
3	-0.32	3.11	0.96	0.13	-0.76	-0.14	-1.04	-2.26	0.33
4	-0.34	3.27	1.01	0.14	-0.80	-0.15	-1.10	-2.38	0.35
5	-0.36	3.41	1.05	0.15	-0.84	-0.15	-1.14	-2.48	0.36
Roof 1	-0.28	2.66	0.82	0.11	-0.65	-0.12	-0.89	-1.94	0.28
Top	-0.08	0.81	0.25	0.03	-0.20	-0.04	-0.27	-0.59	0.09

Floor Level	Wind Perpendicular to Long and Short Side — Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	3.86	14.45	15.67	3.42	6.48	3.26	6.22	4.25	8.54
1	7.22	30.58	31.07	6.75	11.85	6.21	11.10	6.24	16.88
2	7.57	31.50	31.89	7.08	12.10	6.51	11.31	6.27	17.68
3	8.12	33.37	33.75	7.57	12.79	6.98	11.96	6.61	18.93
4	8.55	34.86	35.23	7.97	13.34	7.35	12.47	6.87	19.93
5	8.91	36.09	36.46	8.30	13.80	7.66	12.90	7.09	20.75
Roof 1	6.67	41.49	42.90	6.59	16.74	5.85	15.79	9.45	16.48
Top	2.14	7.64	7.65	1.97	2.86	1.83	2.66	1.41	4.92

Table AF.2, Wind Case III

Floor Level	Wind Perpendicular to Long Side — Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	11.53	14.76	0.00	7.20	0.00	7.20	6.38	0.00
1	0.00	22.24	28.49	0.00	13.90	0.00	13.90	12.31	0.00
2	0.00	22.79	29.19	0.00	14.24	0.00	14.24	12.61	0.00
3	0.00	24.11	30.89	0.00	15.06	0.00	15.06	13.35	0.00
4	0.00	25.17	32.24	0.00	15.72	0.00	15.72	13.93	0.00
5	0.00	26.04	33.35	0.00	16.27	0.00	16.27	14.41	0.00
Roof 1	0.00	30.95	39.64	0.00	19.33	0.00	19.33	17.13	0.00
Top	0.00	5.44	6.97	0.00	3.40	0.00	3.40	3.01	0.00

Floor Level	Wind Perpendicular to Long Side — Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-1.00	9.62	2.97	0.41	-2.36	-0.43	-3.23	-7.01	1.03
1	-2.23	21.39	6.61	0.91	-5.25	-0.96	-7.17	-15.58	2.28
2	-2.30	21.98	6.79	0.94	-5.40	-0.99	-7.37	-16.00	2.35
3	-2.43	23.25	7.19	0.99	-5.71	-1.05	-7.80	-16.93	2.48
4	-2.53	24.27	7.50	1.04	-5.96	-1.09	-8.14	-17.67	2.59
5	-2.62	25.11	7.76	1.07	-6.17	-1.13	-8.42	-18.29	2.68
Roof 1	-3.12	29.84	9.23	1.27	-7.33	-1.34	-10.01	-21.73	3.19
Top	-0.55	1.29	0.43	0.06	-0.34	-0.06	-0.46	-1.01	0.15

Floor Level	Wind Perpendicular to Long Side — Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-1.00	21.15	17.74	0.41	4.84	-0.43	3.97	-0.63	1.03
1	-2.23	43.63	35.11	0.91	8.64	-0.96	6.72	-3.27	2.28
2	-2.30	44.76	35.99	0.94	8.84	-0.99	6.87	-3.39	2.35
3	-2.43	47.36	38.07	0.99	9.35	-1.05	7.26	-3.59	2.48
4	-2.53	49.43	39.74	1.04	9.76	-1.09	7.58	-3.74	2.59
5	-2.62	51.15	41.12	1.07	10.10	-1.13	7.84	-3.87	2.68
Roof 1	-3.12	60.79	48.87	1.27	12.00	-1.34	9.32	-4.60	3.19
Top	-0.55	6.72	7.39	0.06	3.06	-0.06	2.93	2.00	0.15

Floor Level	Wind Perpendicular to Short Side — Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	4.16	0.00	0.00	3.29	0.00	3.39	0.00	0.00	8.23
1	8.10	0.00	0.00	6.40	0.00	6.59	0.00	0.00	15.99
2	8.48	0.00	0.00	6.70	0.00	6.90	0.00	0.00	16.75
3	9.08	0.00	0.00	7.18	0.00	7.39	0.00	0.00	17.94
4	9.57	0.00	0.00	7.56	0.00	7.79	0.00	0.00	18.90
5	9.96	0.00	0.00	7.87	0.00	8.11	0.00	0.00	19.68
Roof 1	7.77	0.00	0.00	6.14	0.00	6.33	0.00	0.00	15.35
Top	2.37	0.00	0.00	1.87	0.00	1.93	0.00	0.00	4.68

Floor Level	Wind Perpendicular to Short Side — Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.33	3.18	0.98	0.14	-0.78	-0.14	-1.07	-2.31	0.34
1	-0.63	6.02	1.86	0.26	-1.48	-0.27	-2.02	-4.38	0.64
2	-0.66	6.35	1.96	0.27	-1.56	-0.29	-2.13	-4.63	0.68
3	-0.71	6.80	2.10	0.29	-1.67	-0.31	-2.28	-4.96	0.73
4	-0.75	7.17	2.22	0.31	-1.76	-0.32	-2.40	-5.22	0.76
5	-0.78	7.46	2.31	0.32	-1.83	-0.34	-2.50	-5.44	0.80
Roof 1	-0.61	5.82	1.80	0.25	-1.43	-0.26	-1.95	-4.24	0.62
Top	-0.19	0.81	0.25	0.03	-0.20	-0.04	-0.27	-0.59	0.09

Floor Level	Wind Perpendicular to Short Side — Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	3.83	3.18	0.98	3.43	-0.78	3.25	-1.07	-2.31	8.57
1	7.47	6.02	1.86	6.65	-1.48	6.32	-2.02	-4.38	16.63
2	7.82	6.35	1.96	6.97	-1.56	6.62	-2.13	-4.63	17.43
3	8.37	6.80	2.10	7.47	-1.67	7.09	-2.28	-4.96	18.67
4	8.82	7.17	2.22	7.87	-1.76	7.46	-2.40	-5.22	19.66
5	9.18	7.46	2.31	8.19	-1.83	7.77	-2.50	-5.44	20.48
Roof 1	7.16	5.82	1.80	6.39	-1.43	6.06	-1.95	-4.24	15.97
Top	2.18	0.81	0.25	1.91	-0.20	1.89	-0.27	-0.59	4.77

Table AF.3, Wind Case II

Wind Perpendicular to Long Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	8.64	11.07	0.00	5.40	0.00	5.40	4.78	0.00
1	0.00	16.68	21.37	0.00	10.42	0.00	10.42	9.23	0.00
2	0.00	17.09	21.89	0.00	10.68	0.00	10.68	9.46	0.00
3	0.00	18.08	23.16	0.00	11.30	0.00	11.30	10.01	0.00
4	0.00	18.87	24.18	0.00	11.79	0.00	11.79	10.45	0.00
5	0.00	19.53	25.02	0.00	12.20	0.00	12.20	10.81	0.00
Roof 1	0.00	23.21	29.73	0.00	14.50	0.00	14.50	12.85	0.00
Top	0.00	4.08	5.22	0.00	2.55	0.00	2.55	2.26	0.00

Wind Perpendicular to Short Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	3.12	0.00	0.00	2.47	0.00	2.54	0.00	0.00	6.17
1	6.07	0.00	0.00	4.80	0.00	4.94	0.00	0.00	11.99
2	6.36	0.00	0.00	5.03	0.00	5.18	0.00	0.00	12.57
3	6.81	0.00	0.00	5.38	0.00	5.55	0.00	0.00	13.46
4	7.17	0.00	0.00	5.67	0.00	5.84	0.00	0.00	14.17
5	7.47	0.00	0.00	5.91	0.00	6.08	0.00	0.00	14.76
Roof 1	5.83	0.00	0.00	4.61	0.00	4.74	0.00	0.00	11.51
Top	1.78	0.00	0.00	1.40	0.00	1.45	0.00	0.00	3.51

Wind Perpendicular to Long Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.75	7.22	2.23	0.31	-1.77	-0.32	-2.42	-5.25	0.77
1	-1.68	16.04	4.96	0.69	-3.94	-0.72	-5.38	-11.68	1.71
2	-1.72	16.48	5.10	0.70	-4.05	-0.74	-5.53	-12.00	1.76
3	-1.82	17.44	5.39	0.74	-4.28	-0.78	-5.85	-12.70	1.86
4	-1.90	18.20	5.63	0.78	-4.47	-0.82	-6.10	-13.25	1.94
5	-1.97	18.83	5.82	0.80	-4.62	-0.85	-6.32	-13.71	2.01
Roof 1	-2.34	22.38	6.92	0.96	-5.50	-1.01	-7.51	-16.30	2.39
Top	-0.41	0.96	0.32	0.04	-0.26	-0.05	-0.35	-0.76	0.11

Wind Perpendicular to Short Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.25	2.38	0.74	0.10	-0.58	-0.11	-0.80	-1.74	0.25
1	-0.47	4.52	1.40	0.19	-1.11	-0.20	-1.51	-3.29	0.48
2	-0.50	4.76	1.47	0.20	-1.17	-0.21	-1.60	-3.47	0.51
3	-0.53	5.10	1.58	0.22	-1.25	-0.23	-1.71	-3.72	0.54
4	-0.56	5.37	1.66	0.23	-1.32	-0.24	-1.80	-3.91	0.57
5	-0.58	5.60	1.73	0.24	-1.37	-0.25	-1.88	-4.08	0.60
Roof 1	-0.46	4.37	1.35	0.19	-1.07	-0.20	-1.46	-3.18	0.47
Top	-0.14	0.61	0.19	0.03	-0.15	-0.03	-0.20	-0.44	0.06

Wind Perpendicular to Long Side and Short									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	2.12	18.24	14.04	2.88	3.04	2.11	2.18	-2.20	7.19
1	3.92	37.24	27.73	5.68	5.37	4.02	3.53	-5.74	14.19
2	4.14	38.34	28.46	5.93	5.46	4.22	3.55	-6.01	14.83
3	4.46	40.63	30.13	6.35	5.76	4.53	3.74	-6.41	15.86
4	4.71	42.45	31.47	6.68	6.00	4.78	3.88	-6.72	16.69
5	4.92	43.96	32.57	6.95	6.20	4.98	4.01	-6.98	17.37
Roof 1	3.03	49.95	38.00	5.75	7.93	3.54	5.53	-6.63	14.37
Top	1.23	5.65	5.73	1.48	2.14	1.37	2.00	1.06	3.69

Table AF.4, Wind Case IV

Maximum Wind Base Shear in Lateral Resisting Elements (Kip)								
AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
76.49	325.00	304.42	63.85	126.60	63.35	121.65	84.03	159.59

Maximum Wind Base Shear in Lateral Resisting Elements (Kip/ft Length)								
AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
7.40	15.42	11.27	7.82	9.62	7.53	9.24	7.20	7.82

Maximum Overturning Moment Shear in Lateral Resisting Elements (Kip)								
AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
3822.4	16608.2	15713.7	3190.7	6495.6	3165.7	6231.1	4251.5	7974.7

Table AF.5, Maximum Element Base Shear and Overturning Moment

Lateral Resisting Member	$\Delta P_{mag-w,level}$ (Kip)						
	0,1	1,2	2,3	3,4	4,5	5, Roof1	Roof1, Top
AV1-X1	10.41	0.49	0.77	0.62	0.51	2.52	6.94
AV1-Y1	43.63	1.13	2.80	2.07	1.71	9.64	51.69
AV2-Y1	40.31	1.01	2.40	1.91	1.58	8.90	46.25
AV2-X1	8.69	0.42	0.65	0.52	0.43	2.35	5.80
AV3-Y1	16.69	0.34	0.99	0.79	0.65	3.68	19.12
AV3-X1	8.62	0.41	0.64	0.51	0.42	2.33	5.75
AV3-Y2	16.02	0.37	0.95	0.76	0.63	3.53	18.34
AV4-Y1	10.99	0.21	0.65	0.52	0.43	2.41	12.51
AV4-X1	21.71	1.04	1.62	1.29	1.07	5.88	14.49

$I_{cr,xx} = t_{eff}\, d_{eff}^{3}/12$

$t_{eff} = 8 - (0.75 + 0.5 + 1/2)2$
$t_{eff} = 4.5''$

$d_{eff} = d_{tot} - 2(0.75 + 0.5 + 1/2)$
$d_{eff} = d_{tot} - 3.5''$

* See excel calculations (Story drift) on following page

Drift Limit = H/400, per AISC 7-05 Section CC.1.2
Drift Limit(h=14') = 14(12)/400 = 0.42"
Drift Limit(h=16') = 16(12)/400 = 0.48"

AMPAD

Lateral Resisting Member	Thickness (in)	Depth (in)	$I_{conc, xx}$ (in^4) [2]	I_{cr} / I_{gross}	$\Delta P_{max\text{-}w, level}$ (Kip)		$\Delta_{drift, story}$ (in) [2] [3]	
					0, 1	1, . . ., Top	0, 1	1, . . ., Top
AV1-X1	8	124.00	317771	0.25	10.41	6.94	0.0062	0.0028
AV1-Y1	8	252.94	2697046	0.25	43.63	51.69	0.0031	0.0024
AV2-Y1	8	324.00	5668704	0.25	40.31	46.25	0.0013	0.0010
AV2-X1	8	98.00	156865	0.25	8.69	5.80	0.010	0.0047
AV3-Y1	8	158.00	657385	0.25	16.69	19.12	0.0048	0.0037
AV3-X1	8	100.94	171398	0.25	8.62	5.75	0.0095	0.0042
AV3-Y2	8	158.00	657385	0.25	16.02	18.34	0.0046	0.0035
AV4-Y1	8	140.00	457333	0.25	10.97	12.51	0.0045	0.0035
AV4-X1	8	244.94	2449146	0.25	21.71	14.49	0.0017	0.00075

C. Irregularity and Seismic.

1) Check for Re-entrant Corner, diaphragm discontinuity irreg.

Floor Type	Dimensions w/o Re-entrant Corners		Re-entrant Corner Dimensions		Gross Area	Void Area
	x	y	x	y	(ft^2)	(ft^2)
A	197.51	73.59	28.00	40.83	25785.62	1929.76
B	225.51	115.43	2	2	26440	2053.77
C	225.51	115.43	2	2	26440	786.43

Floor Type	Re-entrant Corner Extension Percentage		Void Percentage
	x	y	
A	28/197.51 * 100 = 14.2%	40.83/73.59 * 100 = 55.5%	1929.76/25785.62 * 100 = 7.5%
B	2/225.51 * 100 = 0.9%	2/115.43 * 100 = 1.7%	2053.77/26440 * 100 = 7.8%
C	2/225.51 * 100 = 0.9%	2/115.43 * 100 = 1.7%	786.43/26440 * 100 = 3.0%

Floor Type A doesn't have re-entrant corner irregularity because only the y-direction has an extension ≥ 15%

All floor types don't have diaphragm discontinuity irregularity since the voids are ≤ 50% of the gross floor diaphragm area.

*** Due to continuity of lateral force resisting elements there are no out-of-plane offset irreg.

*** Non parallel Systems irreg doesn't exist

Increase in forces due to horizontal forces is not required for SDC A, B, C per ASCE 7-05 §12.3.3.4

2) Check Soft Story and Weight Irregularity.

$K \sim 1/L^3$

Story	Height (ft)	K
1	16	0.00024
2	14	0.00036
3	14	0.00036
4	14	0.00036
5	14	0.00036
6	14	0.00036

→

Storys	K_{avg}
1, 2, 3	$[0.00024 + 0.00036(2)]/3 = 0.00032$
2, 3, 4	0.00036
3, 4, 5	0.00036
4, 5, 6	0.00036

Story	K_i / K_{i+1} (%)	$W_{eff,j} / W_{eff,i}$ (%) [10]
1	$0.00024/0.00036 \times 100 = 67\%$	101.7%
2	100%	101.4%
3	100%	101.8%
4	100%	100.2%
5	100%	
6		

[10] Using effective floor weight determined in seismic analysis.

Only the first story exper. soft story irreg. due to $K_i / K_{i+1} \leq 80\%$.

Extreme soft story irreg. doesn't exist in any story, since $K_i / K_{i+1} > 70\%$

Weight irreg. doesn't exist because $W_{eff,j} / W_{eff,i} \leq 150\%$

In force increase or modifications are required, per ASCE7-05 Table 12.3-2, § 12.3.3.4

Drift Limit Factor = $0.015L$, per ASCE 7-05 Table 12.12-1

→

Appendix G: Structural Computer Modeling

G.1 General Modeling Assumptions and Input

1. All concrete lateral force resisting elements act as if they're are monolithically cast
2. Effective concrete cross-sections is 35% of gross cross-sectional area
3. Rigid panel zone factor 1.0
4. Considered P-Δ effects for drift analysis
5. Seismic importance factor is 1.25
6. All pin connections are perfectly frictionless
7. Non lateral force resisting elements carry no lateral load to the ground
8. Beam end offsets to the pier face
9. Floor diaphragms are considered rigid
10. MEP openings are ignored
11. All material weights are zero
12. Use ACI 318-08 and occasionally ACI 318-05 design criteria

Each assumption and input used in the computer modeling was made to simplify modeling the LMOB's structural behavior whilst keeping an eye out for modeling accuracy. Modeling LMOB's structural behavior in a simple manner reduces the structural modeling software's computational time, as well as computer hardware memory. The first assumption was made to ensure that all beams and columns in the lateral force resisting elements will wholly transfer moments. Intrinsically concrete cracks – to compensate for this behavior – the gross concrete cross section was only assumed to be 35% effective. The result is the application of a 0.35 modification factor to all gross concrete cross section. Let it be clear that the value 35% wasn't pulled out of thin air, instead it is based on the recommendation by ACI 318-11 §10.10.4.1. Whereby ACI 318-11 §10.10.4.1 states that concrete beams and walls can be modeled with 35% of the gross cross section being effective.

Default modeling assumptions programmed into structural modeling software – like ETABS and SAP2000 – must be understood and modified if they impede generally accurate structural behavior simulation. One typical default programmed assumption is centerline modeling (Lepage, 2012). In centerline modeling, the beams extend to the centerline of the piers. This impedes accurate stiffness and deflection analysis, because the moment of inertia is double counted. To prevent the issue, the beam's ends are offset to the pier face. Not all default programmed assumptions were overridden, the panel zone factor was maintained to remain equal to 1.0 to represent fully rigid. For concrete, the typical recommended panel zone factor is between 0.5 and 1.0 (Lepage, 2012). If the lateral force resisting element were steel instead of concrete, then the panel zone factor would be changed to 0.5 (Lepage, 2012). Next the assumption to use ACI 318-08 and occasionally ACI 318-05 design criteria, was necessary because ACI 318-11 design criteria wasn't available in the modeling software used – ETABS and SAP2000. As can be seen in the calculations and computer output, there are slight differences.

Table G.1, Total Diaphragm Mass used in Design I Structural Modeling	
Floor Level	Unit Total Mass (Kip/ft^2)
0	
1	2.64E-06
2	2.68E-06
3	2.65E-06
4	2.60E-06
5	2.60E-06
Roof 1	2.34E-06

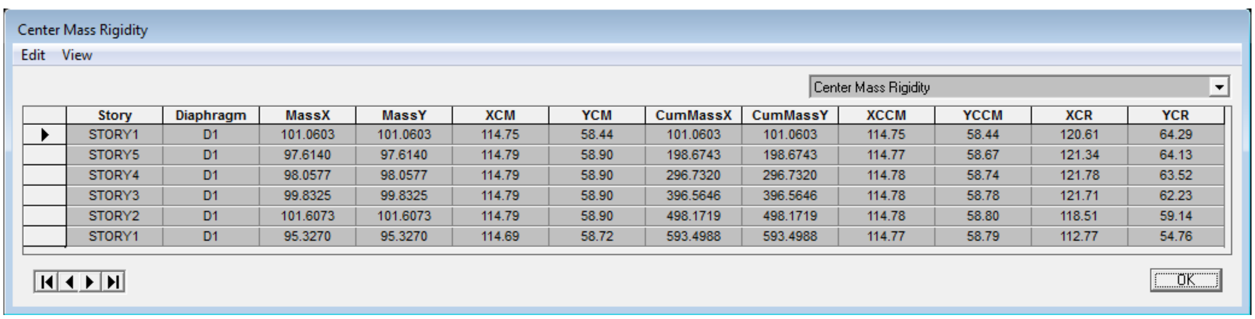
Table G.2, Total Diaphragm Mass used in Design II Structural Modeling	
Floor Level	Unit Total Mass (Kip/ft^2)
0	
1	3.05E-06
2	3.05E-06
3	2.99E-06
4	2.93E-06
5	2.92E-06
Roof 1	2.74E-06

Based upon the predominate theory for modeling lateral force resisting elements – Q-Model (Lollipop Model) – the building mass is concentrated at the floor diaphragms. Modeling by the mentioned method requires all material weights to be set to zero, while the diaphragm is given an equivalent mass. Listed above, in Table G.1 and Table G.2, are the masses applied at the floor diaphragms.

G.2 Non-Formatted Structural Modeling Output

The non-formatted structural modeling output is published in this document as evidence that no computer output was misrepresented when formatting a more compact output. Figure AG.1 to AG.8, are non-formatted structural modeling outputs of Design I and Design II.

<u>G.2.1 Design I</u>


Center Mass Rigidity
Edit View
Center Mass Rigidity

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR
STORY1	D1	101.0603	101.0603	114.75	58.44	101.0603	101.0603	114.75	58.44	120.61	64.29
STORY5	D1	97.6140	97.6140	114.79	58.90	198.6743	198.6743	114.77	58.67	121.34	64.13
STORY4	D1	98.0577	98.0577	114.79	58.90	296.7320	296.7320	114.78	58.74	121.78	63.52
STORY3	D1	99.8325	99.8325	114.79	58.90	396.5646	396.5646	114.78	58.78	121.71	62.23
STORY2	D1	101.6073	101.6073	114.79	58.90	498.1719	498.1719	114.78	58.80	118.51	59.14
STORY1	D1	95.3270	95.3270	114.69	58.72	593.4988	593.4988	114.77	58.79	112.77	54.76

OK

Figure AG.1, Center of Mass and Rigidity for Design I

Point Displacements

Edit View

Point Displacements

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY6	1	WINDDX	0.69	-0.15	0.00	0.00	0.00	0.00
STORY6	1	WINDDY	-0.24	0.82	0.00	0.00	0.00	0.00
STORY6	1	WINDT1DX	0.49	-0.17	0.00	0.00	0.00	0.00
STORY6	1	WINDT1DY	-0.33	0.28	0.00	0.00	0.00	0.00
STORY6	1	WINDT2	0.07	-0.02	0.00	0.00	0.00	0.00
STORY6	1	WINDDXY	0.33	0.51	0.00	0.00	0.00	0.00
STORY6	1	WINDT1DNX	0.54	-0.04	0.00	0.00	0.00	0.00
STORY6	1	WINDT1DNY	-0.03	0.95	0.00	0.00	0.00	0.00
STORY6	2	WINDDX	0.69	-0.13	0.00	0.00	0.00	0.00

Figure AG.2, Corner Point 1 Displacement for Design I

Point Displacements

Edit View

Point Displacements

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY6	6	WINDDX	0.69	-0.08	-0.10	0.01	0.01	0.00
STORY6	6	WINDDY	-0.24	0.62	1.99	-0.14	-0.19	0.00
STORY6	6	WINDT1DX	0.49	0.00	0.06	0.00	-0.01	0.00
STORY6	6	WINDT1DY	-0.33	0.77	2.41	-0.16	-0.23	0.00
STORY6	6	WINDT2	0.07	0.67	2.12	-0.14	-0.20	0.00
STORY6	6	WINDDXY	0.33	0.40	1.42	-0.10	-0.14	0.00
STORY6	6	WINDT1DNX	0.54	-0.12	-0.22	0.01	0.02	0.00
STORY6	6	WINDT1DNY	-0.03	0.16	0.58	-0.04	-0.06	0.00
STORY6	7	WINDDX	0.70	-0.08	-0.01	0.00	0.00	0.00

Figure AG.3, Corner Point 6 Displacement for Design I

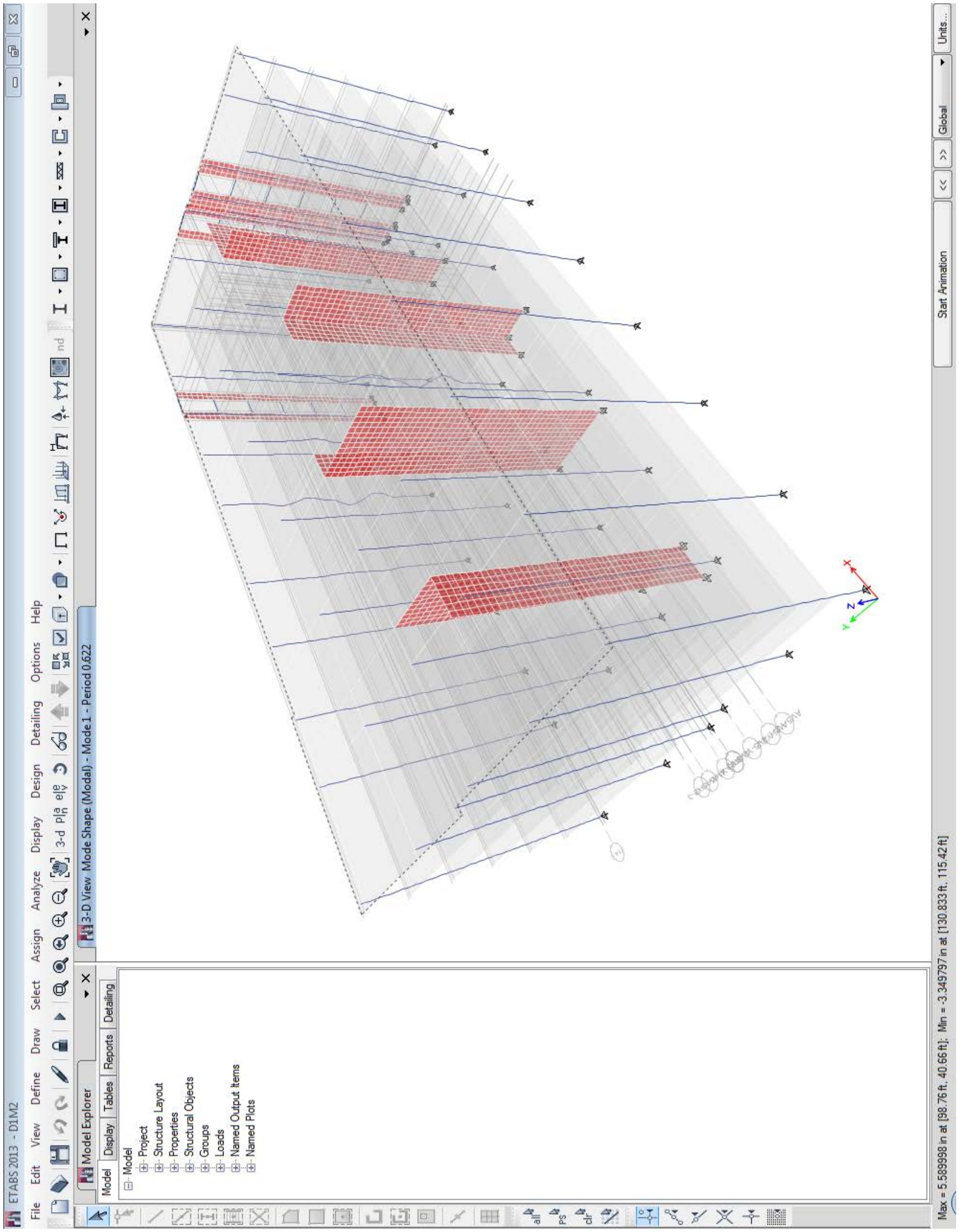


Figure AG.4, Fundamental Building Period (Long)

G.2.2 Design II

Center Mass Rigidity

Edit View

Center Mass Rigidity

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR
STORY1	D1	101.5554	101.5554	114.77	58.42	101.5554	101.5554	114.77	58.42	116.80	59.52
STORY5	D1	98.4901	98.4901	114.83	58.89	200.0455	200.0455	114.80	58.65	117.01	59.08
STORY4	D1	98.9337	98.9337	114.83	58.89	298.9792	298.9792	114.81	58.73	117.23	58.76
STORY3	D1	100.7083	100.7083	114.83	58.89	399.6875	399.6875	114.82	58.77	117.40	58.60
STORY2	D1	102.4829	102.4829	114.83	58.89	502.1705	502.1705	114.82	58.79	117.47	58.77
STORY1	D1	97.5386	97.5386	114.74	58.71	599.7091	599.7091	114.81	58.78	116.62	59.10

OK

Figure AG.5, Center of Mass and Rigidity for Design II

Point Displacements

Edit View

Point Displacements

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY6	1	WINDDX	0.56	0.00	0.00	0.00	0.00	0.00
STORY6	1	WINDDY	0.01	0.84	-0.07	0.00	0.00	0.00
STORY6	1	WINDT1DX	0.41	-0.03	0.00	0.00	0.00	0.00
STORY6	1	WINDT1DY	-0.06	0.50	-0.04	0.00	0.00	0.00
STORY6	1	WINDT2	0.25	0.32	-0.03	0.00	0.00	0.00
STORY6	1	WINDDXY	0.43	0.63	-0.05	0.00	0.00	0.00
STORY6	1	WINDT1DNX	0.44	0.02	0.00	0.00	0.00	0.00
STORY6	1	WINDT1DNY	0.07	0.75	-0.06	0.00	0.00	0.00
STORY6	2	WINDDX	0.56	0.00	0.00	0.00	0.00	0.00

OK

Figure AG.6, Corner Point 1 for Design II

Point Displacements

Edit View

Point Displacements

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY6	6	WINDDX	0.56	0.00	0.00	0.00	0.00	0.00
STORY6	6	WINDDY	0.01	0.81	-0.07	0.00	0.00	0.00
STORY6	6	WINDT1DX	0.41	0.02	0.00	0.00	0.00	0.00
STORY6	6	WINDT1DY	-0.06	0.73	-0.07	0.00	0.00	0.00
STORY6	6	WINDT2	0.25	0.60	-0.06	0.00	0.00	0.00
STORY6	6	WINDDXY	0.43	0.61	-0.06	0.00	0.00	0.00
STORY6	6	WINDT1DNX	0.44	-0.02	0.00	0.00	0.00	0.00
STORY6	6	WINDT1DNY	0.07	0.49	-0.04	0.00	0.00	0.00
STORY6	7	WINDDX	0.56	0.00	0.01	0.00	0.00	0.00

OK

Figure AG.7, Corner Point 6 for Design II

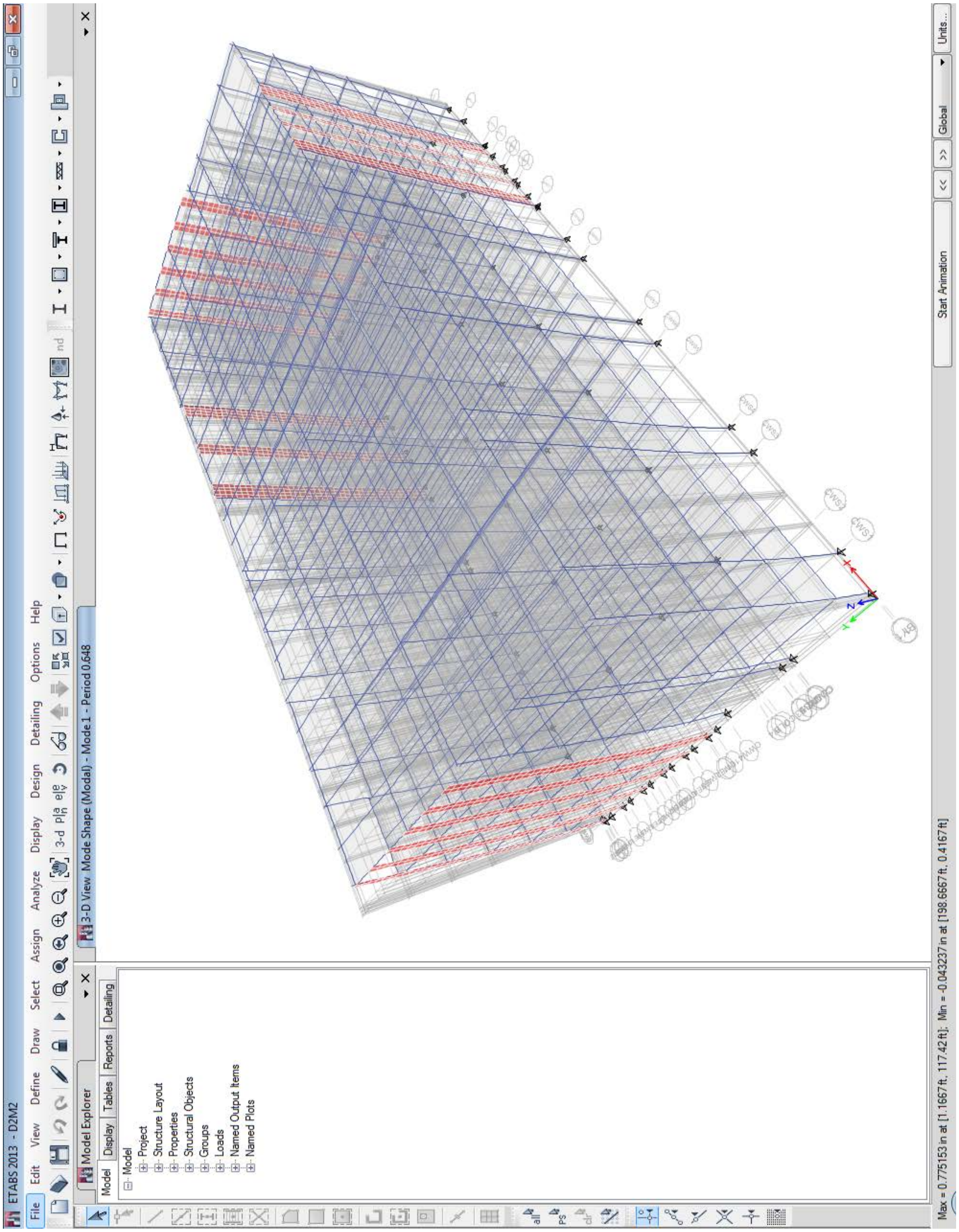


Figure AG.8, Fundamental Building Period (Long)

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Appendix H: Lateral Design for Design I and II

H.1 Re-Design I: Exterior Lateral Resisting Elements

H.1.1 Loads Applied

Table AH.1., Wind Load Case Nomenclature	
Wind Load Case	Description
WINDDX	Wind Case I, wind perpendicular to east and west walls
WINDDY	Wind Case I, wind perpendicular to north and south walls
WINDT1DX	Wind Case II, wind perpendicular to east and west walls, CCW moment
WINDT1DNX	Wind Case II, wind perpendicular to east and west walls, CW moment
WINDT1DY	Wind Case II, wind perpendicular to north and south walls, CCW moment
WINDT1DNY	Wind Case II, wind perpendicular to north and south walls, CW moment
WINDDXY	Wind Case III
WINDT2	Wind Case IV

(a) Columns

Table AH.2, Lateral Load Applied							
Pier Assembly	Load Case	$F_{level,I}$ (Kip)					
		1	2	3	4	5	Roof
P5X1 + P5X2	WINDDX	-1.8	-1.94	0.25	1.32	1.68	15.28
	WINDDY	0.56	0.27	1.07	0.14	0.85	-7.28
	WINDT1DX	-1.79	-1.37	0.25	0.98	1.22	10.78
	WINDT1DY	-1.68	0.65	1.12	0.05	0.69	-9.19
	WINDT2	-3.23	-0.43	1.13	0.75	1.43	0.1
	WINDDXY	-0.94	-1.24	0.99	1.09	1.9	6
	WINDT1DNX	-0.91	-1.54	0.14	1	1.3	12.13
	WINDT1DNY	2.5	-0.23	0.47	0.16	0.6	-1.73
P5Y1 + P5Y2	WINDDX	-0.42	0.06	0.16	-0.07	-0.17	-1.78
	WINDDY	6.66	-1.76	-1.84	0.00	-0.93	16.6
	WINDT1DX	0.48	-0.12	-0.06	-0.02	-0.11	0.13
	WINDT1DY	8.81	-2.24	-2.27	0.00	-1.02	20.5
	WINDT2	8.11	-2.04	-2.02	-0.03	-0.89	17.84
	WINDDXY	4.68	-1.27	-1.26	-0.04	-0.83	11.11
	WINDT1DNX	-1.11	0.21	0.28	-0.06	-0.15	-2.8
	WINDT1DNY	1.2	-0.4	-0.48	0	-0.39	4.41
	WINDDX	7.07	-3.11	-6.85	-6.28	-4.69	-0.63

P5Y3 + P5Y4	WINDDY	-31.85	21.07	38.18	33.18	37.69	17.78
	WINDT1DX	2.55	0.07	-1.06	-1.16	0.02	0.65
	WINDT1DY	-37.64	26.97	47.64	41.16	46.43	21.38
	WINDT2	-30.45	23.67	40.71	34.96	40.27	18.81
	WINDDXY	-18.6	13.48	23.5	20.17	24.76	12.86
	WINDT1DNX	8.05	-4.72	-9.23	-8.26	-7.06	-1.58
	WINDT1DNY	-10.15	4.64	9.63	8.62	10.1	5.3

Table AH.3, Axial Load Break Down for Piers of Interior Lateral Force Resisting Elements

Story	Pier	Axial Load, F_p (Kip)		
		Wind Induced	Live	Non-SW Dead
STORY6	P1X	29.23	3.03	12.6
STORY5		50.56	8.74	12.6
STORY4		72.26	8.74	12.6
STORY3		94.3	8.74	12.8
STORY2		120.49	8.74	13.0
STORY1		145.32	8.74	12.9
STORY6	P1Y	-29.23	3.62	15.1
STORY5		-50.56	11.7	15.1
STORY4		-72.26	11.7	15.1
STORY3		-94.3	11.7	15.3
STORY2		-120.49	11.7	15.5
STORY1		-145.32	11.7	15.4
STORY6	P2Y	42.04	5.07	21.1
STORY5		63.19	15.74	21.1
STORY4		84.39	15.74	21.1
STORY3		104.15	15.74	21.5
STORY2		119.6	15.74	21.8
STORY1		127.97	15.74	21.6
STORY6	P2X	-42.04	20.26	84.4
STORY5		-63.19	54.2	84.4
STORY4		-84.39	54.2	84.4
STORY3		-104.15	54.2	85.7
STORY2		-119.6	54.2	87.1
STORY1		-127.97	54.2	86.4
STORY6	P3X	-20.36	11.92	49.7
STORY5		-33.11	32.5	49.7
STORY4		-46.05	32.5	49.7
STORY3		-58.68	32.5	50.5
STORY2		-82.03	32.5	51.3
STORY1		-123.49	32.5	50.9

STORY6	P3Y2	20.36	14.8	61.7
STORY5		33.11	40.47	61.7
STORY4		46.05	40.47	61.7
STORY3		58.68	40.47	62.7
STORY2		82.03	40.47	63.6
STORY1		123.49	40.47	63.2
STORY6	P4Y	30.39	7.51	31.3
STORY5		49.92	20.73	31.3
STORY4		76.95	20.73	31.3
STORY3		101.84	20.73	31.8
STORY2		117.94	20.73	32.3
STORY1		110.7	20.73	32.0
STORY6	P4X	-30.39	19.03	79.3
STORY5		-49.92	52.9	79.3
STORY4		-76.95	52.9	79.3
STORY3		-101.84	52.9	80.6
STORY2		-117.94	52.9	81.8
STORY1		-110.7	52.9	81.2

Table AH.4, Axial Load Break Down for Piers of Perimeter Lateral Force Resisting Elements				
Story	Pier	Axial Load, F_p (Kip)		
		Wind Induced	Live	Non-SW Dead
STORY6	P5X1	10.11	0	0
STORY5		20.48	0	0
STORY4		21.91	0	0
STORY3		22.47	0	0
STORY2		21.08	0	0
STORY1		16.91	0	0
STORY6	P5X2	-10.11	0	0
STORY5		-20.48	0	0
STORY4		-21.91	0	0
STORY3		-22.47	0	0
STORY2		-21.08	0	0
STORY1		-16.91	0	0
STORY6	P5Y1	12.97	0	0
STORY5		24.89	0	0
STORY4		24.25	0	0
STORY3		22.83	0	0
STORY2		20.7	0	0
STORY1		19.8	0	0

STORY6	P5Y2	-12.97	0	0
STORY5		-24.89	0	0
STORY4		-24.24	0	0
STORY3		-22.83	0	0
STORY2		-20.7	0	0
STORY1		-19.8	0	0
STORY6	P5Y3	0.79	0	0
STORY5		44.66	0	0
STORY4		84.25	0	0
STORY3		129.4	0	0
STORY2		180.8	0	0
STORY1		166.0	0	0
STORY6	P5Y4	-0.79	0	0
STORY5		-44.66	0	0
STORY4		-84.26	0	0
STORY3		-129.4	0	0
STORY2		-180.8	0	0
STORY1		-166.0	0	0

(b) Beams

Table AH.5, Flexural and Shear Loads

Beam Spanning Piers	Story	Maximum Moment (Kip-ft)	Shear (Kip)
P5Y1 + P5Y2	STORY5	84.77	10.90
	STORY3	134.03	17.28
	STORY1	194.12	24.89
P5Y3 + P5Y4	STORY5	274.64	44.66
	STORY3	800.50	129.40
	STORY1	1110.00	271.13

H.1.2 Design Loads and Limtations

(a) Columns

Table AH.6, Base In-Plane Shear and Overturning in Pier Assemblies

Pier Assembly	Load Case	V_{base} (Kip)	M_{base} (Kip-ft)	Pier Assembly	Load Case	V_{base} (Kip)	M_{base} (Kip-ft)
P5X1 + P5X2	WINDDX	14.8	1435.6	P5Y3 + P5Y4	WINDDX	-14.5	-1037.6
	WINDDY	-4.4	-492.6		WINDDY	116.1	7969.6
	WINDT1DX	10.1	1013.0		WINDT1DX	1.1	-13.6
	WINDT1DY	-8.4	-695.8		WINDT1DY	145.9	9871.9

	WINDT2	-0.3	140.2		WINDT2	127.9	8558.9
	WINDDXY	7.8	707.3		WINDDXY	76.2	5199.3
	WINDT1DNX	12.1	1140.1		WINDT1DNX	-22.8	-1542.2
	WINDT1DNY	1.8	-42.5		WINDT1DNY	28.1	2083.4
P5Y1 + P5Y2	WINDDX	-2.2	-167.2				
	WINDDY	18.7	1333.4				
	WINDT1DX	0.3	3.5				
	WINDT1DY	23.8	1663.4				
	WINDT2	20.9	1448.1				
	WINDDXY	12.4	874.7				
	WINDT1DNX	-3.6	-254.2				
	WINDT1DNY	4.3	337.2				

Table AH.7, Maximum Factored In-Plane Shear and Moments

Pier	Floor Level	Width (in)	Maximum Shear (Kip)	Maximum Moment (Kip-ft)
P5Y1	1	48	20.78	181.22
	3		14.14	89.35
	5		16.02	81.11
P5Y2	1	46.5	17.26	143.51
	3		13.39	80.71
	5		15.15	74.75
P5Y3	1	64.5	54.21	454.16
	3		34.64	236.84
	5		15.66	103.95
P5Y4	1	85.5	45.84	964.62
	3		41.51	599.75
	5		40.56	262.67

(b) Beams

Table AH.8, Maximum Factored In-Plane Shear and Moments

Beam	Floor Level	Length (ft)	Height (in)	Maximum Shear (Kip)	Maximum Moment (Kip-ft)	Length-to-Height Ratio	Beam Type
BL5Y1T5Y2	1	7.25	48	24.89	194.12	1.81	Deep
	3		48	17.28	134.03	1.81	Deep
	5		48	10.90	84.77	1.81	Deep
BL5Y3T5Y2	1	6.92	48	271.13	1110.00	1.73	Deep
	3		48	129.40	800.50	1.73	Deep

	5		48	44.66	274.64	1.73	Deep

H.1.3 Structural Lateral System Design

(a) Columns

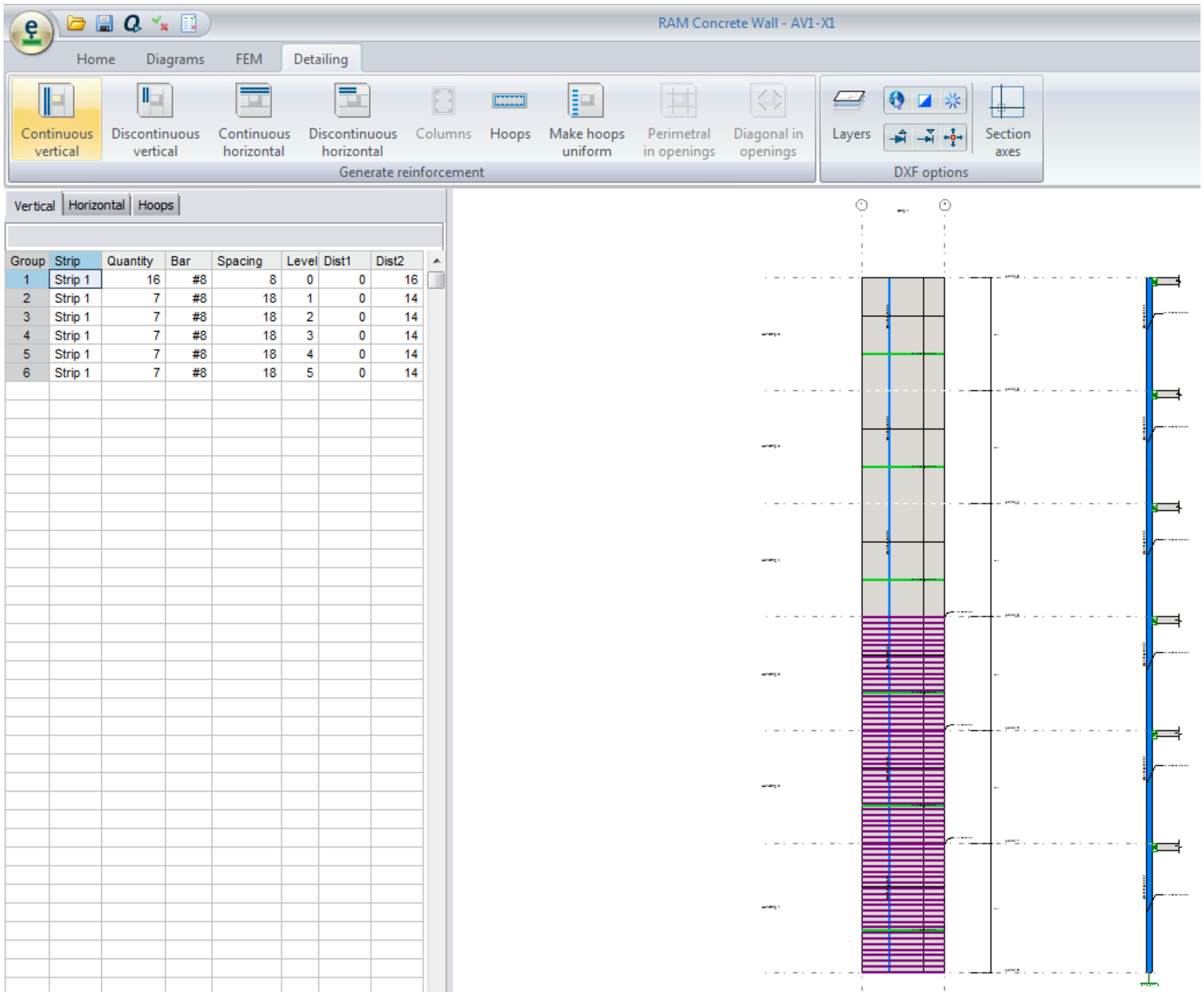
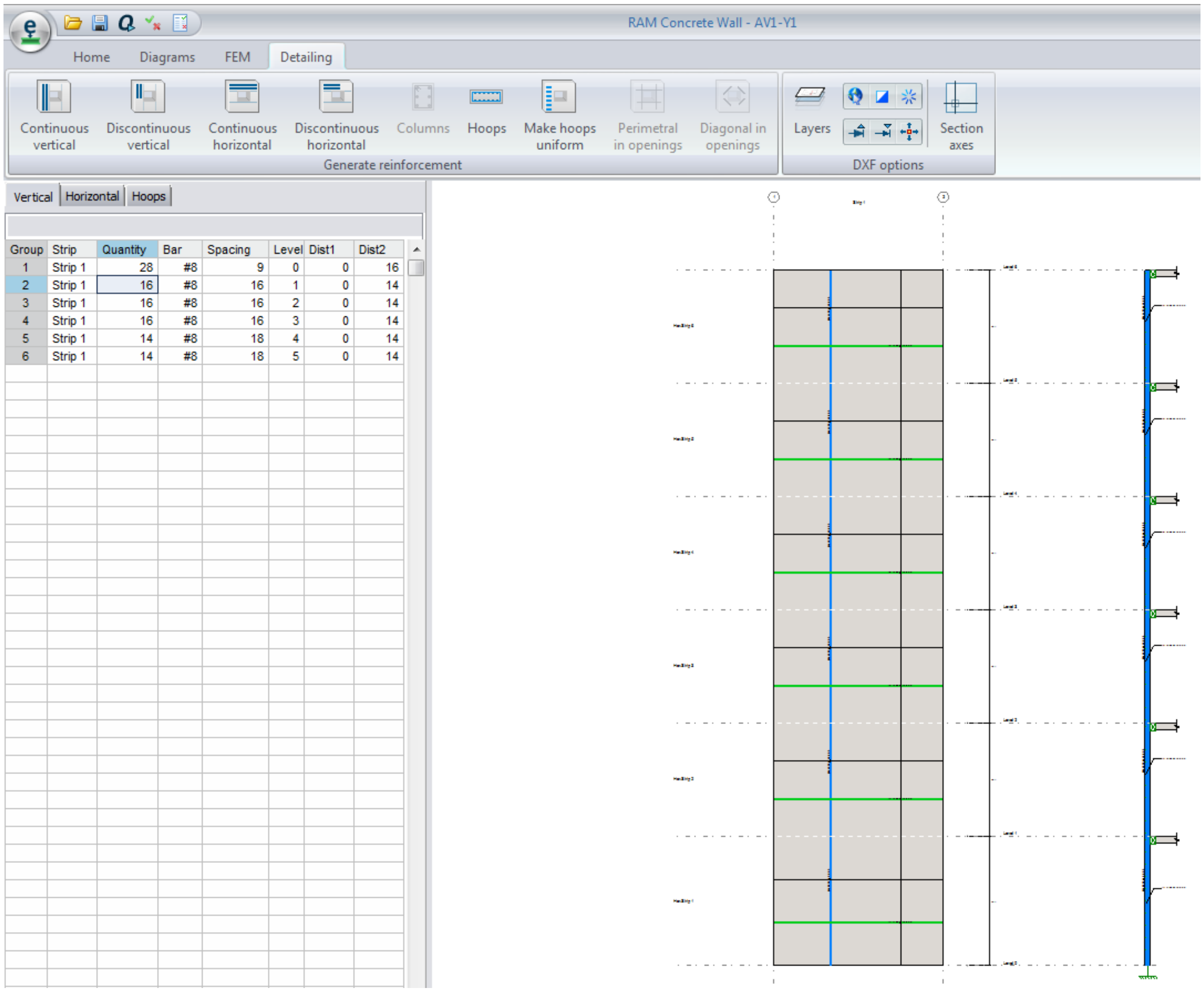


Figure AH.1, RAM Elements Flexural Design Output of AV1-X1



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	28	#8	9	0	0	16
2	Strip 1	16	#8	16	1	0	14
3	Strip 1	16	#8	16	2	0	14
4	Strip 1	16	#8	16	3	0	14
5	Strip 1	14	#8	18	4	0	14
6	Strip 1	14	#8	18	5	0	14

Figure AH.2, RAM Elements Flexural Design Output of AV1-Y1

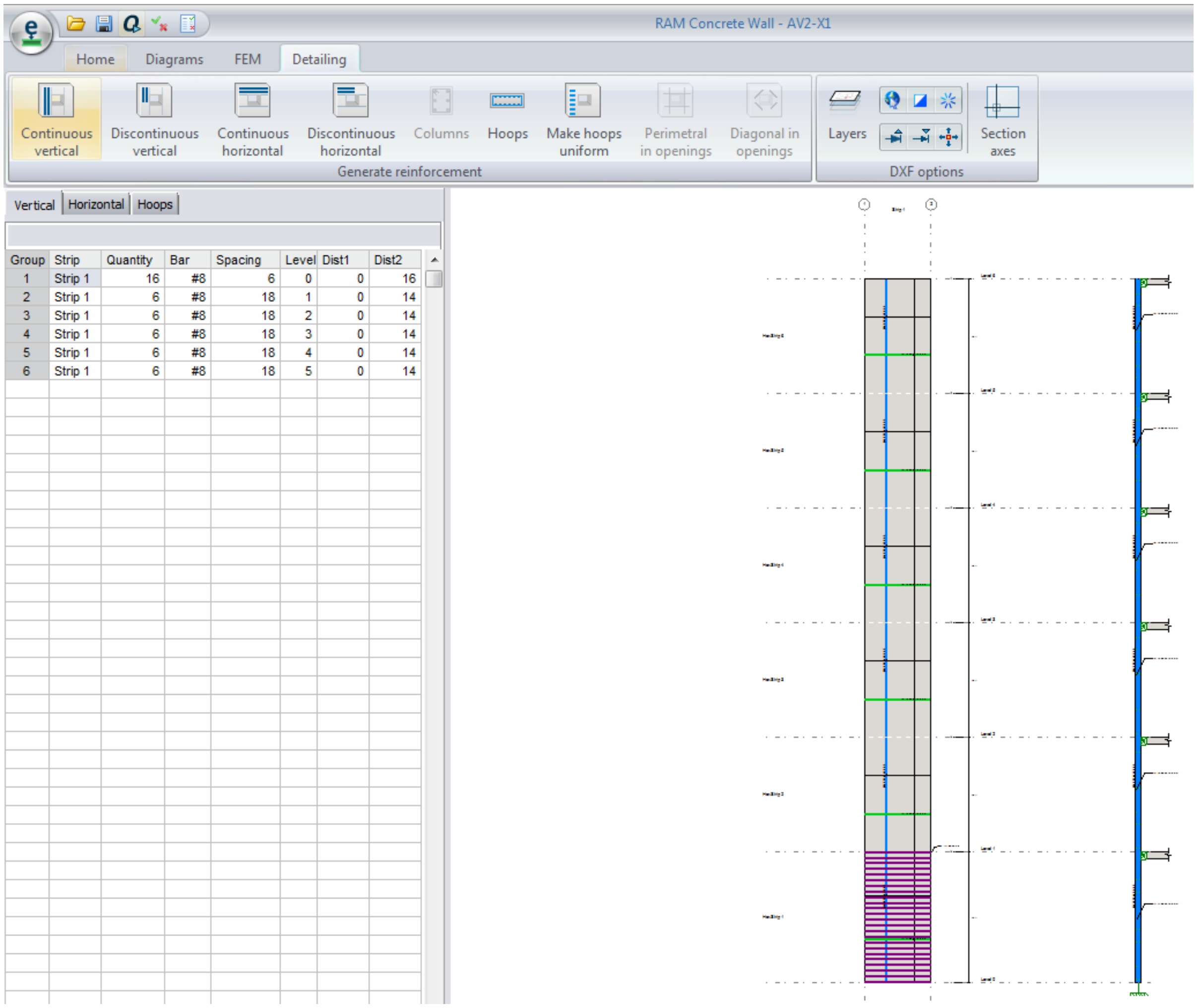


Figure AH.3, RAM Elements Flexural Design Output of AV2-X1

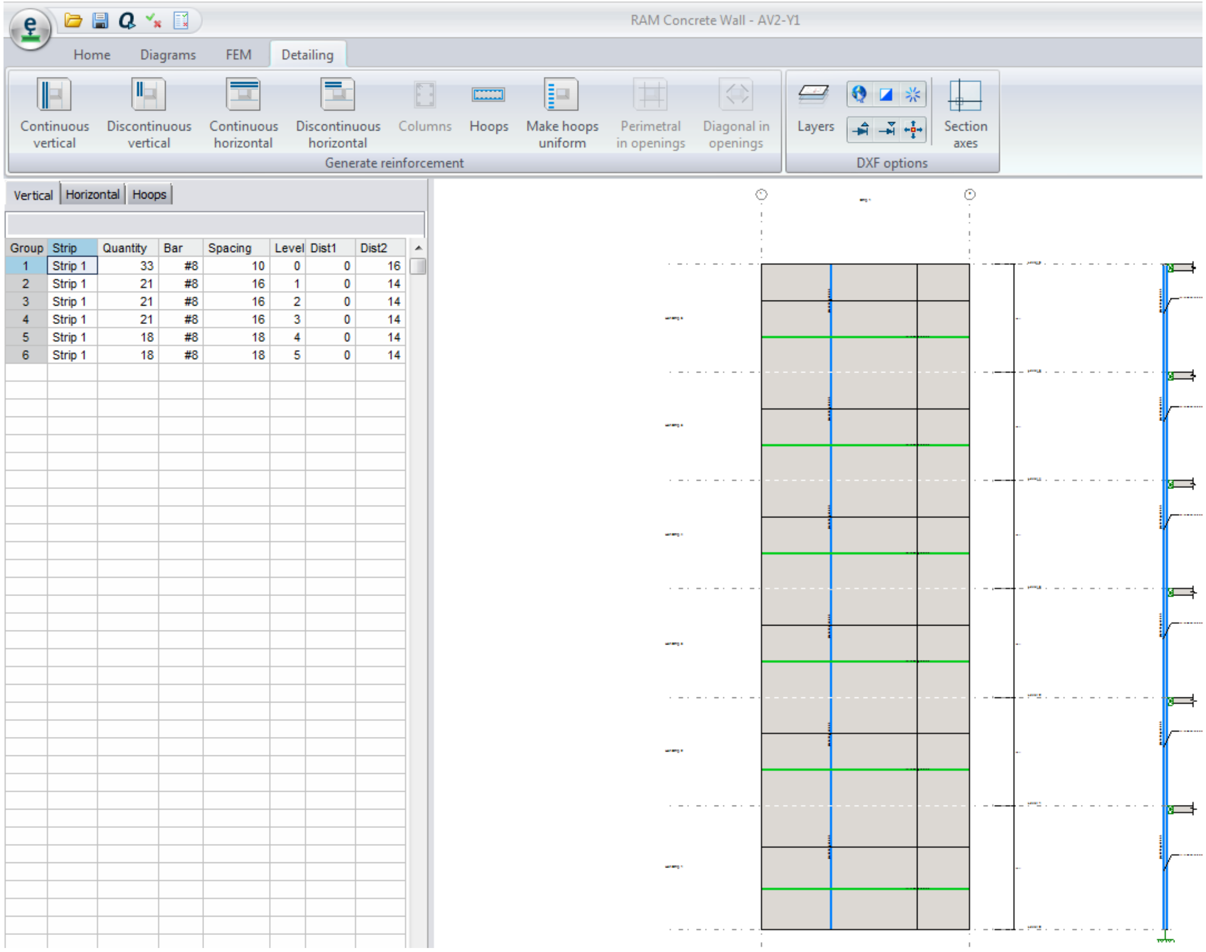
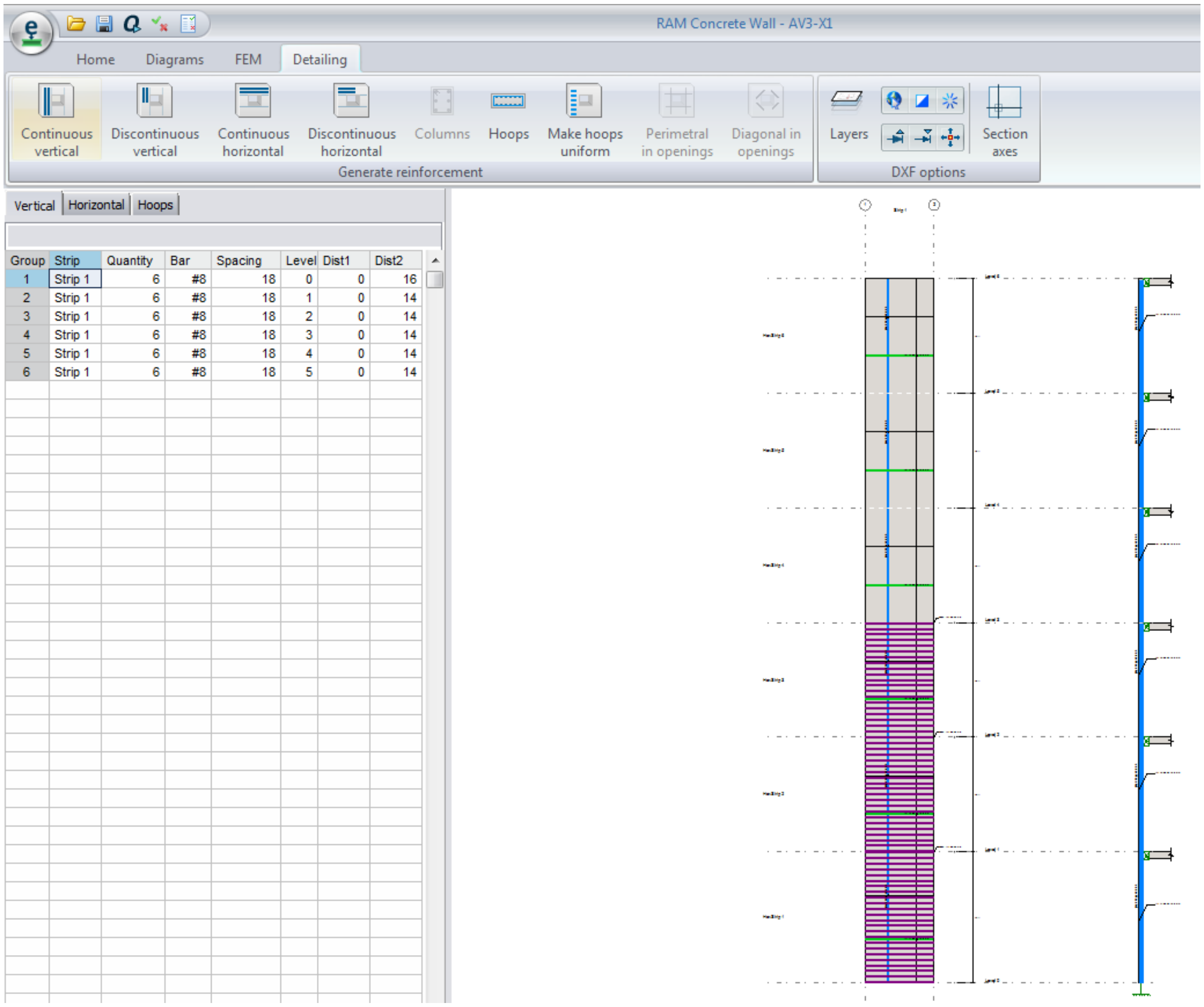
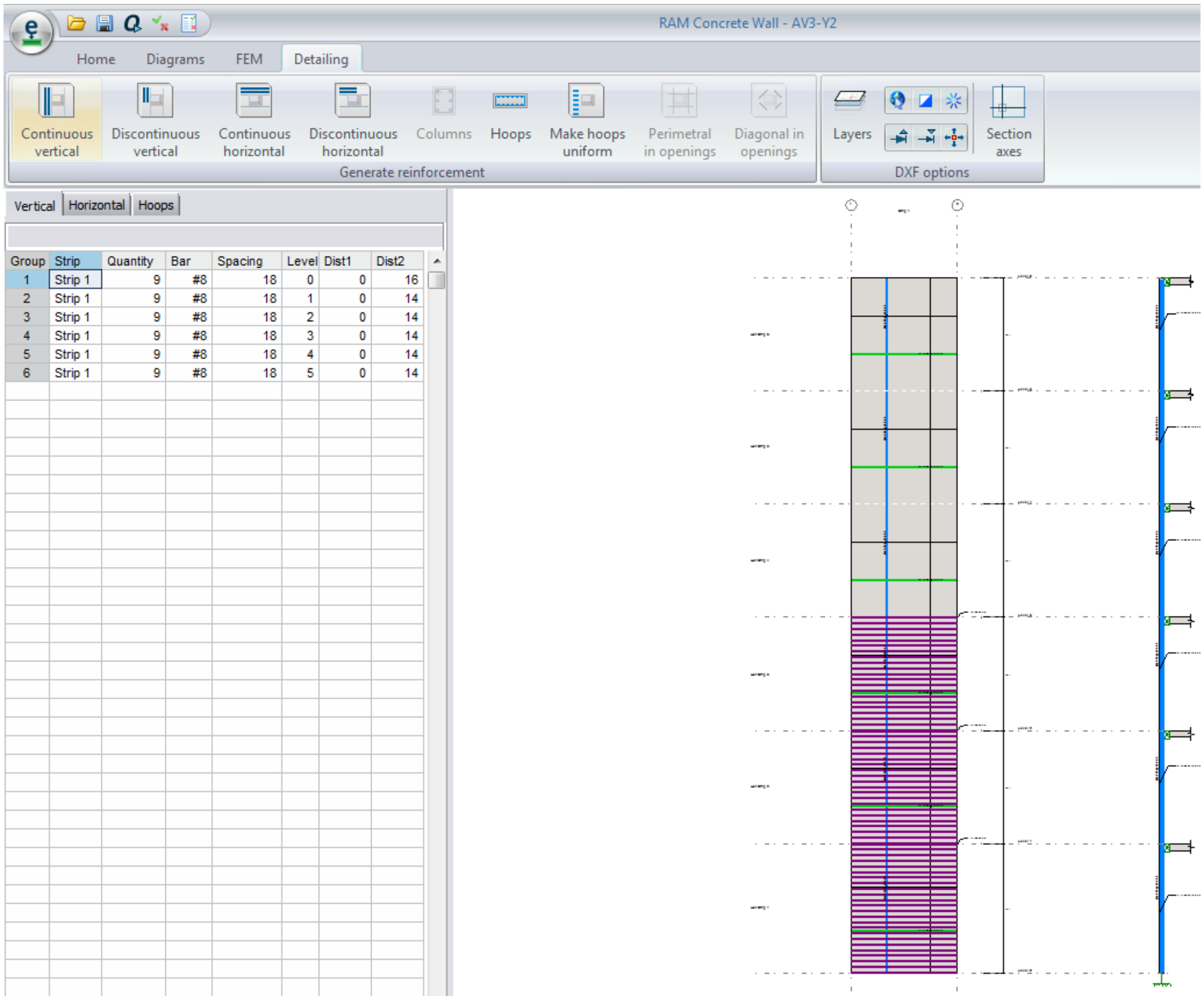


Figure AH.4, RAM Elements Flexural Design Output of AV2-Y1



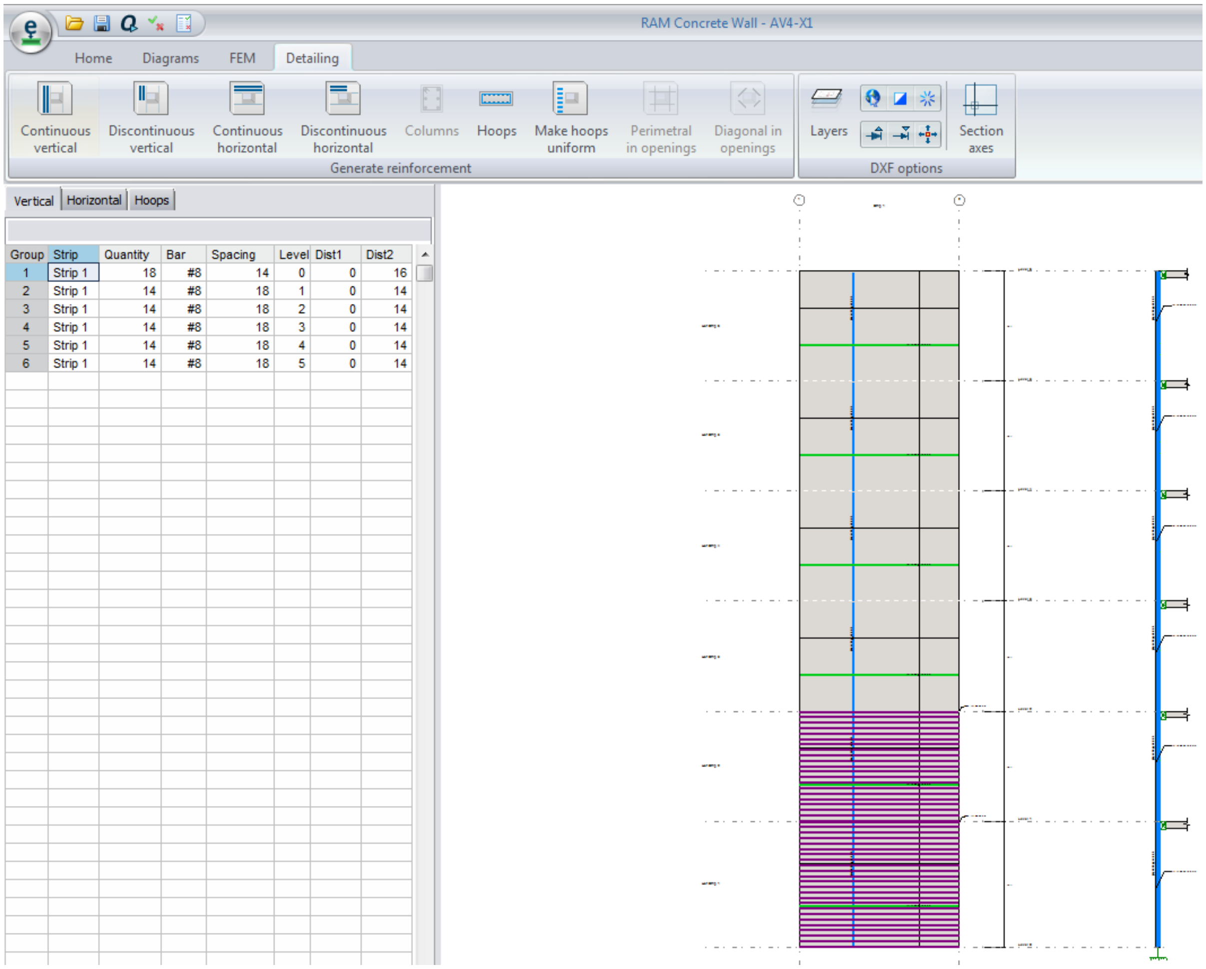
Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	6	#8	18	0	0	16
2	Strip 1	6	#8	18	1	0	14
3	Strip 1	6	#8	18	2	0	14
4	Strip 1	6	#8	18	3	0	14
5	Strip 1	6	#8	18	4	0	14
6	Strip 1	6	#8	18	5	0	14

Figure AH.5, RAM Elements Flexural Design Output of AV3-X1



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	9	#8	18	0	0	16
2	Strip 1	9	#8	18	1	0	14
3	Strip 1	9	#8	18	2	0	14
4	Strip 1	9	#8	18	3	0	14
5	Strip 1	9	#8	18	4	0	14
6	Strip 1	9	#8	18	5	0	14

Figure AH.6, RAM Elements Flexural Design Output of AV3-Y2



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	18	#8	14	0	0	16
2	Strip 1	14	#8	18	1	0	14
3	Strip 1	14	#8	18	2	0	14
4	Strip 1	14	#8	18	3	0	14
5	Strip 1	14	#8	18	4	0	14
6	Strip 1	14	#8	18	5	0	14

Figure AH.7, RAM Elements Flexural Design Output of AV4-X1

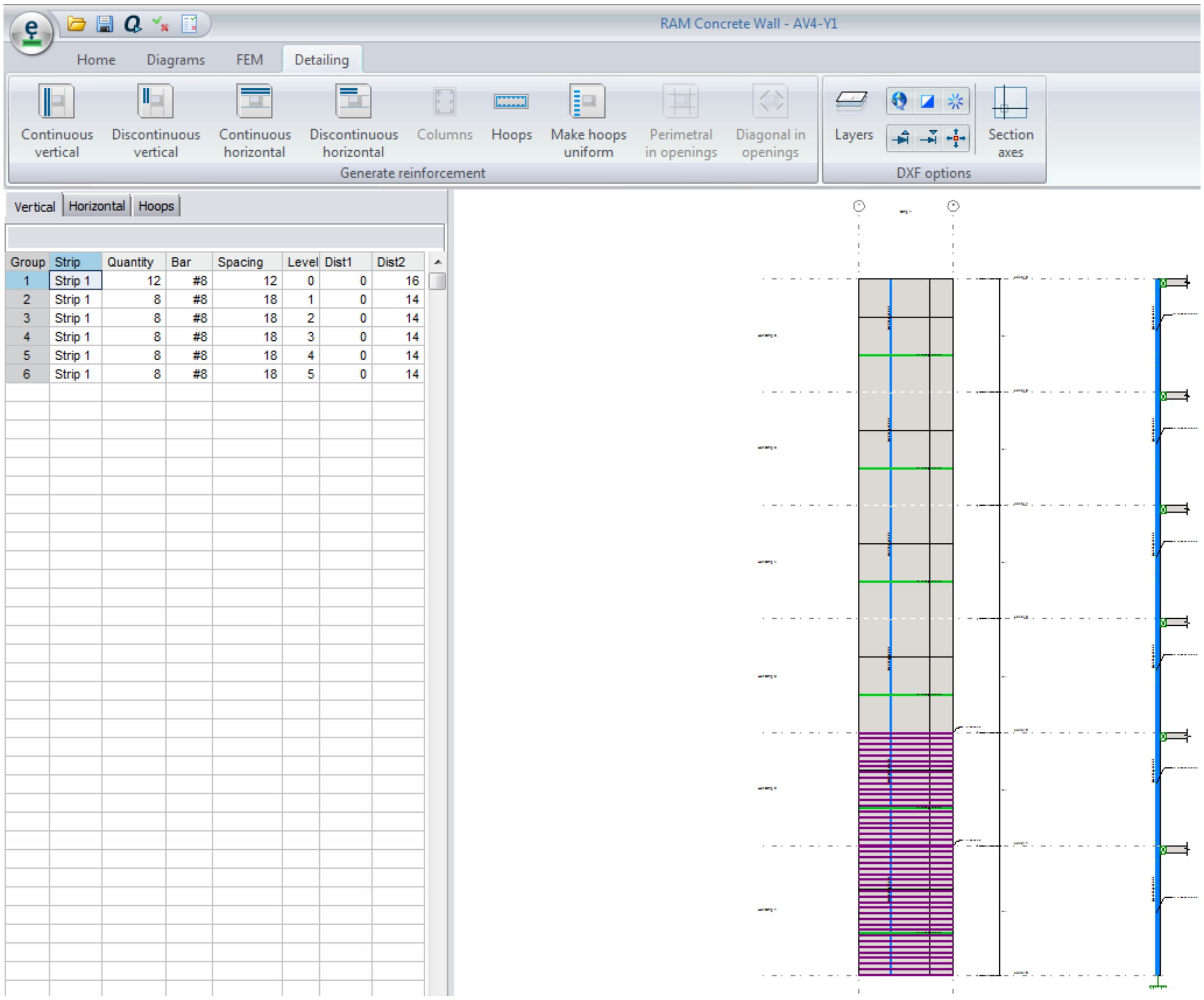
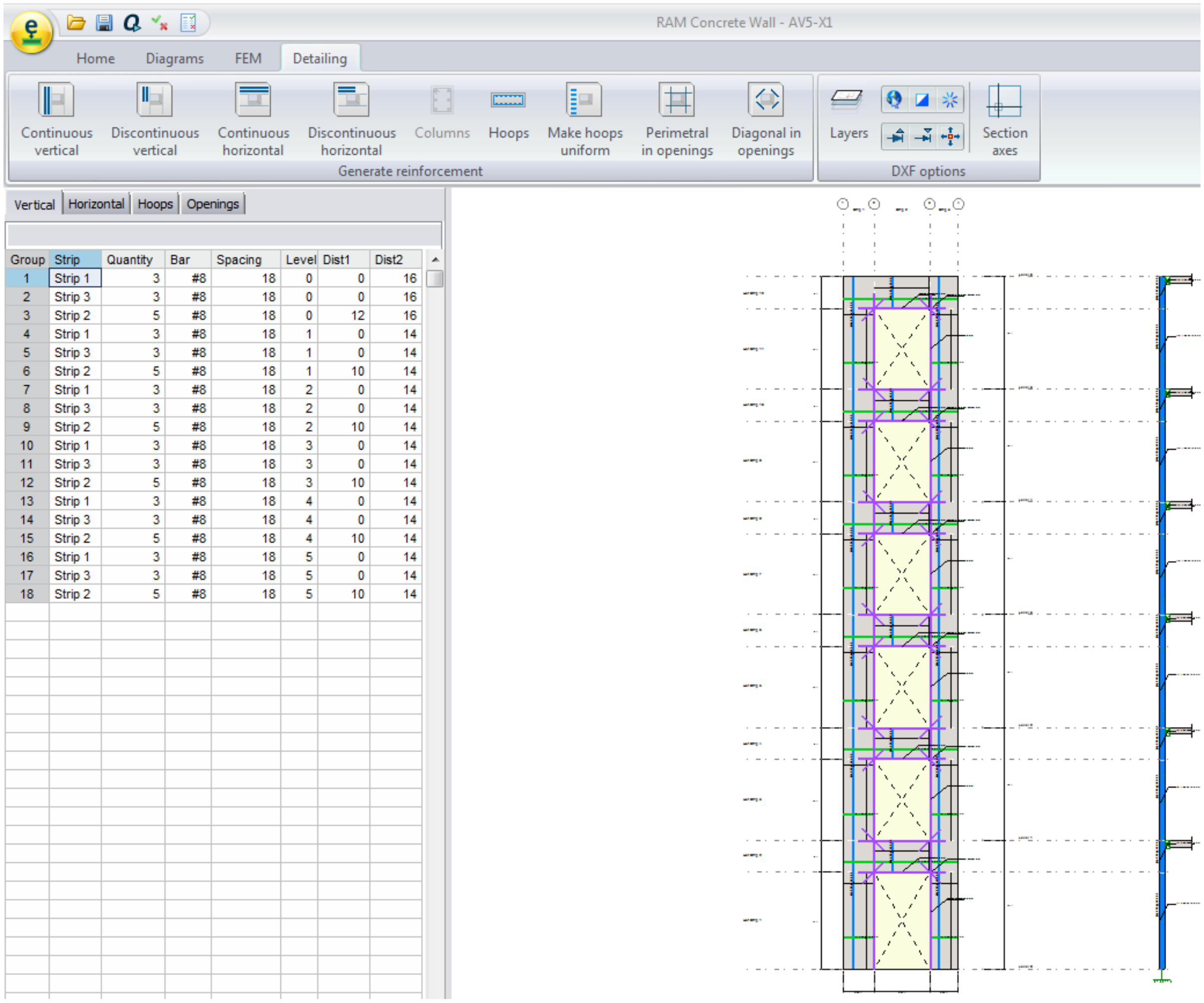
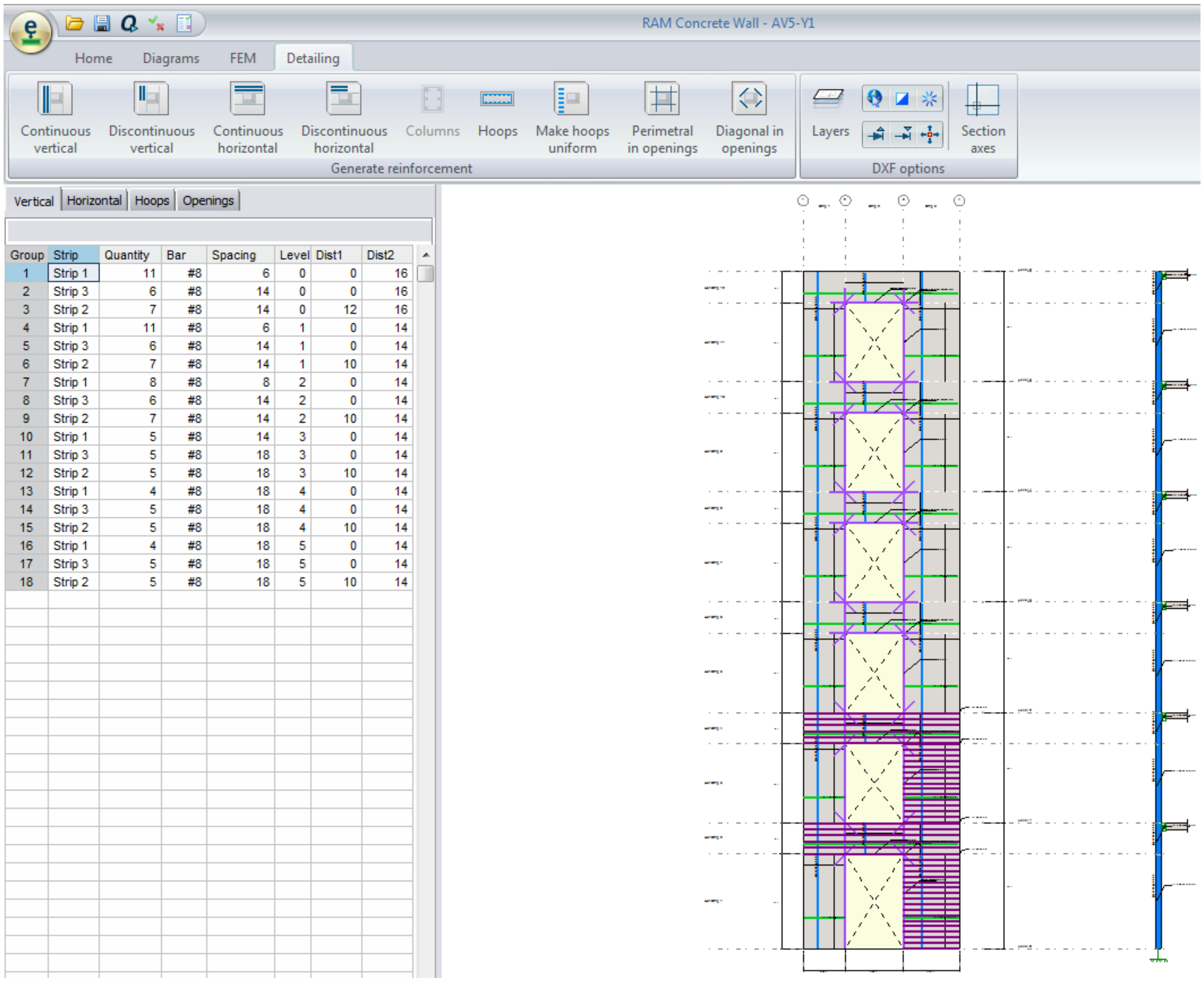


Figure AH.8, RAM Elements Flexural Design Output of AV4-Y1



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	3	#8	18	0	0	16
2	Strip 3	3	#8	18	0	0	16
3	Strip 2	5	#8	18	0	12	16
4	Strip 1	3	#8	18	1	0	14
5	Strip 3	3	#8	18	1	0	14
6	Strip 2	5	#8	18	1	10	14
7	Strip 1	3	#8	18	2	0	14
8	Strip 3	3	#8	18	2	0	14
9	Strip 2	5	#8	18	2	10	14
10	Strip 1	3	#8	18	3	0	14
11	Strip 3	3	#8	18	3	0	14
12	Strip 2	5	#8	18	3	10	14
13	Strip 1	3	#8	18	4	0	14
14	Strip 3	3	#8	18	4	0	14
15	Strip 2	5	#8	18	4	10	14
16	Strip 1	3	#8	18	5	0	14
17	Strip 3	3	#8	18	5	0	14
18	Strip 2	5	#8	18	5	10	14

Figure AH.9, RAM Elements Flexural Design Output of AV5-X1



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	11	#8	6	0	0	16
2	Strip 3	6	#8	14	0	0	16
3	Strip 2	7	#8	14	0	12	16
4	Strip 1	11	#8	6	1	0	14
5	Strip 3	6	#8	14	1	0	14
6	Strip 2	7	#8	14	1	10	14
7	Strip 1	8	#8	8	2	0	14
8	Strip 3	6	#8	14	2	0	14
9	Strip 2	7	#8	14	2	10	14
10	Strip 1	5	#8	14	3	0	14
11	Strip 3	5	#8	18	3	0	14
12	Strip 2	5	#8	18	3	10	14
13	Strip 1	4	#8	18	4	0	14
14	Strip 3	5	#8	18	4	0	14
15	Strip 2	5	#8	18	4	10	14
16	Strip 1	4	#8	18	5	0	14
17	Strip 3	5	#8	18	5	0	14
18	Strip 2	5	#8	18	5	10	14

Figure AH.10, RAM Elements Flexural Design Output of AV5-Y1

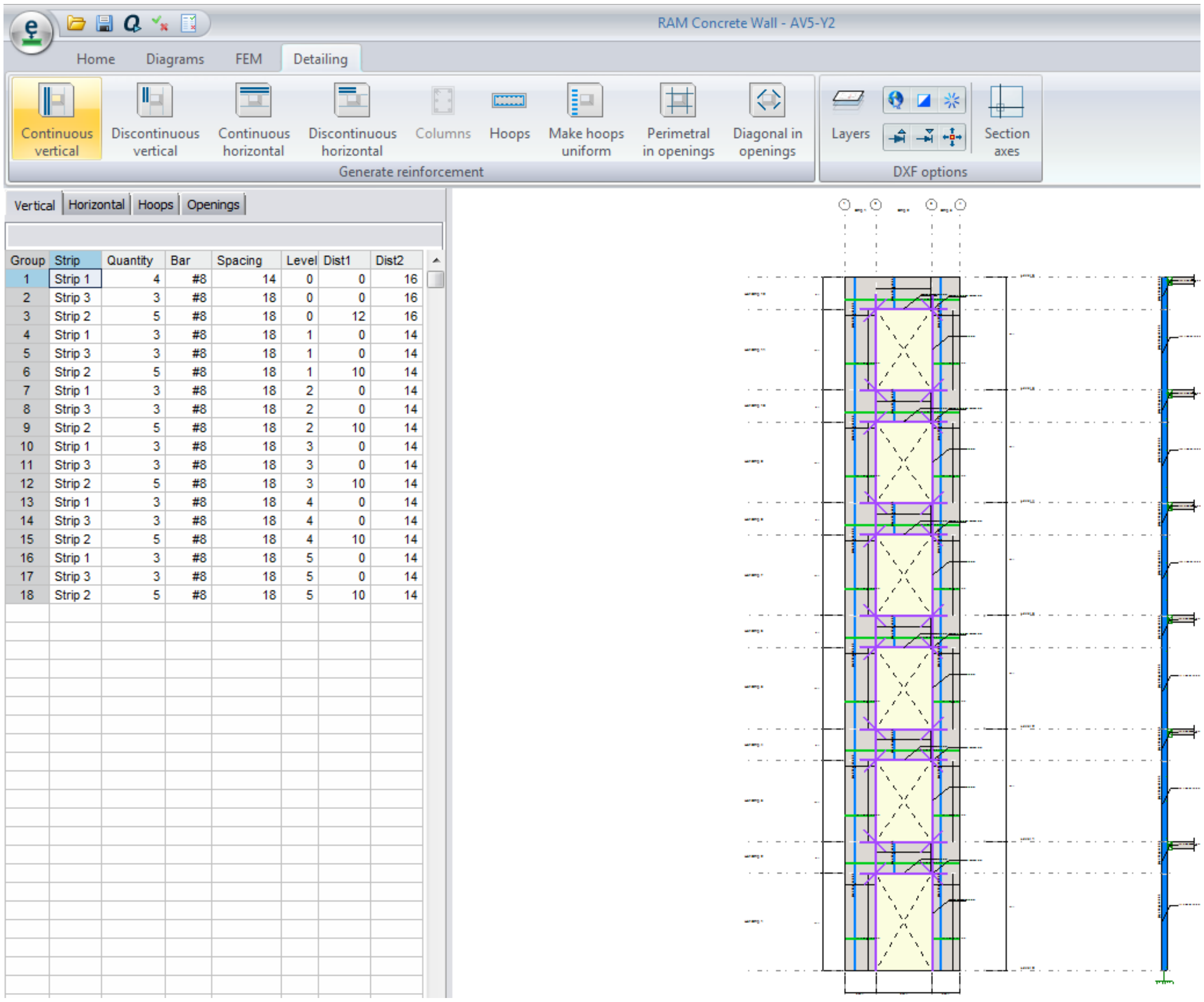


Figure AH.11, RAM Elements Flexural Design Output of AV5-Y2

Thaison Nguyen

Design 1: SPOT CHECK MEMBER
AVI-Y1

$M_u = 19234.9$ kip-ft | Derived From ETABS Output
$V_u = 366.5$ kip
$P_u = 357.6$ kip, self dead weight

Member Dimensions	
Length (in)	252
Width (in)	8
Cover (in)	0.75
$d_{extreme}$ (in)	250.25

** Use #8 rebar
$f'_c = 4000$ psi
$f_y = 60$ ksi

a) Determine number of Rows Req
** Assume rebar rows are 6" O.C.

$\phi M_n = 0.9[A_s f_y (d - a/2)]$

$\frac{M_u}{0.9 f_y} = A_s d - a A_s/2$

$4274.4 = 2nA_{s,1}\left[d_{extreme} - \left(\frac{n-1}{2}\right)6\right] - \frac{2nA_{s,1} f_y}{0.85 f'_c b} \cdot \frac{2nA_{s,1}}{2}$

$\frac{4274.4}{2A_{s,1}} = [d_{extreme} n - 3(n-1)n] - \frac{n^2 f_y A_{s,1}}{0.85 f'_c b}$

$\frac{2137.2}{A_{s,1}} = d_{extreme} n - 3(n^2 - n) - 2.21 A_{s,1} n^2$

$2705.3 = 250.25n - 3n^2 + 3n - 1.75n^2$

$2705.3 = 4.75n^2 + 253.5n$

$0 = n^2 - 53.4n + 569.5$

$n = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$n = \frac{53.4 \pm \sqrt{53.4^2 - 4(569.5)}}{2}$

$n = \frac{53.4 \pm 23.9}{2}$

$n = 14.7$ rows

$n_{actual} = 15$ rows

$\phi M_n = 0.9(2 \times 15 \times 0.79)(60)(208.25 - 52.3/2)$

$\phi M_n = 0.9[(1422 \times 182.1)]$

$\phi M_n = 19410.9$ kip-ft > 19234.9 ✓

$a = \frac{nA_{s,1} f_y}{0.85 f'_c b} \cdot 2$, 2 = # of rebar per row.

$A_s = nA_{s,1} \cdot 2$, n = number of rows

$d = d_{extreme} - \left(\frac{n-1}{2}\right) \cdot 6$

$c = a/\beta_1$
$c = 61.5"$

$\varepsilon_{extreme} = \frac{0.003}{c}(d_{extreme} - c)$

$\varepsilon_{extreme} = 0.009 > 0.005$ ✓, use of $\phi = 0.9$ permitted.

$d = 250.25 - \left(\frac{14}{2}\right)6$

$d = 208.25$

→

Thaison Nguyen | Design I: SPOT CHECK MEMBER AVI-YI

b) Check if 28 rows eq. spaced passes

*** Initially assume compression reinforcement doesn't yield, $\varepsilon_y < 0.00207$

Rebar Row	Position (in) from Compression face
1	1.75
2	10.95
3	20.16
4	29.36
5	38.56
6	47.77
7	56.97
8	66.18
9	75.38
10	84.58
11	93.79
12	102.99
13	112.19
14	121.40

Rebar Row	Position (in) from Compression face
15	130.6
16	139.81
17	149.01
18	158.21
19	167.42
20	176.62
21	185.82
22	195.03
23	204.23
24	213.44
25	222.64
26	231.84
27	241.05
28	250.25

1) Determining C (in)

Assuming 20 rebar rows are in tension

$A_s f_y = 0.85 f'_c ba + A_s' \varepsilon_s' E_s$

$1896 = 0.85 f'_c b(\beta_1 C) + A_s' E_s \left[\frac{0.003}{c}(d' + c)\right]$

$1896 = 23.12C + 365400\left[-0.003d'/c + 0.003\right]$

$1896C = 23.12C^2 + 365400(-0.003d' + 0.003C)$

$1896C = 23.12C^2 - 1096.2(33.97) + 1096.2C$

$0 = 23.12C^2 - 796.32C + 37237.9$

$0 = C^2 - 34.44C - 1610.61$

$C = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$C = 60.95 < 66.18$, doesn't include 8 compression rebar

Assumption wrong

$d = \frac{75.38 + 250.25}{2}$

$d = 162.8''$

$d' = \frac{1.75 + 66.18}{2}$

$d' = 33.97''$

$A_s = 2(20)(0.79)$

$A_s = 31.6 \text{ in}^2$

$A_s' = 2(8)(0.79)$

$A_s' = 12.6 \text{ in}^2$

$c = a/\beta_1$

$c = \frac{A_s f_y}{0.85 f'_c b}$

$a = \beta_1 C$

→

Thaison Nguyen | Design I: SPOT CHECK MEMBER AV1-Y1

Assuming 21 rebar rows are in tension

$1991C = 23.12c^2 + 320740(0.003 * 29.4 + 0.003C)$
$1991C = 23.12c^2 + 962.2C - 28289.3$
$0 = 23.12c^2 - 1028.8C - 28289.3$
$0 = C^2 - 44.5C - 1223.6$
$C \approx 63.7" > 56.97$ ✓, incl 7 compression rebar

Assumption correct

$\phi M_n = 0.9[A_s'\varepsilon_s'E_s + 0.85f'_c ba]$
$\phi M_n = 0.9[11.06(29000)(0.0016) + 0.85(4)(8)(0.85*63.7)]$
$\phi M_n = 19493$ kip-ft > 19234.9 kip-ft ✓

$d = (66.18 + 250.25)/2$
$d = 158.2"$

$d' = (1.75 + 56.97)/2$
$d' = 29.4"$

$A_s = 2(21)(0.79)$
$A_s = 33.18 in^2$

$A_s' = 2(7)(0.79)$
$A_s' = 11.06 in^2$

$\varepsilon_s = \frac{0.003}{63.7}(158.2 - 63.7)$
$\varepsilon_s = 0.0045 > 0.004$ ✓

$\varepsilon_s' = \frac{0.003}{63.7}(-29.4 + 63.7)$
$\varepsilon_s' = 0.0016 < 0.00207$ ✓, compression rebar doesn't yield.

$\varepsilon_{s,max} = \frac{0.003}{63.7}(250.25 - 63.7)$
$\varepsilon_{s,max} = 0.00879 > 0.005$ ✓, can use $\phi = 0.9$

C) Interaction

Pure Axial

$P_o = 0.85f'_c[(A_{gross} - A_{s,gross})] + A_{s,gross}f_y$
$P_o = 0.85(4)[252(8) - 28(0.79)(2)] + 28(0.79)(2)(60)$
$P_o = 9358.4$ kip

$\phi P_n = 0.65(9358.4)$
$\phi P_n = 6082.9$ kip

$0.8\phi P_n = 4866.4$ kip

Balance Codition

$\varepsilon_y = 0.00207$

$C = 0.003/(0.003 + 0.00207) * 250.25$
$C = 148.1"$

*** Each contribution of rebar rows were determined in excel,

→

Table AH.8A. Reinforcement Contribution to Axial and Bending Capacity						
Rebar Row	Position (in)	ε_{si}	f_{si} (Ksi)	M_{bi} (Kip-in)	M_b (Kip-ft)	P_b (Kip)
1	1.75	0.00296	60	14910.0	31724.28	3999.86
2	10.95	0.00278	60	13805.6		
3	20.16	0.00259	60	12701.1		
4	29.36	0.00241	60	11596.7		
5	38.56	0.00222	60	10492.2		
6	47.77	0.00203	58.94	9222.0		
7	56.97	0.00185	53.53	7390.6		
8	66.18	0.00166	48.13	5758.4		
9	75.38	0.00147	42.72	4325.1		
10	84.58	0.00129	37.31	3090.9		
11	93.79	0.00110	31.91	2055.7		
12	102.99	0.00091	26.50	1219.6		
13	112.19	0.00073	21.10	582.5		
14	121.40	0.00054	15.69	144.4		
15	130.60	0.00035	10.28	-94.6		
16	139.81	0.00017	4.88	-134.6		
17	149.01	-0.00002	-0.53	24.4		
18	158.21	-0.00020	-5.94	382.5		
19	167.42	-0.00039	-11.34	939.6		
20	176.62	-0.00058	-16.75	1695.7		
21	185.82	-0.00076	-22.16	2650.9		
22	195.03	-0.00095	-27.56	3805.1		
23	204.23	-0.00114	-32.97	5158.3		
24	213.44	-0.00132	-38.37	6710.6		
25	222.64	-0.00151	-43.78	8461.9		
26	231.84	-0.00170	-49.19	10412.2		
27	241.05	-0.00188	-54.59	12561.6		
28	250.25	-0.00207	-60	14910.0		

Thaison Nguyen

Design I : SPOT CHECK MEMBER AV1-Y1

$\emptyset P_n = 0.65(3999.86)$

$\emptyset P_n = 2599.9$ Kip

$\emptyset M_n = 0.65(31724.28)$

$\emptyset M_n = 20620.8$ Kip-ft

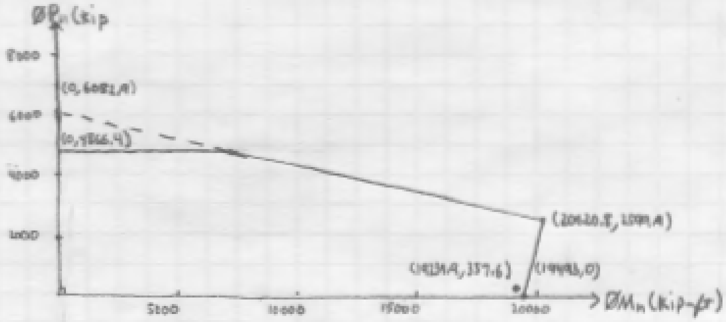


Table AH.9, Part I of #3 Shear Hoop Reinforcement Design: Required Shear Strength

Story	Pier	Length (in)	Thk (in)	d (in)	f'_c (Kip/in^2)	$F_{V\text{-}MAX}$ (Kip)	$V_{u,max}$ (Kip)
STORY6	P1X	124	8	99.2	4	12.2	19.52
STORY5						10.14	35.74
STORY4						11.68	54.43
STORY3						10.48	71.20
STORY2						10.26	87.62
STORY1						7.26	99.23
STORY6	P1Y	252	8	201.6	4	44.2	70.72
STORY5						39.7	134.24
STORY4						36.65	192.88
STORY3						37.28	252.53
STORY2						32.57	304.64
STORY1						38.69	366.54
STORY6	P2Y	324	8	259.2	4	103.92	166.27
STORY5						53.17	251.34
STORY4						55.26	339.76
STORY3						51.61	422.34
STORY2						48.77	500.37
STORY1						22.65	536.61
STORY6	P2X	98	8	78.4	4	13.54	21.66
STORY5						4.86	29.44
STORY4						6.42	39.71
STORY3						6.75	50.51
STORY2						6.29	60.58
STORY1						5.8	69.86
STORY6	P3X	101	8	80.75	4	8.08	12.93
STORY5						4.64	20.35
STORY4						4.92	28.22
STORY3						5.45	36.94
STORY2						10.26	53.36
STORY1						10.16	69.62
STORY6	P3Y2	158	8	126.4	4	15.4	24.64
STORY5						8.92	38.91
STORY4						9.19	53.62
STORY3						7.07	64.93
STORY2						15.9	90.37
STORY1						45.55	163.25
STORY6	P4Y	140	8	112	4	18.05	28.88
STORY5						4.54	36.14
STORY4						7.34	47.89

STORY3						9.57	63.20
STORY2						6.42	73.47
STORY1						43.62	143.26
STORY6	P4X	245	8	195.95	4	40.07	64.11
STORY5						37.2	123.63
STORY4						39.48	186.80
STORY3						37.66	247.06
STORY2						32.26	298.67
STORY1						0.72	299.82
STORY6	P5X1	48	10	37.2	6	7.79	12.46
STORY5						0.9	13.90
STORY4						0.69	15.01
STORY3						0.16	15.26
STORY2						-1.03	13.62
STORY1						-0.58	12.69
STORY6	P5X2	46.5	10	34.4	6	7.49	11.98
STORY5						0.78	13.23
STORY4						0.63	14.24
STORY3						0.09	14.38
STORY2						-0.91	12.93
STORY1						-1.22	10.98
STORY6	P5Y1	48	10	37.2	6	10.55	16.88
STORY5						-0.54	16.02
STORY4						0.02	16.05
STORY3						-1.19	14.14
STORY2						-1.28	12.10
STORY1						5.43	20.78
STORY6	P5Y2	46.5	10	34.4	6	9.95	15.92
STORY5						-0.48	15.15
STORY4						-0.02	15.12
STORY3						-1.08	13.39
STORY2						-0.96	11.86
STORY1						3.38	17.26
STORY6	P5Y3	64.5	10	51.6	6	3.92	6.27
STORY5						5.87	15.66
STORY4						5.73	24.83
STORY3						6.13	34.64
STORY2						4.03	41.09
STORY1						8.2	54.21
STORY6	P5Y4	85.5	10	68.4	6	17.46	27.94
STORY5						40.56	92.83

STORY4						35.43	149.52
STORY3						41.51	215.94
STORY2						22.94	252.64
STORY1						-45.84	179.30

Table AH.10, Part II of #3 Shear Hoop Reinforcement Design: Req. Steel Shear Resistance				
Story	Pier	V_c (Kip) ACI 318-11 §11.4.6.1	$\Phi V_{c,n}$ (Kip) ACI 318-11 § 11.2.1.1	$V_{s,req}$ (Kip) ACI 318-11 § 11.4.7.2
STORY6	P1X	100.4	37.6	0.00
STORY5				0.00
STORY4				34.93
STORY3				57.29
STORY2				79.18
STORY1				94.67
STORY6	P1Y	204.0	76.5	0.00
STORY5				102.48
STORY4				180.67
STORY3				260.20
STORY2				329.68
STORY1				412.22
STORY6	P2Y	262.3	98.4	123.34
STORY5				236.77
STORY4				354.65
STORY3				464.76
STORY2				568.80
STORY1				617.12
STORY6	P2X	79.3	29.8	0.00
STORY5				0.00
STORY4				23.20
STORY3				37.60
STORY2				51.02
STORY1				63.39
STORY6	P3X	81.7	30.6	0.00
STORY5				0.00
STORY4				0.00
STORY3				18.62
STORY2				40.50
STORY1				62.18
STORY6	P3Y2	127.9	48.0	0.00
STORY5				0.00

STORY4				23.52
STORY3				38.61
STORY2				72.53
STORY1				169.70
STORY6	P4Y	113.3	42.5	0.00
STORY5				0.00
STORY4				21.35
STORY3				41.77
STORY2				55.46
STORY1				148.52
STORY6	P4X	198.3	74.4	0.00
STORY5				90.48
STORY4				174.71
STORY3				255.05
STORY2				323.87
STORY1				325.41
STORY6	P5X1	57.6	21.6	0.00
STORY5				0.00
STORY4				0.00
STORY3				0.00
STORY2				0.00
STORY1				0.00
STORY6	P5X2	53.3	20.0	0.00
STORY5				0.00
STORY4				0.00
STORY3				0.00
STORY2				0.00
STORY1				0.00
STORY6	P5Y1	57.6	21.6	0.00
STORY5				0.00
STORY4				0.00
STORY3				0.00
STORY2				0.00
STORY1				0.00
STORY6	P5Y2	53.3	20.0	0.00
STORY5				0.00
STORY4				0.00
STORY3				0.00
STORY2				0.00
STORY1				0.00
STORY6	P5Y3	79.9	30.0	0.00

STORY5				0.00
STORY4				0.00
STORY3				16.21
STORY2				24.81
STORY1				42.30
STORY6	P5Y4	106.0	39.7	0.00
STORY5				84.04
STORY4				159.62
STORY3				248.18
STORY2				297.12
STORY1				199.32

Table AH.11, Part III of #3 Shear Hoop Reinforcement Design: Spacing

Story	Pier	S_{max} (in)		S_{design} (in) ACI 318-11 §11.4.7.2	$A_{v,min}$ (in^2) ACI 318-11 §11.4.6.3	S_{actual} (in)
		ACI 318-11 §11.4.5.1, 11.4.5.3	ACI 318-11 §14.3.5			
STORY6	P1X	24	18	N/A	0.120	18.0
STORY5		24		N/A	0.120	18.0
STORY4		24		37.49	0.120	18.0
STORY3		24		22.86	0.120	18.0
STORY2		24		16.54	0.110	16.0
STORY1		24		13.83	0.092	13.0
STORY6	P1Y	24		N/A	0.120	18.0
STORY5		24		25.97	0.120	18.0
STORY4		24		14.73	0.098	14.0
STORY3		24		10.23	0.068	10.0
STORY2		24		8.07	0.054	8.0
STORY1		12		6.46	0.043	6.0
STORY6	P2Y	24		27.74	0.120	18.0
STORY5		24		14.45	0.096	14.0
STORY4		24		9.65	0.064	9.0
STORY3		24		7.36	0.049	7.0
STORY2		12		6.02	0.040	6.0
STORY1		12		5.54	0.037	5.0
STORY6	P2X	24		N/A	0.120	18.0
STORY5		24		N/A	0.120	18.0
STORY4		24		44.61	0.120	18.0
STORY3		24		27.52	0.120	18.0
STORY2		24		20.28	0.120	18.0
STORY1		24		16.33	0.109	16.0
STORY6	P3X	24		N/A	0.120	18.0

STORY5		24		N/A	0.120	18.0
STORY4		24		N/A	0.120	18.0
STORY3		24		57.26	0.120	18.0
STORY2		24		26.32	0.120	18.0
STORY1		24		17.14	0.114	17.0
STORY6	P3Y2	24		N/A	0.120	18.0
STORY5		24		N/A	0.120	18.0
STORY4		24		70.93	0.120	18.0
STORY3		24		43.22	0.120	18.0
STORY2		24		23.01	0.120	18.0
STORY1		24		9.83	0.066	9.0
STORY6	P4Y	24		N/A	0.120	18.0
STORY5		24		N/A	0.120	18.0
STORY4		24		69.25	0.120	18.0
STORY3		24		35.40	0.120	18.0
STORY2		24		26.66	0.120	18.0
STORY1		24		9.95	0.066	9.0
STORY6	P4X	24		N/A	0.120	18.0
STORY5		24		28.59	0.120	18.0
STORY4		24		14.80	0.099	14.0
STORY3		24		10.14	0.068	10.0
STORY2		24		7.99	0.053	7.0
STORY1		24		7.95	0.053	7.0
STORY6	P5X1	18.6		N/A	0.174	18.0
STORY5		18.6		N/A	0.174	18.0
STORY4		18.6		N/A	0.174	18.0
STORY3		18.6		N/A	0.174	18.0
STORY2		18.6		N/A	0.174	18.0
STORY1		18.6		N/A	0.174	18.0
STORY6	P5X2	17.2		N/A	0.167	17.0
STORY5		17.2		N/A	0.167	17.0
STORY4		17.2		N/A	0.167	17.0
STORY3		17.2		N/A	0.167	17.0
STORY2		17.2		N/A	0.167	17.0
STORY1		17.2		N/A	0.167	17.0
STORY6	P5Y1	18.6		N/A	0.174	18.0
STORY5		18.6		N/A	0.174	18.0
STORY4		18.6		N/A	0.174	18.0
STORY3		18.6		N/A	0.174	18.0
STORY2		18.6		N/A	0.174	18.0
STORY1		18.6		N/A	0.174	18.0

STORY6	P5Y2	17.2		N/A	0.167	17.0
STORY5		17.2		N/A	0.167	17.0
STORY4		17.2		N/A	0.167	17.0
STORY3		17.2		N/A	0.167	17.0
STORY2		17.2		N/A	0.167	17.0
STORY1		17.2		N/A	0.167	17.0
STORY6	P5Y3	24		N/A	0.174	18.0
STORY5		24		N/A	0.174	18.0
STORY4		24		N/A	0.174	18.0
STORY3		24		42.02	0.174	10.0
STORY2		24		27.46	0.174	10.0
STORY1		24		16.10	0.156	10.0
STORY6	P5Y4	24		N/A	0.174	18.0
STORY5		24		10.74	0.104	10.0
STORY4		24		5.66	0.055	5.0
STORY3		12		3.64	0.035	3.0
STORY2		12		3.04	0.029	3.0
STORY1		24		4.53	0.044	4.0

(b) Beams

Table AH.12, Part I of #3 Shear Hoop Reinforcement Design: Required Shear Strength

Story	Beam Spanning Between Piers	Length (in)	Thk (in)	d (in)	f'_c (Kip/in^2)	$V_{u,max}$ (Kip)	
						Col. Face	2" From Col. Face
STORY5	P5Y1 + P5Y2	7.250	10	45.0	6	10.90	10.40
STORY3				45.0		17.28	16.49
STORY1				45.0		24.89	23.75
STORY5	P5Y3 + P5Y4	6.917	10	45.0	6	44.66	40.55
STORY3				42.0		129.40	117.50
STORY1				39.0		271.13	246.20

Table AH.13, Part II of #3 Shear Hoop Reinforcement Design: Req. Steel Shear Resistance

Story	Beam Spanning Between Piers	V_c (Kip) ACI 318-11 §11.4.6.1	$\Phi V_{c,n}$ (Kip) ACI 318-11 §11.2.1.1	$V_{s,req}$ (Kip) ACI 318-11 § 11.4.7.2
STORY5	P5Y1 + P5Y2	69.71	26.14	0.00
STORY3		69.71	26.14	0.00
STORY1		69.71	26.14	0.00
STORY5	P5Y3 + P5Y4	69.71	26.14	18.92

STORY3		65.07	24.40	65.49
STORY1		60.42	22.66	213.14

Table AH.14, Part III of #3 Shear Hoop Reinforcement Design: Spacing				
Story	Beam Spanning Between Piers	S_{max} (in) ACI 318-11 §11.4.5.1,11.4.5.3	S_{design} (in) ACI 318-11 §11.4.7.2	S_{actual} (in)
STORY5	P5Y1 + P5Y2	22.50	N/A	18.00
STORY3		22.50	N/A	18.00
STORY1		22.50	N/A	18.00
STORY5	P5Y3 + P5Y4	22.50	31.40	18.00
STORY3		21.00	8.50	8.50
STORY1		9.75	2.50	2.50

(c) Servicability and Irregularity Check

Table AH.15, Wind Induced Displacements at Roof						
Story	Corner Point	Load Case	UX	UY	RX	RY
6	1	WINDDX	0.69	-0.15	0	0
	1	WINDDY	-0.24	0.82	0	0
	1	WINDT1DX	0.49	-0.17	0	0
	1	WINDT1DY	-0.33	0.28	0	0
	1	WINDT2	0.07	-0.02	0	0
	1	WINDDXY	0.33	0.51	0	0
	1	WINDT1DNX	0.54	-0.04	0	0
	1	WINDT1DNY	-0.03	0.95	0	0
	6	WINDDX	0.69	-0.08	0.01	0
	6	WINDDY	-0.24	0.62	-0.19	0
	6	WINDT1DX	0.49	0	-0.01	0
	6	WINDT1DY	-0.33	0.77	-0.23	0
	6	WINDT2	0.07	0.67	-0.2	0
	6	WINDDXY	0.33	0.4	-0.14	0
	6	WINDT1DNX	0.54	-0.12	0.02	0
	6	WINDT1DNY	-0.03	0.16	-0.06	0

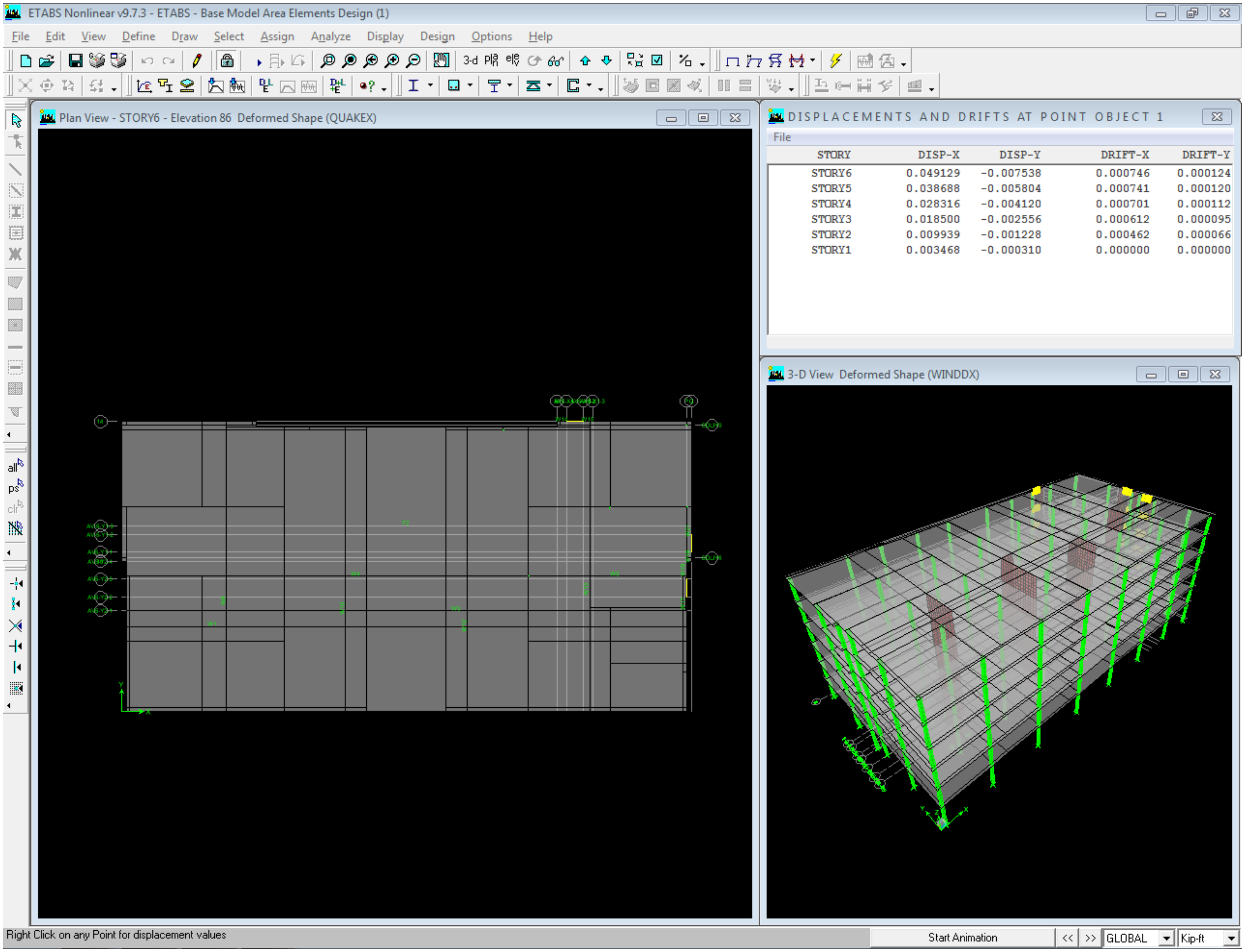
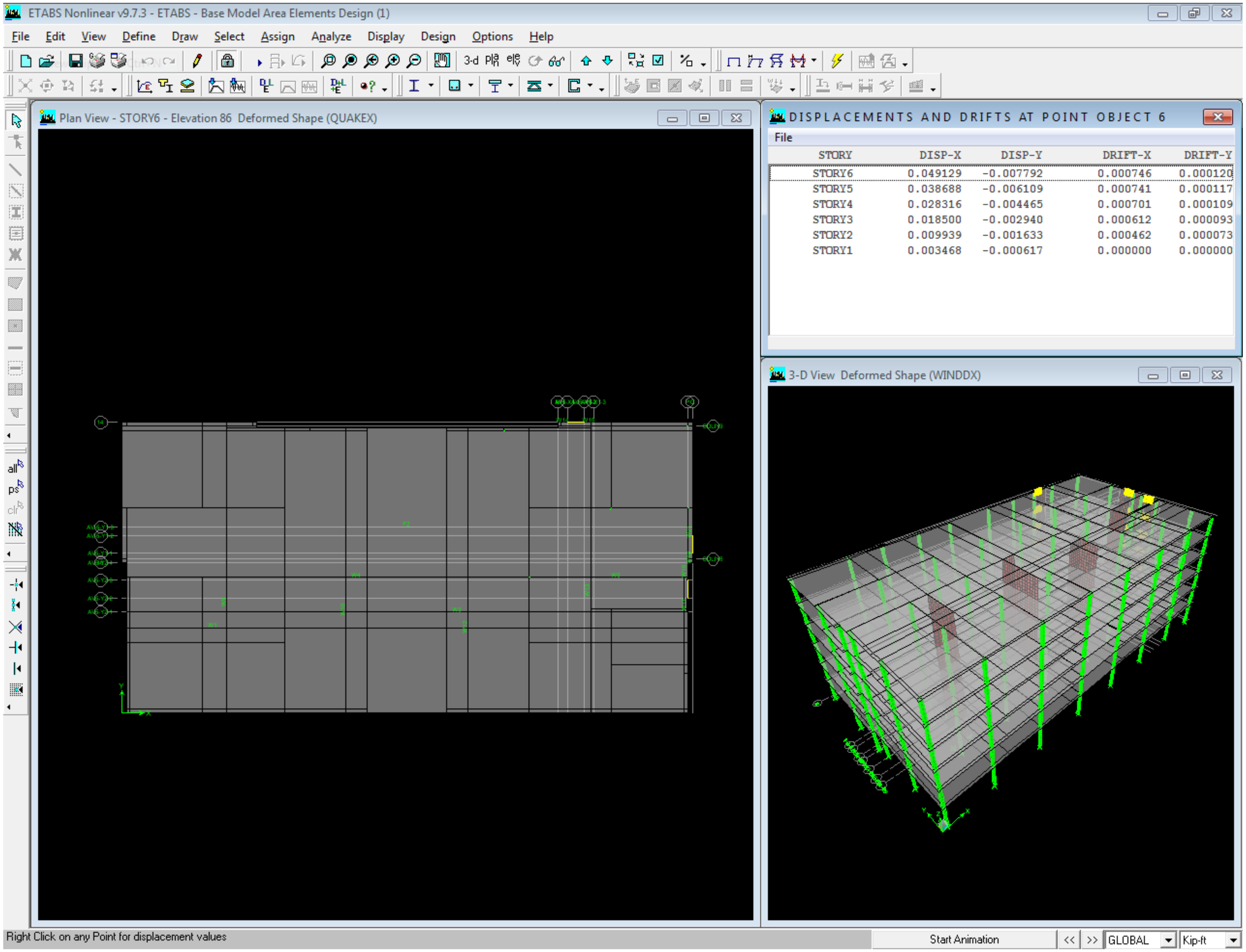


Figure AH.14, Northwest Corner Displacement for Quake in Long Direction



STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY6	0.049129	-0.007792	0.000746	0.000120
STORY5	0.038688	-0.006109	0.000741	0.000117
STORY4	0.028316	-0.004465	0.000701	0.000109
STORY3	0.018500	-0.002940	0.000612	0.000093
STORY2	0.009939	-0.001633	0.000462	0.000073
STORY1	0.003468	-0.000617	0.000000	0.000000

Figure AH.15, Northeast Corner Displacement for Quake in Long Direction

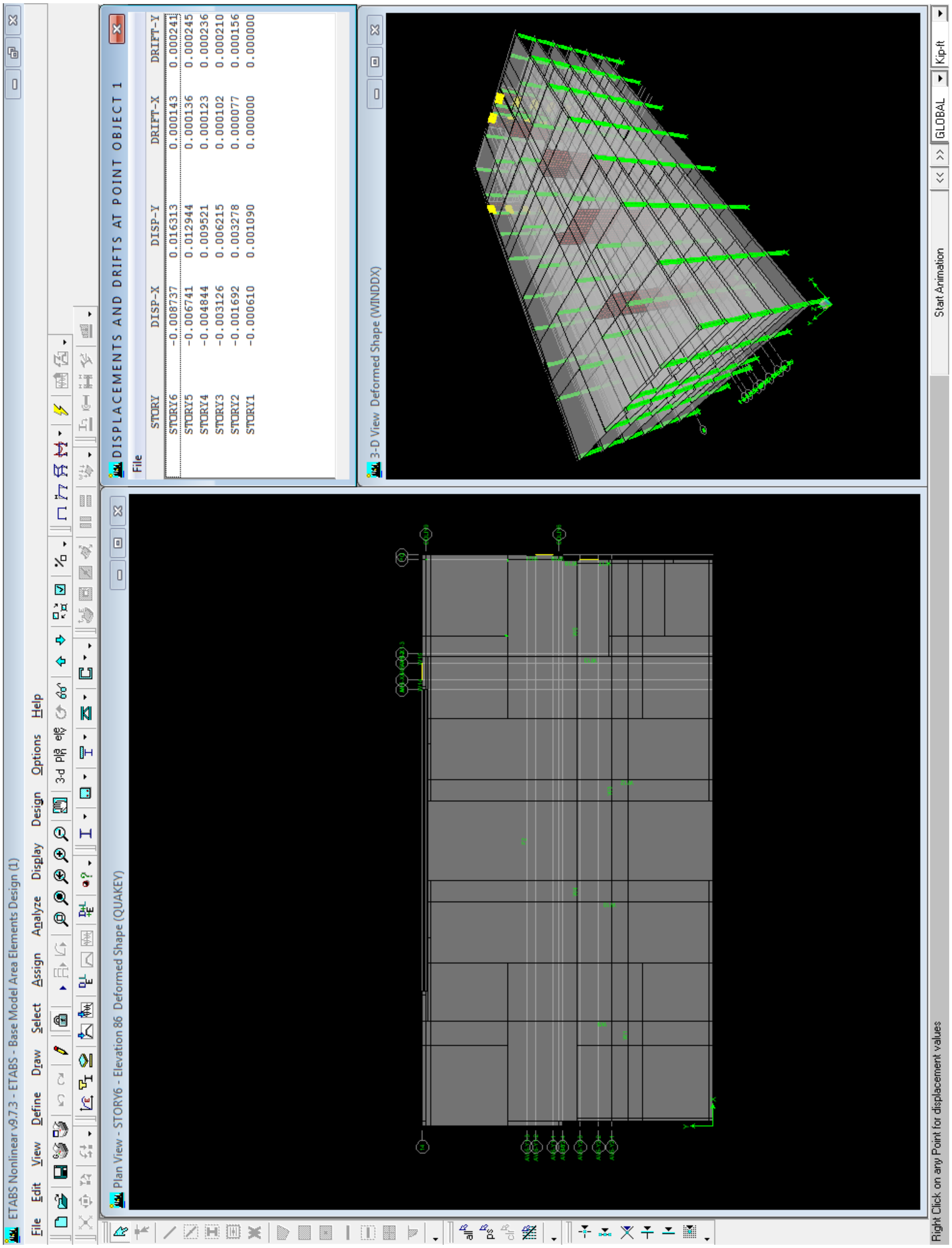


Figure AH.16, Northwest Corner Displacement for Quake in Short Direction

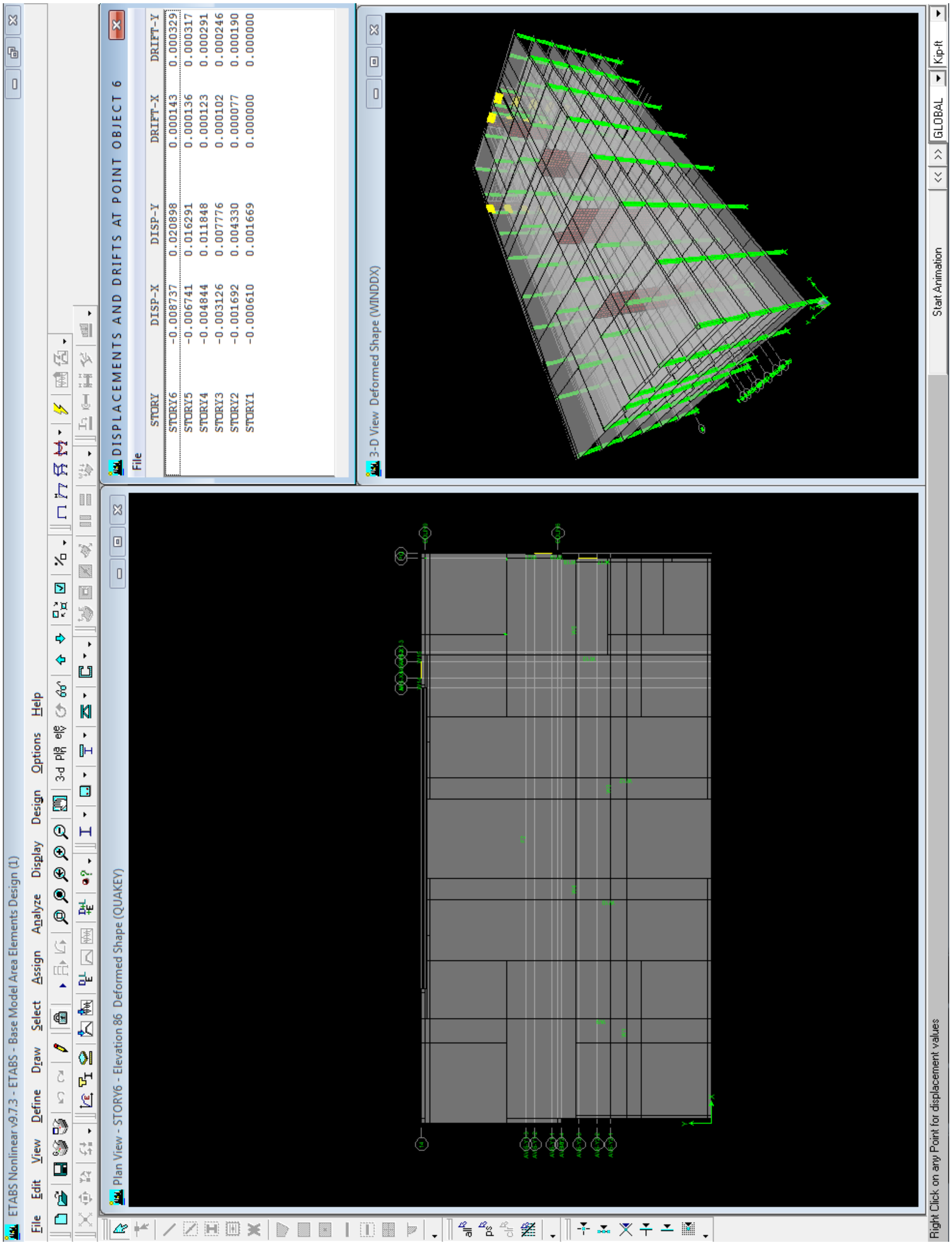


Figure AH.17, Northeast Corner Displacement for Quake in Short Direction

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*** ASCE7-05 Table 12.3-1 Horizontal Irreg. defines torsional irreg. as drift at a structure's ends > 1.2 times the average drift of a structure's two ends

Story	Avg. Drift (Quake in Long Direction w/ Torsion Ac.)		Avg Drift (Quake in Short Direction w/ Torsion Ac.)	
	x	y	x	y
2	0.000462	0.000070	0.000077	0.000173
3	0.000612	0.000094	0.000102	0.000228
4	0.000701	0.000111	0.000123	0.000264
5	0.000741	0.000119	0.000136	0.000281
6	0.000746	0.000122	0.000143	0.000285

a) Check Quake in Long Direction w/ Torsion AC.

1) Story 2

1.2(0.000070) > 0.000073
0.000084 > 0.000073 √, no torsion irreg.

2) Story 3

1.2(0.000094) > 0.000095
0.000113 > 0.000095 √, no torsion irreg.

3) Story 4

1.2(0.000111) > 0.000112
0.000133 > 0.000112 √, no torsion irreg.

4) Story 5

1.2(0.000119) > 0.00012
0.000143 > 0.00012 √, no torsion irreg.

5) Story 6

1.2(0.000122) > 0.000124
0.000146 > 0.000124 √, no torsion irreg.

b) Check Quake in short Direction w/ Torsion Ac.

1) Story 2

1.2(0.000173) > 0.000190
0.000208 > 0.000190 √, no torsion irreg.

→

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Design I: TORSION IRREG. CHECK

2) Story 3

$1.2(0.000228) > 0.000246$

$0.000274 > 0.000246$ ✓, no torsion Irreg.

3) Story 4

$1.2(0.000264) > 0.000291$

$0.000317 > 0.000291$ ✓, no torsion Irreg.

4) Story 5

$1.2(0.000281) > 0.000317$

$0.000337 > 0.000317$ ✓, no torsion Irreg.

5) Story 6

$1.2(0.000285) > 0.000329$

$0.000342 > 0.000329$ ✓, no torsion Irreg.

H.2 Re-Design II: Structural Lateral Resisting Tilt-Up Walls

H.2.1 Loads Applied

(a) Columns

Table AH.16, Lateral Load Applied

Pier Assembly	Load Case	$F_{level,I}$ (Kip)					
		1	2	3	4	5	Roof
CS1P1 + CS1P2 + CS2P1 + CS2P2	WINDDX	25.01	23.97	28.71	29.39	31.36	27.53
	WINDDY	0.37	-1.9	-0.1	-0.5	-0.59	0.15
	WINDT1DX	22.45	18.12	22.09	22.55	24.33	21.03
	WINDT1DY	18.51	-0.82	2.52	1.93	3.24	2.85
	WINDT2	36.18	13.19	19.25	19.07	21.79	18.69
	WINDDXY	19.04	16.54	21.46	21.66	23.08	20.76
	WINDT1DNX	15.04	17.83	20.98	21.52	22.72	20.27
	WINDT1DNY	-17.91	-2.06	-2.65	-2.72	-4.11	-2.62
CN3P1 + CN2P1 + CN1P1	WINDDX	30.1	24.5	27.05	28.68	28.65	35.27
	WINDDY	-0.31	1.84	0.12	0.5	0.58	-0.14
	WINDT1DX	18.84	18.25	19.72	21.01	20.7	26.05
	WINDT1DY	-18.62	0.88	-2.52	-1.94	-3.17	-2.9
	WINDT2	-5.36	14.18	12.11	13.6	12.03	16.57
	WINDDXY	22.34	19.76	20.37	21.9	21.91	26.35
	WINDT1DNX	26.31	18.51	20.84	22.04	22.27	26.84
	WINDT1DNY	18.15	1.88	2.69	2.7	4.04	2.69
CE4P1 + CE5P1 + CE5P2 + CE5P3 + CE5P4 + CE6P1	WINDDX	0.32	0.33	-0.7	-0.39	-0.82	1.54
	WINDDY	61.62	62.03	65.79	68.83	70.20	99.72
	WINDT1DX	0.54	2.41	1.61	1.99	1.59	3.65
	WINDT1DY	47.52	56.62	59.17	62.04	62.77	90.21
	WINDT2	36.44	47.33	48.56	51.21	51.33	74.9
	WINDDXY	46.44	46.78	48.82	51.33	52.04	75.95
	WINDT1DNX	-0.07	-1.9	-2.64	-2.56	-2.85	-1.33
	WINDT1DNY	44.94	36.43	39.5	41.19	42.56	59.37
CW4P1 + CW5P1 + CW5P2 + CW5P3 + CW5P4 + CW6P1	WINDDX	-0.29	-0.36	0.72	0.37	0.84	-1.55
	WINDDY	61.28	60.73	65.67	68.31	70.26	99.46
	WINDT1DX	-0.51	-2.44	-1.57	-1.98	-1.62	-3.65
	WINDT1DY	44.86	35.33	39.45	40.8	42.53	59.23
	WINDT2	32.92	21.56	25.41	25.94	27.58	37.22
	WINDDXY	45.73	45.29	49.78	51.52	53.33	73.43
	WINDT1DNX	0.05	1.9	2.66	2.56	2.86	1.32

	WINDT1DNY	47.03	55.81	59.05	61.66	62.87	89.96

Table AH.17, Gravity Load on Piers			
Pier Type	Lateral Function (Y or N)	Maximum Dead Load (kip)	Maximum Live Load (kip)
CE-1	N	84.97	39.67
CE-2	N	290.89	134.24
CE-3	N	441.14	199.56
CE-4	Y	148.46	64.13
CE-5	Y	204.16	92.96
CE-6	Y	224.78	100.26
CN-2	Y	125.03	56.62
CN-3	Y	368.24	166.52
CN-4	Y	378.57	170.75
CS-1	Y	343.13	155.38
CS-2	Y	607.68	266.88

(b) Beams

Table AH.18, Applied Gravity Loads							
Member Designation	Level	Concentrated Gravity Loads				Distributed Gravity Load	
		Dead		Live		Dead (Kip/ft)	Live (Kip/ft)
		Pos. (ft)	Mag. (Kip)	Pos. (ft)	Mag. (Kip)		
BLS2P2T1P2	1	0.92	116.07	0.92	55.18	0	0
	2	0.92	115.04	0.92	57.93	0	0
	3	0.92	111.56	0.92	57.28	0	0
	4	0.92	108.66	0.92	57.20	0	0
	5	0.92	108.49	0.92	57.28	0	0
	6	0.92	113.72	0.92	14.55	0	0
BLN1P1T2P1	1	9	37.61	9	18.21	0	0
	2	9	35.82	9	18.33	0	0
	3	9	34.52	9	18.01	0	0
	4	9	33.62	9	18.03	0	0
	5	9	33.75	9	18.00	0	0
	6	9	34.13	9	4.54	0	0
BLN2P1T3P1	1	6	38.87	6	18.78	0	0
	2	6	35.90	6	18.28	0	0
	3	6	32.07	6	16.60	0	0
	4	6	31.24	6	16.64	0	0

	5	6	31.53	6	16.59	0	0
	6	6	33.08	6	4.47	0	0
BLEW	1	0	0	0	0	0.81	0.41
	2	0	0	0	0	0.80	0.43
	3	0	0	0	0	0.78	0.43
	4	0	0	0	0	0.76	0.43
	5	0	0	0	0	0.76	0.43
	6	0	0	0	0	0.78	0.11

H.2.2 Design Loads and Limtations

(a) Columns

Table AH.19, Base In-Plane Shear and Overturning

Pier Assembly	Load Case	V_{base} (Kip)	M_{base} (Kip-ft)	Pier Assembly	Load Case	V_{base} (Kip)	M_{base} (Kip-ft)
CS1P1 + CS1P2 + CS2P1 + CS2P2	WINDDX	165.9	8712.6	CE4P1 + CE5P1 + CE5P2 + CE5P3 + CE5P4 + CE6P1	WINDDX	0.2	35.0
	WINDDY	-2.5	-114.0		WINDDY	428.1	23364.0
	WINDT1DX	130.5	6743.0		WINDT1DX	11.7	695.5
	WINDT1DY	28.2	972.7		WINDT1DY	378.3	20938.2
	WINDT2	128.1	6103.8		WINDT2	309.7	17246.9
	WINDDXY	122.5	6448.4		WINDDXY	321.3	17550.2
	WINDT1DNX	118.3	6325.8		WINDT1DNX	-11.3	-642.3
	WINDT1DNY	-32	-1143.9		WINDT1DNY	263.9	14109.1
CN3P1 + CN2P1 + CN1P1	WINDDX	174.2	9166.2	CW4P1 + CW5P1 + CW5P2 + CW5P3 + CW5P4 + CW6P1	WINDDX	-0.3	-35.1
	WINDDY	2.5	114.2		WINDDY	425.7	23266.1
	WINDT1DX	124.5	6665.9		WINDT1DX	-11.7	-695.8
	WINDT1DY	-28.2	-972.5		WINDT1DY	262.2	14035.8
	WINDT2	63.1	3952.4		WINDT2	170.6	8982.7
	WINDDXY	132.6	6960.3		WINDDXY	319.0	17423.6
	WINDT1DNX	136.8	7083.2		WINDT1DNX	11.3	642.7
	WINDT1DNY	32.1	1143.9		WINDT1DNY	376.3	20864.4

Table AH.20, Maximum Factored In-Plane Shear and Moments

Pier	Floor Level	Width (in)	Maximum Shear (Kip)	Maximum Moment (Kip-ft)
CE1	1	30	0.00	0.00
	3		0.00	0.00
	5		0.00	0.00
CE2	1	27	0.00	0.00

	3		0.00	0.00
	5		0.00	0.00
CE3	1	28	0.00	0.00
	3		0.00	0.00
	5		0.00	0.00
CE4	1	46.5	113.53	604.48
	3		69.63	325.70
	5		39.64	160.71
CE5	1	43	196.30	706.72
	3		172.80	604.79
	5		126.11	431.40
CE6	1	37.5	69.77	339.22
	3		57.11	243.60
	5		36.53	136.24
CN2	1	76	101.52	647.88
	3		59.03	255.86
	5		34.87	221.70
CN3	1	57	86.67	416.40
	3		59.38	269.97
	5		23.41	129.77
CN4	1	72	90.99	639.01
	3		31.49	157.91
	5		8.64	138.43
CS1	1	27	47.77	167.66
	3		41.56	162.33
	5		27.40	97.13
CS2	1	87	121.65	867.33
	3		82.57	401.42
	5		50.19	267.11

Table AH.21, Axial Load

Pier	Lateral Function (Y or N)	Maximum Dead Load (kip)	Maximum Live Load (kip)
CE1	N	84.97	39.67
CE2	N	290.89	134.24
CE3	N	441.14	199.56
CE4	Y	148.46	64.13
CE5	Y	204.16	92.96
CE6	Y	224.78	100.26

CN2	Y	125.03	56.62
CN3	Y	368.24	166.52
CN4	Y	378.57	170.75
CS1	Y	343.13	155.38
CS2	Y	607.68	266.88

(b) Beams

Table AH.22, Maximum Factored In-Plane Shear and Moments

Beam	Floor Level	Length (ft)	Height (in)	Maximum Shear (Kip)	Maximum Moment (Kip-ft)	Length-to-Height Ratio	Beam Type
BLS2P2T1P2	1	7.167	48	197.98	780.02	1.79	Deep
	3	7.167	36	133.66	472.54	2.39	Deep
	5	7.167	36	125.44	439.08	2.39	Deep
BLN1P1T2P1	1	14	48	107.46	550.95	3.50	Deep
	3	14	36	66.21	298.06	4.67	
	5	14	36	57.15	237.83	4.67	
BLN2P1T3P1	1	14	48	80.97	429.96	3.50	Deep
	3	14	36	41.58	212.62	4.67	
	5	14	36	30.17	133.15	4.67	
BLEW	1	6.667	48	279.89	925.03	1.67	Deep
	3	6.667	36	210.33	692.27	2.22	Deep
	5	6.667	36	189.46	622.56	2.22	Deep
BG	1		48	148.94	456.21		
	3		36	177.72	367.62		
	5		36	159.64	371.37		

H.2.3 Structural Tilt-Up Wall Design

(a) Columns

Note: It was determined that loads acting on the tilt-up walls during construction were much greater than during full occupancy. As a result the column design by RAM Elements wasn't used, because the software doesn't consider loads during the lifting process. The design done in Microsoft Excel and by RAM Elements – in this section – are for comparision to show the significant effect of the lifting process. Actual design of columns in the tilt-up wall can be found in Appendix I.

Table AH.23, Pier Shear Reinforcement Design (Part 1)

Story	Pier	Length (in)	Thk (in)	d (in)	f'_c (Kip/in^2)	$F_{V\text{-}MAX}$ (Kip)
Story 6	CE-4	46.5	10	38.40	6	4.64
Story 5						9.65
Story 4						7.50
Story 3						8.39
Story 2						5.15
Story 1						32.22
Story 6	CE-5	43	10	36.80	6	28.17
Story 5						13.88
Story 4						14.11
Story 3						13.26
Story 2						14.35
Story 1						7.89
Story 6	CE-6	37.5	10	29.60	6	4.06
Story 5						6.68
Story 4						6.00
Story 3						6.16
Story 2						6.26
Story 1						11.32
Story 6	CN-2	76	10	62.40	6	6.99
Story 5						9.35
Story 4						7.91
Story 3						8.14
Story 2						7.18
Story 1						20.05
Story 6	CN-3	57	10	48.00	6	21.43
Story 5						9.68
Story 4						12.76
Story 3						10.57
Story 2						10.04
Story 1						-11.27
Story 6	CN-4	72	10	60.00	6	6.85
Story 5						9.62
Story 4						8.01
Story 3						8.34
Story 2						7.28
Story 1						21.32
Story 6	CS-1	27	10	24.00	6	10.62
Story 5						4.19

Story 4						5.41
Story 3						4.58
Story 2						2.15
Story 1						12.79
Story 6	CS-2	87	10	67.20	6	3.24
Story 5						11.64
Story 4						9.37
Story 3						9.88
Story 2						9.99
Story 1						25.84

Table AH.24, Pier Shear Reinforcement Design (Part 2)

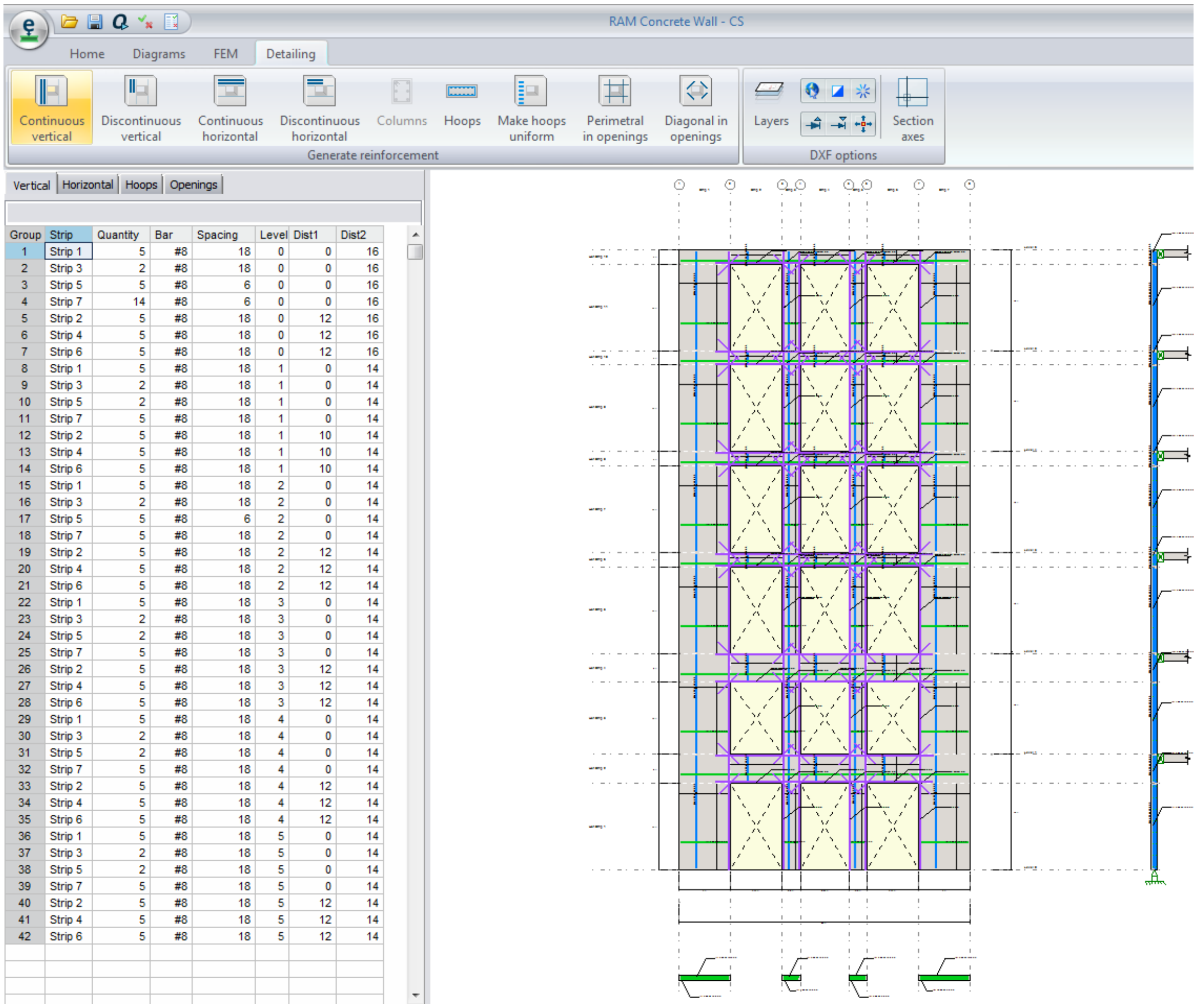
Story	Pier	$V_{u,max}$ (Kip)	V_c (Kip) ACI 318-11 §11.4.6.1	$\Phi V_{c,n}$ (Kip) ACI 318-11 §11.2.1.1	$V_{s,req}$ (Kip) ACI 318-11 §11.4.7.2
Story 6	CE-4	4.64	59.49	22.31	0.00
Story 5		14.29			0.00
Story 4		21.79			0.00
Story 3		30.18			17.93
Story 2		35.33			24.80
Story 1		67.55			67.76
Story 6	CE-5	28.17	57.01	21.38	16.18
Story 5		42.05			34.69
Story 4		56.16			53.50
Story 3		69.42			71.18
Story 2		83.77			90.31
Story 1		91.66			100.83
Story 6	CE-6	4.06	45.86	17.20	0.00
Story 5		10.74			0.00
Story 4		16.74			0.00
Story 3		22.90			13.34
Story 2		29.16			21.68
Story 1		40.48			36.78
Story 6	CN-2	6.99	96.67	36.25	0.00
Story 5		16.34			0.00
Story 4		24.25			0.00
Story 3		32.39			0.00
Story 2		39.57			16.51
Story 1		59.62			43.24
Story 6	CN-3	21.43	74.36	27.89	0.00
Story 5		31.11			13.59

Story 4		43.87			30.61
Story 3		54.44			44.70
Story 2		64.48			58.09
Story 1		53.21			43.06
Story 6	CN-4	6.85	92.95	34.86	0.00
Story 5		16.47			0.00
Story 4		24.48			0.00
Story 3		32.82			0.00
Story 2		40.10			18.61
Story 1		61.42			47.04
Story 6	CS-1	10.62	37.18	13.94	0.00
Story 5		14.81			5.80
Story 4		20.22			13.02
Story 3		24.80			19.12
Story 2		26.95			21.99
Story 1		39.74			39.04
Story 6	CS-2	3.24	104.11	39.04	0.00
Story 5		14.88			0.00
Story 4		24.25			0.00
Story 3		34.13			0.00
Story 2		44.12			19.79
Story 1		69.96			54.24

Table AH.25, Pier Shear Reinforcement Design (Part 3)

Story	Pier	S_{max} (in)		S_{design} (in) ACI 318-11 § 11.4.7.2	S_{actual} (in)
		ACI 318-11 § 11.4.5.1,11.4.5.3	ACI 318-11 §7.10.5.2		
Story 6	CE-4	19.2	10	N/A	10.0
Story 5		19.2		N/A	10.0
Story 4		19.2		N/A	10.0
Story 3		19.2		28.27	10.0
Story 2		19.2		20.44	10.0
Story 1		19.2		7.48	7.0
Story 6	CE-5	18.4		30.02	10.0
Story 5		18.4		14.00	10.0
Story 4		18.4		9.08	9.0
Story 3		18.4		6.82	6.0
Story 2		18.4		5.38	5.0
Story 1		18.4		4.82	4.0
Story 6	CE-6	14.8		N/A	10.0
Story 5		14.8		N/A	10.0

Story 4		14.8		N/A	10.0
Story 3		14.8		29.30	10.0
Story 2		14.8		18.02	10.0
Story 1		14.8		10.62	10.0
Story 6	CN-2	24		N/A	10.0
Story 5		24		N/A	10.0
Story 4		24		N/A	10.0
Story 3		24		N/A	10.0
Story 2		24		49.89	10.0
Story 1		24		19.05	10.0
Story 6	CN-3	24		N/A	10.0
Story 5		24		46.61	10.0
Story 4		24		20.70	10.0
Story 3		24		14.17	10.0
Story 2		24		10.91	10.0
Story 1		24		14.71	10.0
Story 6	CN-4	24		N/A	10.0
Story 5		24		N/A	10.0
Story 4		24		N/A	10.0
Story 3		24		N/A	10.0
Story 2		24		42.56	10.0
Story 1		24		16.84	10.0
Story 6	CS-1	12		N/A	10.0
Story 5		12		54.58	10.0
Story 4		12		24.34	10.0
Story 3		12		16.57	10.0
Story 2		12		14.41	10.0
Story 1		12		8.11	8.0
Story 6	CS-2	24		N/A	10.0
Story 5		24		N/A	10.0
Story 4		24		N/A	10.0
Story 3		24		N/A	10.0
Story 2		24		44.83	10.0
Story 1		24		16.35	10.0



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	5	#8	18	0	0	16
2	Strip 3	2	#8	18	0	0	16
3	Strip 5	5	#8	6	0	0	16
4	Strip 7	14	#8	6	0	0	16
5	Strip 2	5	#8	18	0	12	16
6	Strip 4	5	#8	18	0	12	16
7	Strip 6	5	#8	18	0	12	16
8	Strip 1	5	#8	18	1	0	14
9	Strip 3	2	#8	18	1	0	14
10	Strip 5	2	#8	18	1	0	14
11	Strip 7	5	#8	18	1	0	14
12	Strip 2	5	#8	18	1	10	14
13	Strip 4	5	#8	18	1	10	14
14	Strip 6	5	#8	18	1	10	14
15	Strip 1	5	#8	18	2	0	14
16	Strip 3	2	#8	18	2	0	14
17	Strip 5	5	#8	6	2	0	14
18	Strip 7	5	#8	18	2	0	14
19	Strip 2	5	#8	18	2	12	14
20	Strip 4	5	#8	18	2	12	14
21	Strip 6	5	#8	18	2	12	14
22	Strip 1	5	#8	18	3	0	14
23	Strip 3	2	#8	18	3	0	14
24	Strip 5	2	#8	18	3	0	14
25	Strip 7	5	#8	18	3	0	14
26	Strip 2	5	#8	18	3	12	14
27	Strip 4	5	#8	18	3	12	14
28	Strip 6	5	#8	18	3	12	14
29	Strip 1	5	#8	18	4	0	14
30	Strip 3	2	#8	18	4	0	14
31	Strip 5	2	#8	18	4	0	14
32	Strip 7	5	#8	18	4	0	14
33	Strip 2	5	#8	18	4	12	14
34	Strip 4	5	#8	18	4	12	14
35	Strip 6	5	#8	18	4	12	14
36	Strip 1	5	#8	18	5	0	14
37	Strip 3	2	#8	18	5	0	14
38	Strip 5	2	#8	18	5	0	14
39	Strip 7	5	#8	18	5	0	14
40	Strip 2	5	#8	18	5	12	14
41	Strip 4	5	#8	18	5	12	14
42	Strip 6	5	#8	18	5	12	14

Figure AH.17A, Vertical Reinforcement Design for CS

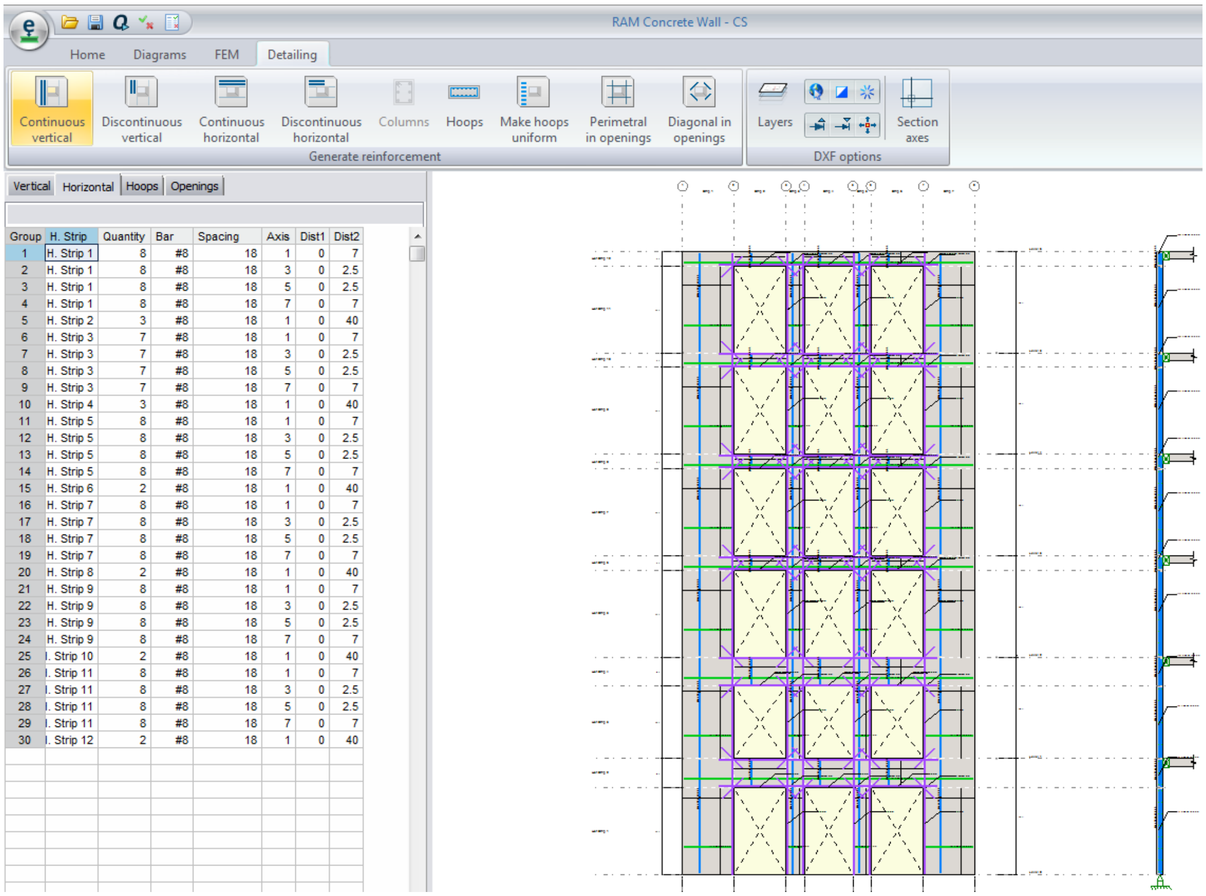
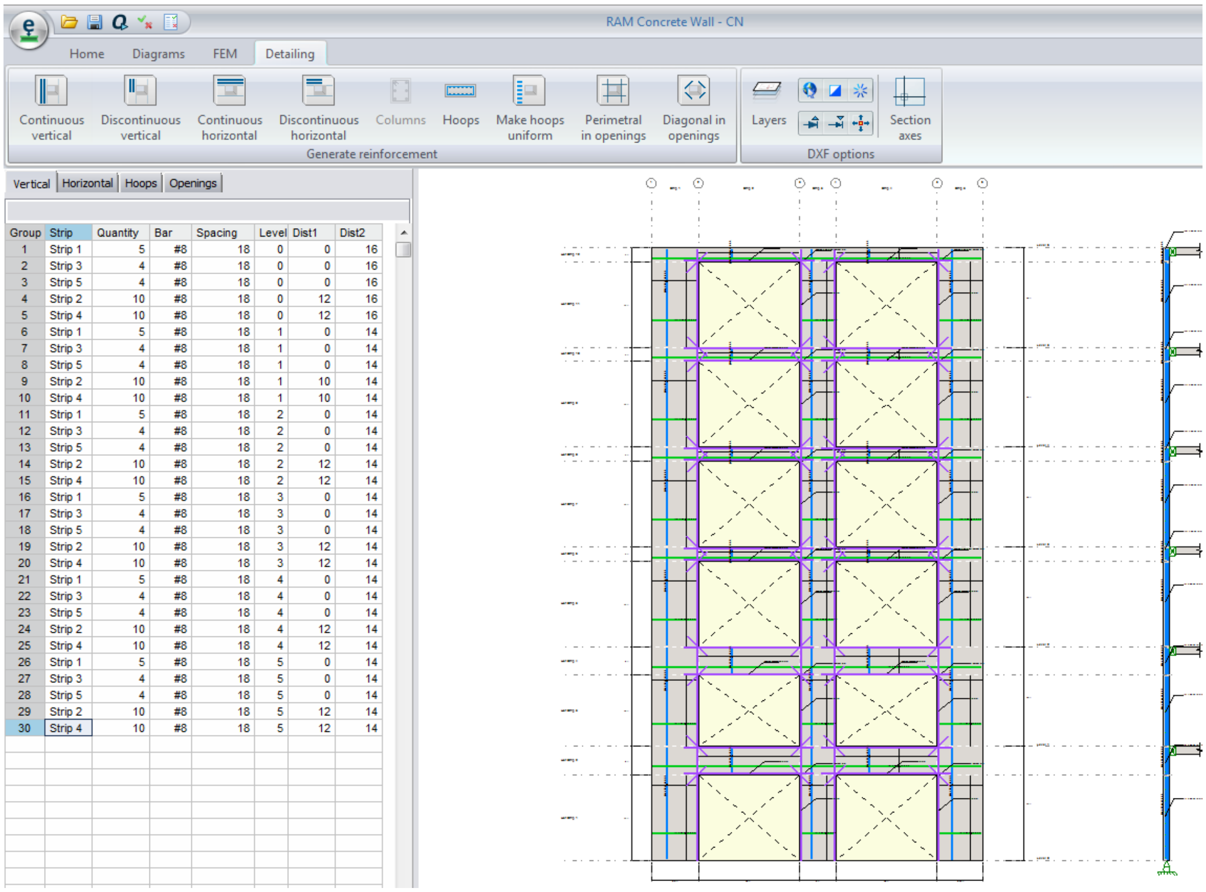
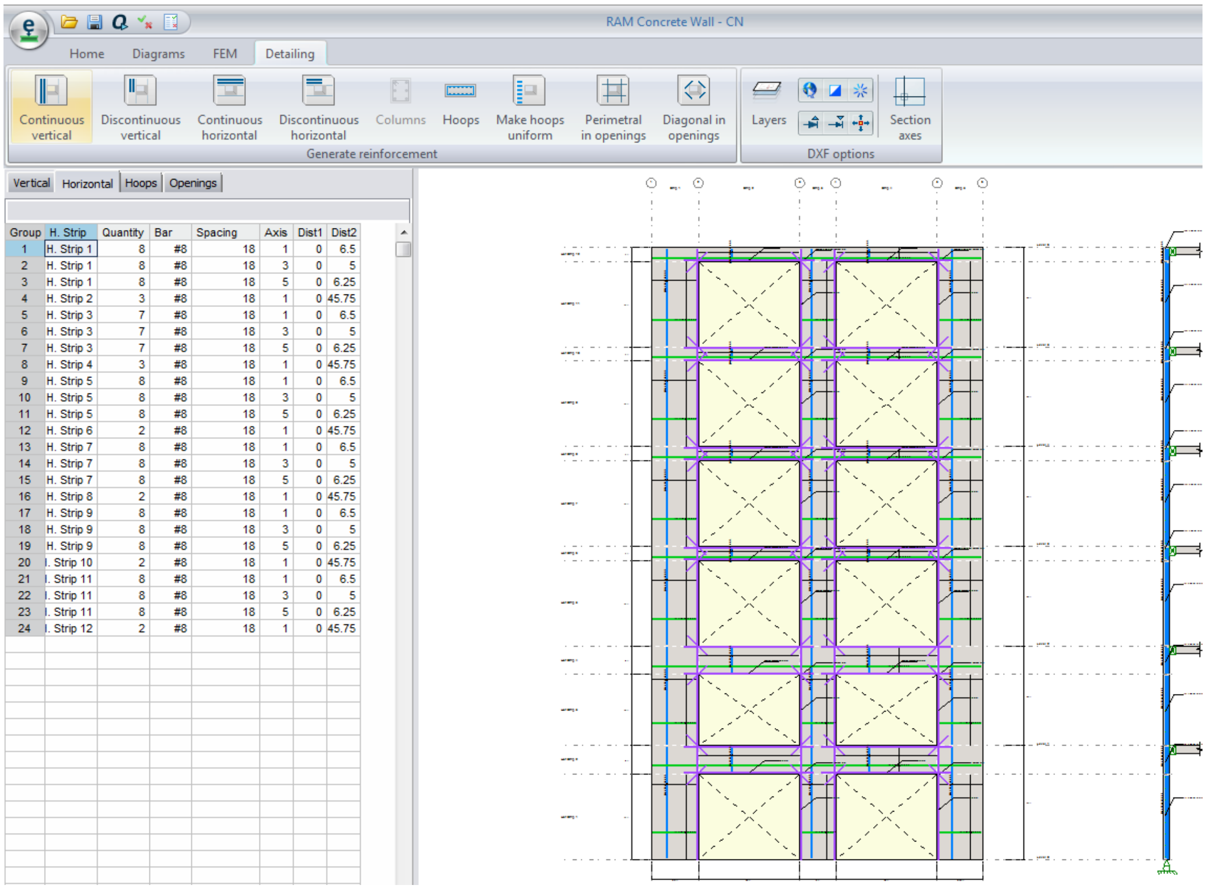


Figure AH.17B, Horizontal Reinforcement Design for CS



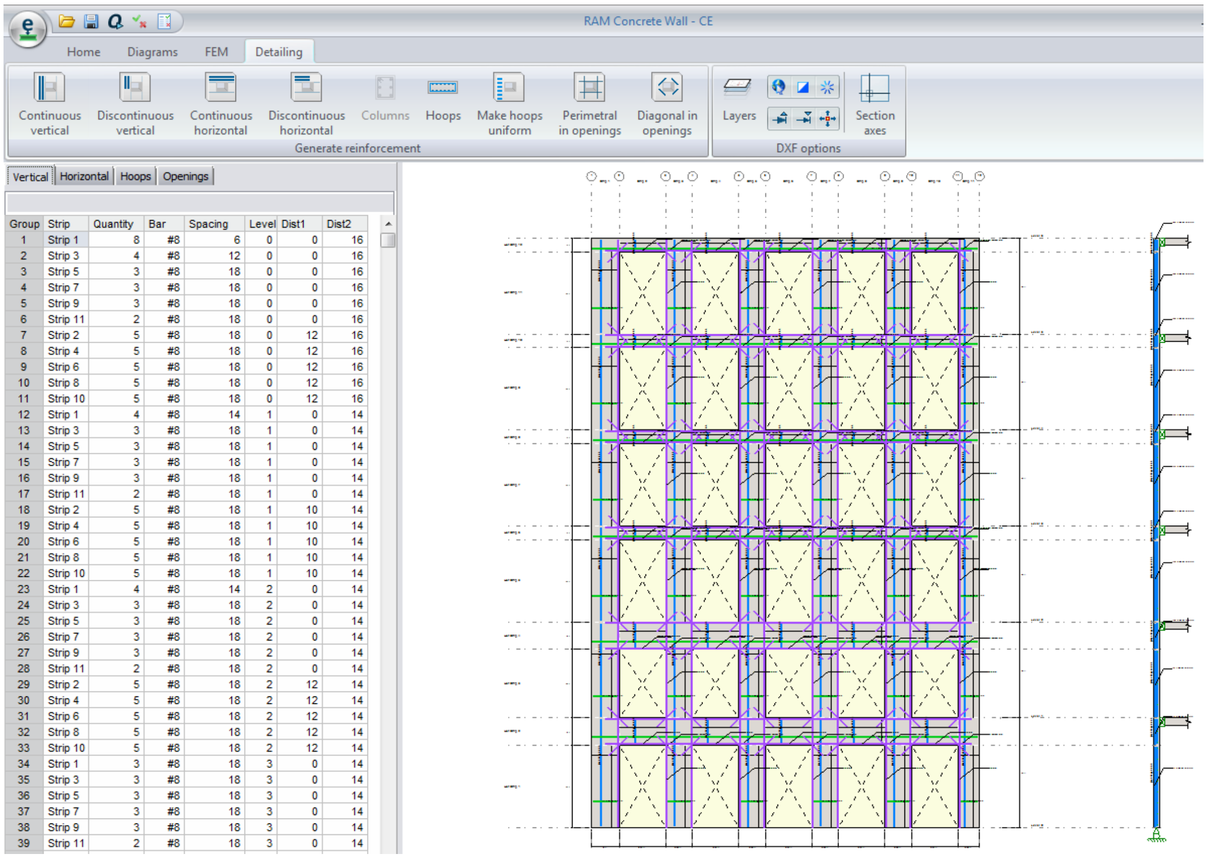
Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	5	#8	18	0	0	16
2	Strip 3	4	#8	18	0	0	16
3	Strip 5	4	#8	18	0	0	16
4	Strip 2	10	#8	18	0	12	16
5	Strip 4	10	#8	18	0	12	16
6	Strip 1	5	#8	18	1	0	14
7	Strip 3	4	#8	18	1	0	14
8	Strip 5	4	#8	18	1	0	14
9	Strip 2	10	#8	18	1	10	14
10	Strip 4	10	#8	18	1	10	14
11	Strip 1	5	#8	18	2	0	14
12	Strip 3	4	#8	18	2	0	14
13	Strip 5	4	#8	18	2	0	14
14	Strip 2	10	#8	18	2	12	14
15	Strip 4	10	#8	18	2	12	14
16	Strip 1	5	#8	18	3	0	14
17	Strip 3	4	#8	18	3	0	14
18	Strip 5	4	#8	18	3	0	14
19	Strip 2	10	#8	18	3	12	14
20	Strip 4	10	#8	18	3	12	14
21	Strip 1	5	#8	18	4	0	14
22	Strip 3	4	#8	18	4	0	14
23	Strip 5	4	#8	18	4	0	14
24	Strip 2	10	#8	18	4	12	14
25	Strip 4	10	#8	18	4	12	14
26	Strip 1	5	#8	18	5	0	14
27	Strip 3	4	#8	18	5	0	14
28	Strip 5	4	#8	18	5	0	14
29	Strip 2	10	#8	18	5	12	14
30	Strip 4	10	#8	18	5	12	14

Figure AH.17C, Vertical Reinforcement Design for CN



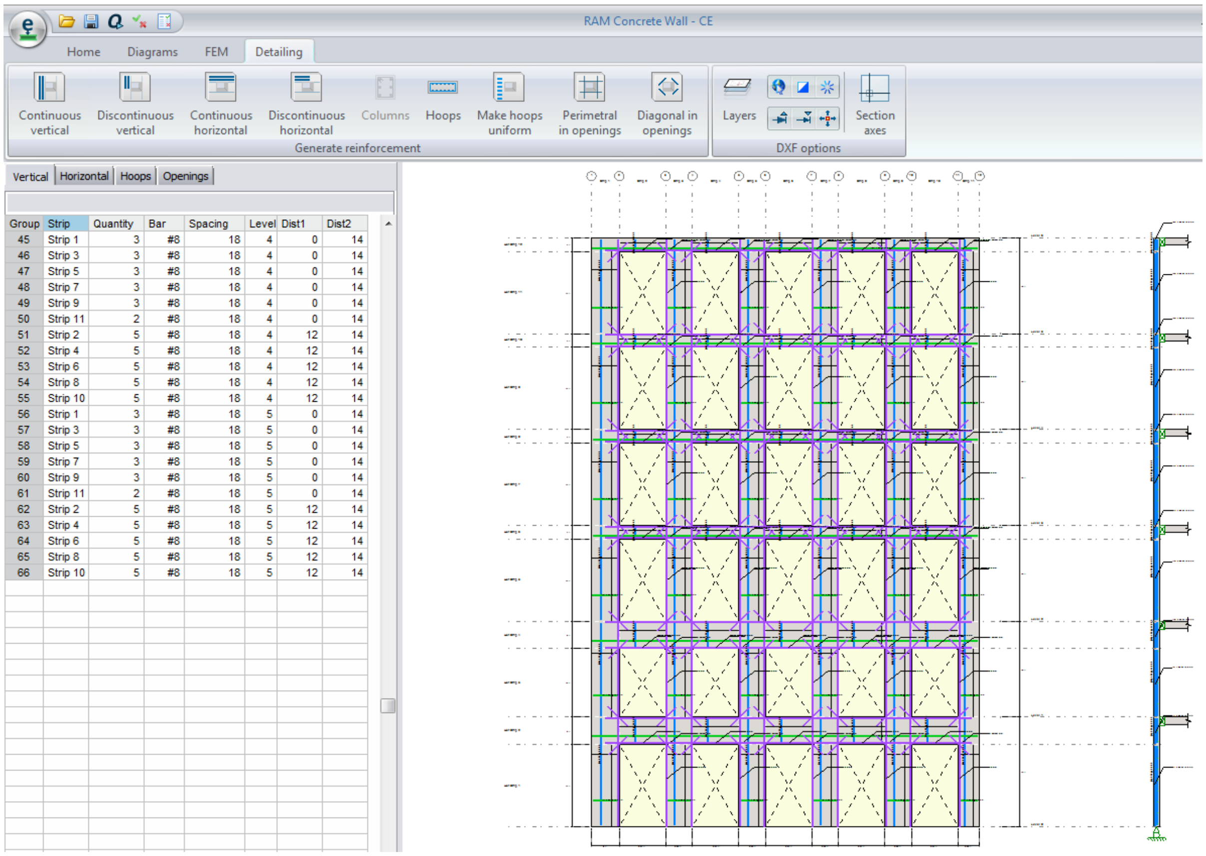
Group	H. Strip	Quantity	Bar	Spacing	Axis	Dist1	Dist2
1	H. Strip 1	8	#8	18	1	0	6.5
2	H. Strip 1	8	#8	18	3	0	5
3	H. Strip 1	8	#8	18	5	0	6.25
4	H. Strip 2	3	#8	18	1	0	45.75
5	H. Strip 3	7	#8	18	1	0	6.5
6	H. Strip 3	7	#8	18	3	0	5
7	H. Strip 3	7	#8	18	5	0	6.25
8	H. Strip 4	3	#8	18	1	0	45.75
9	H. Strip 5	8	#8	18	1	0	6.5
10	H. Strip 5	8	#8	18	3	0	5
11	H. Strip 5	8	#8	18	5	0	6.25
12	H. Strip 6	2	#8	18	1	0	45.75
13	H. Strip 7	8	#8	18	1	0	6.5
14	H. Strip 7	8	#8	18	3	0	5
15	H. Strip 7	8	#8	18	5	0	6.25
16	H. Strip 8	2	#8	18	1	0	45.75
17	H. Strip 9	8	#8	18	1	0	6.5
18	H. Strip 9	8	#8	18	3	0	5
19	H. Strip 9	8	#8	18	5	0	6.25
20	I. Strip 10	2	#8	18	1	0	45.75
21	I. Strip 11	8	#8	18	1	0	6.5
22	I. Strip 11	8	#8	18	3	0	5
23	I. Strip 11	8	#8	18	5	0	6.25
24	I. Strip 12	2	#8	18	1	0	45.75

Figure AH.17D, Horizontal Reinforcement Design for CN



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
1	Strip 1	8	#8	6	0	0	16
2	Strip 3	4	#8	12	0	0	16
3	Strip 5	3	#8	18	0	0	16
4	Strip 7	3	#8	18	0	0	16
5	Strip 9	3	#8	18	0	0	16
6	Strip 11	2	#8	18	0	0	16
7	Strip 2	5	#8	18	0	12	16
8	Strip 4	5	#8	18	0	12	16
9	Strip 6	5	#8	18	0	12	16
10	Strip 8	5	#8	18	0	12	16
11	Strip 10	5	#8	18	0	12	16
12	Strip 1	4	#8	14	1	0	14
13	Strip 3	3	#8	18	1	0	14
14	Strip 5	3	#8	18	1	0	14
15	Strip 7	3	#8	18	1	0	14
16	Strip 9	3	#8	18	1	0	14
17	Strip 11	2	#8	18	1	0	14
18	Strip 2	5	#8	18	1	10	14
19	Strip 4	5	#8	18	1	10	14
20	Strip 6	5	#8	18	1	10	14
21	Strip 8	5	#8	18	1	10	14
22	Strip 10	5	#8	18	1	10	14
23	Strip 1	4	#8	14	2	0	14
24	Strip 3	3	#8	18	2	0	14
25	Strip 5	3	#8	18	2	0	14
26	Strip 7	3	#8	18	2	0	14
27	Strip 9	3	#8	18	2	0	14
28	Strip 11	2	#8	18	2	0	14
29	Strip 2	5	#8	18	2	12	14
30	Strip 4	5	#8	18	2	12	14
31	Strip 6	5	#8	18	2	12	14
32	Strip 8	5	#8	18	2	12	14
33	Strip 10	5	#8	18	2	12	14
34	Strip 1	3	#8	18	3	0	14
35	Strip 3	3	#8	18	3	0	14
36	Strip 5	3	#8	18	3	0	14
37	Strip 7	3	#8	18	3	0	14
38	Strip 9	3	#8	18	3	0	14
39	Strip 11	2	#8	18	3	0	14

Figure AH.17E, Vertical Reinforcement (Part I) Design for CE



Group	Strip	Quantity	Bar	Spacing	Level	Dist1	Dist2
45	Strip 1	3	#8	18	4	0	14
46	Strip 3	3	#8	18	4	0	14
47	Strip 5	3	#8	18	4	0	14
48	Strip 7	3	#8	18	4	0	14
49	Strip 9	3	#8	18	4	0	14
50	Strip 11	2	#8	18	4	0	14
51	Strip 2	5	#8	18	4	12	14
52	Strip 4	5	#8	18	4	12	14
53	Strip 6	5	#8	18	4	12	14
54	Strip 8	5	#8	18	4	12	14
55	Strip 10	5	#8	18	4	12	14
56	Strip 1	3	#8	18	5	0	14
57	Strip 3	3	#8	18	5	0	14
58	Strip 5	3	#8	18	5	0	14
59	Strip 7	3	#8	18	5	0	14
60	Strip 9	3	#8	18	5	0	14
61	Strip 11	2	#8	18	5	0	14
62	Strip 2	5	#8	18	5	12	14
63	Strip 4	5	#8	18	5	12	14
64	Strip 6	5	#8	18	5	12	14
65	Strip 8	5	#8	18	5	12	14
66	Strip 10	5	#8	18	5	12	14

Figure AH.17F, Vertical Reinforcement (Part 2) Design for CE

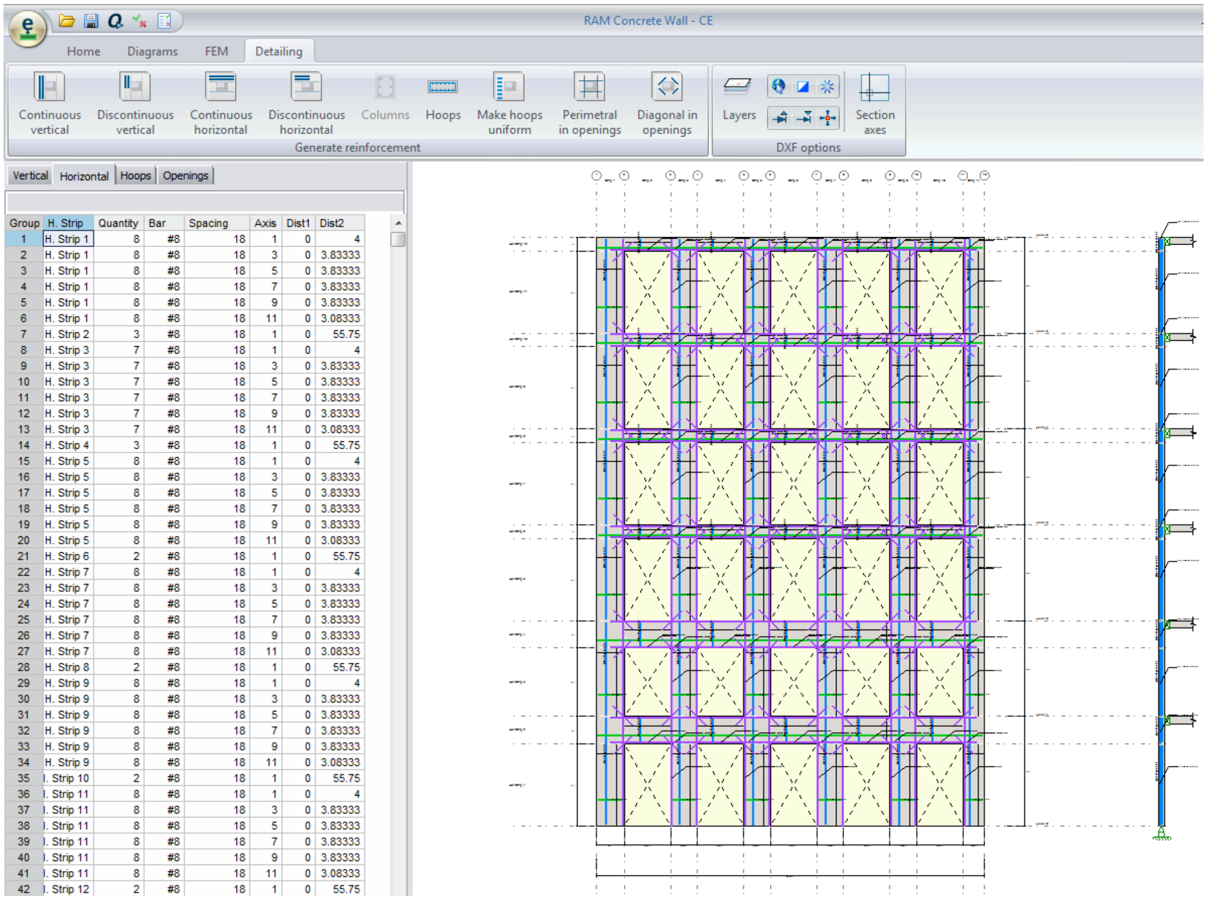


Figure AH.17G, Horizontal Reinforcement Design for CE

(b) Beams

Table AH.23, Beam Flexural Design Based on spBeam and RAM Elements Output				
Beam	Level	h (in)	Top Reinforcement	Bottom Reinforcement
BLS2P2T1P2	1	48	(10) #8	(10) #8
	3	36	(6) #8	(6) #8
	5	36	(6) #8	(6) #8
BLN1P1T2P1	1	48	(6) #8	(6) #8
	3	36	(2) #8	(2) #8
	5	36	(2) #8	(2) #8
BLN2P1T3P1	1	48	(6) #8	(6) #8
	3	36	(2) #8	(2) #8
	5	36	(2) #8	(2) #8
BLEW	1	48	(12) #8	(12) #8
	3	36	(8) #8	(8) #8
	5	36	(8) #8	(8) #8

(c) Servicability and Irregularity Check

Table AH.24, Wind Induced Deflection						
Story	Corner Point	Load Case	UX	UY	RX	RY
6	1	WINDDX	0.56	0	0	0
	1	WINDDY	0.01	0.84	0	0
	1	WINDT1DX	0.41	-0.03	0	0
	1	WINDT1DY	-0.06	0.5	0	0
	1	WINDT2	0.25	0.32	0	0
	1	WINDDXY	0.43	0.63	0	0
	1	WINDT1DNX	0.44	0.02	0	0
	1	WINDT1DNY	0.07	0.75	0	0
	6	WINDDX	0.56	0	0	0
	6	WINDDY	0.01	0.81	0	0
	6	WINDT1DX	0.41	0.02	0	0
	6	WINDT1DY	-0.06	0.73	0	0
	6	WINDT2	0.25	0.6	0	0
	6	WINDDXY	0.43	0.61	0	0
	6	WINDT1DNX	0.44	-0.02	0	0
	6	WINDT1DNY	0.07	0.49	0	0

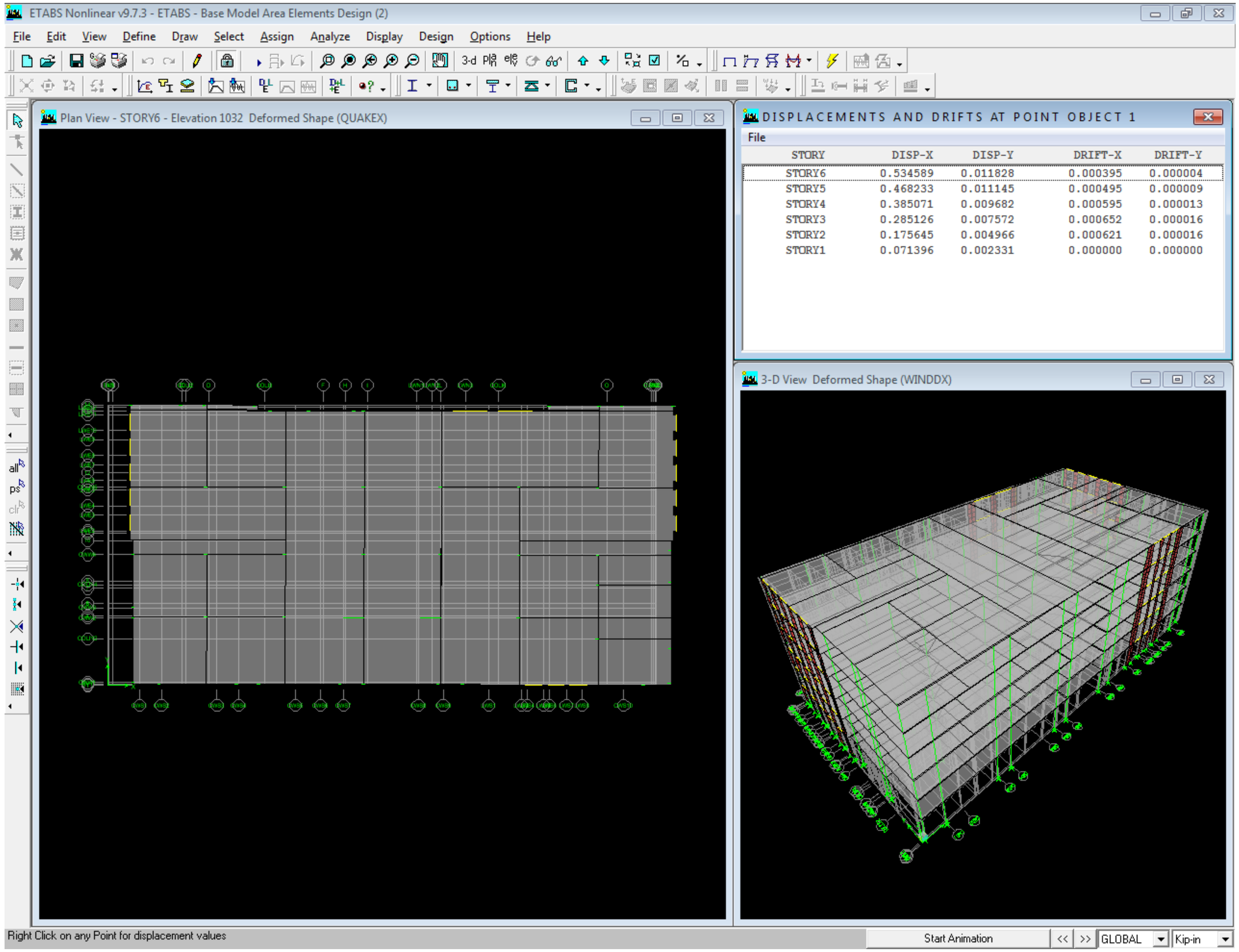


Figure AH.18, Northwest Corner Displacement for Quake in Long Direction
(Incl. Accidental Torsion from 5% Eccentricity)

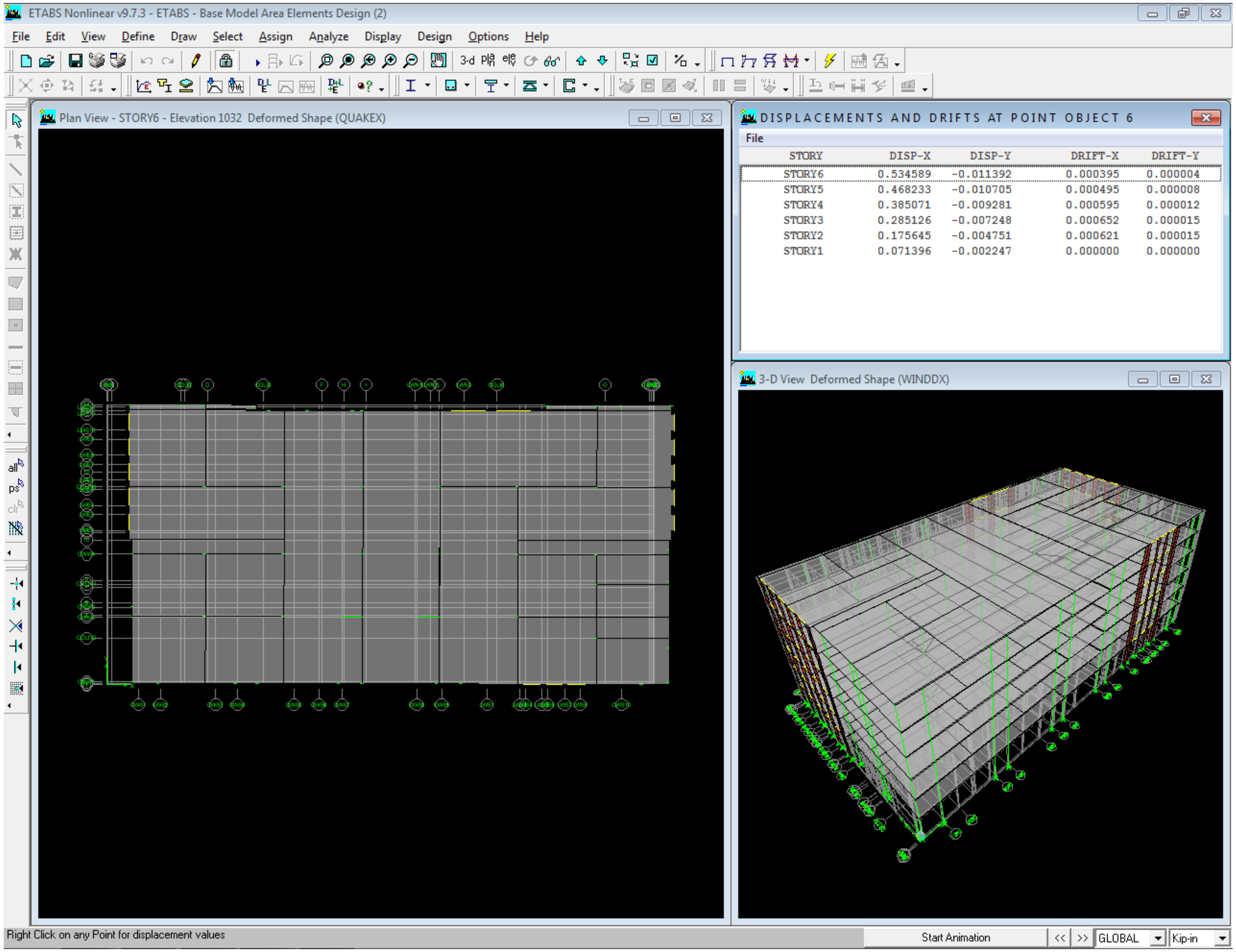


Figure AH.19, Northeast Corner Displacement for Quake in Long Direction
(Incl. Accidental Torsion from 5% Eccentricity)

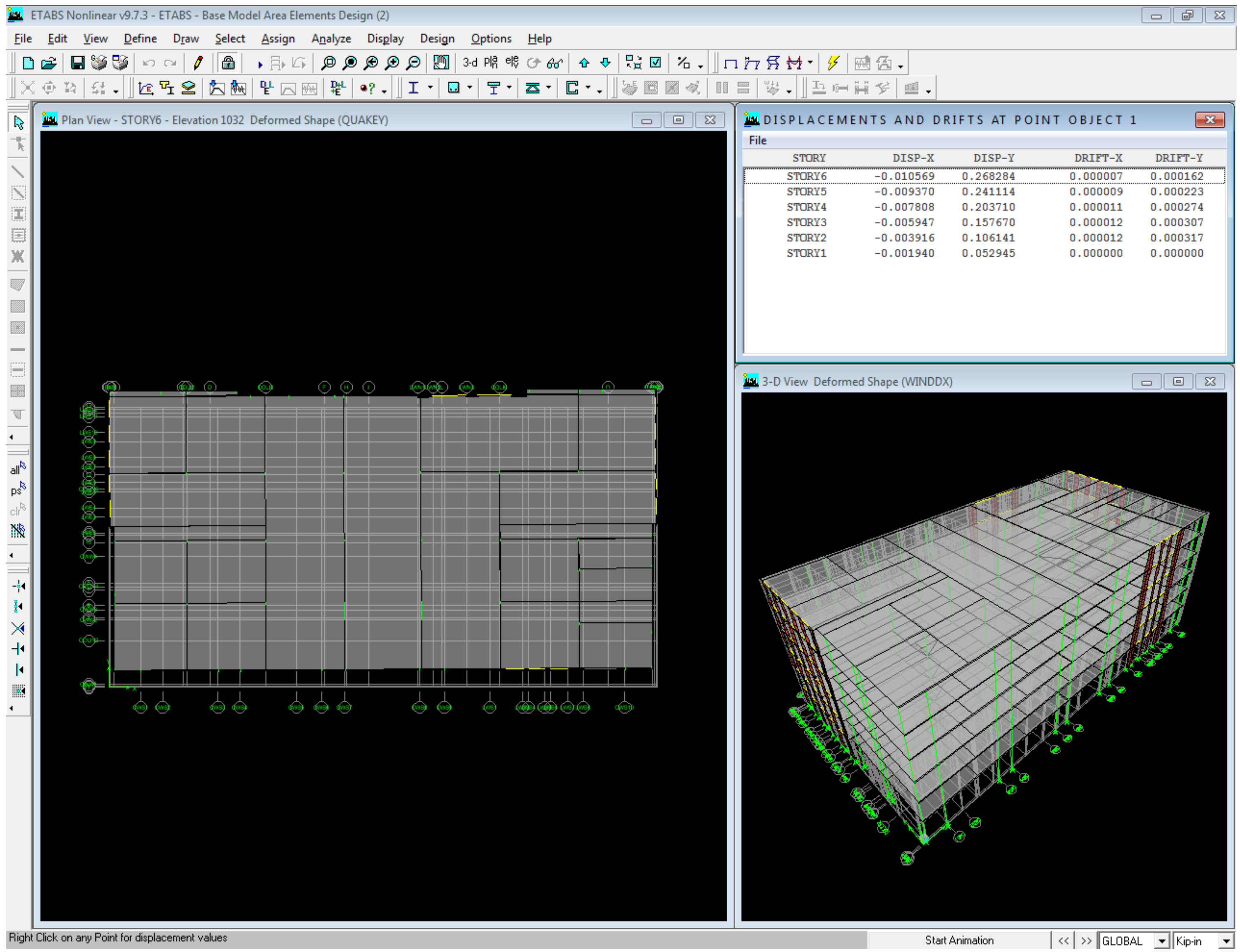


Figure AH.20, Northwest Corner Displacement for Quake in Short Direction
(Incl. Accidental Torsion from 5% Eccentricity)

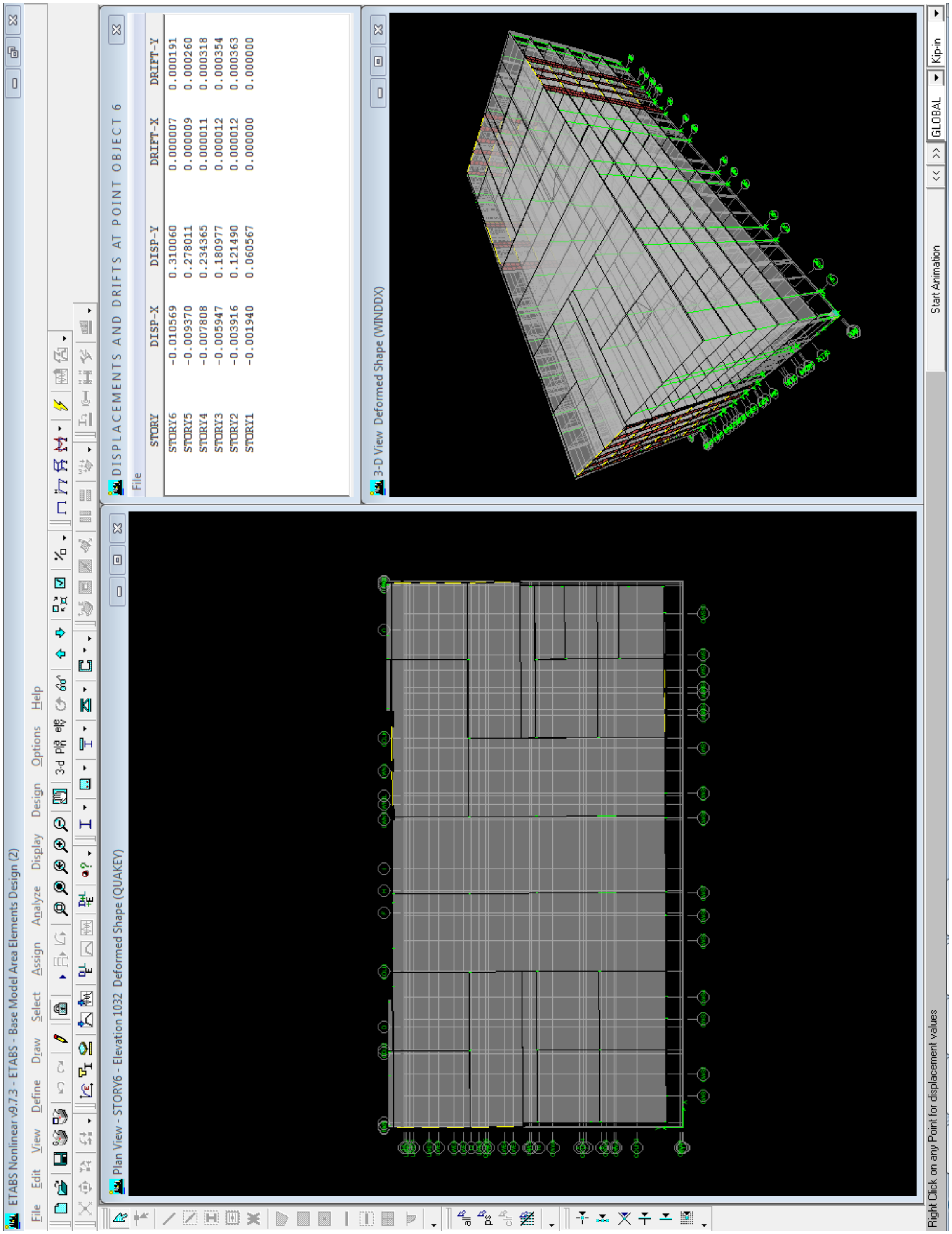


Figure AH.21, Northeast Corner Displacement for Quake in Short Direction
(Incl. Accidental Torsion from 5% Eccentricity)

Thaison Nguyen | Design II: TORSION IRREG. CHECK

** ASCE 7-05 Table 12.3-1 Horizontal Irreg. defines Torsional irreg. as drift at a structure's ends > 1.2 times the average drift of a structure's two ends.

Story	Average Drift (Quake in Long Direction w/ Torsion Ac.)		Average Drift (Quake in Short Direction w/ Torsion Ac.)	
	x	y	x	y
2	0.000621	0.000015	0.000012	0.000340
3	0.000652	0.000015	0.000012	0.000331
4	0.000595	0.000012	0.000011	0.000296
5	0.000495	0.000008	0.000009	0.000242
6	0.000395	0.000004	0.000007	0.000177

a) Check Quake in Long Direction w/ Torsion Ac.

*** By visual inspection there is no torsion irreg. at any story when bldg. is exposed to quake in long bldg. direction (w/ torsion Ac.)

b) Check Quake in Short Direction w/ Torsion Ac.

1) Story 2

$1.2(0.00034) > 0.000363$

$0.000408 > 0.000363$ ✓, no torsion irreg.

2) Story 3

$1.2(0.000331) > 0.000354$

$0.000397 > 0.000354$ ✓, no torsion irreg.

3) Story 4

$1.2(0.000296) > 0.000318$

$0.000355 > 0.000318$ ✓, no torsion irreg.

4) Story 5

$1.2(0.000242) > 0.000260$

$0.000290 > 0.000260$ ✓, no torsion irreg.

5) Story 6

$1.2(0.000177) > 0.000191$

$0.000212 > 0.000191$ ✓, no torsion irreg.

Appendix I: Gravity Design for Design II

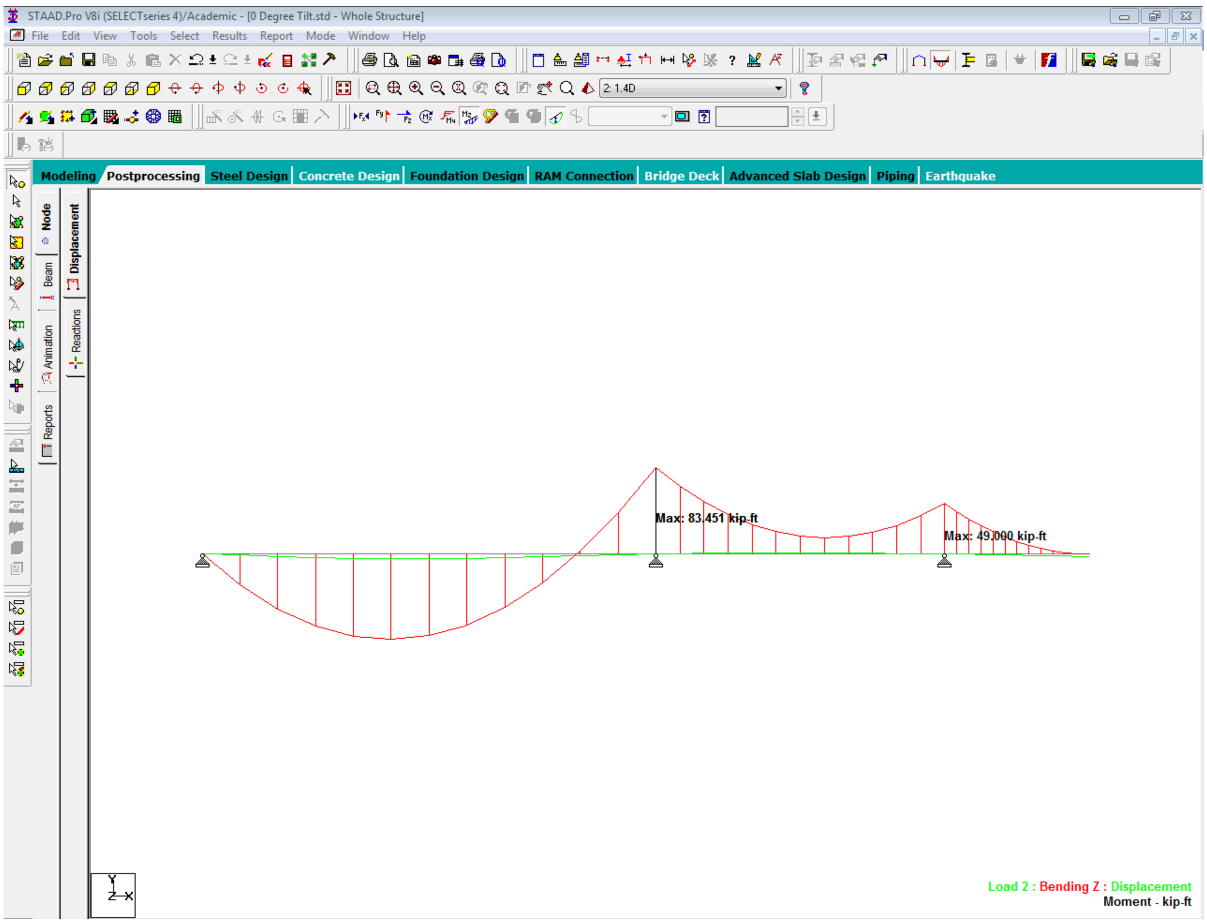
I.1 Structural Tilt-Up Wall

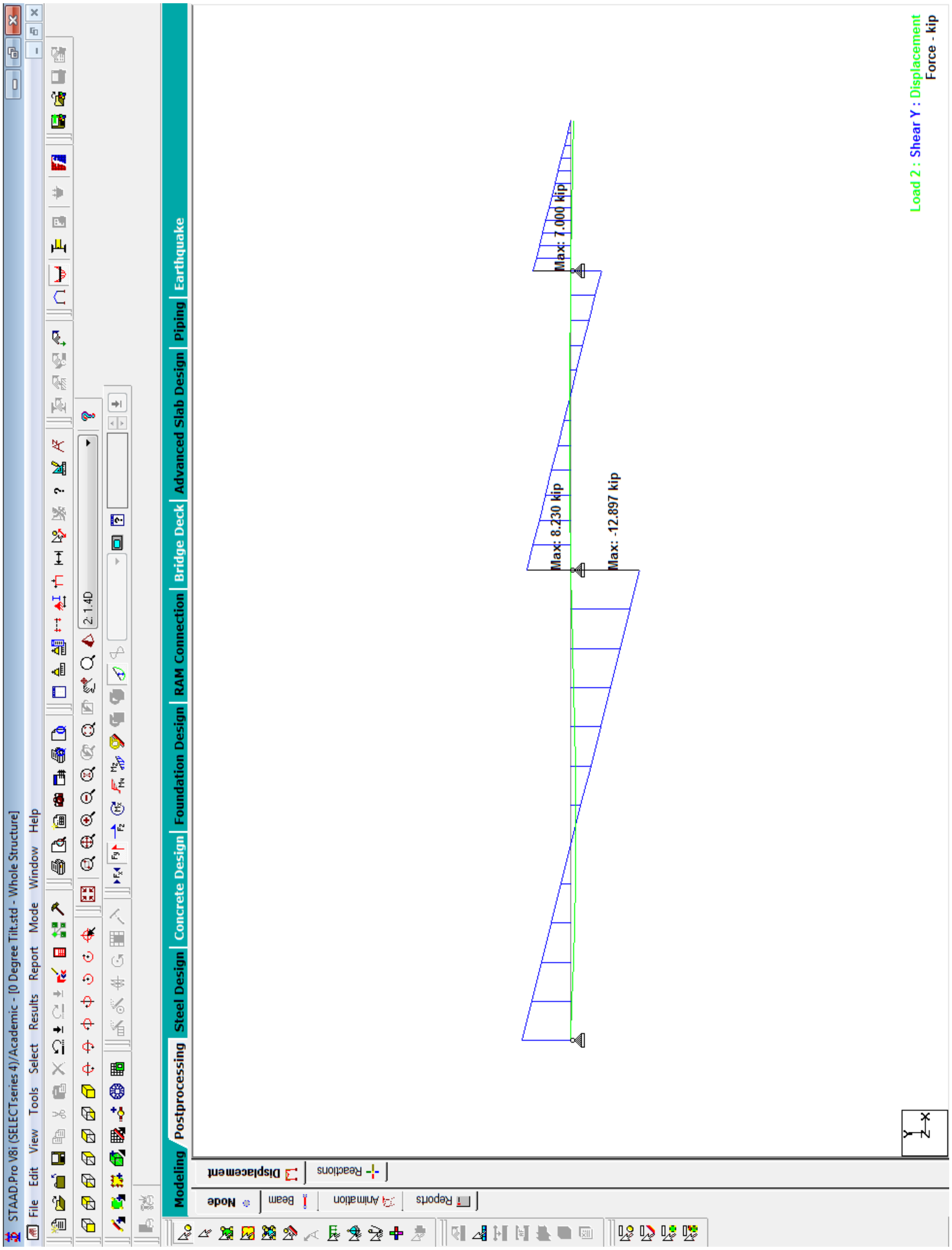
I.1.1 Loads Acting on Structural Tilt-Up Walls

Note: Some gravity load data was combined with lateral load data, and can be found in Appendix H

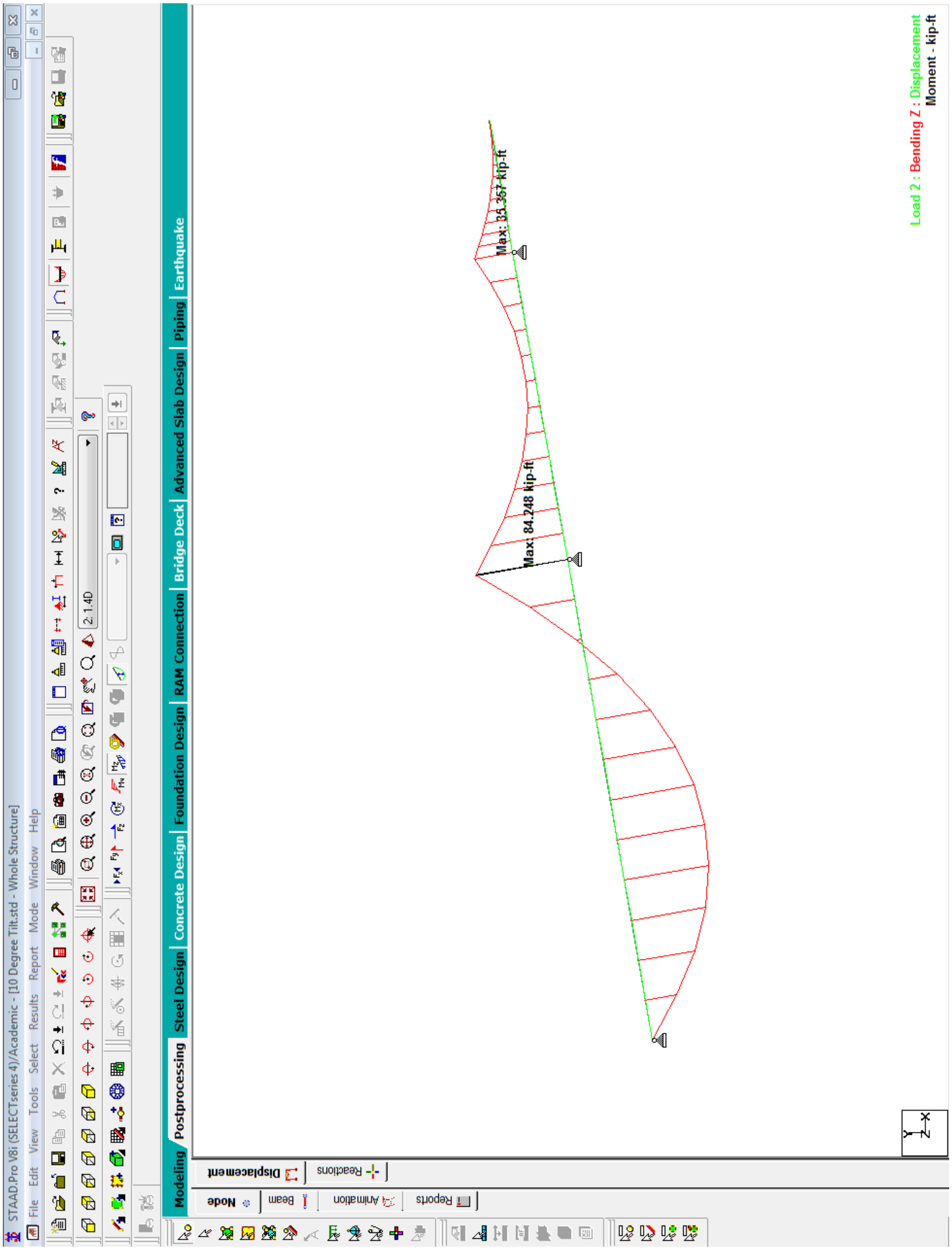
(a) STAAD Output

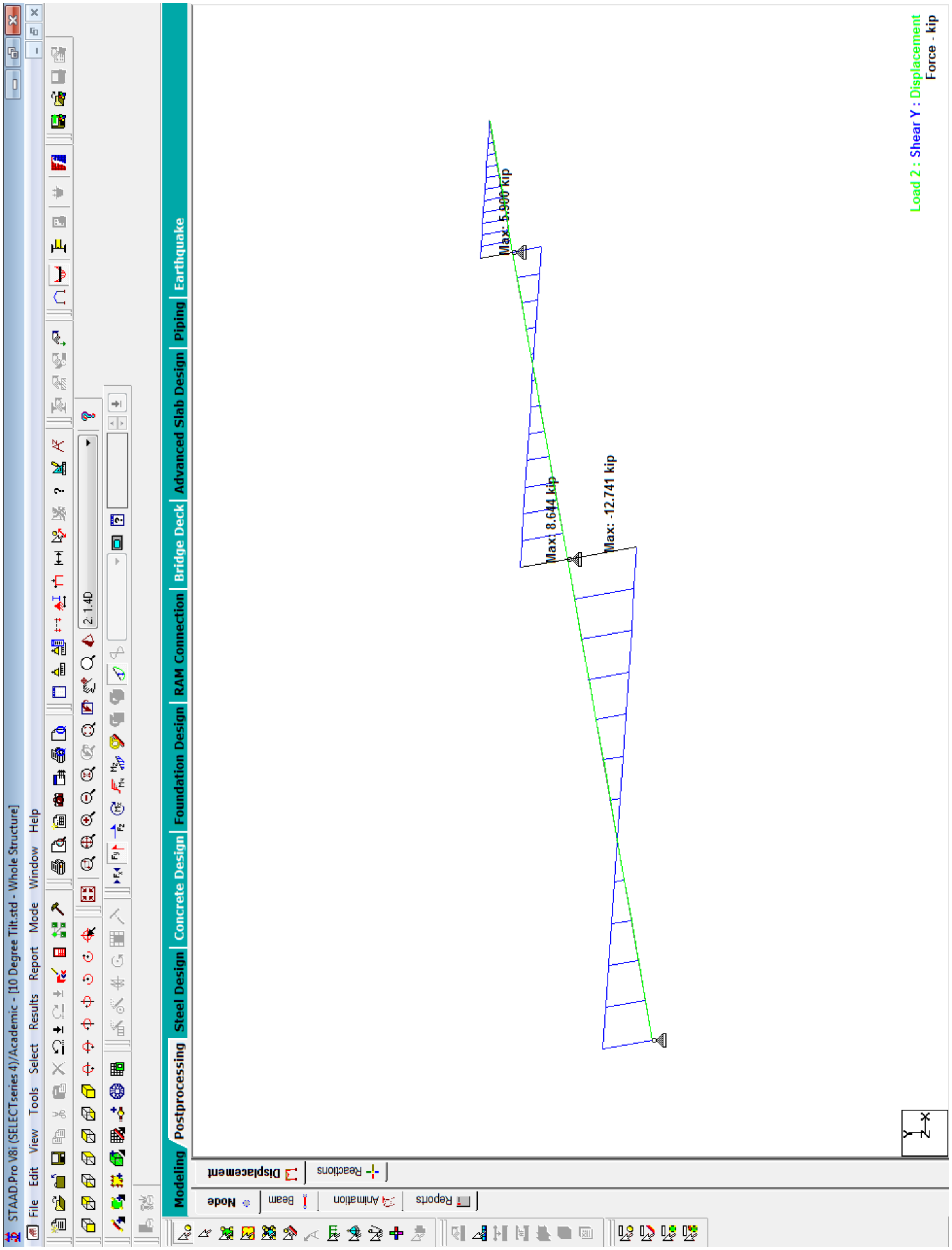
0-Degree Tilt

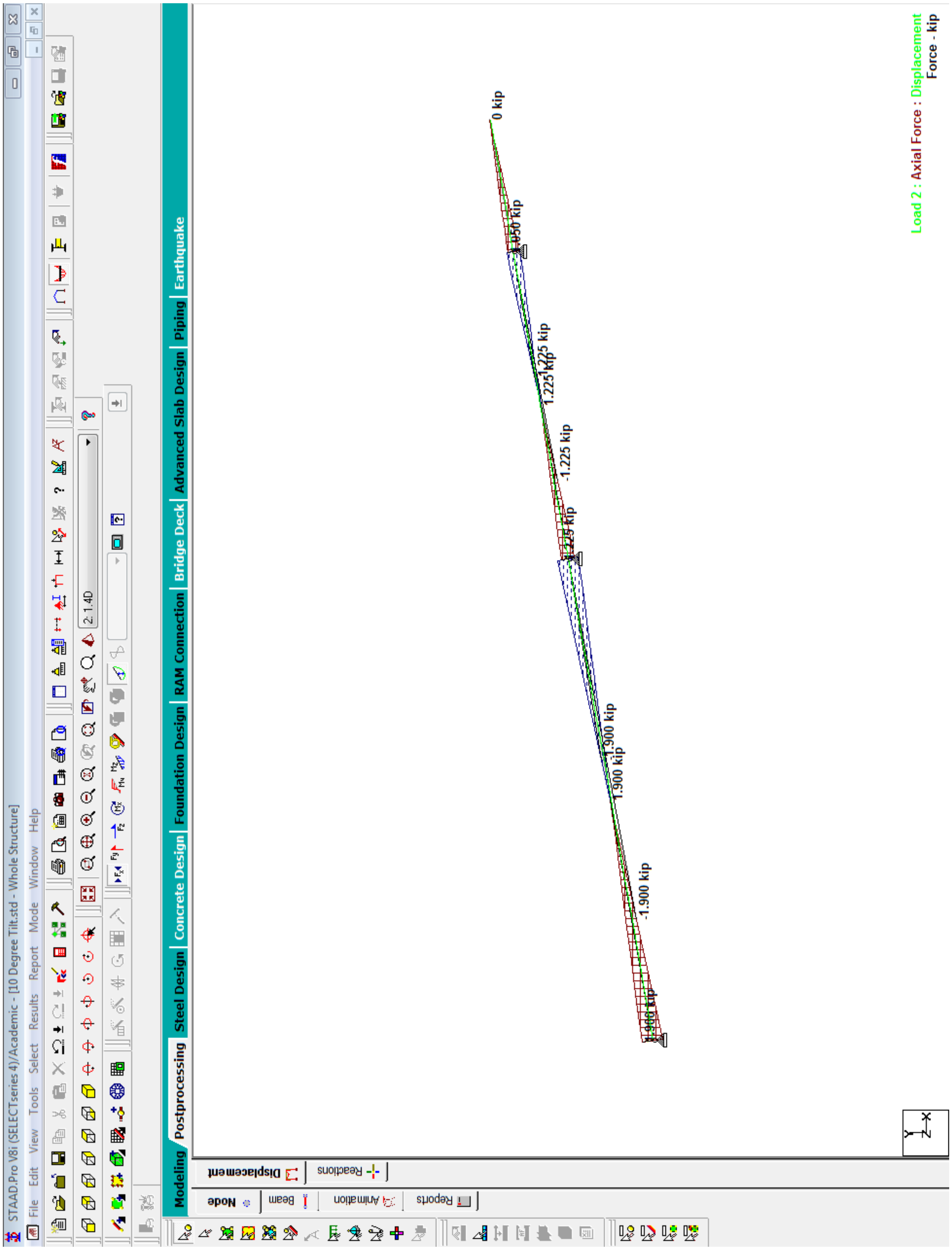



STAAD.Pro V8i (SELECTseries 4)/Academic - [0 Degree Tilt.std - Whole Structure]
Max: 7.000 kip
Max: 8.230 kip
Max: -12.897 kip
Load 2 : Shear Y : Displacement
Force - kip

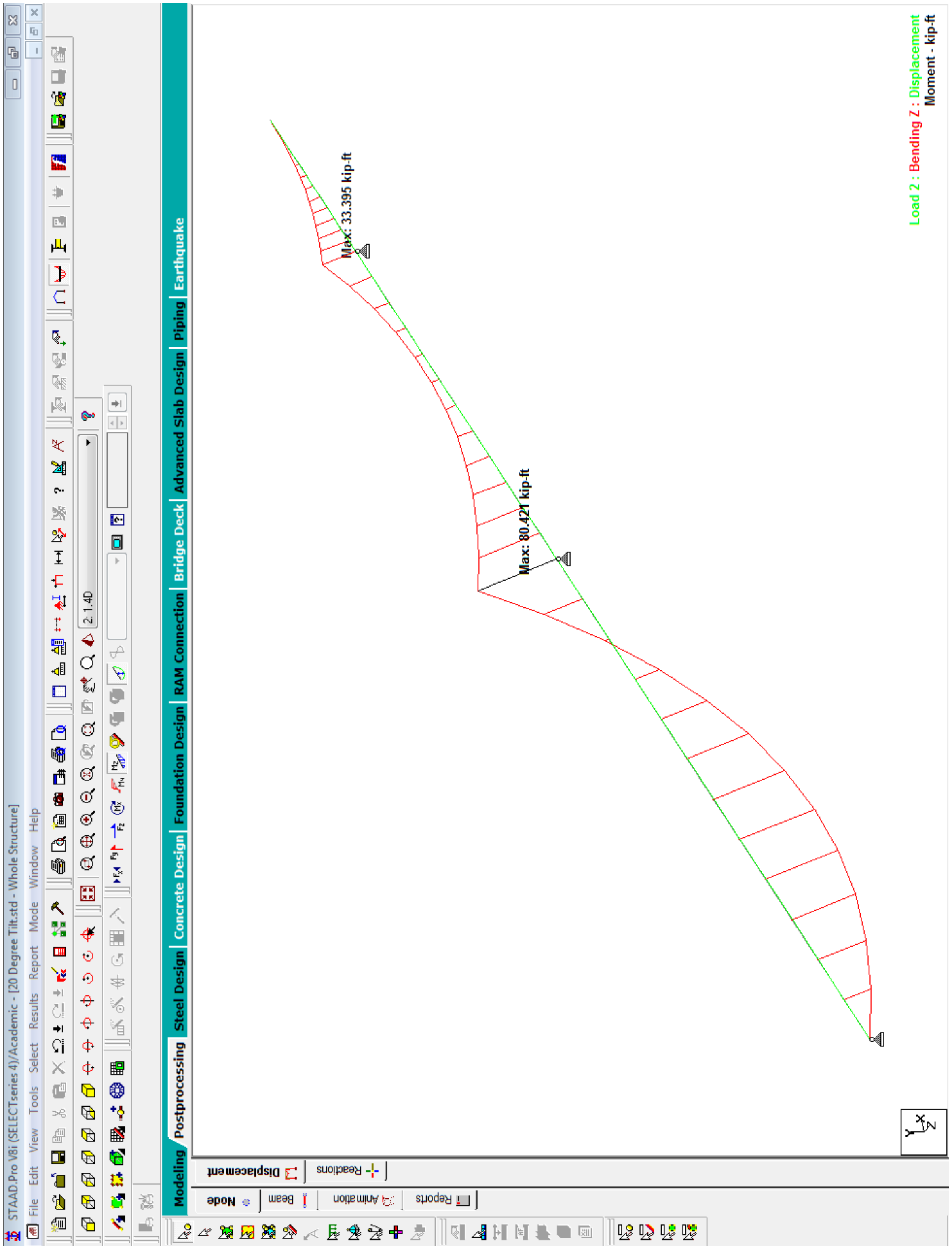
10-Degree Tilt

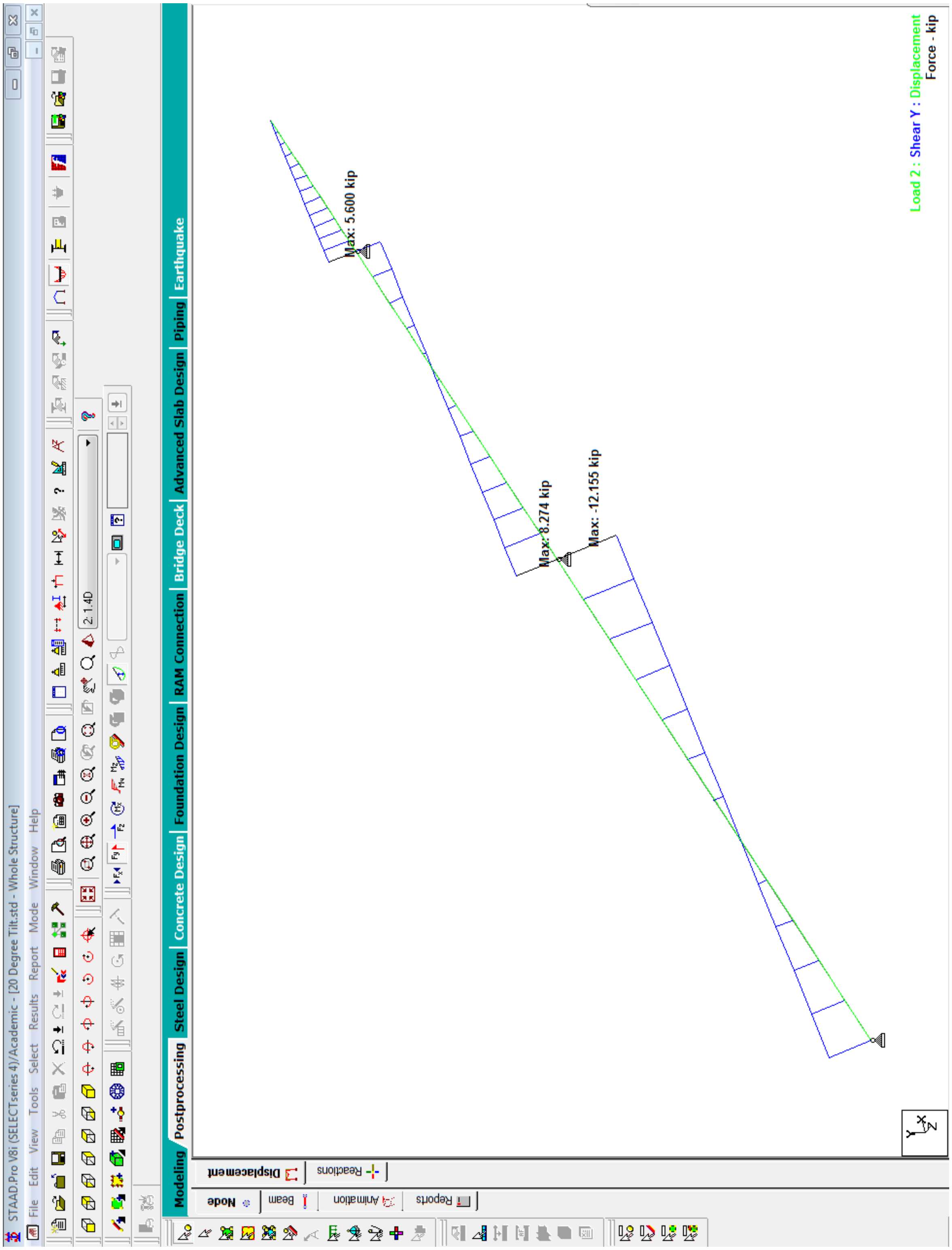


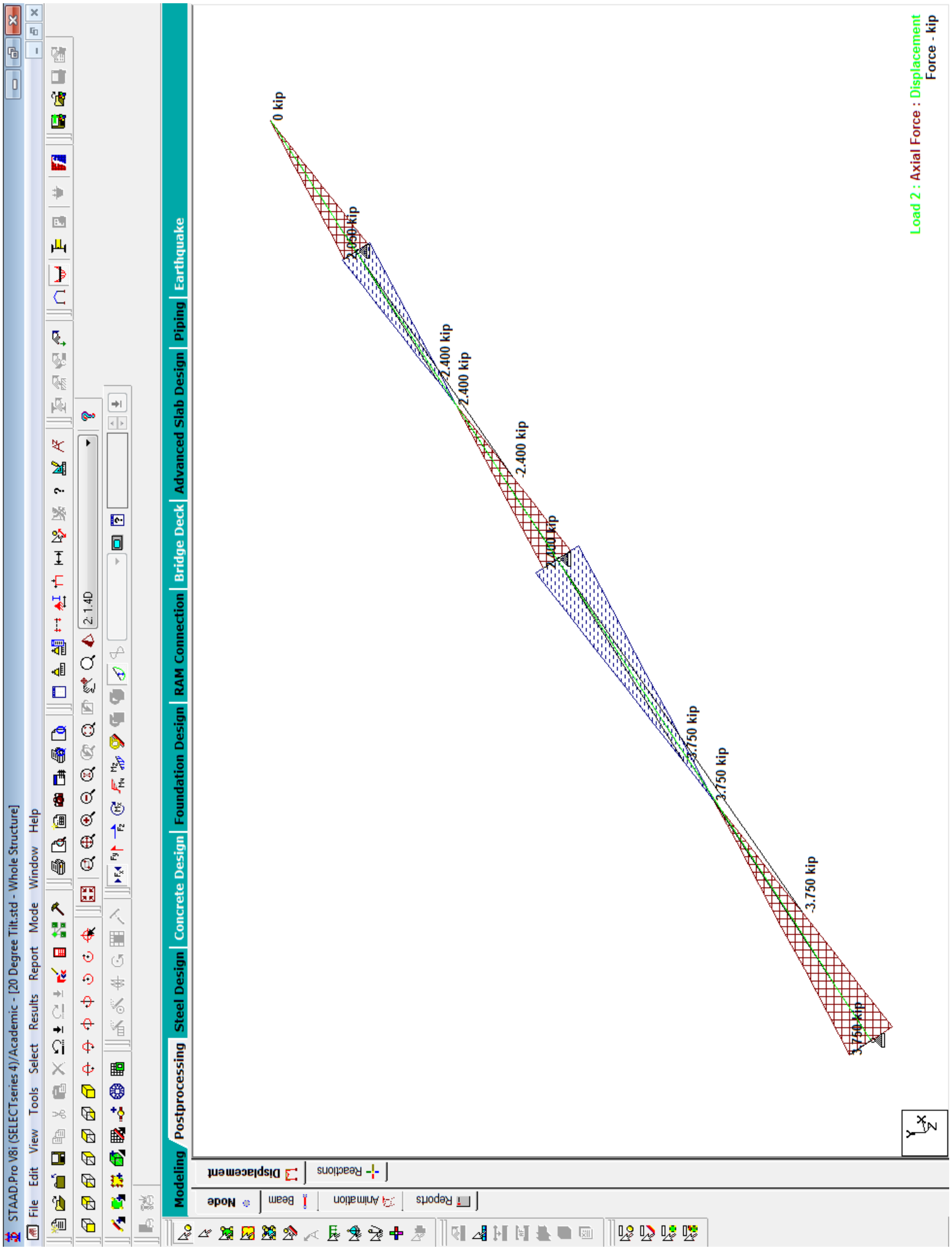

STAAD.Pro V8i (SELECTseries 4)/Academic - [10 Degree Tilt.std - Whole Structure]
File Edit View Tools Select Results Report Mode Window Help
Modeling Postprocessing Steel Design Concrete Design Foundation Design RAM Connection Bridge Deck Advanced Slab Design Piping Earthquake
Max: 5.900 kip
Max: 8.644 kip
Max: -12.741 kip
Load 2 : Shear Y : Displacement
Force - kip


STAAD.Pro V8i (SELECTseries 4)/Academic - [10 Degree Tilt.std - Whole Structure]
Modeling
Postprocessing
Steel Design
Concrete Design
Foundation Design
RAM Connection
Bridge Deck
Advanced Slab Design
Piping
Earthquake
Load 2 : Axial Force : Displacement
Force - kip
0 kip
-1.225 kip
-1.900 kip
Node
Beam
Animation
Reports
Reactions
Displacement

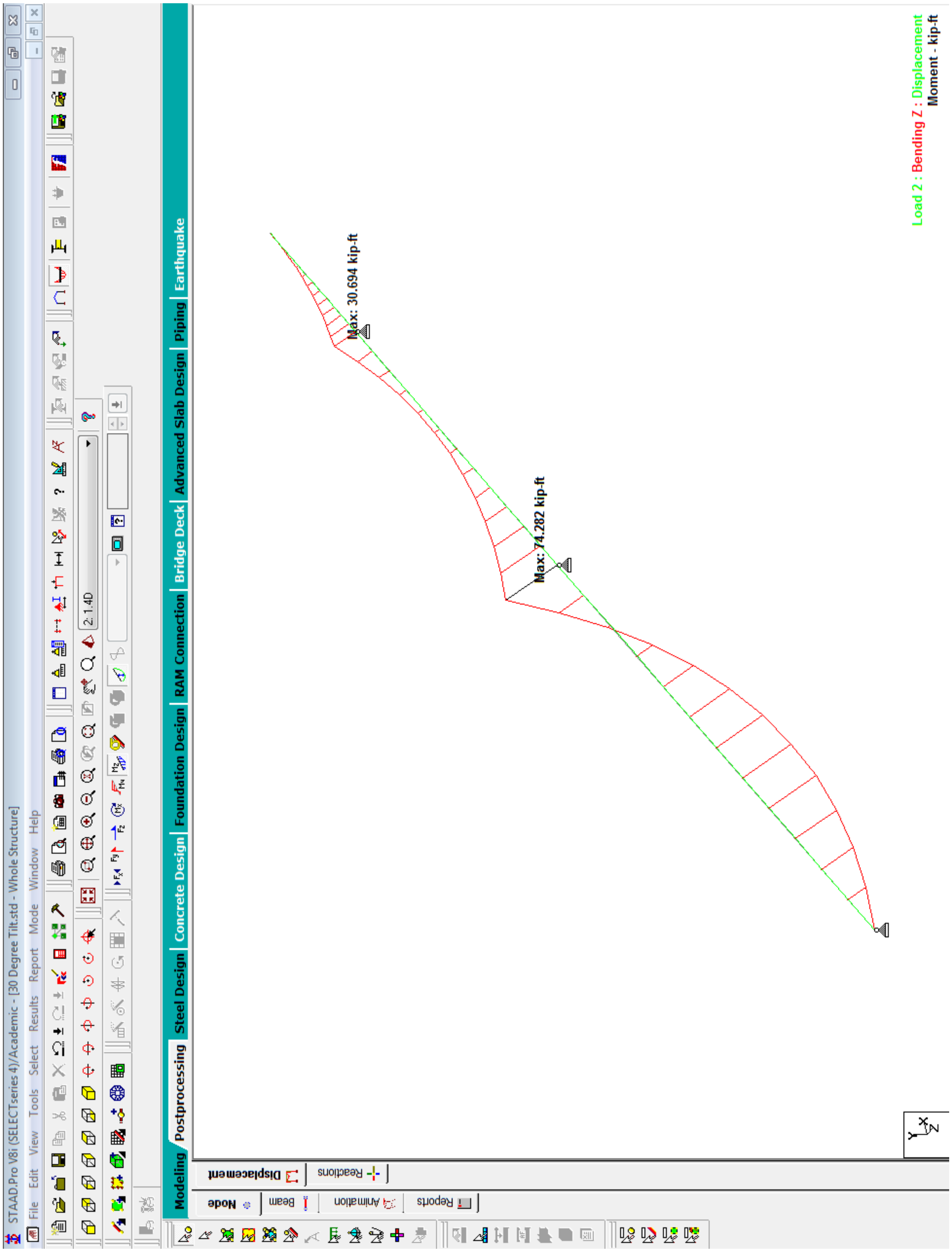
20-Degree Tilt

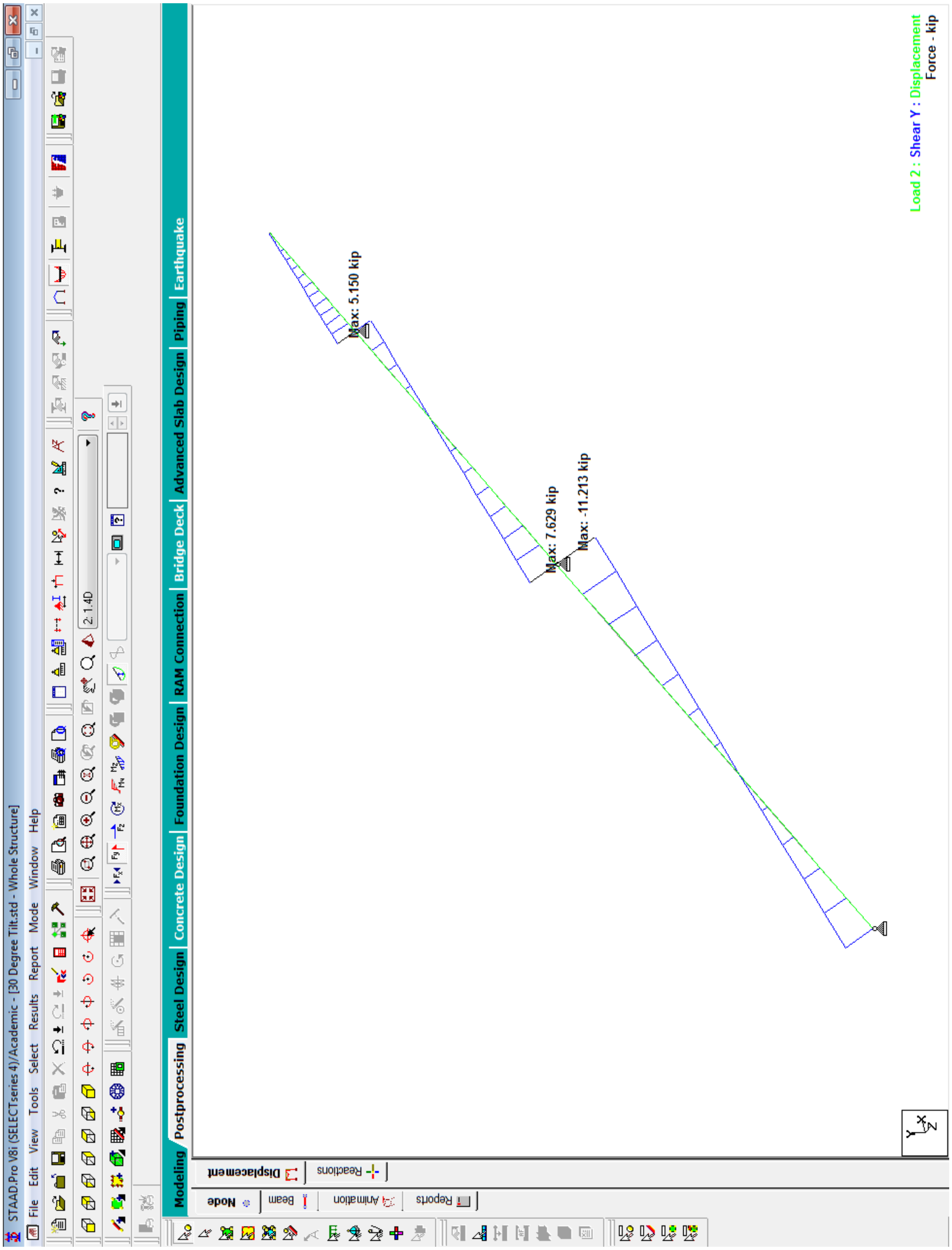


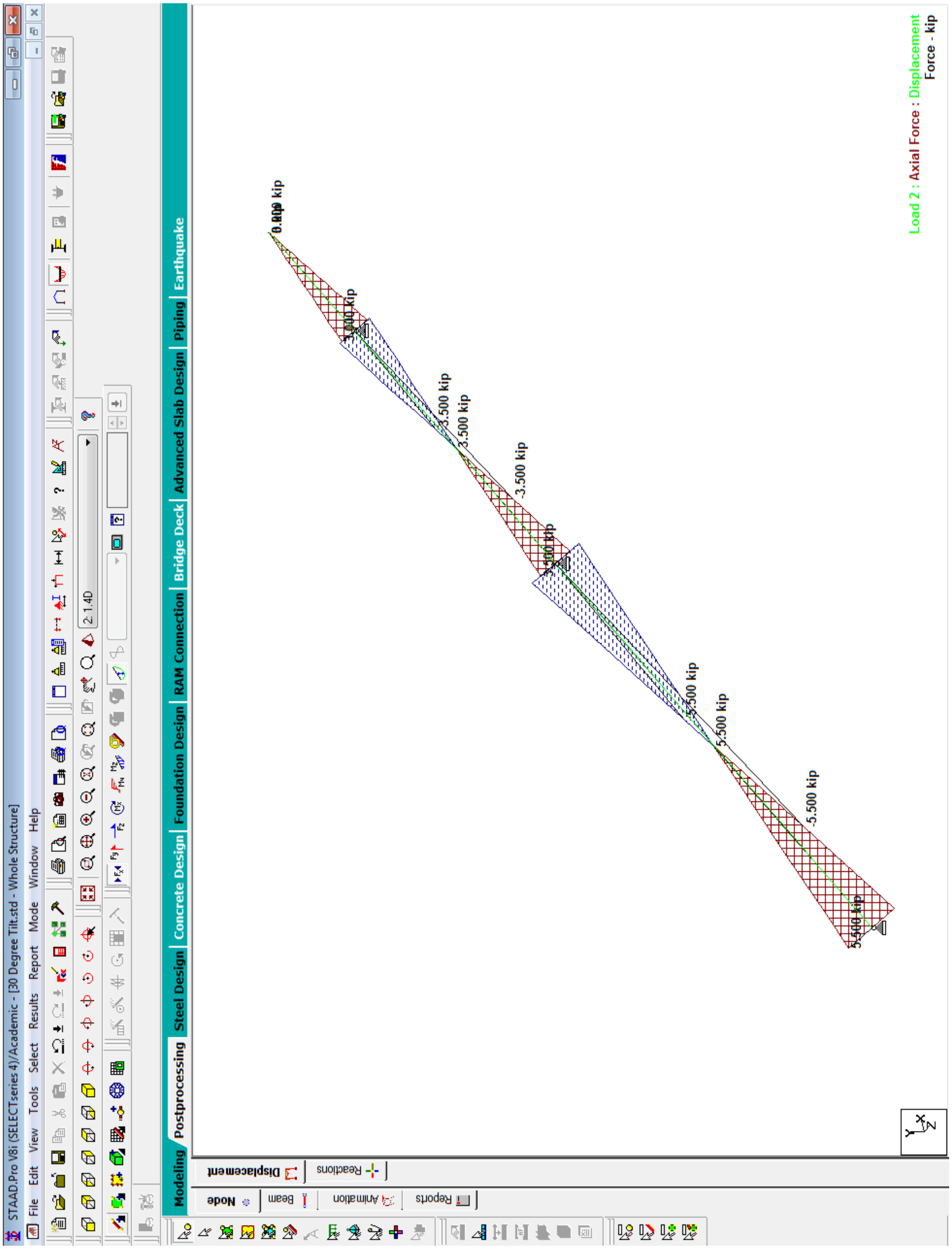

STAAD.Pro V8i (SELECTseries 4)/Academic - [20 Degree Tilt.std - Whole Structure]
Max: 5.600 kip
Max: 8.274 kip
Max: -12.155 kip
Load 2 : Shear Y : Displacement
Force - kip


Load 2 : Axial Force : Displacement
Force - kip

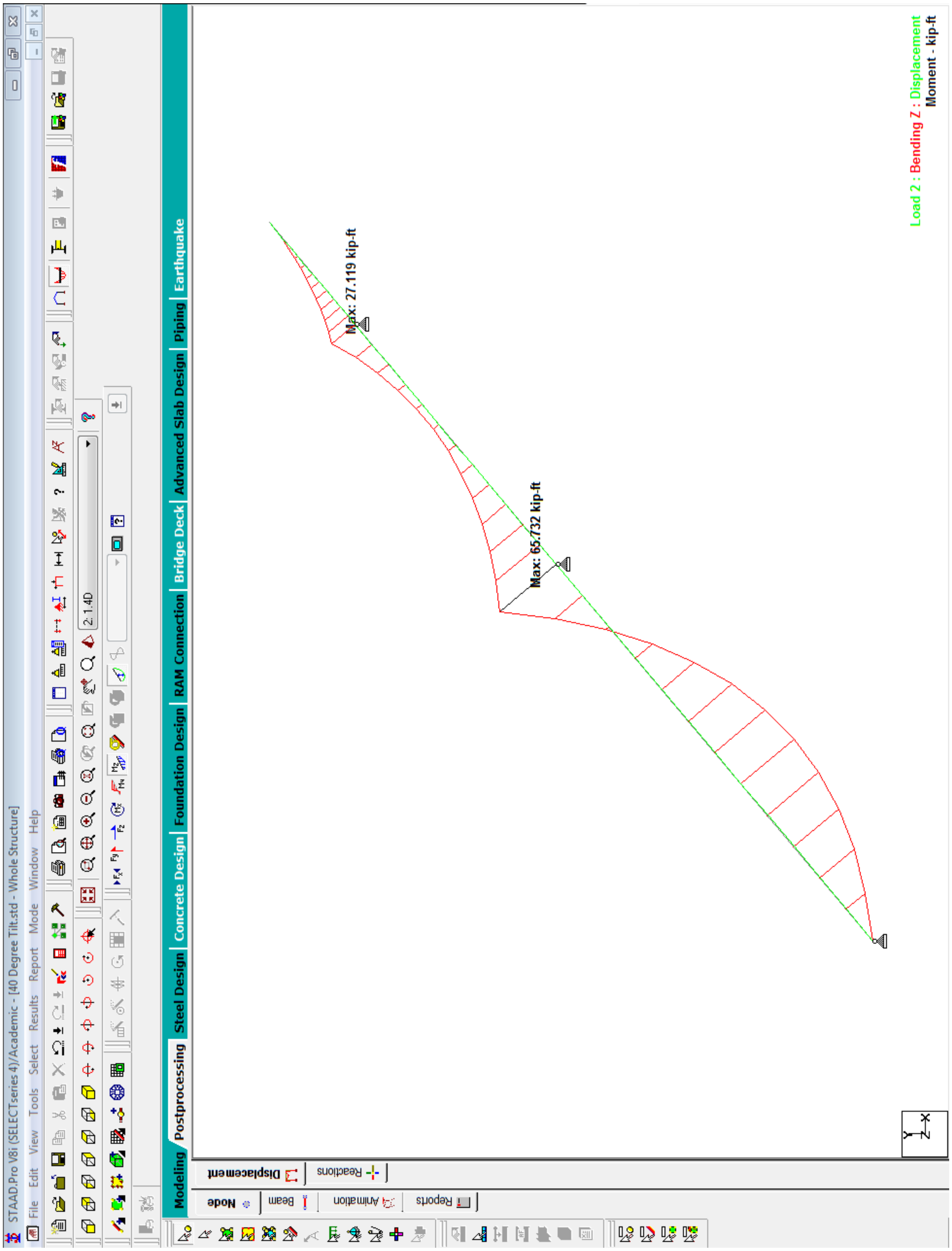
30-Degree Tilt

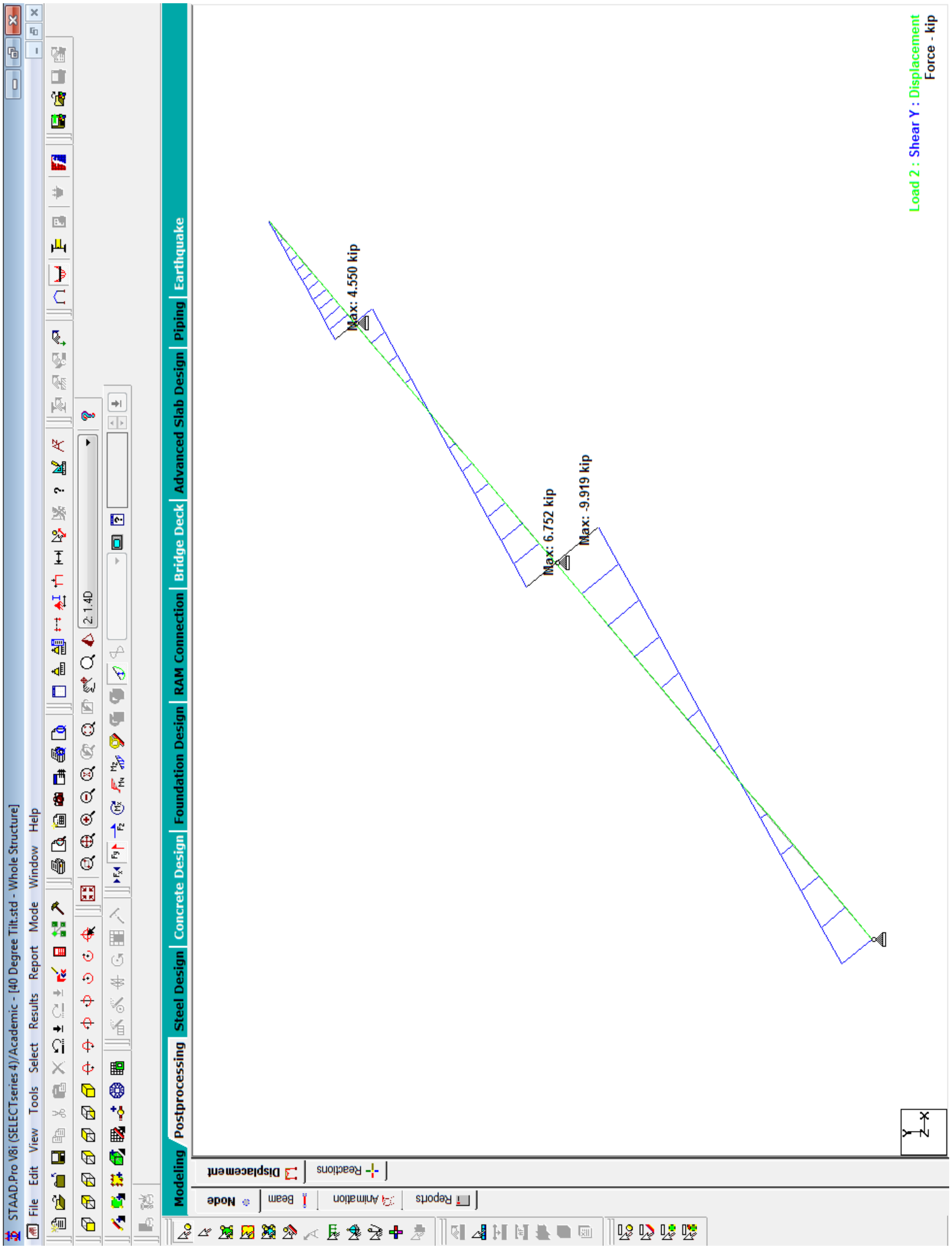


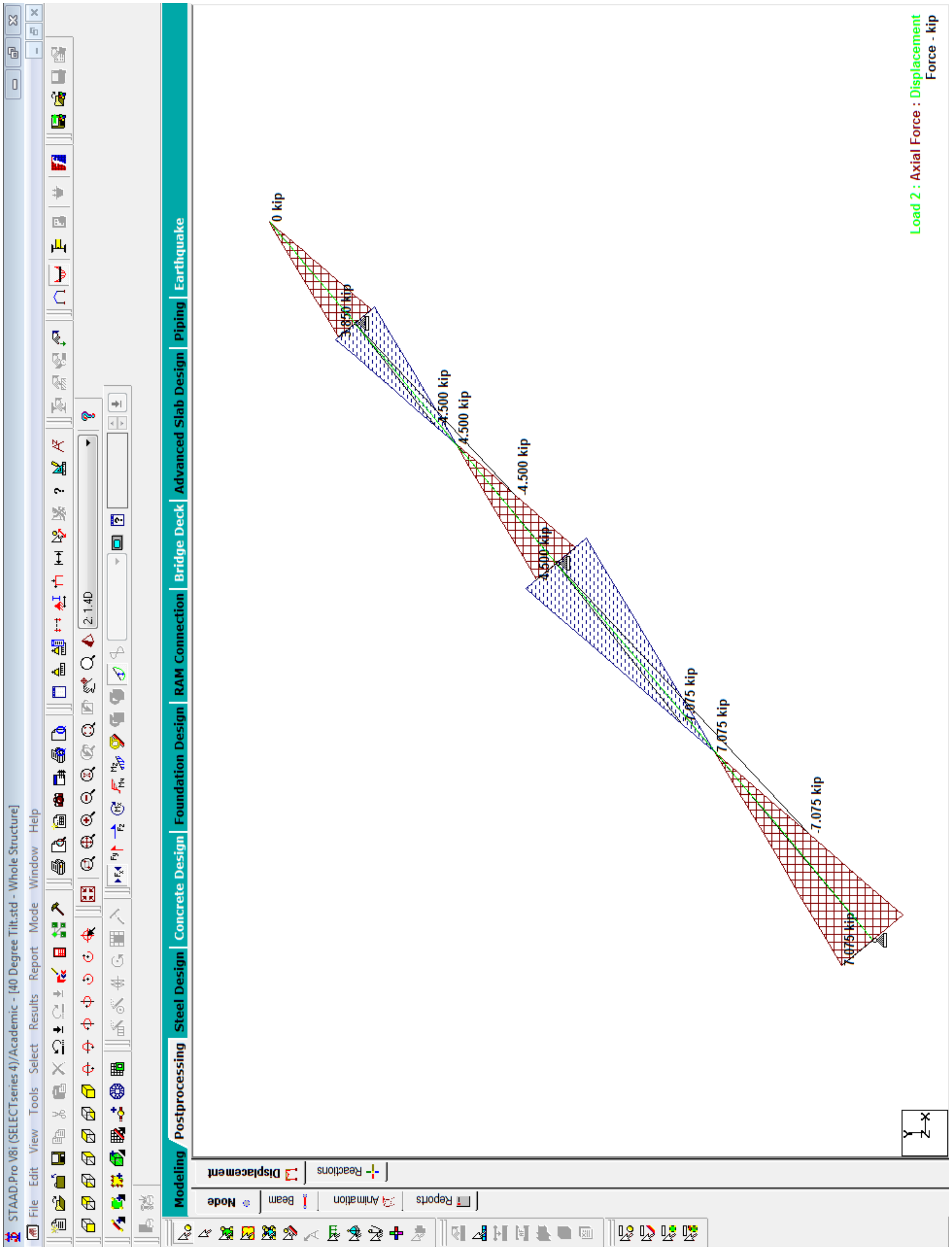

STAAD.Pro V8i (SELECTseries 4)/Academic - [30 Degree Tilt.std - Whole Structure]
Max: 5.150 kip
Max: 7.629 kip
Max: -11.213 kip
Load 2 : Shear Y : Displacement
Force - kip


STAAD.Pro V8i (SELECTseries 4)/Academic - [30 Degree Tilt.std - Whole Structure]
Modeling
Postprocessing
Steel Design
Concrete Design
Foundation Design
RAM Connection
Bridge Deck
Advanced Slab Design
Piping
Earthquake
Load 2 : Axial Force : Displacement
Force - kip
3.500 kip
-3.500 kip
5.500 kip
-5.500 kip

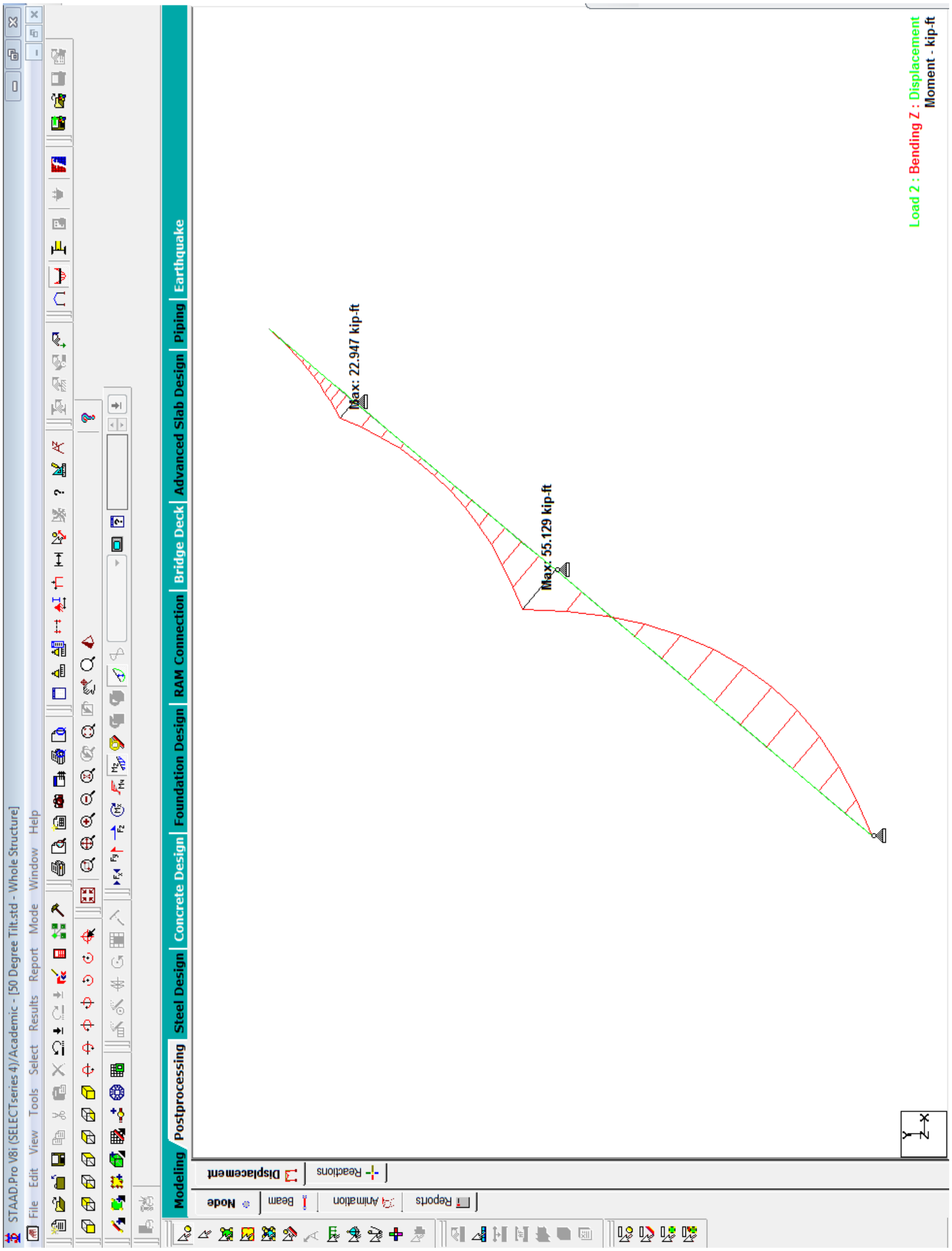
40-Degree Tilt

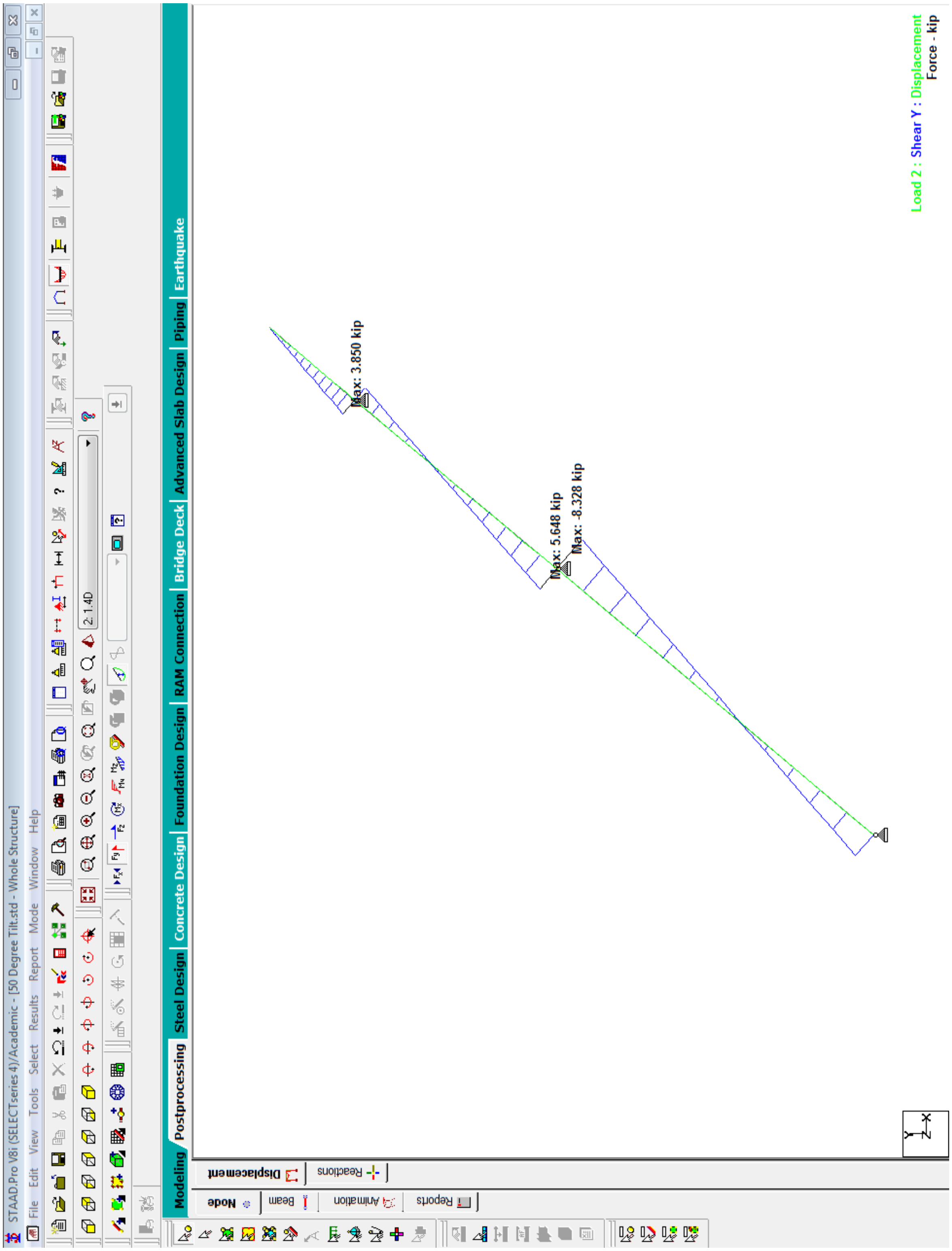


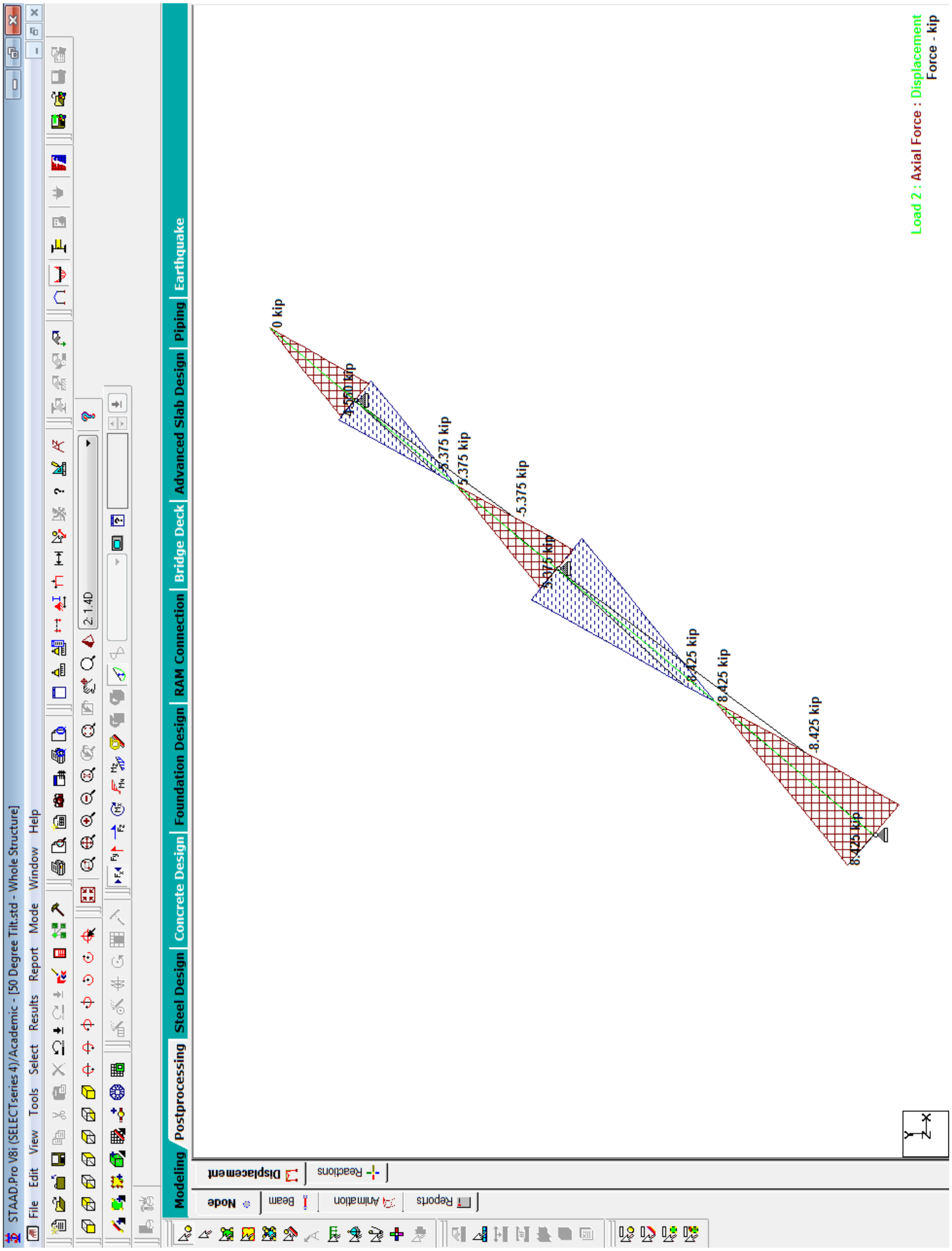

STAAD.Pro V8i (SELECTseries 4)/Academic - [40 Degree Tilt.std - Whole Structure]
Max: 4.550 kip
Max: 6.752 kip
Max: -9.919 kip
Load 2 : Shear Y : Displacement
Force - kip


Load 2 : Axial Force : Displacement
Force - kip

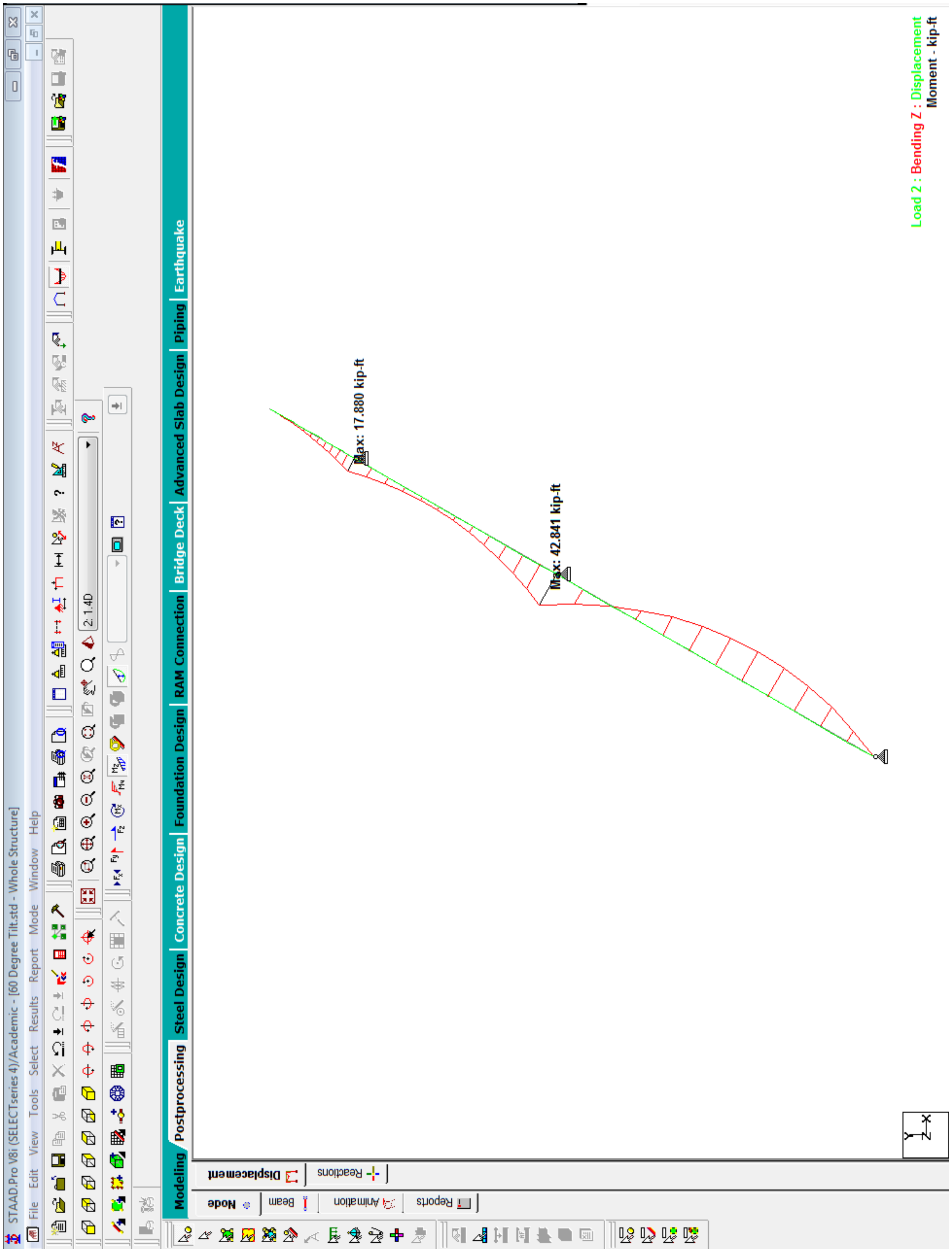
50-Degree Tilt

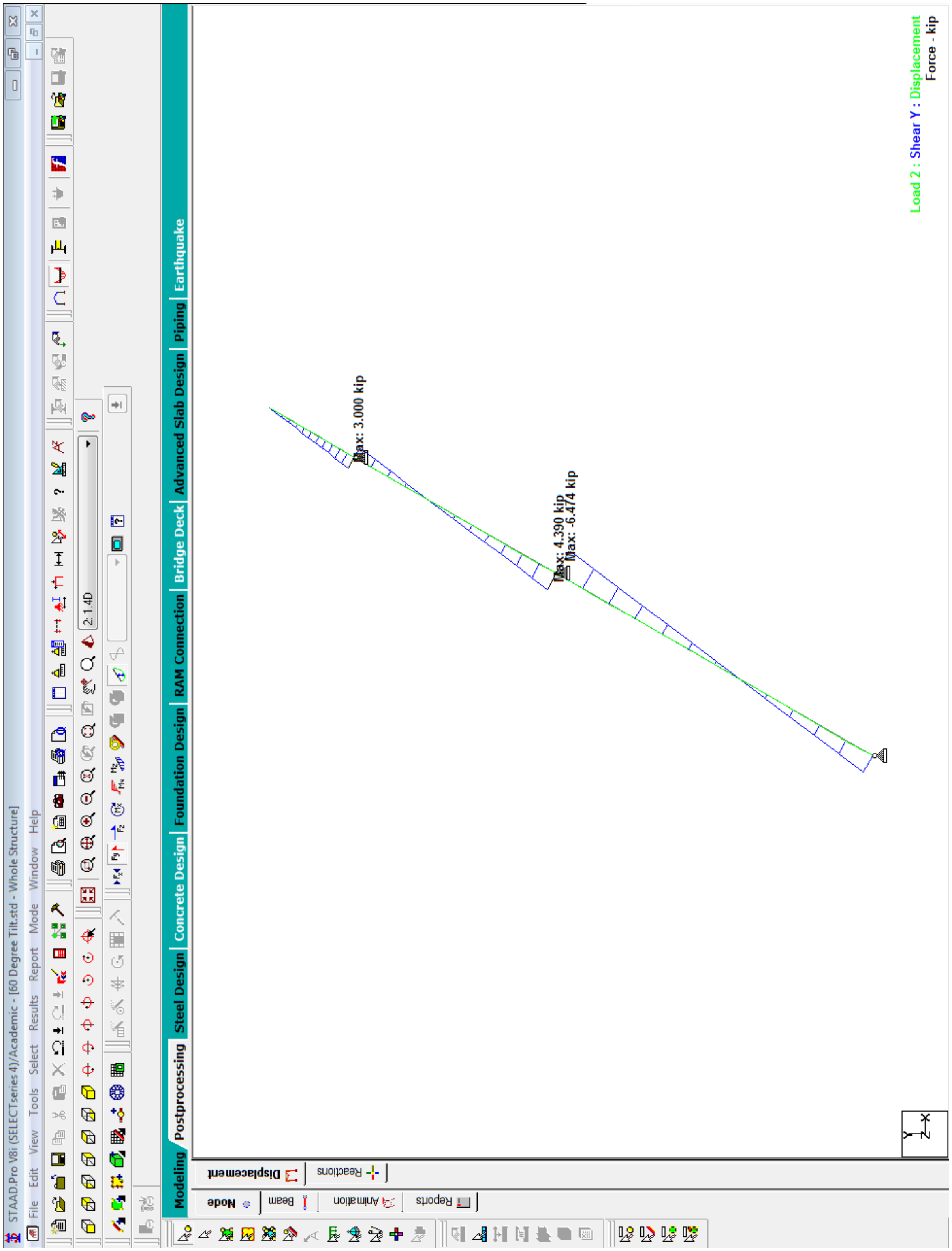


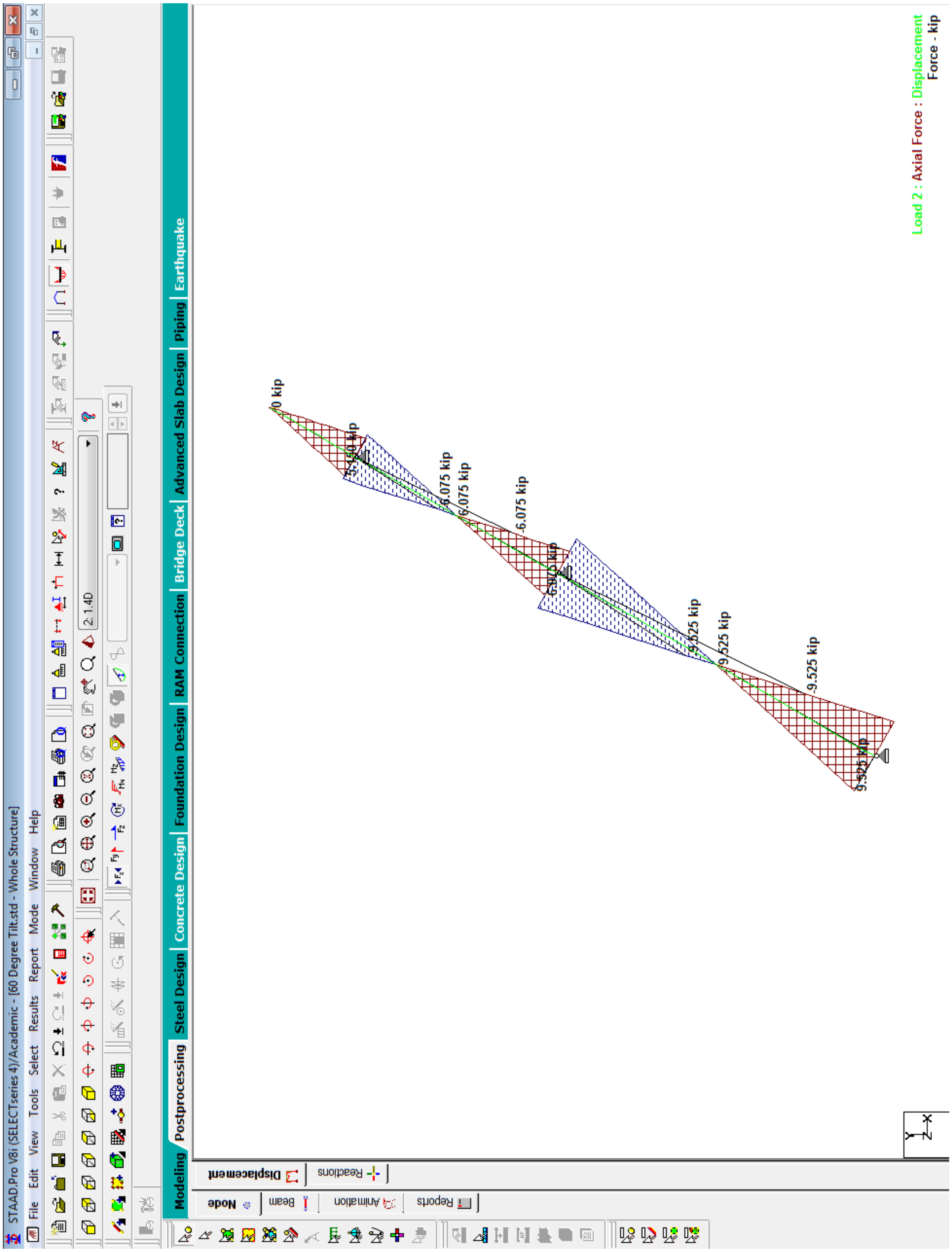

STAAD.Pro V8i (SELECTseries 4)/Academic - [50 Degree Tilt.std - Whole Structure]
Max: 3.850 kip
Max: 5.648 kip
Max: -8.328 kip
Load 2 : Shear Y : Displacement
Force - kip


STAAD.Pro V8i (SELECTseries 4)/Academic - [50 Degree Tilt.std - Whole Structure]
Modeling
Postprocessing
Steel Design
Concrete Design
Foundation Design
RAM Connection
Bridge Deck
Advanced Slab Design
Piping
Earthquake
Load 2 : Axial Force : Displacement
Force - kip

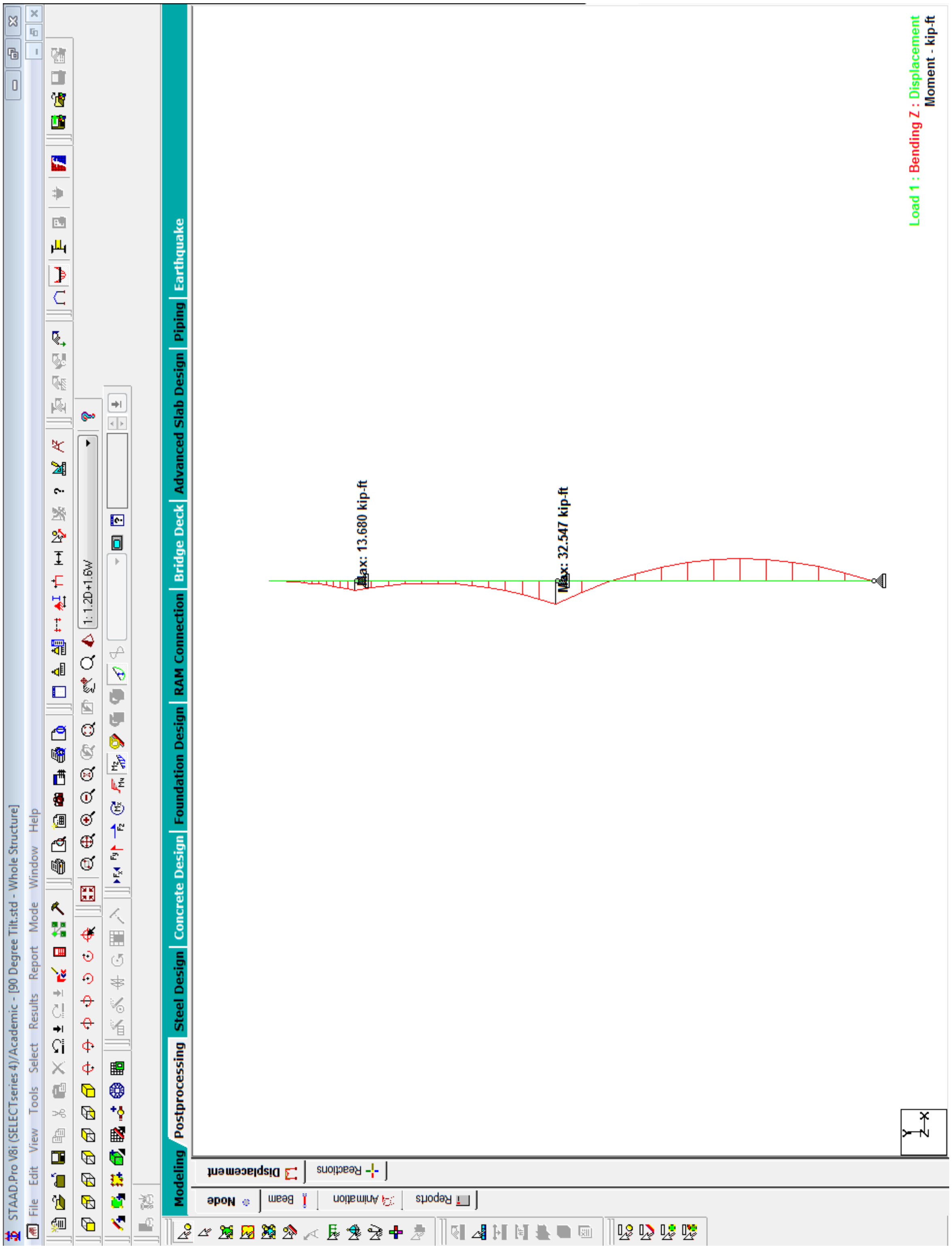
60-Degree Tilt

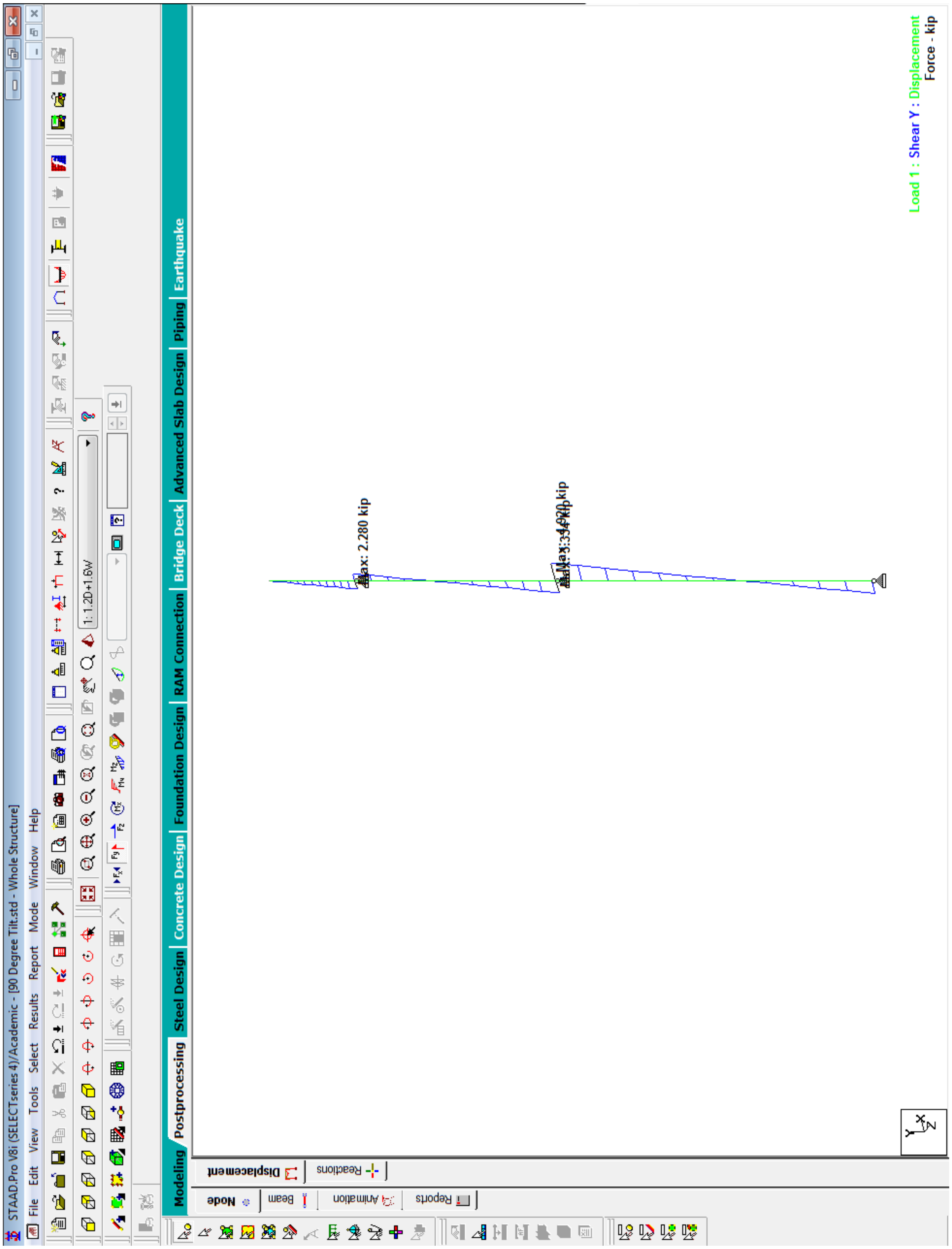


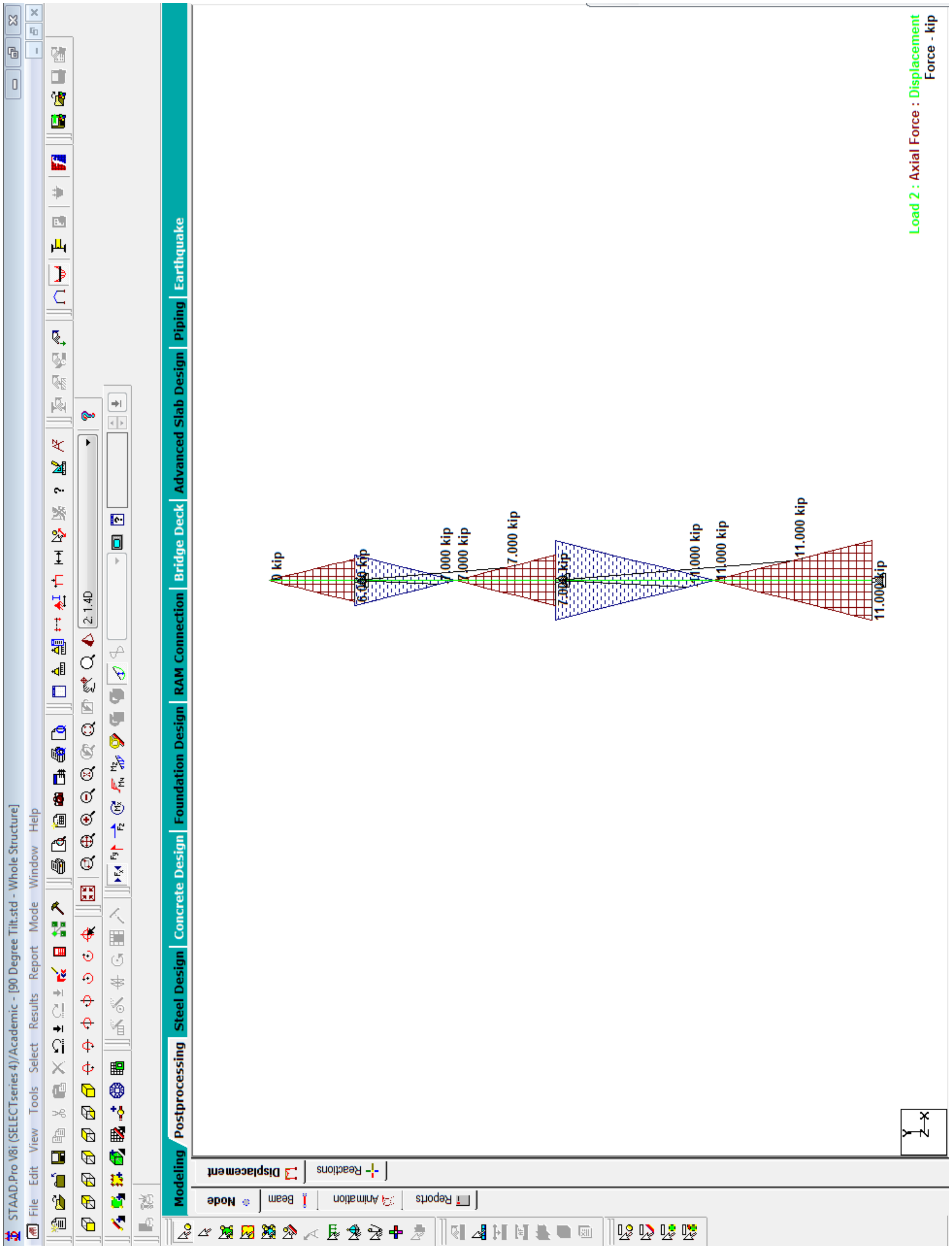

STAAD.Pro V8i (SELECTseries 4)/Academic - [60 Degree Tilt.std - Whole Structure]
Max: 3.000 kip
Max: 4.390 kip
Max: -6.474 kip
Load 2 : Shear Y : Displacement
Force - kip


STAAD.Pro V8i (SELECTseries 4)/Academic - [60 Degree Tilt.std - Whole Structure]
Load 2 : Axial Force : Displacement
Force - kip
0 kip
6.075 kip
6.075 kip
6.075 kip
9.525 kip
9.525 kip
-9.525 kip

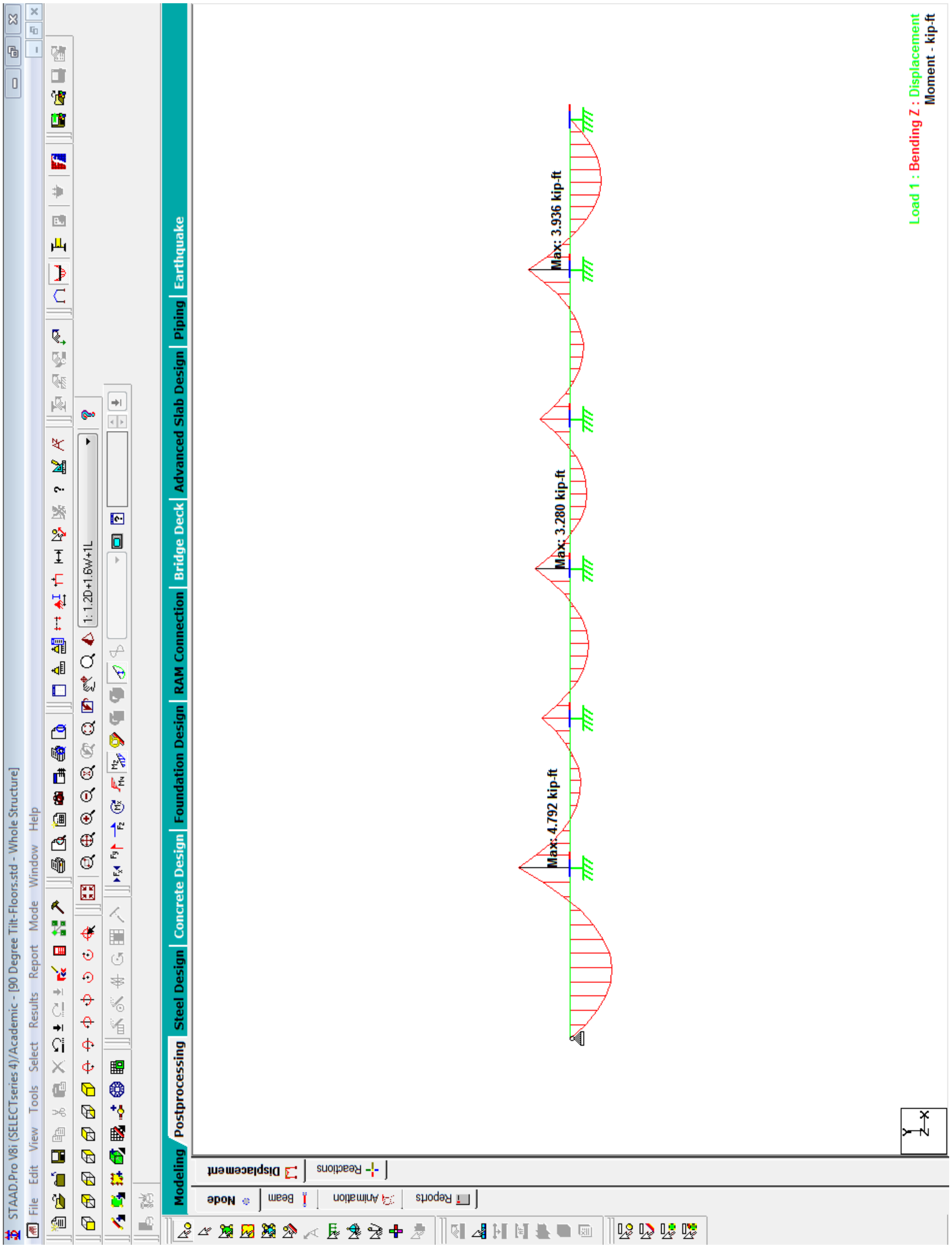
90-Degree Tilt

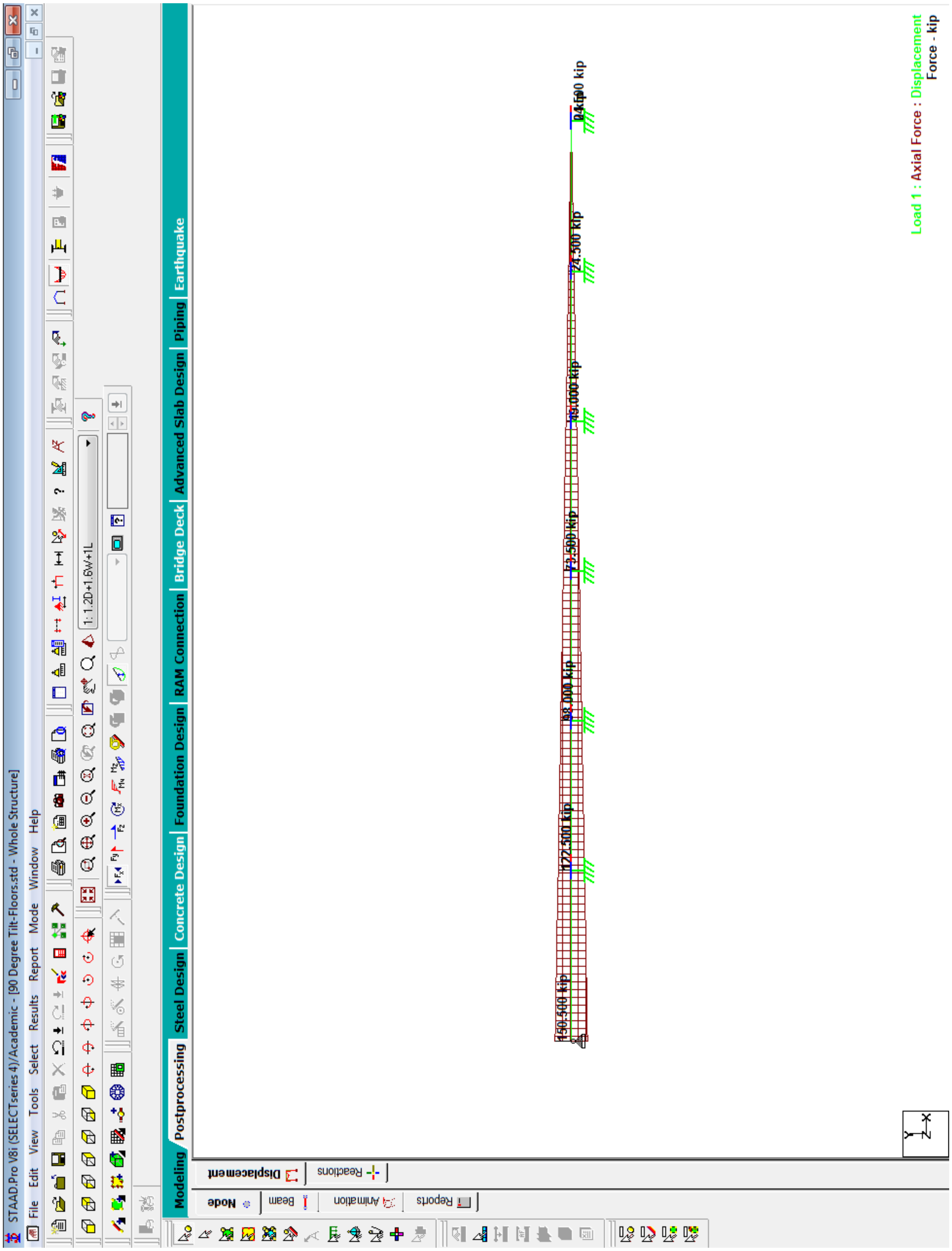



STAAD.Pro V8i (SELECTseries 4)/Academic - [90 Degree Tilt.std - Whole Structure]
File Edit View Tools Select Results Report Mode Window Help
1: 1.2D+1.6W
Modeling Postprocessing Steel Design Concrete Design Foundation Design RAM Connection Bridge Deck Advanced Slab Design Piping Earthquake
Max: 2.280 kip
Load 1 : Shear Y : Displacement
Force - kip
Node Beam Animation Reports
Displacement Reactions


STAAD.Pro V8i (SELECTseries 4)/Academic - [90 Degree Tilt.std - Whole Structure]
Load 2 : Axial Force : Displacement
Force - kip
Postprocessing
Steel Design
Concrete Design
Foundation Design
RAM Connection
Bridge Deck
Advanced Slab Design
Piping
Earthquake
7.000 kip
11.000 kip

90 Degree Tilt w/ Floors




Load 1 : Axial Force : Displacement
Force - kip
24.500 kip
49.000 kip
73.500 kip
98.000 kip
122.500 kip
150.500 kip

I.1.2 Structural Tilt-Up Wall Design

(a) Columns

Table I.1, Tilt-Up Wall Panel Characteristics

Structural Panel	Column Area (ft^2)	Total Panel Area (ft^2)	Total Panel-to-Column Area Ratio
NN1	401.0	829.6	2.07
NN2	401.0	829.6	2.07
NN3	989.0	1349.0	1.36
NN4	739.0	1293.8	1.75
NN5	601.5	1216.9	2.02
SN1	401.0	1093.8	2.73
SN2	401.0	1135.5	2.85
SN3	401.0	1127.7	2.81
SN4	1060.5	1994.0	1.88
SN5	1060.5	1918.8	1.81
EN1	410.5	714.6	1.74
EN2	401.0	669.9	1.67
EN3	1003.4	1319.3	1.31
EN4	945.4	1261.3	1.33
WN1	945.4	1261.3	1.33
WN2	1003.4	1319.3	1.31
WN3	401.0	667.2	1.66
WN4	401.0	724.8	1.81

Table I.2, Unit Strip Dimensions

Length (in)	12
Width (in)	10
Cover (in)	0.75
	2
$d_{extreme}$ (in)	7.125

Table I.3, Factored Loads (lb/ft^2) used in Unit Strip

1.2D + 1.6W + 1.0L			1.4D
150	67	80	175

Table I.4, Load per Unit Strip (kip/ft)

Total Panel -to-Column Area Ratio	Load on Unit-Strip of Wall (Kip/ft)			
	1.2D + 1.6W + 1.0L			1.4D
2.85	0.43	0.19	1.32	0.50

Table I.5, Moment Magnification Factor			
Phase	I_{cr} (in^4) ACI 318-11 §10.10.4.1	$P_{c,min}$ (kip) ACI 318-11 §10.10.6	Δ ACI 318-11 §10.10.7.4
Construction	350	54.71	Varies
Full Occupancy		413.73	1.04

Table I.6, Part 1 of 60,000 lb/in² Flexural Reinforcement Design w/o Compression Reinforcement per Unit Strip Design								
Total Panel -to- Column Area Ratio	Panel Angle (°)	δ	$M_{u,p\Delta\text{-max}}$ (kip-ft)		$P_{u,p\Delta\text{-max}}$ (kip)		$V_{u,p\Delta\text{-max}}$ (kip)	
			1.2D + 1.6W	1.4D	1.2D + 1.6W	1.4D	1.2D + 1.6W	1.4D
2.85	0	1.00		83.5				12.9
	10	1.05		84.2		1.9		12.7
	20	1.10		80.4		3.8		12.2
	30	1.15		74.3		5.5		11.2
	40	1.21		65.7		7.1		9.9
	50	1.26		55.1		8.4		8.3
	60	1.30		42.8		9.5		6.5
	90	1.37	44.4		9.5	11.0	4.9	

Table I.7, Part 2 of 60,000 lb/in² Flexural Reinforcement Design w/o Compression Reinforcement per Unit Strip Design						
Total Panel -to- Column Area Ratio	Flexural Reinforcement Requirement					
	d (in)	$Dx^2 + Ex + F = 0$			$A_{s,req}$ (in^2)	Notes
		D	E	F		
2.85	7.125	-0.49	7.125	-18.6	3.4	Requires doubly reinforced section to reduce congestion

Table I.8, Part 1 of 75,000 lb/in² Flexural Reinforcement Design w/o Compression Reinforcement per Unit Strip Design								
Total Panel -to- Column Area Ratio	Panel Angle (°)	δ	$M_{u,p\Delta\text{-max}}$ (kip-ft)		$P_{u,p\Delta\text{-max}}$ (kip)		$V_{u,p\Delta\text{-max}}$ (kip)	
			1.2D + 1.6W	1.4D	1.2D + 1.6W	1.4D	1.2D + 1.6W	1.4D
2.85	0	1.00		83.50				12.90
	10	1.00		84.20		1.89		12.70
	20	1.00		80.40		3.80		12.20
	30	1.00		74.30		5.48		11.19
	40	1.00		65.70		7.10		9.88

	50	1.00		55.10		8.41		8.33
	60	1.00		42.80		9.47		6.45
	90	1.00	44.40		9.50	11.00	4.9	

Table I.9, Part 2 of 75,000 lb/in^2 Flexural Reinforcement Design w/o Compression Reinforcement per Unit Strip Design						
Total Panel -to- Column Area Ratio	Flexural Reinforcement Requirement					
	d (in)	$Dx^2 + Ex + F = 0$			$A_{s,req}$ (in^2)	Notes
		D	E	F		
2.85	7.125	-0.61	7.125	-14.8	2.702	Requires doubly reinforced section to reduce congestion

Table I.10, Part 1 of 60,000 lb/in^2 Flexural Reinforcement Design w/ Compression Reinforce per Unit Strip						
Total Panel-to-Column Area Ratio	Tension Reinforcement			Compression Reinforcement		
	n	A_s (in^2)	d (in)	n	A_s' (in^2)	d' (in)
2.85	3	2.37	7.125	3	2.37	1.625
	4	3.16		3	2.37	
	4	3.16		4	3.16	

Table I.11, Part 2 of 60,000 lb/in^2 Flexural Reinforcement Design w/ Compression Reinforce per Unit Strip							
Total Panel -to-Column Area Ratio	$Dx^2 + Ex + F = 0$			c (in)	a (in)	ε_s	ε_s'
	D	E	F				
2.85	45.90	63.99	-335.06	2.09	1.57	0.0073	0.0007
		16.59	-335.06	2.53	1.90	0.0055	0.0011
		85.32	-446.75	2.33	1.75	0.0062	0.0010

Table I.12, Part 3 of 60,000 lb/in^2 Flexural Reinforcement Design w/ Compression Reinforce per Unit Strip			
Total Panel -to-Column Area Ratio	M_n (kip-ft)	Φ	ΦM_n (kip-ft)
2.85	72.8	0.9	65.5
	94.5	0.9	85.1
	97.8	0.9	88.0

Table I.13, Part 1 of 60,000 lb/in^2 Shear Reinforcement Design per Unit Strip							
Total Panel -to- Column Area Ratio	$V_{u,p\Delta}$ (Kip)		Length (ft)	Thk (in)	d (in)	f'_c (lb/in^2)	V_c (Kip) ACI 318-11 § 11.4.6.1
	Position (ft)	Magnitude					
2.85	44	12.90	44	10	7.125	6000	13.2

Table I.14, Part 2 of 60,000 lb/in^2 Shear Reinforcement Design per Unit Strip		
Total Panel -to-Column Area Ratio	$\Phi V_{c,n}$ (Kip) ACI 318-11 §11.2.1.1	$V_{s,req}$ (Kip) ACI 318-11 §11.4.7.2
2.85	4.97	3.95

Table I.15, Part 3 of 60,000 lb/in^2 Shear Reinforcement Design per Unit Strip				
Total Panel -to-Column Area Ratio	S_{max} (in)		S_{design} (in) ACI 318-11 §11.4.7.2	S_{actual} (in)
	ACI 318-11 §11.4.5.1,11.4.5.3	ACI 318-11 §7.10.5.2		
2.85	2-Y1	10	23.78	3.5

Table I.16, 60,000 lb/in^2 Flexural Rebar Quantity per Unit Strip		
Structural Panel	Total Panel-to-Column Area Ratio	Rebar Quantity per Unit Strip
NN1	2.07	2.258
NN2	2.07	2.258
NN3	1.36	1.627
NN4	1.75	2.089
NN5	2.02	2.204
SN1	2.73	3.044
SN2	2.85	3.190
SN3	2.81	3.145
SN4	1.88	2.243
SN5	1.81	2.159
EN1	1.74	2.077
EN2	1.67	1.993
EN3	1.31	1.569
EN4	1.33	1.592
WN1	1.33	1.592
WN2	1.31	1.569
WN3	1.66	1.985
WN4	1.81	2.156

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Required Axial and Bending Capacities		
Phase	P_u (Kip)	M_u (Kip-ft)
Construction	0	83.5
Construction	1.9	84.2
Construction	3.8	80.4
Construction	5.5	74.3
Construction	7.1	65.7
Construction	8.4	55.1
Construction	9.5	42.8
Construction	11.0	44.4
Full Occupancy	312.4	5

Unit Strip Dimensions	
Length (in)	12
Width (in)	10
Cover (in)	0.75 2
$d_{extreme}$ (in)	7.125

a) Interaction (Out-of-Plane)

Pure Axial

$P_0 = 0.85f'_c\,[A_{gross} - A_{s,gross}] + A_{s,gross}f_y$
$P_0 = 0.85(6)\,[12(10) - 8 \times 0.79] + 8(0.79)(60)$
$P_0 = 958.97$ kip

$\phi P_0 = 0.65\,(958.97)$
$\phi P_0 = 623.3$ kip

$0.8\phi P_0 = 0.8(623.3)$
$0.8\phi P_0 = 498.7$ Kip

Balance Condition

$\varepsilon_y = 0.00207$

$C = \dfrac{0.003(7.125)}{0.003 + 0.00207}$
$C = 4.2''$

*** Initially assume compression doesn't yield

$\varepsilon_{si,row1} = \dfrac{0.003}{4.2}(4.2 - 1.63)$
$\varepsilon_{si,row1} = 0.00184 < 0.00207$ ✓, compression rebar doesn't yield

$f_{si,row1} = 0.00189 \times 29000 = 53.5$ ksi

→

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$\varepsilon_{si,row2} = \frac{0.003}{4.2}(7.125-4.2)$

$\varepsilon_{si,row2} = 0.00207$

$f_{si,row2} = 60$ ksi

$M_{bi,row1} = 4(53.5)\cdot(5-1.63) = 721.2$ kip-in

$M_{bi,row2} = 4(60)\cdot(10/2 - 7.125) = 510.0$ kip-in

$M_b = 0.85f'_c\, b\, a\,[h/2 - c/2] + \Sigma M_{bi,rowi}$

$M_b = 0.85(6)(12)(0.75\cdot 4.2)[5-4.2/2] + 721.2 + 510.0$

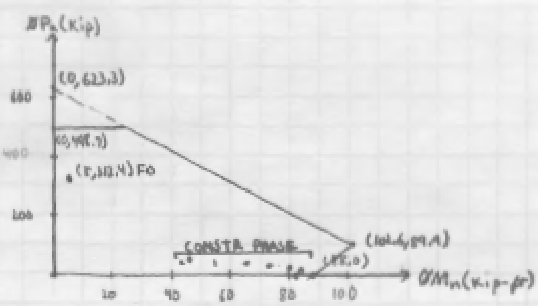
$M_b = 158$ kip-ft

$\phi M_b = 0.65(158)$

$\phi M_b = 102.7$ kip-ft

$P_b = 135.19$ kip

$\phi P_b = 87.9$ kip.



(a) Beams

Table I.17, Beam Flexural Design Based on spBeam and RAM Elements Output				
Beam	Level	h (in)	Top Reinforcement	Bottom Reinforcement
BG	1	48	(6) #8	(6) #8
	3	36	(4) #8	(4) #8
	5	36	(4) #8	(4) #8

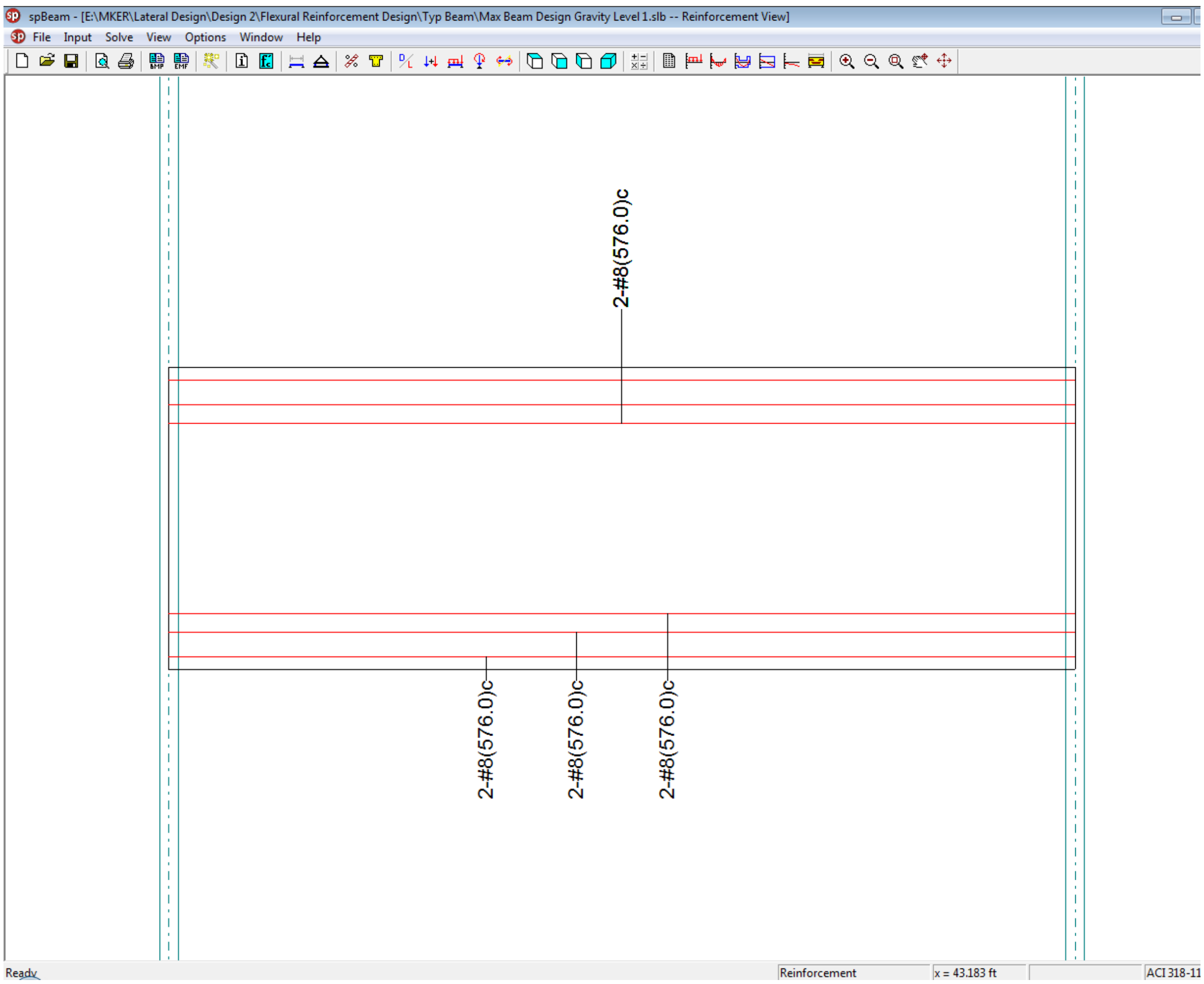


Figure I.2, spBeam Flexural Reinforcement Design of BG at Lvl. 1

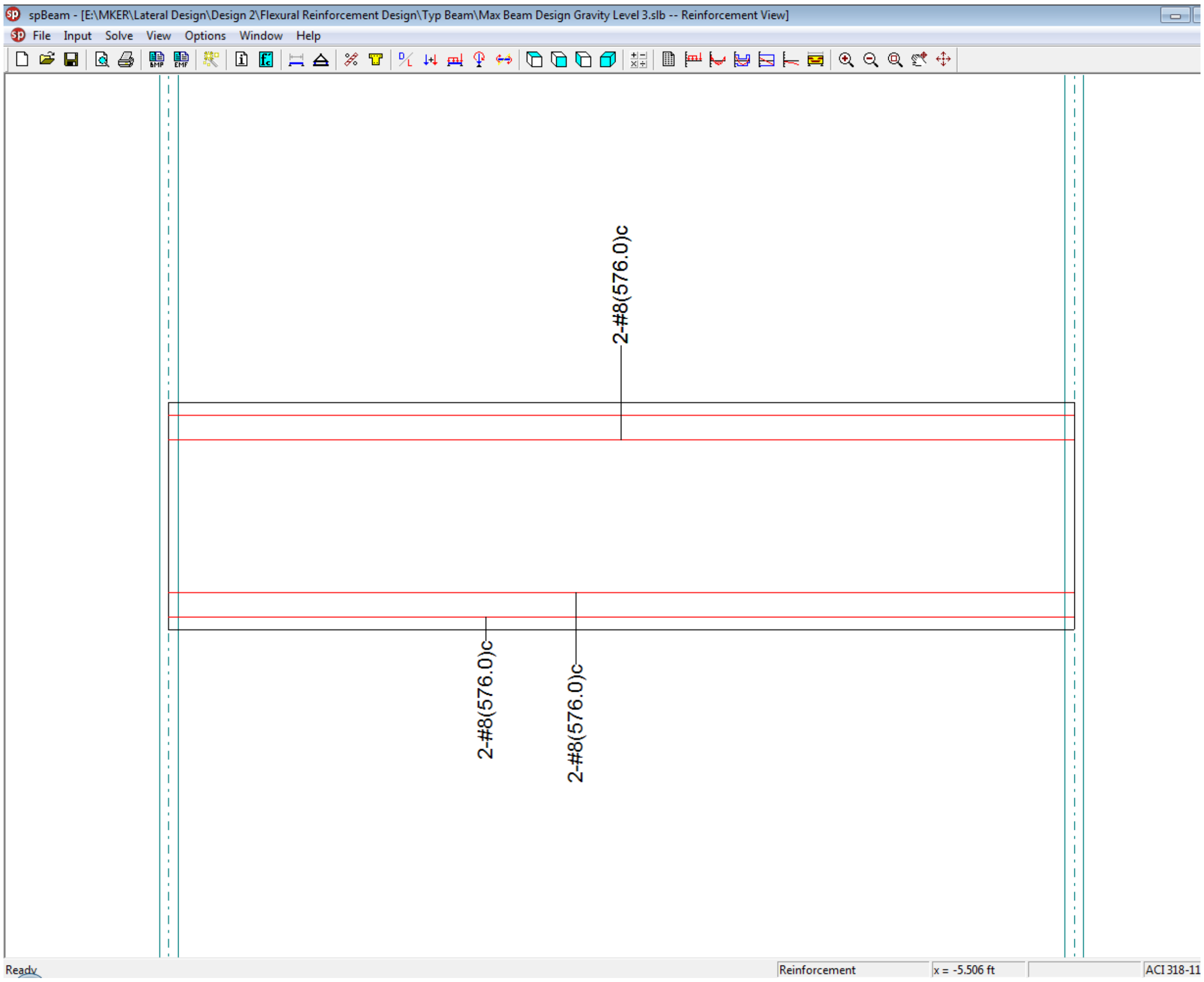


Figure I.3, spBeam Flexural Reinforcement Design of BG at Lvl. 3

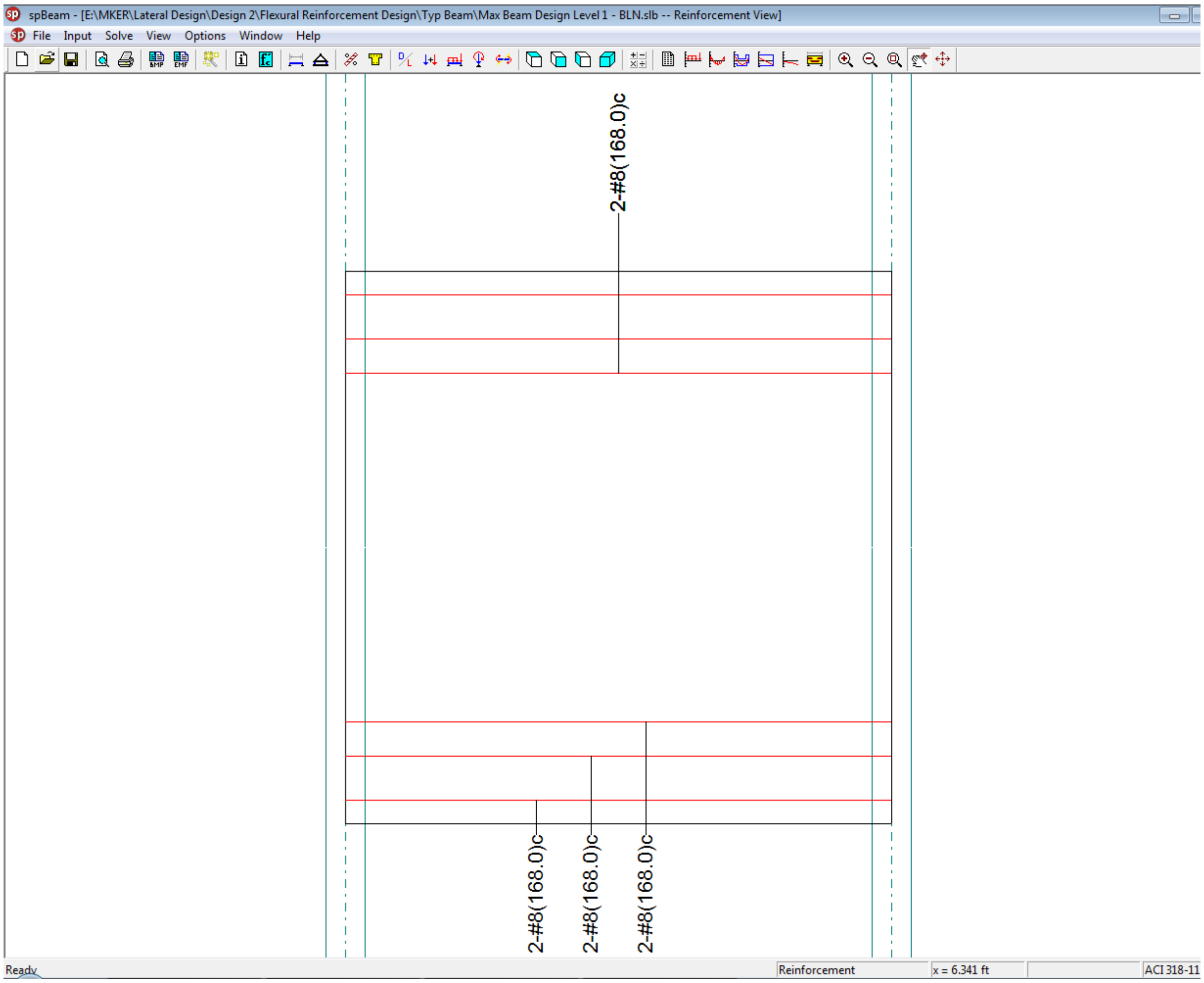


Figure I.4, spBeam Flexural Reinforcement Design of BLN Lvl. 1

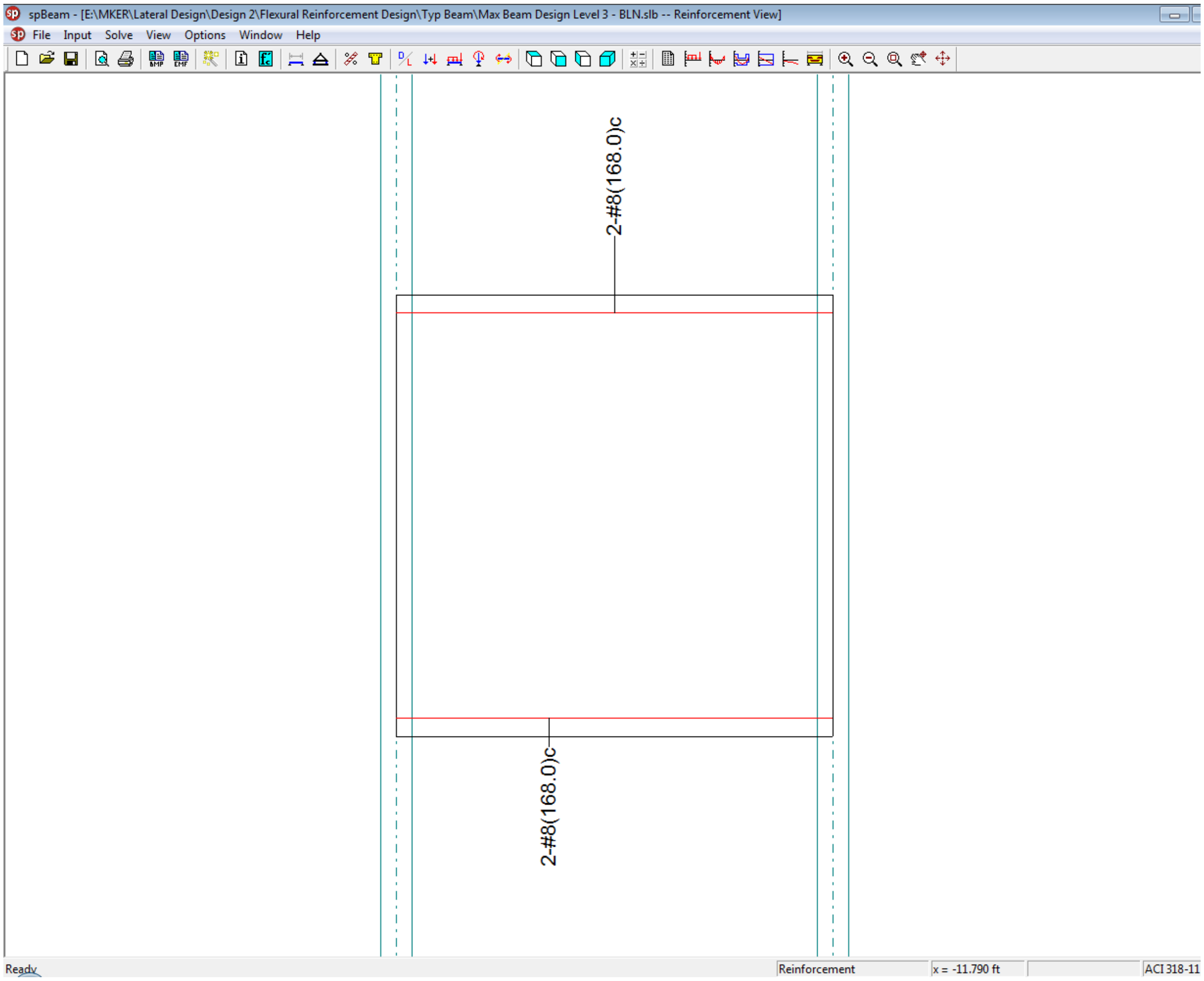


Figure I.5, spBeam Flexural Reinforcement Design of BLN Lvl. 3

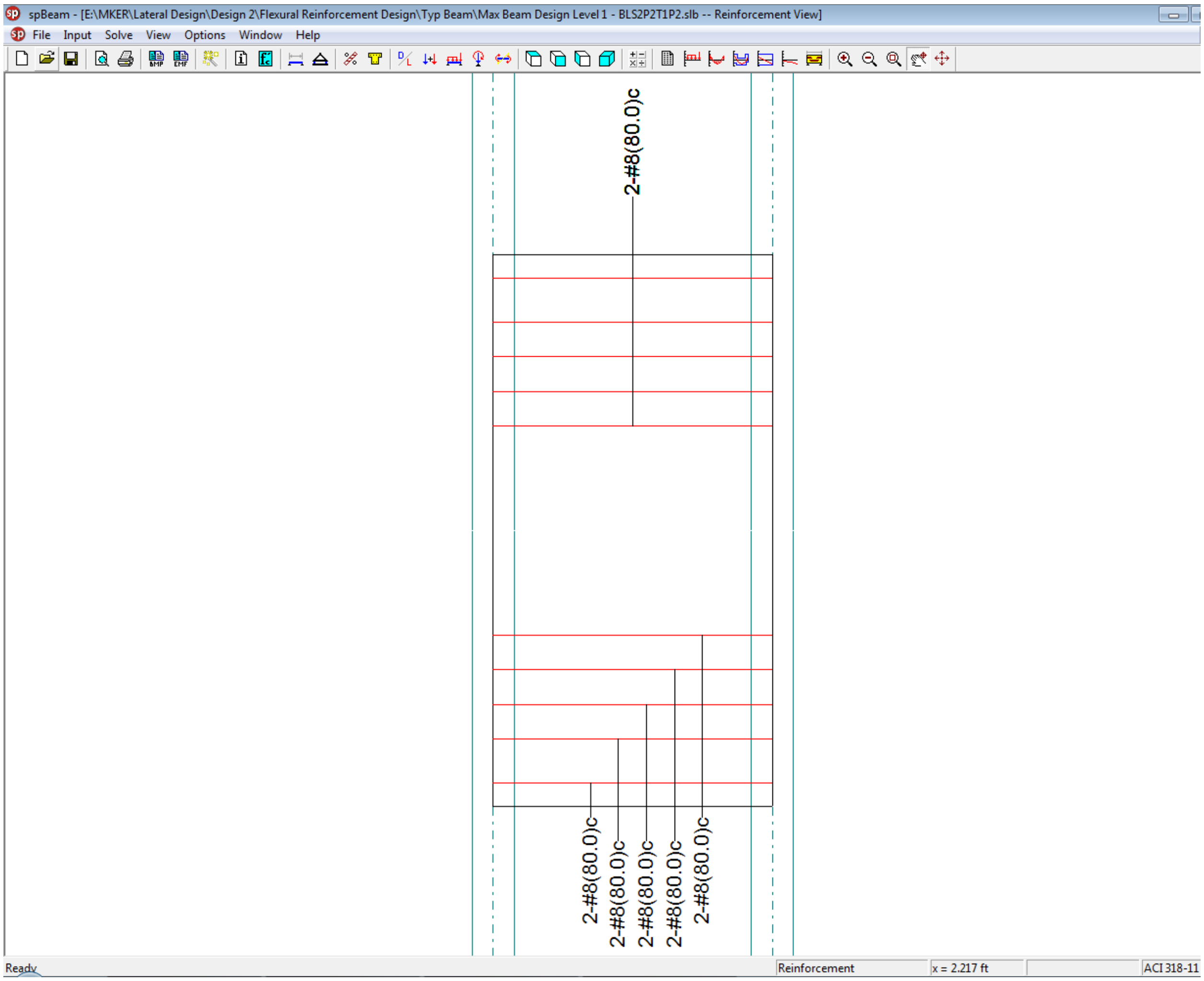


Figure I.6, spBeam Flexural Reinforcement Design of BLS2P2T1P2 Lvl. 1

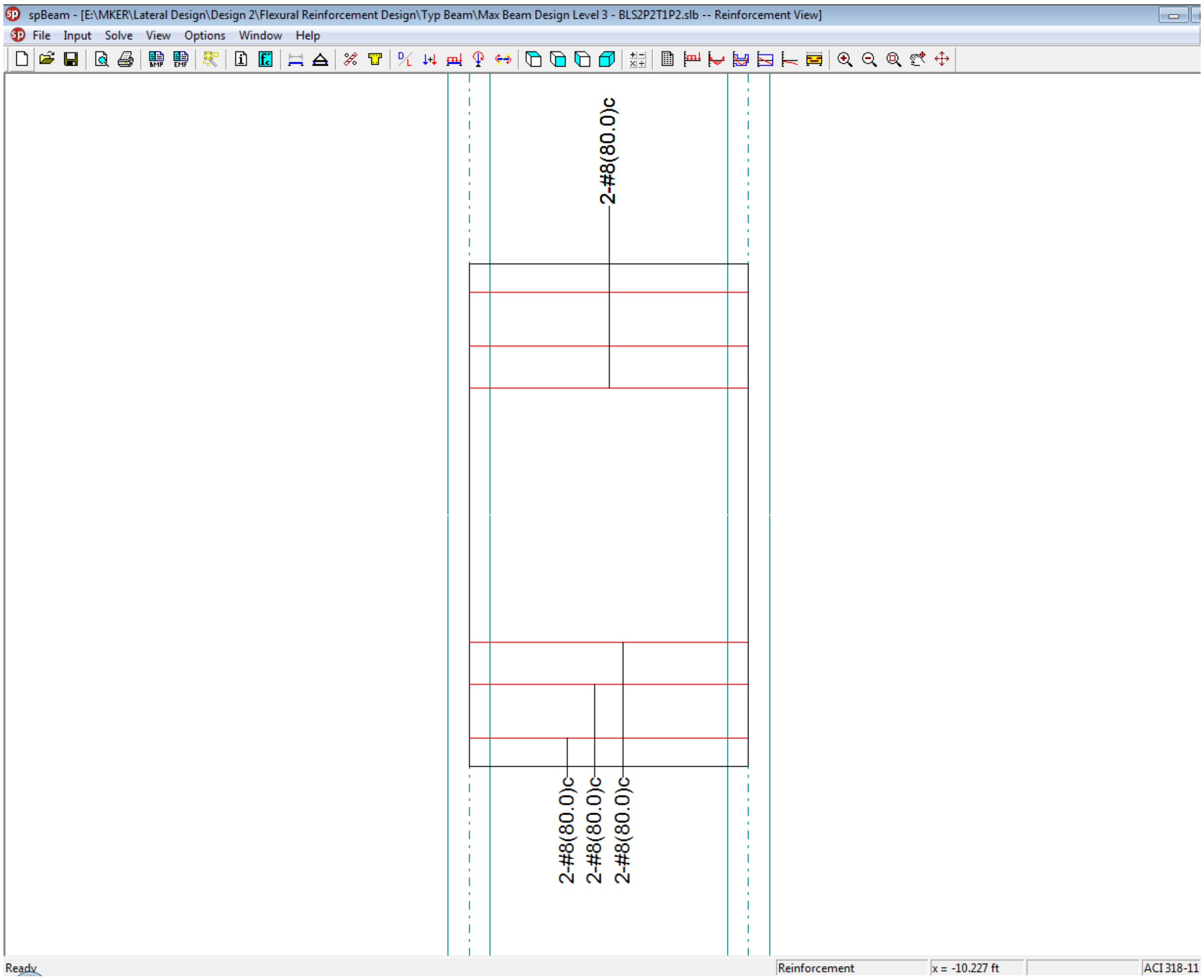


Figure I.7, spBeam Flexural Reinforcement Design of BLS2P2T1P2 Lvl. 3

Appendix J: Construction Breadth Calculations and Details

J.1 Quantity Take-Offs and Cost Estimate

J.1.1 Quantity Take-Offs

Figure AJ.1, Typical Lap Splice and Hook Lengths Determination			
A_{tr} =	0.22	in^2	
$S_{max, BM}$ =	5.00	in	
$S_{max, COL}$ =	10.0	in	
n_{BM} =	8.0		
n_{COL} =	4.0		
$K_{tr,BM}$ =	0.22		
$K_{tr,BM}$ =	0.22		
$c_{b,min}$ =	1.25	in	
$L_{d60,T}$ =	6.42	ft	, tension ACI 318-11 §12.2.3 Equation 12-1
$L_{d75,T}$ =	8.03	ft	
$L_{d60,C}$ =	1.29	ft	, compression members ACI 318-11 §12.3.2
$L_{d75,C}$ =	1.61	ft	
L_{hook60} =	1.55	ft	, 90° hooks ACI 318-11 §12.5.1, 12.5.2
$12\ d_B$ =	1	ft	

(a) Original Design and Design I

*** Only wall components unique to the wall system is counted and included in cost analysi:

Major Facade Quantities											
Concrete and Masonry Wall Designation	Concrete f'c = 6000 lb/in^2			Stucco	Rigid Insulation		Vapor Retarder (Fluid Applied)	Cement Board Sheathing	Mtl. Stud/Furring		Acoustical Insulation
	Layer Thk.	Height (ft.)	Area (ft^2)	Area (ft^2)	Thk. (in.)	Area (ft^2)	Area (ft^2)	Area (ft^2)	Quantity	Length (ft)	Area (ft^2)
NN1	10.0	86.0	1551.9	3696.9	1.0	3696.9	3696.9	2145.0	54.0	2835.0	2145.0
NN2	10.0	86.0	1488.7	3070.3	1.0	3070.3	3070.3	1581.7	34.0	1785.0	1581.7
NN3	10.0	86.0	1373.0	2241.0	1.0	2241.0	2241.0	868.0	26.0	1092.0	868.0
NN4	10.0	86.0	1277.3	1943.0	1.0	1943.0	1943.0	665.6	27.0	1134.0	665.6
NN5	10.0	86.0	1092.7	2439.1	1.0	2439.1	2439.1	1346.4	32.0	1344.0	1346.4
SN1	10.0	86.0	1117.3	3769.0	1.0	3769.0	3769.0	2651.6	47.0	2796.5	2651.6
SN2	10.0	86.0	1021.3	3225.5	1.0	3225.5	3225.5	2204.2	30.0	1785.0	2204.2
SN3	10.0	86.0	991.3	3053.3	1.0	3053.3	3053.3	2062.0	33.0	1963.5	2062.0
SN4	10.0	86.0	1473.7	3019.5	1.0	3019.5	3019.5	1545.8	30.0	1785.0	1545.8
SN5	10.0	86.0	1624.7	3397.5	1.0	3397.5	3397.5	1772.8	25.0	1487.5	1772.8
EN-1	10.0	83.5	941.3	2743.8	1.0	2743.8	2743.8	1802.5	35.0	2450.0	1802.5
EN-2	10.0	86.0	683.5	1416.0	1.0	1416.0	1416.0	732.5	11.0	770.0	732.5
EN-3	10.0	86.0	1286.0	1906.0	1.0	1906.0	1906.0	620.0	15.0	630.0	620.0
EN-4	10.0	86.0	1228.7	1642.1	1.0	1642.1	1642.1	413.4	10.0	420.0	413.4
WN1	10.0	86.0	1228.7	1642.1	1.0	1642.1	1642.1	413.4	10.0	420.0	413.4
WN2	10.0	86.0	1286.0	1906.0	1.0	1906.0	1906.0	620.0	15.0	630.0	620.0
WN3	10.0	86.0	888.7	2229.9	1.0	2229.9	2229.9	1341.2	34.0	2380.0	1341.2
WN4	10.0	86.0	660.0	1251.7	1.0	1251.7	1251.7	591.7	12.0	840.0	591.7

*** Generally detailed flashing layout was not provided, thus it was designed
*** Both redesign and original façade wall systems have the same flashing layout and design
*** Assume flashings are 2' high and bendable/foldable

Flashing Quantities		
Designatio	Length (ft.)	Area (ft^2)
Stick-On	1407.36	2815
AL	2172.00	4344
	2218.75	2773

Structural Component Quantities						
Major Facade Quantities	60 Ksi Rebar				75 Ksi Rebar	
	#3 Rebar		#8 Rebar		#8 Rebar	
	Shear Rebar Length (ft)	Weight (lb)	Flex. Rebar Length (ft)	Weight (lb)	Flex. Rebar Length (ft)	Weight (ft)
NN1	7679	2887	14272	38106		
NN2	7679	2887	11584	30929		
NN3	12169	4576	13044	34827		
NN4	13453	5058	10684	28526		
NN5	11518	4331	9696	25888		
SN1	7986	3003			8768	23411
SN2	7371	2772			7808	20847
SN3	7525	2829			7808	20847
SN4	9568	3597	13900	37113		
SN5	9568	3597	13996	37369		
EN-1	7924	2980	7808	20847		
EN-2	7679	2887	5600	14952		
EN-3	18398	6918	12512	33407		
EN-4	18152	6825	11604	30983		
WN1	18152	6825	11604	30983		
WN2	18398	6918	12512	33407		
WN3	9214	3465	8000	21360		
WN4	6143	2310	5408	14439		

*** Structural plates for connecting the tilt-up walls are not included because their design is beyond the scope of the project

Structural Steel and Fire Proofing Quantities					
Steel Member Designation	Unit Weight	Length (ft)	Total Weight (lb)	Quantity	Total Cementitious Fireproofing (ft^2)
W14x120	120	1634	196080		11955
W16x67	67	13740	920580		
W24x76	76	4056	308256		
L8x6x1	44.2	136	6024		
HSS9x9x5/16	36.1	1490	53789		
HSS12x12x3/8	58.1	1052	61112		
Anchor Bolts 1-1/2"				88	

Temporary Infrastructure Materials		
Material	Quantity	Units
Crushed Stone Base	4994	Yd^3
Asphalt	4994	Yd^3
Pre-Cast Foundations (24"x44"x20'	22	Each

*** Apart from the systems and equipment for tilt-up wall construction, all temporary infrastructure materials are the same for both the redesign and original

(c) Design II

*** Quantity of vapor retarder, stucco, and rigid insulation are the same
*** Only wall components unique to the wall system is counted and included in cost analysis

Major Facade Wall Quantities											
Concrete and Masonry Wall Designation	Concrete						Masonry			Mtl. Stud/Furring	
	f'c = 4000 lb/in^2			f'c = 6000 lb/in^2							
	Layer Thk.	Height (ft.)	Area (ft^2)	Layer Thk.	Height (ft.)	Area (ft^2)	Layer Thk.	Height (ft.)	Area (ft^2)	Quantity	Length (ft)
N1							7.6	86.0	2435.8	31	1946
AV5-X1				10.0	86.0	769.0				10	680
N2							7.6	86.0	6405.0	94	5240
N3							7.6	86.0	3357.8	41	2626
S1							7.6	86.0	16658.8	176	13243
E1							7.6	83.5	2594.3	31	2090
AV5-Y1				10.0	86.0	833.7				16	1190
AV5-Y2				10.0	83.5	1257.4				13	938
E2							7.6	86.0	2586.3	31	1946
W1							7.6	86.0	3573.3	47	3556
W2							7.6	86.0	4116.0	44	2884
AV1-X1	8.0	86.0	888.7								
AV1-Y1	8.0	86.0	1806.0								
AV2-Y1	8.0	86.0	2322.0								
AV2-X1	8.0	86.0	702.3								
AV3-X1	8.0	86.0	723.8								
AV3-Y2	8.0	86.0	1132.3								
AV4-Y1	8.0	86.0	1003.3								
AV4-X1	8.0	86.0	1755.8								

*** Flashing layout and design is the same as redesign

Flashing Quantities		
Designatio	Length (ft.)	Area (ft^2)
Stick-On	1407.36	2815
AL	2172.00	4344
	2218.75	2773

Structural Concrete and Masonry Component Quantities

Concrete and Masonry Wall Designation	Rebar: #3 Rebar: Rebar Length (ft): Shear	#3 Rebar: Weight (lb.)	#5 Rebar: Rebar Length (ft): Horizontal	#5 Rebar: Rebar Length (ft): Vertical	#5 Rebar: Weight (lb.)	#8 Rebar: Rebar Length (ft): Flexure	#8 Rebar: Weight (lb.)	#10 Rebar: Rebar Length (ft): Flexure	#10 Rebar: Weight (lb.)	Lintel/Bond Beams: Bond Beam (ft.)	Lintel/Bond Beams: Relieving Angle (ft.)
N1			1207	1201	2508					486	
AV5-X1	1143	430				1272	3396				
N2			3312	3216	6802					1274	531
N3			1935	1656	3742					650	
S1			8328	8272	17297					2706	135
E1			1195	1214	2510					490	73
AV5-Y1	1143	430						1272	5473		
AV5-Y2	4249	1598						3432	14768		
E2			1325	1288	2723					504	
W1			1770	1682	3597					680	
W2			2077	2076	4328					729	
AV1-X1	1377	518				1492	3984				
AV1-Y1	4320	1624				3024	8074				
AV2-Y1	7172	2697				3828	10221				
AV2-X1	1056	397				1352	3610				
AV3-X1	1043	392				1032	2755				
AV3-Y2	1904	716				1548	4133				
AV4-Y1	1700	639				1504	4016				
AV4-X1	4252	1599				2536	6771				

Structural Steel and Fire Proofing Quantities

Steel Member Designation	Unit Weight	Length (ft)	Total Weight (lb)	Total Cementitious Fireproofing (ft^2)
W14x120	120	3526	423120	25798.6
W16x67	67	15132	1013844	
W24x76	76	6768	514368	

*** Temporary infrastructure is the same as the redesign

Temporary Infrastructure

Material	Quantity (Yd^3)
Crushed Stone Base	4994
Asphalt	4994

J.1.2 Cost Estimate

Immediately below are the items used in all the cost estimates, all of which are taken from 2013 and 2014 R.S. Means. The purpose of including these snapshots in the report serve as proof that no values were in any way made up .

Line Number	Description	Unit	Crew	Daily Output	Labor Hours	Bare Material	Bare Labor	Bare Equipment
042210141150	Concrete block, back-up, normal weight, tooled joint one side, 2000 psi, 8" x 8	S.F.	D8	395.00	0.101	2.37	2.56	
015423751510	Scaffolding Specialties, sidewalk bridge, tubular steel scaffold frames, incl. plan	L.F.	3 Ca	45.00	0.533	5.57	15.17	
040516300700	Grout, cavity walls, 6" space, 0.500 C.F./S.F., pumped, excludes blockwork	S.F.	D4	800.00	0.050	2.21	1.17	0.16
042210162100	Concrete block, bond beam, normal weight, 2000 psi, 8" x 8" x 16", includes mortar	L.F.	D8	300.00	0.133	4.37	3.37	
051223451000	Lintel angle, structural, unpainted, over 2000 lb., shop fabricated, for galvanizing	Lb.		0.00		0.22		
033105350300	Structural concrete, ready mix, normal weight, 4000 psi, includes local aggregate, s	C.Y.		0.00		106.28		
033105350411	Structural concrete, ready mix, normal weight, 6000 PSI, includes local aggregate, :	C.Y.		0.00		128.17		
033105351420	Structural concrete, ready mix, for high-range water reducer/superplasticizer, add	C.Y.		0.00		6.36		
033105705000	Structural concrete, placing, walls, with crane and bucket, 8" thick, includes level	C.Y.	C7	80.00	0.900		19.52	14.88
031113852800	C.I.P. concrete forms, wall, job built, plywood, over 16' high, 3 use, includes er	SFC/	C2	315.00	0.152	0.90	3.75	
092423401000	Stucco, exterior, with bonding agent, 3 coats, on walls, excl. mesh	S.Y.	J1	200.00	0.200	3.48	4.96	0.70
092423400300	Stucco, 3 coats, float finish, 3/4" thick, excl. lath, for trowel finish, add	S.Y.	1 Pla	170.00	0.047		1.23	
032110600700	Reinforcing Steel, in place, walls, #3 to #7, A615, grade 60, incl labor for accessories,	Ton	4 Ro	3.00	10.667	953.00	392.80	
032110600750	Reinforcing Steel, in place, walls, #8 to #18, A615, grade 60, incl labor for acces	Ton	4 Ro	4.00	8.000	953.00	293.62	
032116100100	Epoxy coating, for reinforcing steel, add to fabricated & delivered price for coating with	Ton		0.00		443.15		
072129100350	Insulation, polyurethane foam, 2#/CF density, 5" thick, R32.5, sprayed	S.F.	G2A	1200.00	0.020	2.20	0.55	0.59
072113102100	Wall insulation, rigid, expanded polystyrene, 1" thick, R3.85	S.F.	1 Ca	800.00	0.010	0.24	0.37	
076510100020	Sheet metal flashing, aluminum, flexible, mill finish, .013" thick, including up to 4 benc	S.F.	1 Rot	145.00	0.055	0.73	1.20	
076513100750	Laminated sheet flashing, aluminum, mill finish, mastic-backed, self adhesive	S.F.	1 Rot	460.00	0.017	3.46	0.38	
075113400900	Felt, asphalt, #15, 4 square per roll, no mopping	Sq.	1 Rot	58.00	0.138	4.87	3.24	
051223177400	W14 x 120	L.F.	E2	960.00	0.050	154.63	1.90	1.57
051223753140	x 67	L.F.	E2	760.00	0.063	86.30	2.40	1.99
051223755500	x 76	L.F.	E5	1110.00	0.065	97.99	2.46	1.48

Line Number	Description	Unit	Crew	Daily Output	Labor Hours	Bare Material	Bare Labor	Bare Equipment	
321123230050	Base course drainage layers, aggregate base course for roadways and large pave	S.Y.	B36C	5200.00	0.008	3.09	0.23	0.77	
320610100100	Sidewalks, driveways, and patios, side walks, asphaltic concrete, 2-1/2" thick, ex	S.Y.	B37	660.00	0.073	10.21	1.90	0.23	
054113304340	Partition, galv LB studs, 16 ga x 3-5/8" W studs 16" O.C. x 8' H, incl galv top & I	L.F.	2 Ca	66.00	0.242	9.37	8.58		
092813100200	3' x 6' x 5/8" sheets	S.F.	2 Ca	350.00	0.046	0.87	1.29		
032110601200	Reinforcing Steel, for high strength, Grade 75, add to base	Ton		0.00		88.15			
032105750300	Splice rebar, standard, self-aligning type, taper threaded, #8 bars	Ea.	C25	115.00	0.278	15.82	8.10		
031519100510	Anchor bolt, L-type, 2-bolt set, plain steel, 1-1/2" dia x 24" L, incl nut & washer,	Set	2 Ca	32.00	0.500	52.75	13.85		
072726100100	Fluid applied membrane air barrier, 25 S.F./Gallon, spray	S.F.	1 Po	1375.00	0.006	0.01	0.19		
092213130030	Furring, beams & columns, galvanized, 7/8" channels, 12" O.C.	S.F.	1 Lat	155.00	0.052	0.38	1.29		
015433601400	Rent crawler mounted, lattice boom crane, cable, 200 ton, 70' boom	Ea.		0.00		126.54	2423.05	7254.32	
034105101300	Precast beam, rectangular, 20' span, 24" x 44", includes material only	Ea.	C11	22.00	2.545	3612.00	97.54	83.57	
051223770800	3 to 6 stories	Ton	E6	14.40	8.333	2382.35	316.58	127.58	
015433203320	Rent, sheepsfoot vibratory roller, 340 H.P.	Ea.		0.00	88.32	1532.95	4579.07	13747.10	1621.96
015433602800	Rent crane self-propelled, 4x4 telescoping boom 5 ton	Ea.		0.00	16.96	225.49	677.47	2027.45	271.18

(a) Original Design

Original Façade Wall System														
Cost Code	Item	Units	Required Quantity	Material Unit Cost [1] [2]	Material Cost	Labor Unit Cost [1] [2]	Daily Output per Crew [1] [2]	Unit Crew Size	Number of Crews	Labor Cost	Equipment Unit Cost [1] [2]	Equipment Cost	Total w/ Waste Factor	Notes
	CMU													
042210141150	8"	ft²	44587	$2.33	$103,888.64	$2.56	395.00	5	4	$114,143.74			$223,226.82	[3] [4]
015423751510	Scaffolding	ft.	3486	$5.60	$19,521.60	$18.48	45.00	3	5	$64,421.28			$84,918.96	[4]
040516300700	Grout	ft²	44587	$2.19	$97,646.41	$1.17	800.00	6	2	$52,167.26	$0.16	$7,133.98	$181,359.25	[5]
	Bond Beams													
042210162100	8"	ft.	7663	$4.31	$33,027.53	$3.37	300.00	5	1	$25,824.31			$60,503.22	[4]
	Lintels													
051223451000	Relieving Angles	lb.	883	$0.23	$203.09								$213.24	[4] [6]
	Reinforcement													
032110600700	#3 to #7 Rebar (60 Ksi)	Ton	25	$953.00	$23,976.91	$392.80	3.00	4	2	$9,882.61			$36,257.21	10% Waste Factor
032116100100	Epoxy Coating	Ton	25	$443.15	$11,149.39								$12,264.33	10% Waste Factor
	Barriers to Moisture Infiltration													
076510100020	Aluminum Flashing	ft²	7117	$0.72	$5,124.24	$1.20	145.00	1	7	$8,540.40			$13,920.85	[4] [7]
076513100750	Laminated Sheet Flashing, Self Adhered	ft²	2815	$3.46	$9,739.90	$0.38	460.00	1	1	$1,069.70			$11,296.60	[4] [7]
075113400900	Vapor Retarder/Waterproofing	100 ft²	446	$4.90	$2,185.03	$3.24	58.00	1	2	$1,444.80			$3,848.33	10% Waste Factor
	Misc.													
092213130030	Furring Strips/Resilient Channels	ft²	44587	$0.40	$17,834.96	$1.29	155.00	1	10	$57,517.75			$76,244.45	[4]
	Subtotals				$324,297.70					$335,011.85		$7,133.98	$704,053.26	
	Sales Tax (6%)				$19,457.86									[8]
	Overhead & Profit (10%)				$34,375.56					$33,501.18				[9]
	Subtotal				$378,131.11					$368,513.03		$7,133.98		
	Contingency (10%)				$37,813.11					$36,851.30		$713.40		
	Adjustments				-$413.71					-$403.19		-$7.81		[10]
	Total				$415,530.51					$404,961.15		$7,839.58	$869,747.80	

Original Structural System														
Cost Code	Item	Units	Required	Material Unit	Material Cost	Labor Unit	Daily Output	Unit Crew	Number of	Labor Cost	Equipment Unit	Equipment	Total w/ Waste	Notes
	Reinforcement													
032110600700	#3 to #7 Rebar (60 Ksi)	Ton	4	$953.00	$4,089.32	$392.80	3.00	4	2	$1,685.50			$6,183.76	10% Waste Factor
032110600750	#8 to #18 Rebar (60 Ksi)	Ton	22	$953.00	$20,758.25	$293.62	4.00	4	2	$6,395.63			$29,229.70	10% Waste Factor
032116100100	Epoxy Coating	Ton	26	$443.15	$11,554.25								$11,554.25	
	Concrete													
033105350300	4000 psi	Yd^3	255	$104.75	$26,728.58								$29,401.44	10% Waste Factor
033105351420	Superplasticizer	Yd^3	255	$6.26	$1,597.34								$1,757.07	10% Waste Factor
033105705000	Crane and Bucket for Walls	Yd^3	255			$19.58	80.00	13	1	$4,996.14	$15.00	$3,827.48	$8,823.62	
031113852800	Job Built Formwork Over 16' High, 3 Use	ft^2	20668	$0.92	$19,014.93	$3.76	315.00	6	4	$77,713.18			$98,629.60	10% Waste Factor
	Structural Steel													
051223177400	W14x120	ft.	3526	$154.63	$545,225.38	$1.90	960.00	7	1	$6,699.40	$1.57	$5,535.82	$557,460.60	
051223753140	W16x67	ft.	15132	$86.30	$1,305,891.60	$2.40	760.00	7	1	$36,316.80	$1.99	$30,112.68	$1,372,321.08	
051223755500	W24x76	ft.	6768	$97.99	$663,196.32	$2.46	1110.00	11	1	$16,649.28	$1.48	$10,016.64	$689,862.24	
	Fireproofing													
078116100700	Sprayed Cementitious	ft^2	25799	$0.61	$15,737.39	$0.67	1100.00	4	4	$17,285.33	$0.12	$3,095.88	$37,692.34	10% Waste Factor
	Misc.													
321123230050	Base Course Crushed Graded Stone	Yd^2	4994	$3.09	$15,431.46	$0.23	5200.00	10	1	$1,148.62	$0.77	$3,845.38	$21,968.61	10% Waste Factor
320610100100	2-1/2" Asphalt Road Topping	Yd^2	4994	$10.21	$50,988.74	$0.07	660.00	7	1	$364.56	$0.23	$1,148.62	$55,051.36	[4]
015433203320	Sheepsfoot Drum Vibratory Roller/Compactor	Per Day	18			$1,621.96				$29,195.28	$1,532.95	$27,593.10	$56,788.38	
015433602800	Self-Propelled 5 Ton Crane w/ Telesc. Boom	Per Day	80			$271.18				$21,694.40	$225.49	$18,039.20	$39,733.60	
	Subtotals				$2,680,213.55					$220,144.13		$103,214.80	$3,016,457.65	
	Sales Tax (6%)				$160,812.81									[8]
	Overhead & Profit (10%)				$284,102.64					$22,014.41				[9]
	Subtotal				$3,125,129.00					$242,158.54		$103,214.80		
	Contingency (10%)				$312,512.90					$24,215.85		$10,321.48		
	Adjustments				-$3,419.18					-$264.94		-$112.93		[10]
	Total				$3,434,222.72					$266,109.45		$113,423.36	$4,004,443.31	

(b) Design I

Revised Original Façade Wall System														
Cost Code	Item	Units	Required Quantity	Material Unit Cost [1] [2]	Material Cost	Labor Unit Cost [1] [2]	Daily Output per Crew [1] [2]	Unit Crew Size	Number of Crews	Labor Cost	Equipment Unit Cost [1] [2]	Equipment Cost	Total w/ Waste Factor	Notes
	CMU													
042210141150	8"	ft²	41727	$2.33	$97,224.61	$2.56	395.00	5	4	$106,821.89			$208,907.73	[3] [4]
015423751510	Scaffolding	ft.	3486	$5.60	$19,521.60	$18.48	45.00	3	5	$64,421.28			$84,918.96	[4]
040516300700	Grout	ft²	41727	$2.19	$91,382.79	$1.17	800.00	6	2	$48,820.94	$0.16	$6,676.37	$169,725.79	[5]
	Bond Beams													
042210162100	8"	ft.	7519	$4.31	$32,406.89	$3.37	300.00	5	1	$25,339.03			$59,366.26	[4]
	Lintels													
051223451000	Relieving Angles	lb.	739	$0.23	$169.97								$178.47	[4] [6]
	Reinforcement													
032110600700	#3 to #7 Rebar (60 Ksi)	Ton	22	$953.00	$20,731.09	$392.80	3.00	4	2	$8,544.77			$31,348.97	10% Waste Factor
032116100100	Epoxy Coating	Ton	22	$443.15	$9,640.06								$10,604.07	10% Waste Factor
	Insulation													
072113102100	1" Expanded Polystyrene	ft²	44593	$0.25	$11,148.13	$0.37	800.00	1	3	$16,499.23			$28,204.76	[4]
	Barriers to Moisture Infiltration													
076510100020	Aluminum Flashing	ft²	7117	$0.72	$5,124.24	$1.20	145.00	1	7	$8,540.40			$13,920.85	[4] [7]
076510100020	Laminated Sheet Flashing, Self Adhered	ft²	2815	$3.46	$9,739.90	$0.38	460.00	1	1	$1,069.70			$11,296.60	[4] [7]
075113400900	Vapor Retarder/Waterproofing	100 ft²	446	$4.90	$2,185.03	$3.24	58.00	1	2	$1,444.80			$3,848.33	10% Waste Factor
	Misc.													
092213130030	Furring Strips/Resilient Channels	ft²	41727	$0.40	$16,690.92	$1.29	155.00	1	10	$53,828.22			$71,353.68	[4]
	Subtotals				$315,965.22					$335,330.25		$6,676.37	$693,674.47	
	Sales Tax (6%)				$18,957.91									[8]
	Overhead & Profit (10%)				$33,492.31					$33,533.03				[9]
	Subtotal				$368,415.45					$368,863.28		$6,676.37		
	Contingency (10%)				$36,841.54					$36,886.33		$667.64		
	Adjustments				-$403.08					-$403.57		-$7.30		[10]
	Total				$404,853.91					$405,346.04		$7,336.70	$858,413.48	

Structural Solution (1)

Cost Code	Item	Units	Required	Material Unit	Material Cost	Labor Unit	Daily Output	Unit Crew	Number of	Labor Cost	Equipment Unit	Equipment	Total w/ Waste	Notes
	Reinforcement													
032110600700	#3 to #7 Rebar (60 Ksi)	Ton	6	$953.00	$5,260.56	$392.80	3.00	4	2	$2,168.26			$7,954.87	10% Waste Factor
032110600750	#8 to #18 Rebar (60 Ksi)	Ton	34	$953.00	$32,021.28	$293.62	4.00	4	2	$9,865.78			$45,089.18	10% Waste Factor
032116100100	Epoxy Coating	Ton	39	$443.15	$17,336.25								$17,336.25	
	Concrete													
033105350300	4000 psi	Yd^3	255	$104.75	$26,728.58								$29,401.44	10% Waste Factor
033105350411	6000 psi	Yd^3	71	$126.32	$8,920.69								$9,812.76	10% Waste Factor
033105351420	Superplasticizer	Yd^3	326	$6.26	$2,039.42								$2,243.36	10% Waste Factor
033105705000	Crane and Bucket for Walls	Yd^3	326			$19.58	80.00	13	1	$6,378.87	$15.00	$4,886.78	$11,265.65	
031113852800	Job Built Formwork Over 16' High, 3 Use	ft^2	26389	$0.92	$24,277.51	$3.76	315.00	6	4	$99,221.14			$125,926.40	10% Waste Factor
	Structural Steel													
051223177400	W14x120	ft.	3526	$154.63	$545,225.38	$1.90	960.00	7	1	$6,699.40	$1.57	$5,535.82	$557,460.60	
051223753140	W16x67	ft.	15132	$86.30	$1,305,891.60	$2.40	760.00	7	1	$36,316.80	$1.99	$30,112.68	$1,372,321.08	
051223755500	W24x76	ft.	6768	$97.99	$663,196.32	$2.46	1110.00	11	1	$16,649.28	$1.48	$10,016.64	$689,862.24	
	Fireproofing													
078116100700	Sprayed Cementitious	ft^2	25799	$0.61	$15,737.39	$0.67	1100.00	4	4	$17,285.33	$0.12	$3,095.88	$37,692.34	10% Waste Factor
	Misc.													
321123230050	Base Course Crushed Graded Stone	Yd^2	4994	$3.09	$15,431.46	$0.23	5200.00	10	1	$1,148.62	$0.77	$3,845.38	$21,968.61	10% Waste Factor
320610100100	2-1/2" Asphalt Road Topping	Yd^2	4994	$10.21	$50,988.74	$0.07	660.00	7	1	$364.56	$0.23	$1,148.62	$55,051.36	[4]
015433203320	Sheepsfoot Drum Vibratory Roller/Compactor	Per Day	18			$1,621.96				$29,195.28	$1,532.95	$27,593.10	$56,788.38	
015433602800	Self-Propelled 5 Ton Crane w/ Telesc. Boom	Per Day	80			$271.18				$21,694.40	$225.49	$18,039.20	$39,733.60	
	Subtotals				$2,713,055.17					$246,987.72		$104,274.10	$3,079,908.11	
	Sales Tax (6%)				$162,783.31									[8]
	Overhead & Profit (10%)				$287,583.85					$24,698.77				[9]
	Subtotal				$3,163,422.33					$271,686.49		$104,274.10		
	Contingency (10%)				$316,342.23					$27,168.65		$10,427.41		
	Adjustments				-$3,461.07					-$297.25		-$114.09		[10]
	Total				$3,476,303.49					$298,557.89		$114,587.42	$4,083,921.23	

(b) Design II

Redesigned Façade Wall System														
Cost Code	Item	Units	Quantity	Material Unit Cost [1] [2]	Material Cost	Labor Unit Cost [1] [2]	Daily Output per Crew [1] [2]	Unit Crew Size	Number of Crews	Labor Cost	Equipment Unit Cost [1] [2]	Equipment Cost	Total w/ Waste Factor	Notes
	Sheathing and Framing													
092813100200	5/8" Fiber Cement Board	ft²	23378	$9.37	$219,049.99	$8.58	66.00	2	12	$200,581.52	[redacted]	[redacted]	$430,584.01	[4]
015423751510	Scaffolding	ft.	3486	$5.60	$19,521.60	$18.48	45.00	3	5	$64,421.28	[redacted]	[redacted]	$84,918.96	[4]
054113305430	16 Gauge Mtl. Stud	ft.	26548	$18.15	$481,837.13	$8.98	63.00	2	15	$238,396.55	[redacted]	[redacted]	$744,325.53	[4]
	Insulation													
072113102100	1" Expanded Polystyrene	ft²	44593	$0.25	$11,148.13	$0.37	800.00	1	2	$16,499.23	[redacted]	[redacted]	$28,204.76	[4]
098436100970	3-1/2" Acoustical Insulation	ft²	23378	$0.35	$8,182.23	$0.16	1400.00	1	3	$3,740.45	[redacted]	[redacted]	$12,331.79	[4]
	Barriers to Moisture Infiltration													
076510100020	Aluminum Flashing	ft²	7117	$0.72	$5,124.24	$1.20	145.00	1	6	$8,540.40	[redacted]	[redacted]	$13,920.85	[4] [7]
076510100020	Laminated Sheet Flashing, Self Adhered	ft²	2815	$3.46	$9,739.90	$0.38	460.00	1	1	$1,069.70	[redacted]	[redacted]	$11,296.60	[4] [7]
072726100100	Vapor Retarder/Waterproofing (Fluid)	ft²	44593	$0.10	$4,459.25	$0.19	1375.00	5	4	$8,472.58	[redacted]	[redacted]	$13,154.79	[4] [7]
	Misc.													
092213130030	Furring Strips/Resilient Channels	ft²	41727	$0.40	$16,690.92	$1.29	155.00	1	10	$53,828.22	[redacted]	[redacted]	$71,353.68	[4]
	Subtotals				$775,753.38					$595,549.92		$0.00	$1,410,090.96	
	Sales Tax (6%)				$46,545.20									[8]
	Overhead & Profit (10%)				$82,229.86					$59,554.99				[9]
	Subtotal				$904,528.44					$655,104.91		$0.00		
	Contingency (10%)				$90,452.84					$65,510.49		$0.00		
	Adjustments				-$989.64					-$716.74		$0.00		[10]
	Total				$993,991.64					$719,898.66		$0.00	$1,799,584.81	

[1] Values referenced from R.S. Means 2013
[2] Open shop labor
[3] CMU laid in alternate courses
[4] Waste Factor is assumed to be 5%, unless noted
[5] Grout Waste factor is 25%, Provided by R.S. Means
[6] Wt. of relieving angles is based on stl. density of 490 lb/ft3
[7] All flashing are assumed to be 24" in height
[8] Sales tax is assumed to be 6%
[9] O+P is assumed to be 10%
[10] Location factor is 91.4 (total weighted average) for 2013 and 91.3 (total weighted average) for 2008, valuesfrom R.S. Means

Structural Solution (2)														
Cost Code	Item	Units	Quantity	Material Unit Cost [1] [2]	Material Cost	Labor Unit Cost [1] [2]	Daily Output per Crew [1] [2]	Unit Crew Size	Number of Crews	Labor Cost	Equipment Unit Cost [1] [2]	Equipment Cost	Total w/ Waste Factor	Notes
	Reinforcement													
032110600700	#3 to #7 Rebar (60 Ksi)	Ton	37	$953.00	$35,577.40	$392.80	3.00	4	2	$14,664.01			$53,799.15	10% Waste Factor
032110600750	#8 to #18 Rebar (60 Ksi)	Ton	249	$953.00	$237,412.79	$293.62	4.00	4	4	$73,147.05			$334,301.12	10% Waste Factor
032110601200	Reinforcing in Place, A615 Gr. 75	Ton	33	$88.15	$2,869.50								$3,156.45	10% Waste Factor
032116100100	Epoxy Coating	Ton	286	$443.15	$126,941.87								$139,636.06	10% Waste Factor
031519100510	1-1/2" Anchor Bolts, 24" Long	Set	44	$52.75	$2,321.00	$13.85	32.00	2	1	$609.40			$3,046.45	[4]
	Concrete													
033105350411	6000 psi	Yd³	655	$126.32	$82,711.14								$90,982.25	10% Waste Factor
033105351420	Superplasticizer	Yd³	655	$6.26	$4,098.89								$4,508.78	10% Waste Factor
033105705000	Crane and Bucket for Walls	Yd³	655			$19.58	80.00	13	2	$1,566.40	$15.00	$9,821.62	$11,388.02	
031113852800	Job Built Formwork Over 16' High, 3 Use	ft²	21215	$0.92	$19,517.52	$3.75	315.00	6	6	$79,555.13			$101,024.40	10% Waste Factor
	Structural Steel													
051223177400	W14x120	ft.	1634	$154.63	$252,665.42	$1.90	960.00	7	1	$3,104.60	1.57	2565.38	$258,335.40	
051223753140	W16x67	ft.	13740	$86.30	$1,185,762.00	$2.40	760.00	7	1	$32,976.00	1.99	27342.6	$1,246,080.60	
051223755500	W24x76	ft.	4056	$97.99	$397,447.44	$2.46	1110.00	11	1	$9,977.76	1.48	6002.88	$413,428.08	
051223400400	Angles 4" and Larger	lb.	6024	$0.67	$4,036.08	$2.09	440.00	4	2	$12,590.16	$0.33	$1,987.92	$18,614.16	
	Fireproofing													
078116100700	Sprayed Cementitious	ft²	11955	$0.61	$7,292.55	$0.67	1100.00	4	2	$8,009.85	$0.12	$1,434.60	$17,466.26	10% Waste Factor
	Misc.													
321123230050	Base Course Crushed Graded Stone	Yd³	4994	$3.09	$15,431.46	$0.23	5200.00	10	1	$1,148.62	$0.77	$3,845.38	$21,968.61	10% Waste Factor
320610100100	2-1/2" Asphalt Road Topping	Yd²	4994	$10.21	$50,988.74	$0.07	660.00	7	1	$364.56	$0.23	$1,148.62	$55,051.36	[4]
015433203320	Sheepsfoot Drum Vibratory Roller/Compactor	Per Day	18			$1,621.96				$29,195.28	$1,532.95	$27,593.10	$27,593.10	
015433601400	200 Ton Crawler Crane	Week	4	$126.54	$506.16	$2,423.05				$9,692.20	$12,315.92	$49,263.68	$59,462.04	
015433602800	Self-Propelled 5 Ton Crane w/ Telesc. Boom	Per Day	45			$271.18				$12,203.10	$225.49	$10,147.05	$22,350.15	
034105101300	Pre-Cast Conc. Foundations	Each	22	$3,612.00	$79,464.00	$97.54	22.00	6	1	$2,145.88	$83.57	$1,838.54	$83,448.42	
051223770800	Tilt-Up Temporary Bracing	Ton	58	$2,382.35	$138,176.30	$316.58	14.40	15	1	$18,361.64	$127.58	$7,399.64	$163,937.58	
	Subtotals				$2,643,220.26					$309,311.64		$150,391.01	$3,129,578.43	
	Sales Tax (6%)				$158,593.22									[8]
	Overhead & Profit (10%)				$280,181.35					$30,931.16				[9]
	Subtotal				$3,081,994.82					$340,242.81		$150,391.01		
	Contingency (10%)				$308,199.48					$34,024.28		$15,039.10		
	Adjustments				-$3,371.99					-$372.26		-$164.54		[10]
	Total				$3,386,822.32					$373,894.83		$165,265.57	$4,122,281.85	

J.2 Temporary Bracing

J.2.1 Temporary Bracing Design

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a) Initial Design Parameters

1) Length of Brace

$$L_{\text{ground to pt 1 or 2}} = \sqrt{44^2 + 11.79^2}$$
$$L_{\text{ground to pt 1 or 2}} = 45.54' = 547''$$

$$L_{\text{ground to pt 3 or 4}} = \sqrt{72^2 + 19.29^2}$$
$$L_{\text{ground to pt 3 or 4}} = 74.54' = 894''$$

** Braces are angled 15° from vertical.

2) Minimum Moment of Inertia

Brace w/ 29.8 kip Axial

$$A_g = P_u / F_{cr}$$
$$A_g = \frac{P_u}{0.877 F_e}$$
$$A_g = \frac{P_u}{\dfrac{0.877\pi^2 E}{\left(\frac{KL}{r}\right)^2}}$$
$$A_g = \frac{P_u}{\dfrac{0.9(0.877\pi^2 E)}{\left(\frac{KL}{r}\right)^2}}$$
$$A_g = \frac{P_u}{\dfrac{0.9(0.877\pi^2 E) r^2}{(KL)^2}}$$
$$A_g = \frac{P_u (KL)^2}{0.9(0.877\pi^2 E) r^2}$$
$$A_g = \frac{P_u (KL)^2 A_g}{0.9(0.877\pi^2 E) I_g}$$
$$I_g = \frac{P_u (KL)^2}{0.9(0.877\pi^2 E)}$$
$$I_g = \frac{29.8(1 * 894)^2}{0.9(0.877)(\pi^2)(29000)}$$
$$I_g = 105.5\ \text{in}^4$$

** No shapes will be slender
** All shapes are compact
** Assume $KL/r > 4.71\sqrt{\frac{E}{F_y}}$

$4.71\sqrt{\frac{29000}{46}} = 118.26$, A500 Gr B.

$r = \sqrt{\frac{I_g}{A_g}}$

K = 1, pin-pin ends

Brace w/ 97.9 kip Axial

$$I_g = \frac{97.9(1 * 547)^2}{0.9(0.877)(\pi^2)(29000)}$$
$$I_g = 129.5\ \text{in}^4$$

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→

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Brace w/ 235.3 Kip Axial

$$I_g = \frac{235.3(1 \times 547)^2}{0.9(0.877)(\pi^2)(29000)}$$

$$I_g = 311.2 \text{ in}^4$$

b) Potential Bracing Members

Member	ℓ (in)	Member Properties		
		I (in⁴)	A (in²)	L/r
HSS10x10x3/8	894	202	13.2	228.65
HSS10x10x3/8	547	202	13.2	139.73
HSS12x12x1/2	547	457	20.9	116.90

** Member properties were referenced off of Table 1-12 of AISC STEEL CONSTR. MANUAL

1) Axial & Bending Capacity (LRFD)

HSS10x10x3/8, $\ell = 894''$

$I = 202 > 105.5$ ✓

$P_u = 0.9 \times 0.877 \times F_e A_g$
$P_u = 0.9 \times 0.877 \times 5.47 \times 13.2$
$P_u = 57.04$ Kip

$KL/r > 118.26$
$228.65 > 118.26$ ✓ use E3-3 and E3-4

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$F_e = \frac{\pi^2(29000)}{(228.65)^2}$$

$F_e = 5.47$ kip/in²

$M_u = 0.9 \times F_y Z_x$, AISC STL CONSTR MANUAL §F7
$M_u = 0.9(46)(47.2)/12$
$M_u = 162.8$ kip-ft

HSS10x10x3/8, $\ell = 547$

$I = 202 > 129.5$ ✓

$$P_u = 0.9(0.877)\left[\frac{\pi^2(29000)}{(139.73)^2}\right] \times 13.2$$

$P_u = 152.72$ Kip

$KL/r > 118.26$
$139.73 > 118.26$ ✓ use E3-3 and E3-4

$M_u = 0.9(46)(47.2)/12$
$M_u = 162.8$ kip-ft

$$M_{u,local} = 0.9\left[F_y Z - (F_y Z - F_y S_x)\left(0.305 \frac{b}{t_{w}}\sqrt{\frac{F_y}{E}} - 0.738\right)\right]$$

$$M_{u,local} = 0.9\left[46(47.2) - (46 \cdot 47.2 - 46 \cdot 40.4)(-0.426)\right]$$

$M_{u,local} = 172.83$ kip-ft

→

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HSS12x12x1/2, $\ell = 547$

$I = 457 > 311.2$ ✓

$P_u = 0.9\left[0.658^{F_y/F_e} \times F_y\right]A_g$

$P_u = 0.9\left[0.658^{2.196} \times 46\right]20.9$

$P_u = 345.09$ Kip

$M_u = 0.9(46)(89.6)/12$

$M_u = 309.1$ Kip-ft

$KL/r < 118.26$

$116.9 < 118.26$ ✓ use E3-2

$F_y/F_e = \dfrac{46}{\dfrac{\pi^2(29000)}{(116.9)^2}} = 2.196$

$M_{u,local} = 0.9\left[46(89.6) - (46 \times 89.6 - 46 \times 76.2)(-0.461)\right]$

$M_{u,local} = 330.4$ Kip-ft

2) Non-Translation and Translation Loads

** Translation loads are not present because pin-pin braces are not directly a lateral force resisting system per note in AISC STL CONSTR MANUAL §A8.2 of SPEC

** Only member (brace) self-wt contribute to the moment causing bending

Member	P_{nt} (Kip)	P_r (Kip)	M_{nt} (Kip-ft)	ℓ (in)
HSS10x10x3/8	29.8	29.8	39.9	894
HSS10x10x3/8	97.8	97.8	14.9	547
HSS12x12x1/2	235.3	235.3	23.7	547

3) Load Magnification Factors

** $B_2 = 0$, braces are not directly a lateral force resisting system per note in AISC STL CONSTR MANUAL §A8.2 of SPEC.

$B_1 = \dfrac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$, AISC STL CONSTR MANUAL §8.2.1

$C_m = 1$; transverse loading (brace self-wt) is present btw. supports, conservative

$P_{e1} = \dfrac{\pi^2 EI^*}{(K_1 L)^2}$

** No direct translation at brace ends $\Rightarrow K_1 = 1.0$

$EI^* = 0.8\tau_b EI$

$\tau_b = 1.0$; because $(P_r/P_y)\alpha < 0.5$, per AISC STL CONSTR MANUAL Chapter C (SPEC) §C2.3(2)

→

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HSS10x10x3/8 $\ell = 894''$

$$P_{e1} = \frac{\pi^2 (0.8 \times 1.0 \times 29000 \times 202)}{(1.0 \times 894)^2}$$

$P_{e1} = 57.8$ Kip

$$B_1 = \frac{1}{1 - 1(29.8/57.8)}$$

$B_1 = 2.1$

HSS10x10x3/8, $\ell = 547''$

$$P_{e1} = \frac{\pi^2 (0.8 \times 1.0 \times 29000 \times 202)}{(1.0 \times 547)^2}$$

$P_{e1} = 154.8$ Kip

$$B_1 = \frac{1}{1 - 1(97.9/154.8)}$$

$B_1 = 2.7$

HSS12x12x1/2, $\ell = 547''$

$$P_{e1} = \frac{\pi^2 (0.8 \times 1.0 \times 29000 \times 457)}{(1.0 \times 547)^2}$$

$P_{e1} = 350.2$ Kip

$$B_1 = \frac{1}{1 - 1(135.3/350.2)}$$

$B_1 = 3.0$

4) Axial-Bending Interaction

$M_r = B_1 M_{nt} + B_2 M_{lt}$

$P_r = P_{nt} + B_2 P_{lt}$

HSS10x10x3/8, $\ell = 894''$

$P_r = P_{nt}$

$P_r = 29.8$ Kip

$P_r/P_c = 29.8/57.04$

$P_r/P_c = 0.52 \geq 0.2$, use Eq. H1-1a

$M_r = 2.1(39.9)$

$M_r = 82.4$ Kip·ft

$M_r/M_c = 82.4/162.8$

$M_r/M_c = 0.51$

$$\frac{P_r}{P_c} + \frac{8}{9}\frac{M_r}{M_c} \leq 1$$

$\rightarrow$

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$0.52 + \frac{8}{9}(0.51) \leq 1$

$0.97 \leq 1$ ✓, can use HSS 10×10×3/8 $l = 894''$

HSS10×10×3/8, $l = 547''$

$P_r = 97.9$ kip

$M_r = 2.7(14.9)$

$M_r = 40.6$ kip-ft

$P_r/P_c = 97.9/152.72$

$P_r/P_c = 0.64 \geq 0.2$, use Eq. H1-1a

$M_r/M_c = 40.6/162.8$

$M_r/M_c = 0.25$

$\frac{P_r}{P_c} + \frac{8}{9}\frac{M_r}{M_c} \leq 1$

$0.86 \leq 1$ ✓, can use HSS 10×10×3/8 $l = 547''$

HSS12×12×1/2, $l = 547''$

$P_r = 235.2$ kip

$M_r = 3(23.7)$

$M_r = 72.2$ kip-ft

$P_r/P_c = 235.2/345.09$

$P_r/P_c = 0.68 \geq 0.2$, use Eq H1-1a

$M_r/M_c = 72.2/309.1$

$M_r/M_c = 0.23$

$\frac{P_r}{P_c} + \frac{8}{9}\frac{M_r}{M_c} \leq 1$

$0.89 \leq 1$ ✓, can use HSS12×12×1/2 $l = 547''$

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Appendix K: Façade Breadth Calculations and Details

K.1 Thermal Comfort

K.1.1 Thermal Comfort Benchmark

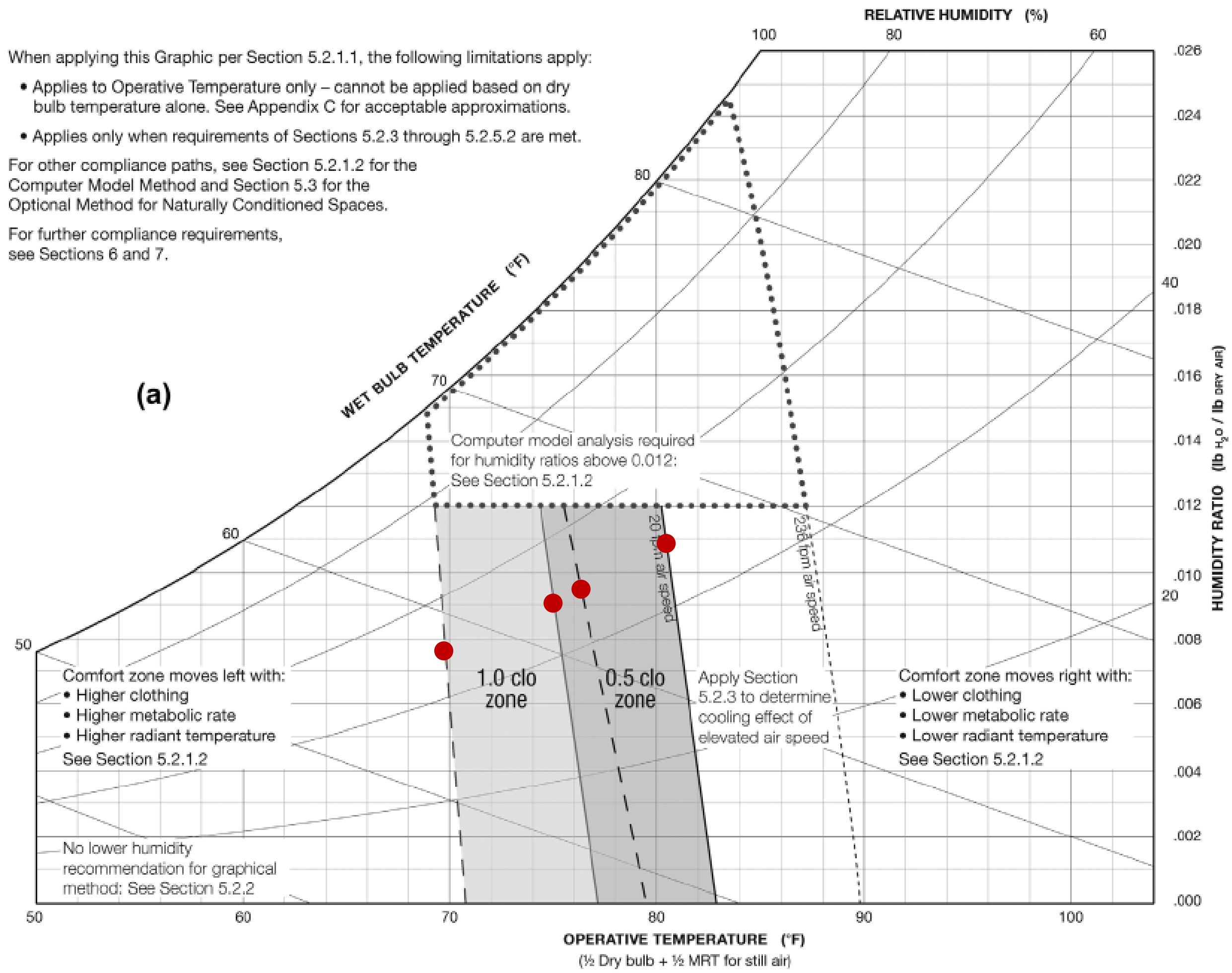

Figure AK.1, Comfort Zones
Source: ASHRAE §5.2.1.1

TABLE 7-1 Design Parameters

Function of Space	Pressure Relationship to Adjacent Areas (n)	Minimum Outdoor ach	Minimum Total ach	All Room Air Exhausted Directly to Outdoors (j)	Air Recirculated by Means of Room Units (a)	RH (k), %	Design Temperature (l), °F/°C
SURGERY AND CRITICAL CARE							
Class B and C operating rooms, (m), (n), (o)	Positive	4	20	N/R	No	20~~30~~–60	68–75/20–24
Operating/surgical cystoscopic rooms, (m), (n), (o)	Positive	4	20	N/R	No	20~~30~~–60	68–75/20–24
Delivery room (Caesarean) (m), (n), (o)	Positive	4	20	N/R	No	20~~30~~–60	68–75/20–24
Treatment room (p)	N/R	2	6	N/R	N/R	20~~30~~–60	70–75/21–24
Trauma room (crisis or shock) (c)	Positive	3	15	N/R	No	20~~30~~–60	70–75/21–24
Laser eye room	Positive	3	15	N/R	No	20~~30~~–60	70–75/21–24
Class A Operating/Procedure room (o), (d)	Positive	3	15	N/R	No	20~~30~~–60	70–75/21–24
DIAGNOSTIC AND TREATMENT							
Gastrointestinal endoscopy procedure room	Positive	2	6	N/R	No	20~~30~~–60	68–73/20–23

Figure AK.1A, Recommended Humidity Levels
Source: Addendum D ASHRAE 170-2008

Activity	MET Units[b]	Btu/h ft²	W/
Resting			
Sleeping	0.7	13	40
Reclining	0.8	15	45
Seated, quiet	1.0	18	60
Standing, relaxed	1.2	22	70
Walking (on the level)			
2 mph (0.9 m/s)	2.0	37	115
3 mph (1.2 m/s)	2.6	48	150
4 mph (1.8 m/s)	3.8	70	220
Office activities			
Reading, seated	1.0	18	60
Writing	1.0	18	60
Typing	1.1	20	65
Filing, seated	1.2	22	70
Filing, standing	1.4	26	80
Walking about	1.7	31	100
Lifting, packing	2.1	39	120
Driving/flying			
Car	1.0–2.0	18–37	60–11
Aircraft, routine	1.2	22	70
Aircraft, instrument landing	1.8	33	105
Aircraft, combat	2.4	44	140
Heavy vehicle	3.2	59	185
Miscellaneous occupational activities			
Cooking	1.6–2.0	29–37	95–11
House cleaning	2.0–3.4	37–63	115–2
Seated, heavy limb movement	2.2	41	130
Handling 110-lb (50-kg) bags	4.0	74	235
Pick and shovel work	4.0–4.8	74–88	235–2

Figure AK.1B, Metabolic Rate of Typical Activities
Source: ASHRAE Handbook – Fundamentals

Ensemble Description[a]	CLO[b]
Walking shorts, short-sleeve shirt	0.36
Trousers, short-sleeve shirt	0.57
Trousers, long-sleeve shirt	0.61
Same as above, plus suit jacket	0.96
Same as above, plus vest and T-shirt	1.14
Trousers, long-sleeve shirt, long-sleeve sweater, T-shirt	1.01
Same as above, plus suit jacket and long underwear bottoms	1.30
Sweat pants, sweat shirt	0.74
Long-sleeve pajama top, long pajama trousers, short ¾-sleeve robe, slippers (no socks)	0.96
Knee-length skirt, short-sleeve shirt, pantyhose, sandals	0.54
Knee-length skirt, long-sleeve shirt, full slip, pantyhose	0.67
Knee-length skirt, long-sleeve shirt, half slip, pantyhose, long-sleeve sweater	1.10
Same as above; replace sweater with suit jacket	1.04
Ankle-length skirt, long-sleeve shirt, suit jacket, pantyhose	1.10
Long-sleeve coveralls, T-shirt	0.72
Overalls, long-sleeve shirt, T-shirt	0.89
Insulated coveralls, long-sleeve thermal underwear, long underwear bottoms	1.37

Figure AK.1C, Metabolic Rate of Typical Activities
Source: ASHRAE Handbook – Fundamentals

Table AK.1, Clothing Level (clo)		
Classification	Season	
	Summer	Winter
Clinic Personnel	0.61	0.96
Patients	0.57	0.96

Table AK.2, Metabolic Rate (Met)	
Walking About	1.70
Seated	1.00

Table AK.3, Interior Target Temperature (°F)	
Summer	Winter
76	72

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Facade - THERMAL COMFORT

a) Clinic Personnel

1) Summer

$T_{min,1.0clo} = 70°F$
$T_{min,0.5clo} = 76°F$
$T_{max,0.5clo} = 81°F$
$T_{max,1.0clo} = 75°F$
Determined using ASHRAE §5.2.1.1 Graph, shown above

$$T_{min,Icl} = [T_{min,1.0clo} * (\text{Clothing Level} - 0.5) + T_{min,0.5clo} * (1 - \text{Clothing Level})] / 0.5$$
$$T_{min,Icl} = [70 * (0.61 - 0.5) + 76 * (1 - 0.61)] / 0.5$$
$$T_{min,Icl} = 74.68°F$$

$$T_{max,Icl} = [T_{max,1.0clo} * (\text{Clothing Level} - 0.5) + T_{max,0.5clo} * (1 - \text{Clothing Level})] / 0.5$$
$$T_{max,Icl} = [75 * (0.61 - 0.5) + 81 * (1 - 0.61)] / 0.5$$
$$T_{max,Icl} = 79.68°F$$

2) Winter

$$T_{min,Icl} = [70 * (0.96 - 0.5) + 76 * (1 - 0.96)] / 0.5$$
$$T_{min,Icl} = 70.48°F$$

$$T_{max,Icl} = [75 * (0.96 - 0.5) + 81 * (1 - 0.96)] / 0.5$$
$$T_{max,Icl} = 75.48°F$$

b) Patients

1) Summer

$$T_{min,Icl} = [70 * (0.57 - 0.5) + 76 * (1 - 0.57)] / 0.5$$
$$T_{min,Icl} = 75.16°F$$

$$T_{max,Icl} = [75 * (0.57 - 0.5) + 81 * (1 - 0.57)] / 0.5$$
$$T_{max,Icl} = 80.16°F$$

2) Winter

$$T_{min,Icl} = [70 * (0.96 - 0.5) + 76 * (1 - 0.96)] / 0.5$$
$$T_{min,Icl} = 70.48°F$$

$$T_{max,Icl} = [75 * (0.96 - 0.5) + 81 (1 - 0.96)] / 0.5$$
$$T_{max,Icl} = 75.48°F$$

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