Voorhees Replacement Hospital

Voorhees, New Jersey

Final Report

Paul Stewart

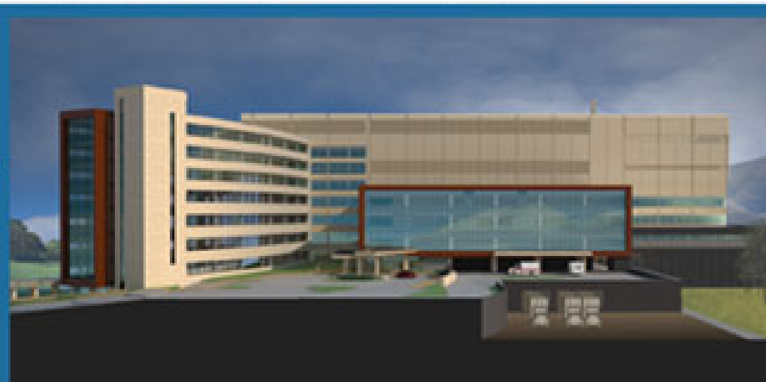
Structural Option

Consultant: Dr. Ali M. Memari

April 7th, 2010

General Building Statistics:

Location: Voorhees, New Jersey
Size: 642,000 ft^2
Total Levels: 9
Construction Dates: March 2008 - March 2011
Construction Cost: $323 Million

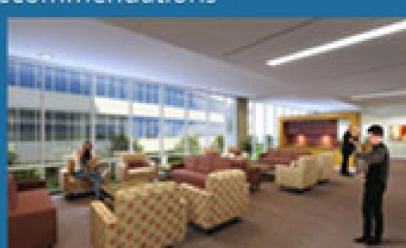


Design Team:

Owner: Virtua Health
Owner's Rep: Hammes Company
Architect/Engineers: HGA Architects and Engineers, LLC
Geotechnical Engineer: Lippincott & Jacobs Consulting
Elevator Consultant: Lerch, Bates, & Associates
Construction Manager: The Turner Company

Lighting/Electrical

(6) 2000 KVA units transform produce 277Y/480 Volt service
(3) 1500 KW generators provide emergency power
Uses a rotary UPS in order to provide uninterrupted power during a power loss
Majority of Interior Lighting is recessed fluorescent fixtures designed to follow IESNA recommendations



Structural

Steel structure
Concrete footings reston on concrete piers
Steel bracing and moment frames
Composite steel/concrete slab system
Geopiers required to densify the loose soils

Mechanical

(10) Air handling units serving various portions of the building
(3) 1,000 ton water cooled chillers, with an area for 2 additional chillers if needed
(4) 2,083 GPM cooling towers, with an area for 2 additional towers if needed

Paul Stewart | Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2010/prs5011/

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Executive Summary

The Voorhees Replacement Hospital is a new hospital replacing the current Voorhees hospital due to its inability to expand and be renovated. The new building is 9 stories tall, approximately 140 feet tall. It consists of two parts, a main bed tower, and a services building.

In the final report the current building's systems will be explained in depth. An in depth description of the mechanical, electrical, lighting, telecommunication, and structural systems will be given. The architecture of the building will also be explained as well as some of the building's sustainable features.

A description of the proposed changes to the building will then be given. An overview look at the potential benefits of the changes will be given, and the process of making the changes will be explained.

The current helipad will be looked at and the possibility of moving it from the parking area to the top of the building will be explored. This report will look into possible problems and requirements that a helipad located on the roof will require. A location will then be determined based on these requirements. In order to move it from its current location, the helipad will be redesigned as a two-way slab with beams spanning from column to column. The beams supporting the slab will also be designed as concrete beams. Due to the added weight on the roof, the gravity columns that support the new helipad will be adjusted for the added weight. Also, since weight is added to the columns, the foundations will also need to be redesigned. These new foundations will then be checked against a RAM model to insure they are accurate.

In order to eliminate moment frames throughout the building, the lateral system will also be changed. The lateral system will be changed in the Southern Building from a combination of braced frames and moment frames, to a system that only uses braced frames. Since changing the lateral system will also change the seismic forces, these forces will need to be redesigned. In order to design the new system a RAM model will be created. This model will assist in the design and analysis of the new building.

Because new braces will be added to the building, a study of their new location will be performed to insure that the architecture is unchanged. The location of these new braces will then be plugged into the RAM model and a design will be performed. The location and sizes of the new braces will then be reported. Since the forces on the foundations will also change, the foundations will also need to be redesigned. The new foundations are redesigned and the sizes are reported.

The new system will also be analyzed for a number of different factors. The analysis will look at the new building's drift, center of mass and center of rigidity, torsion, overturning moment, and constructability. These factors are then compared to the old system's values. It is found that the new systems' seismic forces are reduced due to the new R value. It is also found that by eliminating moment frames, time and money can be saved with the new system. Although it is found that the new system will have a larger drift, torsion, and overturning moment when compared to the old system.

Due to the new type of lateral system a design of the connections for the new braces will be performed. Two brace to column connections, a brace to beam connection, and a brace to brace connection will be designed and reported in this report.

Because of the new location of the helipad, an electrical breadth and an acoustical breadth will be performed. The electrical study will determine new wire sizes for lights located on the new helipad. These wires will also be tied into an existing circuit breaker. The acoustical study will look at the potential sound entering patients' rooms. The study will find that a new façade type will be necessary in order to eliminate sound entering the space.

Acknowledgements

To Virtua Health:
For permitting the use of the Voorhees Replacement Facility for this project.

To The Turner Company:
Especially Bill Swanson and Chris McCrae
For taking time out of your day to track down project information, for providing me with construction documents, providing me with a tour of the site, and answering my questions.

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For helping me keep my sanity throughout the year.

To my girlfriend Kara, family, and friends:
For their patience and understanding when my thesis came first and all your loving care and support.

Introduction

The Virtua Voorhees Replacement Hospital is located in Voorhees, New Jersey (Latitude: 39.84° Longitude: -74.93°), immediately off Rt. 73. It will be replacing the old Voorhees hospital because of its inability to be renovated. The new hospital will have 9 floors, starting with a Garden Level continuing up through Floor 8. The building is broken up into two main areas, the main bed tower (referred to as Building A, or Northern Building in this report), and a services building (referred to as Building B, or Southern Building in this report). The building is also broken up into 7 smaller zones, for ease of reference in the drawings. Figure 1 shows how the building is split up.

For the final report of the Voorhees Replacement Hospital the existing conditions are explained in detail. All of the systems including mechanical, electrical, and lighting are explained to give some background on this building. The structural system is then explained in greater detail so that the current building can be compared to any new designs. A proposal is also introduced in order to fully understand the purpose of this report. The proposal will introduce the conclusions that this report is hoping to make.

After the proposal is introduced a structural study will be conducted. This study will look at the possibility of moving the existing helipad from the parking area and adding it to the roof of the structure. In order to move the helipad, it must first be designed as a concrete two-way slab system using concrete beams that span from column to column. The supporting gravity columns will then be analyzed and increased in size to account for the added load from the helipad. Finally, to ensure that the helipad is a feasible addition, the foundations will need to be redesigned to support the added gravity loads.

This report will also look into changing the lateral system from a combination of steel moment and braced frames to purely braced frames. By making this change this report hopes to eliminate the need of moment connections which can be both costly and timely. In order to ensure that this is a possible option, many factors for the new system will be studied. First, new seismic loads will be calculated in order to find the change in loading that occurs when switching from a combination of lateral systems to a purely braced frame system. A RAM model will then be created to assist in the design of the new lateral system. Following the design an analysis of the structure will be preformed to look at the drift, torsion, and overturning moments of the new structure. These criteria will then be compared to the existing system to see the advantages and disadvantages of the new system.

An advanced study of connections will be preformed to look at the new lateral connections. Four typical connections will be designed to fulfill the MAE requirements of this report. The four connections will include two brace to column connections, a brace to beam connection, and a brace to brace connection.

Two breadths will also be preformed for this report, an electrical breadth and an acoustical breadth. Since the change in location of the helipad will also be moving the light fixtures that are required for it, an electrical study will be required to size the new wires and breaker for the new lights. Also, since the helicopter, now located on the top of the building, will create added noise, an acoustical study will be required to find the noise levels in the patients' rooms. Also, in the case that noise levels are too high, the existing façade will need to be redesigned to reduce the noise levels.

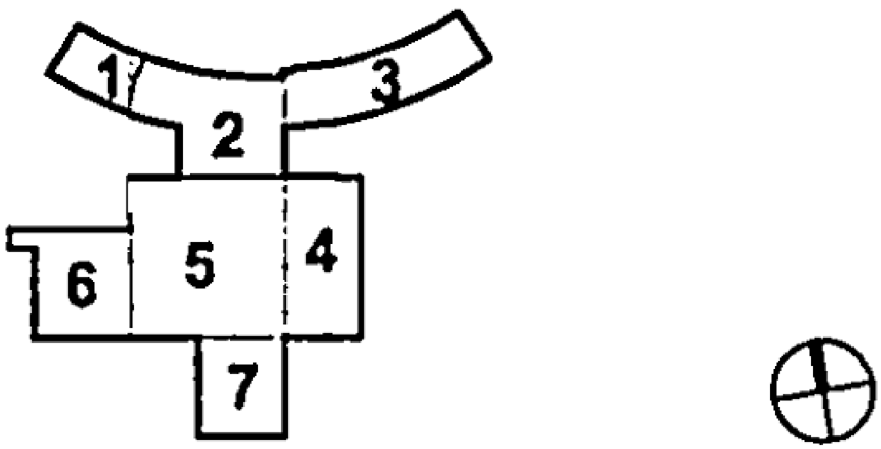


Figure 1 - Separation of Zones

Existing Building Information

Architecture

The new Virtua Replacement Hospital will be a nine story hospital with approximately 360 beds, each in its own private room. The building is split into two main parts, a main bed tower (Figure 2) and a services building (Figure 3). The main bed tower is 8 levels and holds zones 1 through 3. This tower holds the majority of the 360 beds and private rooms. It is a curved building with a curtain wall facing the majority of the site. This curtain wall allows residents to get an excellent view of the site and the natural wetlands that were protected during construction. This particular shape mixed with the curtain wall also helps sunlight enter the building, allowing natural light to fill the patients' rooms.

Figure 2 - Main Bed Tower

The other services building, which holds zones 4 through 7, is attached to the main bed tower via a thin corridor. The corridor has large windows along either side to allow for light to enter the space. The services building houses most of the labs, offices, and other surgical spaces needed in the hospital. The services are located on the ground floor through the 5th floor. Above the 5th floor, the building narrows to match the width of the corridor connecting the bed tower and the services building. Mechanical spaces start on the 6th floor and continue up through the 8th floor. The main features of this building are the light wells which also act as courtyards. These light wells are located in the center of the building allowing natural daylight to enter the center of the building while also allowing the occupants of the building a chance to get some fresh air without leaving the hospital. The services building also allows for future growth, by adding more space on top of zone 6.

Figure 3 - Services building

Sustainable Features

Besides the natural light provided by the curtain wall and light wells, the Virtua Replacement Hospital also takes advantage of some other sustainable techniques. One example is the wetlands located on the 120 acre site. During construction protection was required around certain trees and natural areas to insure that they would stay intact for when the building was complete. For the areas that were not protected during construction, the designers called out various grasses and plants native to the area.

The designers also anticipated a bus route to go to the hospital allowing visitors and staff to bus in rather than drive. Bike racks were added for those who would rather bike to the hospital instead of drive or take the bus.

Electrical

Electrical service is received from Atlantic City Electric (ACE) at 12.47 KV. Currently there is only one utility service, however, a second service is planned to be installed in the future to increase reliability. The hospital has six 2,000 KVA unit substations to transform the utility service to 277Y/480 Volts. This power is then distributed throughout the facility.

For emergency power, the hospital will have three 1,500 KW diesel generators, with provisions to add a fourth generator in the future, when the hospital expands. In the event of a loss of power, these generators will come on line and backup essential electrical branches, which include life safety circuits, critical branch circuits, equipment branch circuits, and the newly added helicopter lights. To provide uninterruptible power for the roughly 10 seconds when utility power is lost and the generators come on line, this hospital will have a rotary UPS. This is basically a large flywheel that is always spinning. If utility power is lost, it will continue to spin for about 30 seconds and generate power for that time while the generators are coming on line.

Lighting

For the interior lighting design, the IESNA recommendations were followed. Recessed fluorescent fixtures of various types, sizes, and styles were used in the majority of the building. Electronically ballasted T8 linear, twin-tube long compact and triple-tube compact fluorescent lamps were used as the building standard.

Lighting for the parking and driving areas are provided by pole-mounted cutoff luminaries. The fixtures were classified as cut-off luminaries by the IESNA and have a peak distribution of less than 90 above nadir. Light trespasses are limited to not more than 0.5 foot-candles crossing the property line. The pole heights were determined by the Voorhees Township Code, and the heights determined the spacing and wattage for each lamp. Typical exterior lights are metal halides.

Mechanical

Building heating, HVAC humidification, domestic water heating, and auxiliary steam demands for the hospital shall be served from a Central Utility Plant (CUP) hybrid heating system located primarily at the 5th floor mechanical area. The steam boilers provide primary service for the facility's demands. The plant capacity is based on an initial installation of 3 boilers to provide a total capacity of 1,500 BHP. The design facilitates future expansion of 2 additional units for a future capacity of 2,500 BHP total. The steam system also provides supplemental capacity and backup fuel redundancy for facility HVAC heating.

Facility cooling will be provided by a centralized chilled water system. Primary chilled water generation will be provided at the CUP. Chilled water will be generated by electric drive, water-cooled, centrifugal water chillers. Base chiller sizing shall be nominal 1,000 cooling tons each. Base plant capacity shall be based on an initial installation of 3, 1,000 ton units to provide a total capacity of 2,000 ton, with one unit always reserved for redundancy. The facility is designed for a future expansion of 2 units to increase capacity to 4,000 tons. Cooling heat will be rejected through 4 induced-draft, cooling tower cells. The cooling towers will be high efficiency design with a nominal capacity of 833 cooling tons heat rejection per cell. There is also room for 2 future cells.

There are 3 sets of air handling units. All the units are located in the 7th floor ancillary mechanical room. The first set contains 2 units and serves non-patient care areas such as Dietary, Environmental Services, Receiving, Lab and Maintenance. The second set contains 2 units and serves the Emergency Department, Peds Emergency, Surgery, C-Section operating rooms, the Pharmacy, and NICU. The Operating and C-Section rooms have pressure monitors to maintain positive room pressure, and will alarm if positive pressure is not maintained. The third set contains 6 units and serves the patient care areas.

Telecommunication

The telecommunication system of the building is on the low-voltage system and is being provided on cat-6 cables. A Public Address (PA) system is utilized in the building. The system is unique in that it must be able to withstand an earthquake, meaning it must remain in place without any separation of parts from the device when subjected to seismic forces specified, and the unit must be fully operational after the seismic event.

Existing Structural System

Foundations

The soil for the Voorhees Replacement Hospital is mainly a sandy soil. To prevent these loose soils from liquefaction, stone piers, or geopiers were required to be put in to densify the soil. These geopiers were required to increase the bearing pressure of the soil to 6,000 psi for the soil below all the building's foundations, canopy foundations, and utility structures. For any soil below the site's retaining walls, the bearing pressure was required to be increased to 3,000 psi. The minimum required equivalent coefficient of friction equals 0.36 for sliding resistance across the entire footing bottom area for the retaining walls, and brace frame foundations.

The foundation system is a series of concrete footings either resting on concrete piers, or resting on grade. The exterior columns are concrete footings with sizes ranging from 4' x 4' x 1' – 6" to 13' x 13' x 3' – 4" with rebar sizes ranging from #6 - #10 both ways. The columns that rest on concrete piers range in size from 2' – 4" x 2' – 4" to 3' x 4' – 6" with rebar sizes ranging from #9 - #11 for the vertical reinforcement, and #4's or #5's for the ties. See Figures 4 and 5 below for typical footing and pier details.

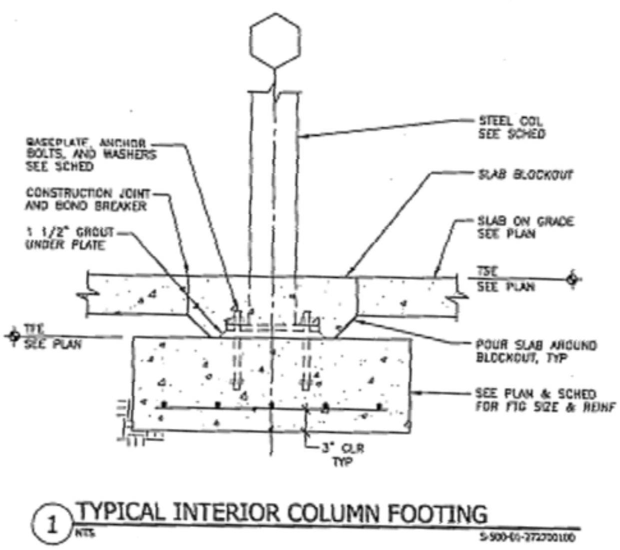


Figure 4 - Typical Concrete Foundation

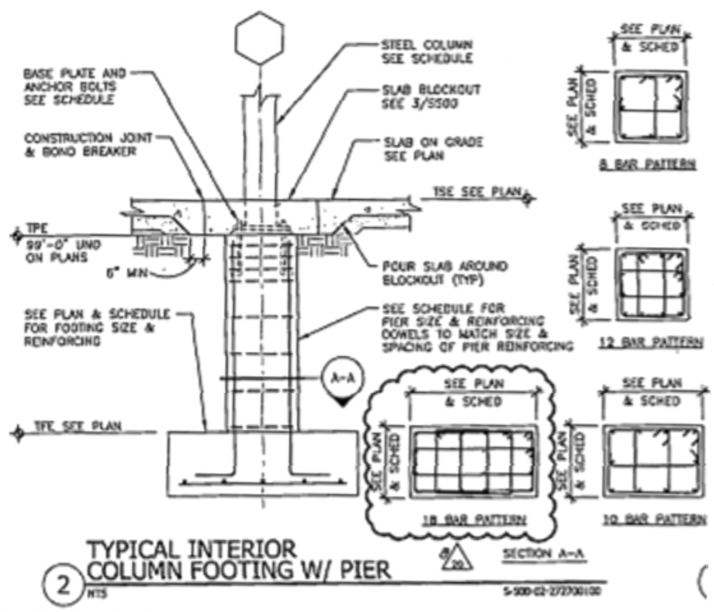


Figure 5 - Typical Foundation Footing With a Concrete Pier

The garden level floor system is constructed, in most places, using a 5" concrete slab on grade, with 6 x 6 – W2.9xW2.9 WWF. In specified spots the size of the concrete slab is increased for specialized equipment, such as refrigerator equipment required for the kitchen and dietary section of the hospital. In zones 4 and 5, a grade beam travels along the perimeter. The grade beam is shown in Figure 6 below.

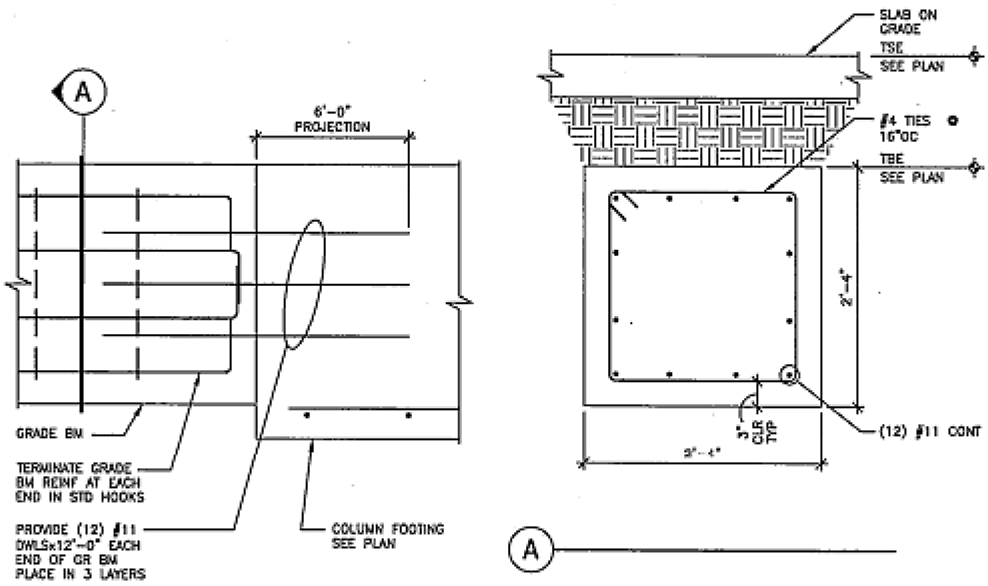


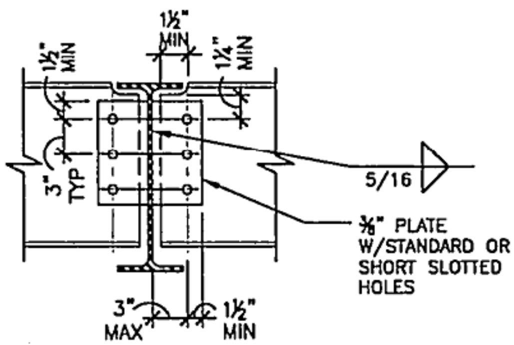
Figure 6 - Typical Concrete Grade Beam Detail

Superstructure

Floor System

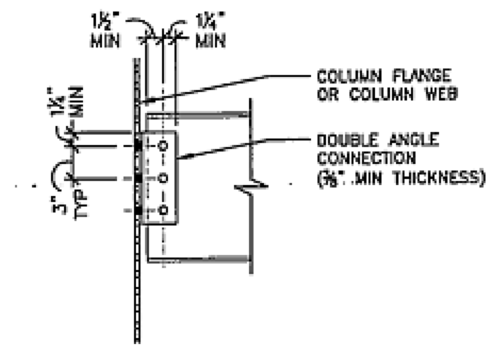
The floor system of the Voorhees hospital is a composite steel/concrete system. In Building A the typical bay sizes are around 30' x 30' or 30' x 10', depending on what area of the building they are located in. In Building B the bay sizes are typically 31' – 4" x 31' – 4" or 31' – 4" x 29' – 4". 3 – ½" light weight concrete sits on top of 3" x 18 Gage composite steel deck. The total thickness of the concrete is 6 – ½" with 6x6-W2.1xW2.1 WWF.

The steel deck is connected to the W-shape beams by ¾” diameter x 5” long shear studs allowing the two systems to work together in composite action. The beams then frame into larger W-shape girders via a single angle connection or a single plate connection. The beams are coped allowing them to connect to the girder’s web so that the composite deck can sit on both the beams and the girders. A typical beam to girder connection is pictured below in Figure 7. The W-shape girders frame into W-shape columns by either double angle connections, or by



CONNECTION SCHEDULE	
BEAM SIZE	NUMBER OF ¾"ø A-325N BOLTS
W12, W14, C12	3
W16	4
W18	5
W21	6
W24, W27	7
W30 & UP	8

Figure 7 - Typical Single Plate Beam to Beam Connection Detail



CONNECTION SCHEDULE	
BEAM SIZE	NUMBER OF ¾"ø A-325N BOLTS
W8	2
W12, W14, W16	3
W18, W21x44, 50, 57	4
W21x62 & UP W24x55, 62, 68	5
W24x76 & UP	6
W30, W33, W36	7

Figure 8 - Typical Double Angle Beam to Column Connection Detail

moment connections. The double angle connection is shown above in Figure 8.

Columns

Typical columns for the Voorhees Replacement Hospital are W14’s. The gravity columns are much lighter than the lateral columns. This is due to the added lateral force that the lateral columns must take. The columns are spliced every two floors, 4’-0” above the floor with either a bolted column splice or a welded splice. The columns located in zone 6 are designed for future expansion to be built above.

Lateral System

The Voorhees Replacement Hospital uses a combination of braced framing and moment connections for its lateral system. Though in both buildings the composite floor system and the roof deck acts as a diaphragm to transfer loads to either the braced frames, or the moment connections. In building A the braced frame supports the N-S lateral forces while the moment connections brace the E-W lateral system. The braced system consists of diagonal, square, HSS connected to W shapes. The braced frames are of two different styles, the bracing either frames from corner to corner, or from lower corner to the midpoint of the top beam. Typical frames can be seen in the Figures 10 and 11 on the next page. The moment frames in the Northern Building support the E-W lateral forces. The moment connections are located at the columns at the perimeter of the building, see Appendix A for a typical floor plan. A typical moment connection can be seen below in Figure 9.

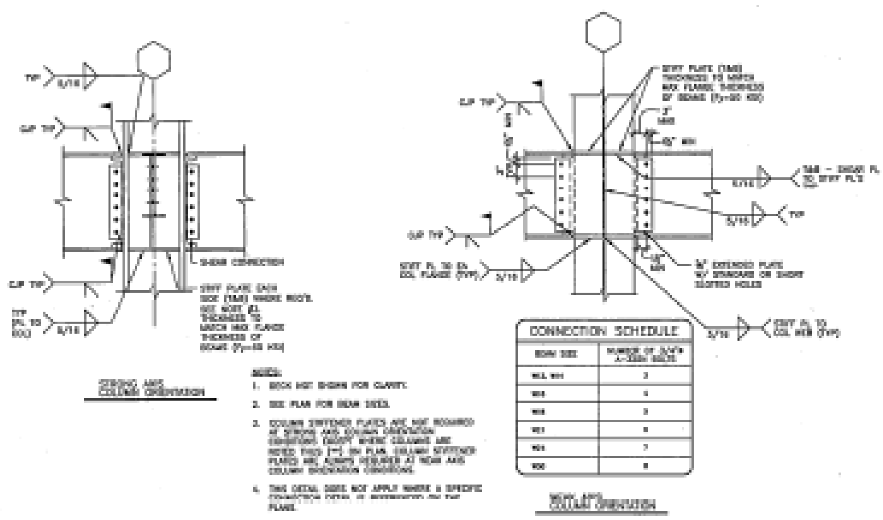


Figure 9 - Typical Moment Connection Detail

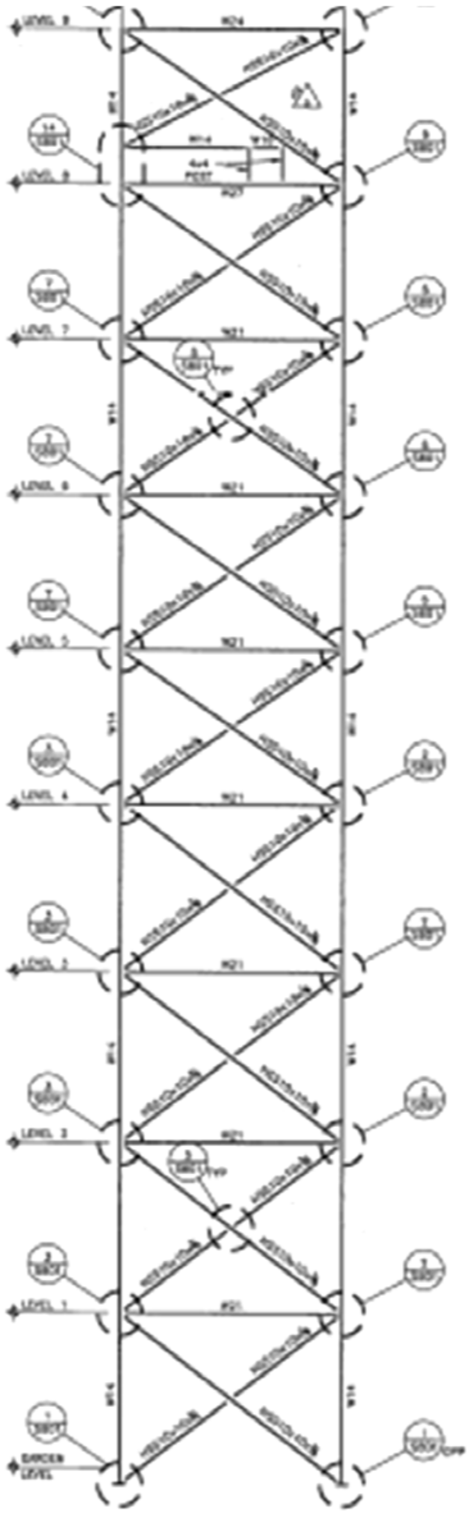


Figure 10 - Typical Braced Frame

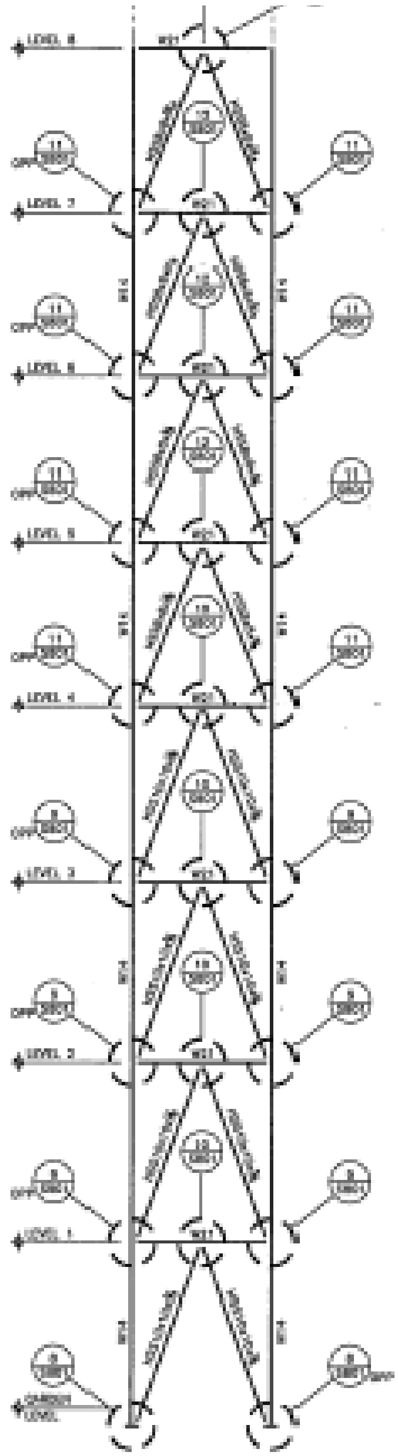


Figure 11 - Typical Braced Frame

In Building B a combination of systems is used. In the N-S direction braced frames are used to resist the lateral forces. In the E-W direction, both braced frames, and moment connections resist the loads. The moment connections, again, are typically exterior columns running along the perimeter of the building. The diagonal braces are typically, like in Building A, diagonal HSS's connected to W shapes.

Roof System

The roof system is composed of 3" x 20 Gage steel roof deck topped with a concrete slab, vapor retarder, and insulation system. In certain areas the roof deck must support the green roof. To support the extra 100 psf of added weight from the green roofs, W shapes are added with a short beam to beam span. A section with added beams can be seen in the Figure 12 below.

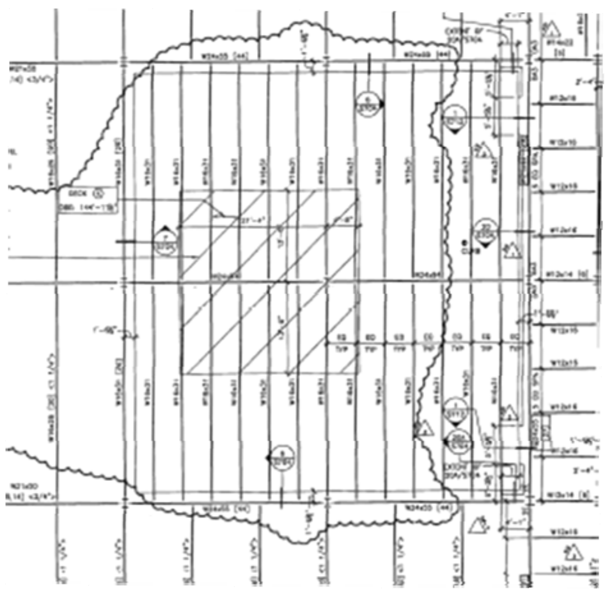


Figure 12 - Typical Framing Plan for Garden Roofs

Proposal

Problem Introduction

For the final report of the Voorhees Replacement Hospital, two features of the building will be studied. The first feature that will be looked at is the hospital's helipad which is currently located near the parking lot on site, as seen in Figure 13. The current helipad which is located far from the building's Emergency Room increases a patients' time until they can receive proper medical care. By locating the helipad away from the building, an ambulance must transfer a patient from the helicopter to the hospital. In order to eliminate this step, this report will look at relocating the helipad from the parking lot to the top of Building B.

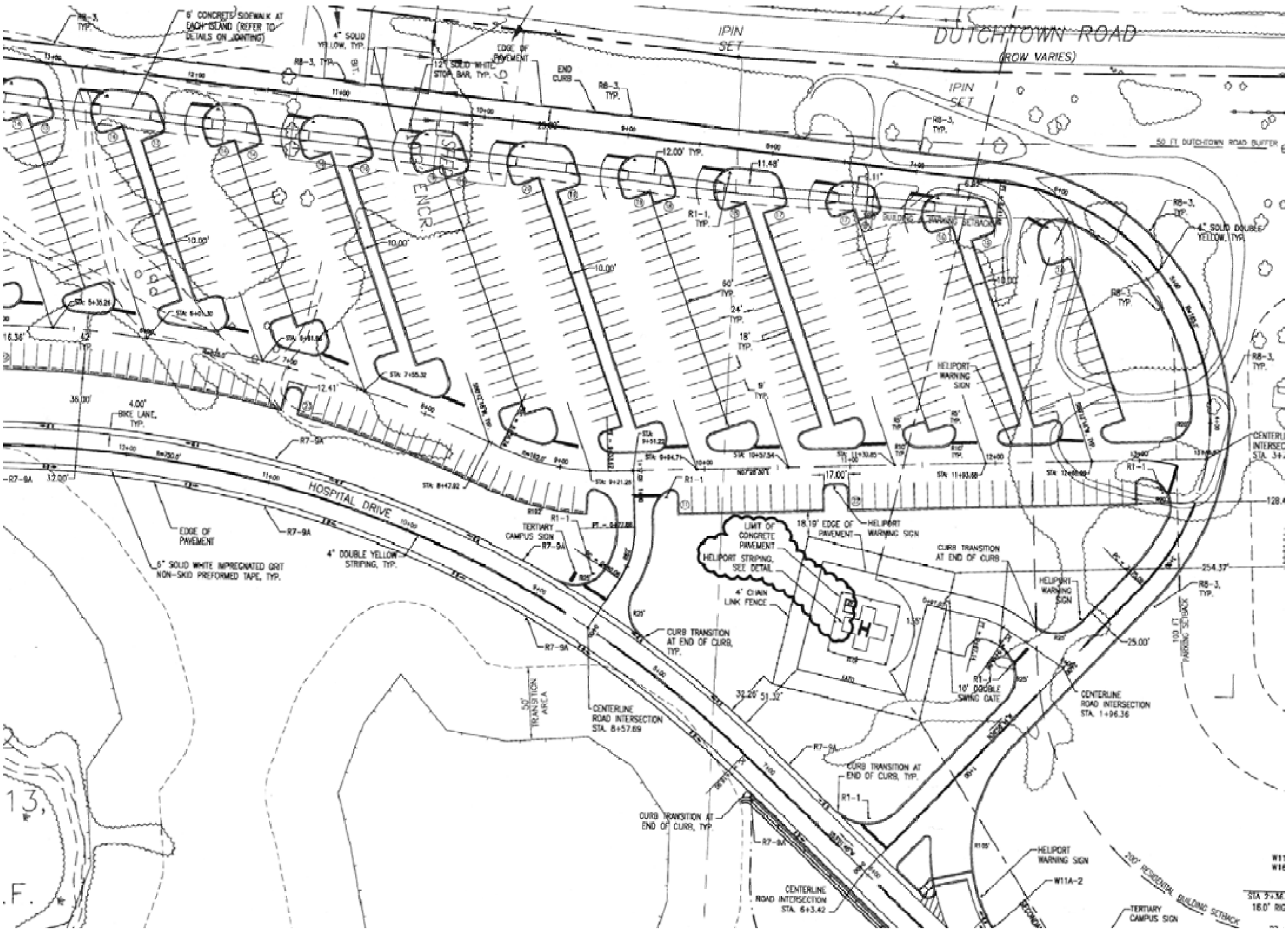


Figure 13 - Location of Current Helipad

The second feature that will be studied in this report is the southern building's lateral system. In Building B a combination of braced frames and moment frames are used. Since moment frames can be much more expensive than braced frames and can take longer to install, an attempt will be made to eliminate all moment frames, replacing them with new braced frames.

Problem Solution

An in-depth study will be preformed to find any and all requirements for helipads that are located on top of hospitals and other building types. Because the helicopter will be refilling on the helipad certain requirements may be necessary for spill containment and fire protection. Due to these added requirements, this report will look at keeping the existing helipad for fueling and storage purposes. There will also be special requirements for fall protection, and other safety concerns that will need to be researched.

After the requirements for the helipad are found, a design of the helipad will be required. A location on the roof will need to be found so that the helipad is located close to an elevator shaft so that the patient can be easily transported from the helipad to the emergency room. After finding a location the helipad will be created using a concrete pad supported by steel members. The concrete pad will be a two – way concrete slab since the helipad is square. The pad will be supported by steel W shapes that frame down to the building's structural system.

The building's gravity system will need to be redesigned to account for the extra gravity loads introduced from the helipad and the helicopter. The structural system will continue to be composite steel beams with steel columns. Member sizes will be increased so that they are acceptable for the new loads. The current bay sizes will be used throughout the building in order to maintain the proper circulation in the building.

The building's lateral system will also need to be redesigned for the added weight of the helipad. It was found in Third Technical Report that the seismic forces in the North – South direction control over the wind forces. Since the seismic forces are directly related to the building's weight, when the extra weight of the helipad and the helicopter are added, the seismic forces will be increased. Therefore the lateral members must be redesigned for the new forces.

In Building B the lateral system will be altered in order to eliminate any moment connections. This will help to eliminate the added costs and time that moment frames can create. The building's circulation will have to be looked at as a result of this. By adding braced frames there is potential that a wall will be added in a hallway resulting in a circulation problem. If this is found to be the case then potential solutions will be looked at, such as moving the hallway, or using a specialized brace that does not affect the entire bay, such as a 'V' brace.

Breadth Topics

The new Voorhees Hospital uses many methods to make sure that their patients are in a healthy stress-free environment. If noise from a helicopter can be heard in the hospital then all of the work the designers have done to create those friendly, stress-free environments will be lost. Therefore, the building envelope will have to be studied in order to ensure that the noise in the building is not a distraction. The noise levels are likely to increase even though there is already a helipad on site. The current helipad is far away from the building in the parking area, if it is moved to directly on top of the building then noise will increase significantly. If the noise levels are found to be too high in the building, then the building envelope will have to be redesigned in order to reduce the noise levels.

The relocation of a helipad to the roof of the building will also require special lights to be added so that the helicopter can see the helipad in the dark. These lighting requirements will be researched, and lights will be chosen and placed on the new helipad per the requirements. The lights will also need a source of power to ensure that they are always available. Possible sources of power will be researched in the existing plans and new wires will be designed for the lights. A path for the wires will be determined and the voltage drop will be checked to ensure that the new wires are a reasonable length. The wire sizes will be changed if the voltage drop is found to be excessive.

MAE Topic

The MAE requirement for this report will be fulfilled through the seismic analysis. Methods taught in AE538: Earthquake Design will be used to determine the seismic forces, and design the lateral members accordingly. These calculations can be found in the Seismic Loads section of this report. Additionally, use of the AE597A: Computer Modeling course will be vital when creating and understanding a RAM model of the hospital. RAM modeling will be used to analyze and design both gravity and lateral members. Details about the RAM model can be found in the RAM Model section of this report. The use of AE 534: Steel Connections will also be used to design typical braced frame connections. Typical connection designs can be found in the Steel Connections section of this report.

Structural Study

Codes & Design Standards

Design Codes

International Building Code (IBC) 2006

American Society of Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, 2005

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

American Institute of Steel Construction (AISC), Steel Construction Manual

Material Strength Requirement Summary:

Cast-in-place Concrete:

f'_c = 3,500 psi @ 28 days for all lightweight concrete on metal decking

f'_c = 4,000 psi @ 28 days for all other concrete types

Concrete Masonry:

Concrete Masonry Units: ASTM C90 Type "N-1"

Masonry Grout: f'_c = 3,000 psi @ 28 days

Masonry Mortar: ASTM C270 (Type S uno)

Steel Reinforcing:

Reinforcing Bars: ASTM A615 (Grade 60)

Welded Bars & Anchors: ASTM A706 (Grade 60)

Deformed Bar Anchors: ASTM A496

Epoxy-Coated Reinforcing Bars: ASTM A775 or ASTM A934

Welded Wire Fabric: ASTM A185

Structural Steel:

W & WT Shapes: ASTM A992, F_y = 50 ksi

Plates & Shapes Other Than W: ASTM A36, F_y = 36 ksi

Rectangular HSS: ASTM A500, Grade B, F_y = 46 ksi

Round HSS: ASTM A500, Grade B, F_y = 42 ksi

Pipes: ASTM A53, Type E or S, Grade B, F_y = 35 ksi

Bolts: ASTM A325, F_y = 36 ksi

Expansion Bolts: Hilti, Rawl, Thunderstuds, or National Fasteners

Adhesive Anchors/Grout: Sika, Hilti, Epcon

Headed Studs/Shear Connectors: ASTM A108

Welds:

All Types: E70XX

Gravity Loads

Building live loads are determined by referencing ASCE 7-05. Table 1 below outlines the findings.

Live Loads				
Load Description	**ASCE 07-05 Load (psf)**	**Reduced?**	**Reducible by Code?**	**Assumed Partition Load (psf)**
Labs	60	Yes	Yes	20
Operating Rooms	60	Yes	Yes	20
Private Rooms/Wards	40	Yes	Yes	20
Offices	50	Yes	Yes	20
Corridors above the 1st floor	80	Yes	Yes	N/A
Lobbies/1st floor corridors	100	Yes	No	N/A
Stairs and Exits	100	No	No	N/A
Storage	125	No	No	N/A
Mechanical Room	125	No	No	N/A
Roof Garden	100	N/A	No	N/A
Roof	20	N/A	Yes	N/A

Table 1 - Live Loads

Building snow loads are determined by referencing ASCE 7-05. Table 2 below outlines the loads used in this report.

Snow Loads	
	Found
Ground Snow Load, P_g	25 psf
Flat Roof Snow Load, P_f	24 psf
Snow Importance Factor	1.2
Snow Exposure Factor, C_e	1.0
Thermal Factor, C_t	1.0

Table 2 - Snow Loads

Helipad Design

Helipad Background Research

There are many regulations for helipads that are located on top of hospitals. The FAA, or Federal Aviation Administration sets these regulations; all requirements for helipads in this report come from the FAA's Advisory Circular, AC 150/5390-2B. For this report many considerations were taken into account when determining the location of the helipad, and designing the helipad.

One of the main requirements found from the FAA report is the size requirements for the helipad. The helipad is broken up into two main areas, the TLOF, or Touchdown and Lift-Off Area, and the FATO, or the Final Approach and Takeoff Area. The TLOF area must be 1.0 rotor diameters in length and width, but not less than 40 feet. There must also be at least one FATO that contains the TLOF, and it must be 1.5 times the overall length of the helicopter. In this report it will be assumed that the maximum size helicopter that will be used has a 42 foot overall length.

The FAA report also defines safety measures that must be taken. One of the more obvious safety measures that must be taken is that there are be no obstacles above the helipad. If there are any obstacles close to the landing area, a helicopter blade might hit it. Another safety procedure for rooftop helipads is a safety net that hangs off of the helipad. Since no obstacles may be above the helipad, in order to keep people from falling, a safety net that hangs below the helipad must be installed. The net must be at least 5 feet wide, and should be able to hold a load of 25 psf. An example of a safety net detail can be found above in Figure 14.

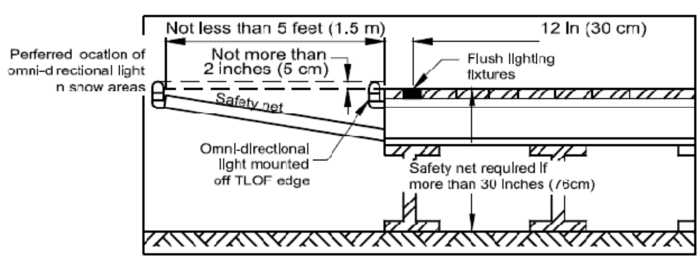


Figure 14 - Typical Safety Net Detail

The helipad must also follow OSHA and fire protection guidelines. Elevated helipads require two separate access points per OSHA guidelines. At least one of these access points should be a ramp in order to safely and quickly move the incoming patient. Any ramps or stairs that are used should be designed according to OSHA guidelines. For helipads the NFPA 418: Standards for Heliports, and NFPA 403: Aircraft Rescue Services should be used. This states that a fire hose or extinguisher should be provided adjacent to, but not above, the TLOF.

Snow removal must also be considered when designing a helipad since swirling snow raised by a helicopter's rotor can cause the pilot to lose sight of the intended landing point. For this report it is assumed that snow removal will be handled by employees of the hospital. Another weather factor that must be considered is the rain. The helipad will be required to have a broomed or roughened finish so that users can maintain traction during wet weather. It is crucial that the helipad is not slippery for safety concerns.

Vibration is also a large concern when dealing with helipads on the top of buildings. A helicopter on top of buildings can cause vibration throughout the building disturbing areas that require minimal vibrations, such as surgical spaces. A vibration analysis is not part of this report, and will need to be looked at further in the future.

The final non-structural consideration for a hospital helipad is the MRI machine. MRI's can create powerful magnetic fields and can interfere with a helicopter's magnetic compass or other navigational systems. When designing a hospital's helipad, the location of the MRI must be taken into account. For this report it was assumed that the MRI machine has no affect on the incoming helicopters or the helipad.

The FAA also defines the load requirements that need to be designed for. In the FAA report it states that the helipad must be designed for a static load equal to the maximum takeoff weight of the helicopter. For this hospital it will be assumed that the maximum weight of any helicopter landing is 12,000 pounds. The FAA report also states that the helipad must be designed for a dynamic load of 150% of the takeoff weight, or for this helipad, 18,000 pounds. For this report it will be assumed that the 18,000 pounds acts as a static load.

Helipad Location Study

Many factors are considered selecting the area for the helipad to be located. The first factor taken into account is the height of the building's roofs. Since the building is not a consistent height, the tallest part of the building is needed in order to eliminate any obstructions for the helicopter. Another factor taken into account was the building's column grid lines. An area is needed where the shape of the helipad slab would fit easily onto the building's existing columns in order to eliminate unnecessary transfer girders. Important factors such as the location of the building's intake air are taken into account when deciding on a location for the helipad. It is important to avoid the building's intake air since a helicopter can release unpleasant and possibly harmful exhaust from its engine. The final consideration for the location of the helipad is its distance compared to the emergency room. Since one of the goals for moving the helipad is to decrease the time it takes the patient to get to the Emergency Room, it is important to locate the helipad so that it is quick and easy to get to the ER.

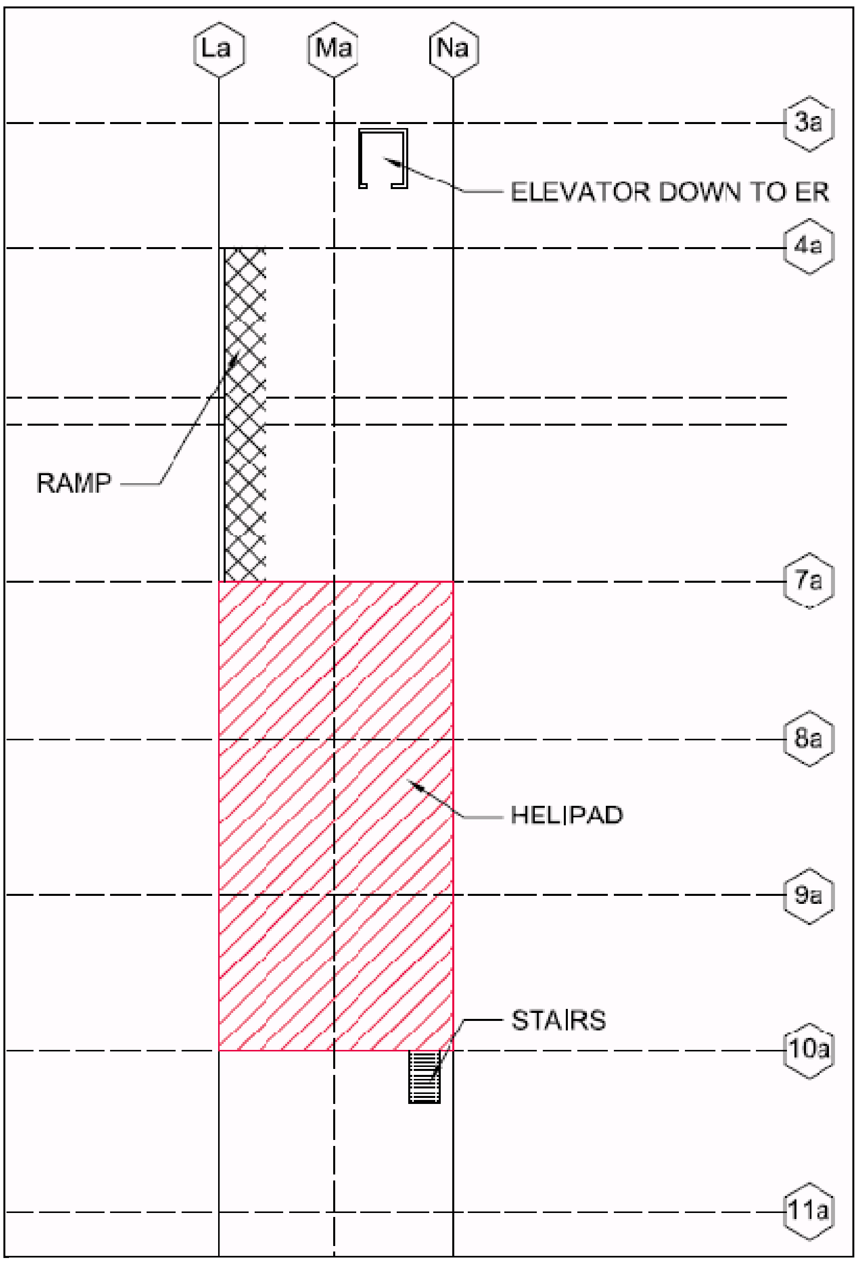


Figure 15 - Helipad Location

Taking all of these factors into consideration, a location in Zone 5 is determined to be ideal. The helipad rests between grid lines La-Na and 7a-10a on Level 9 in Zone 5.The exact location can be seen in Figure 15. This location is ideal for a number of reasons. First, it is the tallest area on the building, with very few objects that are above it. There are communication antennas in Zone 7 that will need to be relocated to another part of the building so that they do not interfere with the helicopter. This location is also ideal because the helipad can easily rest on the building's existing gridline. The grid line can be seen in Figure 15, and it allows for helipad supports to line up directly with the building's supports. The helipad location is ideal also because it is located directly next to the building's main elevator shafts. Along grid line 3a there are 8 elevator shafts, 2 staff elevators, 4 patient elevators, and 2 spaces for future elevators. These elevators are located directly in the corridor that connects the northern bed tower and the southern services building. They are located just down the hall from the 1st floor emergency room. Since the 2 staff elevators and the 4 patient elevators might be in use in an emergency, a new elevator will need to be installed that is specifically designed to service between the helipad and 1st floor where the ER is located. With this location of the helipad, and the new elevator in place it will be quick and easy for personnel to travel from the helipad directly to the 1st floor without making unnecessary stops.

There are also disadvantages to this location however. The main disadvantage for this location is that it is not as wide as other preferred sites. For this location, the building's width limits the width of the helipad, specifically the FATO and safety net. In order to have the proper size helipad, the FATO area of the helipad would have to hang off of the building by a considerable amount. Since this would not be architecturally pleasing, it must be assumed for this location that approaching and departing helicopters will only come from the North or South. The North and South sides of the helipad will be long enough to accommodate an approaching or departing helicopter.

The safety net that hangs of the building creates another disadvantage. Because the building is not large enough, the FATO is taken to the edge of the building to allow for the largest area possible. Since the pad ends at the edge of the building, a safety net must still attach to the end of the pad. The pad will then hang 5 feet off of both the pad and the building allowing it to be seen from the ground. Though this is not ideal, it is assumed to be acceptable because the helipad is 140 feet above grade and a 5 foot overhang is expected not to be extremely noticeable.

Helipad Slab Design

The helipad lab is designed as a two-way slab using the direct design method. The columns are assumed to be attached to 20" x 20" base plates. Beams are assumed to span from column to column and are 10" in depth and 20" in width. A snow load of 25 psf, live load of 100 psf, and a dead load of 136 psf, which includes self weight with a 10" slab thickness, are also assumed. For these loads a load combination of 1.2D + 1.6L + .5S is used. For the design of the helipad, the pad is broken up into 4 sections labeled A-D. The design sections can be seen in Figure 16. The moments in the beam, column strip and middle strip are found for each section. The moments can be found below in Tables 3, 5, 7, and 9. These moments are used to then find the reinforcement needed in each section of the slab. A hand calculation is performed to find the rebar for the Column Strip of Section A, then excel is used to find the remaining rebar requirements. The reinforcement requirements for each of the slab sections can be found in Tables 4, 6, 8, and 10 below. The calculations can be found in Appendix B. The moments from Tables 3, 5, 7, and 9 are then used to find the reinforcement requirements for the beams. Calculations for the rebar in the beams can also be found in Appendix B. The final step for the slab design is a shear check in each of the beams. Where the shear is found to be excessive, shear reinforcement is required. The reinforcement requirements for each beam can also be found in Tables 4, 6, 8, and 10 below. These calculations can be found with the other rebar calculations in Appendix B.

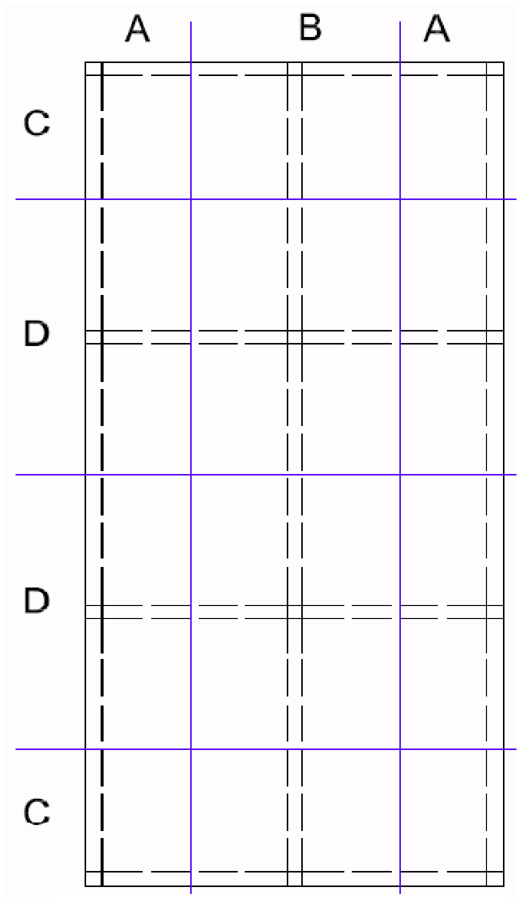


Figure 16 - Design Sections of Helipad

Moment in Frame A (k-ft)									
	M_{edge}	M^+	M^-	M^-	M^+	M^-	M^-	M^+	M_{edge}
M_{total}	-66.5	236.9	-291	-270.2	145.5	-270.2	-291	236.9	-66.5
M_{beam}	-52.5	147.8	-194.4	-180.5	90.8	-180.5	-194.4	147.8	-52.5
$M_{cs\ slab}$	-9.21	26.1	-34.3	-31.9	16.02	-31.9	-34.3	26.1	-9.21
$M_{ms\ slab}$	-5.12	63	-62.3	-57.8	38.7	-57.8	-62.3	63	-5.12

Table 3 - Moment in Frame A

Reinforcement in Frame A									
	M_{edge}	M^+	M^-	M^-	M^+	M^-	M^-	M^+	M_{edge}
Column Strip	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5
Middle Strip	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5
Flexural Beam	(4) #8	(4) #8	(4) #8	(4) #8	(4) #8	(4) #8	(4) #8	(4) #8	(4) #8
Shear Beam	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 4 - Reinforcement in Frame A

Moment in Frame B (k-ft)									
	M_{edge}	M^+	M^-	M^-	M^+	M^-	M^-	M^+	M_{edge}
M_{total}	-132.6	472.5	-580	-538.9	290.2	-538.9	-580	472.5	-132.6
M_{beam}	-108.2	294.8	-387.5	-360	181.1	-360	-387.5	294.8	-108.2
$M_{cs\ slab}$	-19.1	52	-68.4	-63.6	32	-63.6	-68.4	52	-19.1
$M_{ms\ slab}$	-19.1	125.7	-124.12	-115.3	77.2	-115.3	-124.12	125.7	-19.1

Table 5 - Moments in Frame B

Reinforcement in Frame B									
	M_{edge}	M^+	M^-	M^-	M^+	M^-	M^-	M^+	M_{edge}
Column Strip	(8) #5	(8) #5	(8) #5	(8) #5	(8) #5	(8) #5	(8) #5	(8) #5	(8) #5
Middle Strip	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5	(4) #5
Flexural Beam	(9) #8	(9) #8	(9) #8	(9) #8	(9) #8	(9) #8	(9) #8	(9) #8	(9) #8
Shear Beam	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”

Table 6 - Reinforcement in Frame B

Moment in Frame C (k-ft.)						
	M_{edge}	M^+	M^-	M^-	M^+	M_{edge}
M_{total}	-45.6	162.3	-199.3	-199.3	162.3	-45.6
M_{beam}	-34.9	87.2	-127.1	-127.1	87.2	-34.9
$M_{cs\ slab}$	-6.2	15.4	-22.4	-22.4	15.4	-6.2
$M_{ms\ slab}$	-4.5	59.6	-49.8	-49.8	59.6	-4.5

Table 7 - Moments in Frame C

Reinforcement in Frame C						
	M_{edge}	M^+	M^-	M^-	M^+	M_{edge}
Column Strip	(6) #5	(6) #5	(6) #5	(6) #5	(6) #5	(6) #5
Middle Strip	(6) #5	(6) #5	(6) #5	(6) #5	(6) #5	(6) #5
Flexural Beam	(3) #8	(3) #8	(3) #8	(3) #8	(3) #8	(3) #8
Shear Beam	N/A	N/A	N/A	N/A	N/A	N/A

Table 8 - Reinforcement in Frame C

Moment in Frame D (k-ft.)						
	M_{edge}	M^+	M^-	M^-	M^+	M_{edge}
M_{total}	-91.1	324.6	-398.6	-398.6	324.6	-91.1
M_{beam}	-73.6	174.3	-254.1	-254.1	174.3	-73.6
$M_{cs\ slab}$	-13	30.8	-44.9	-44.9	30.8	-13
$M_{ms\ slab}$	-4.5	119.1	-99.7	-99.7	119.1	-4.5

Table 9 - Moments in Frame D

Reinforcement in Frame D						
	M_{edge}	M^+	M^-	M^-	M^+	M_{edge}
Column Strip	(11) #5	(11) #5	(11) #5	(11) #5	(11) #5	(11) #5
Middle Strip	(6) #5	(6) #5	(6) #5	(6) #5	(6) #5	(6) #5
Flexural Beam	(5) #8	(5) #8	(5) #8	(5) #8	(5) #8	(5) #8
Shear Beam	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”	(2) #4 @ 8”

Table 10 - Reinforcement in Frame D

Rebar Location Details

The reinforcement locations are then determined by using Figure 13.3.8 in ACI 318-08. All reinforcement in the slabs uses a 1” clear cover due to the fact that it is exposed to weather conditions. The reinforcement in the beams uses a clear cover of 1 ½” for the same reasons. Figures 17 - 32 shows the reinforcement placement of each slab area A-D.

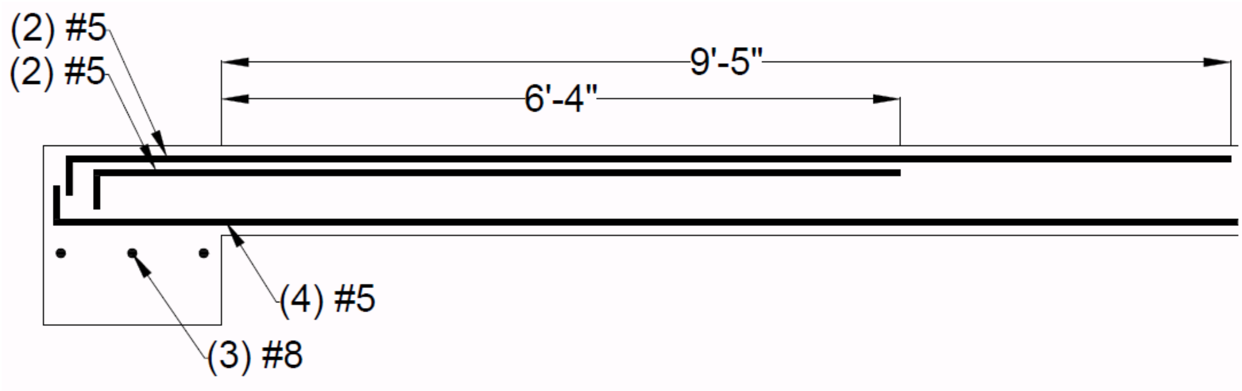


Figure 17: Area A Column Strip

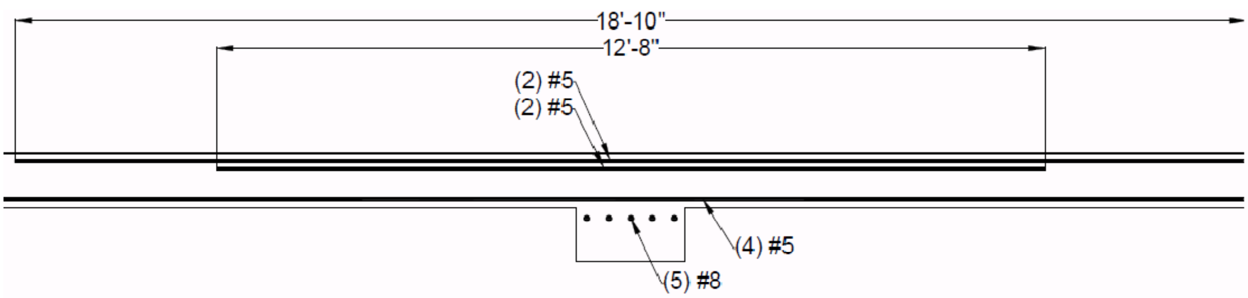


Figure 18: Area A Column Strip

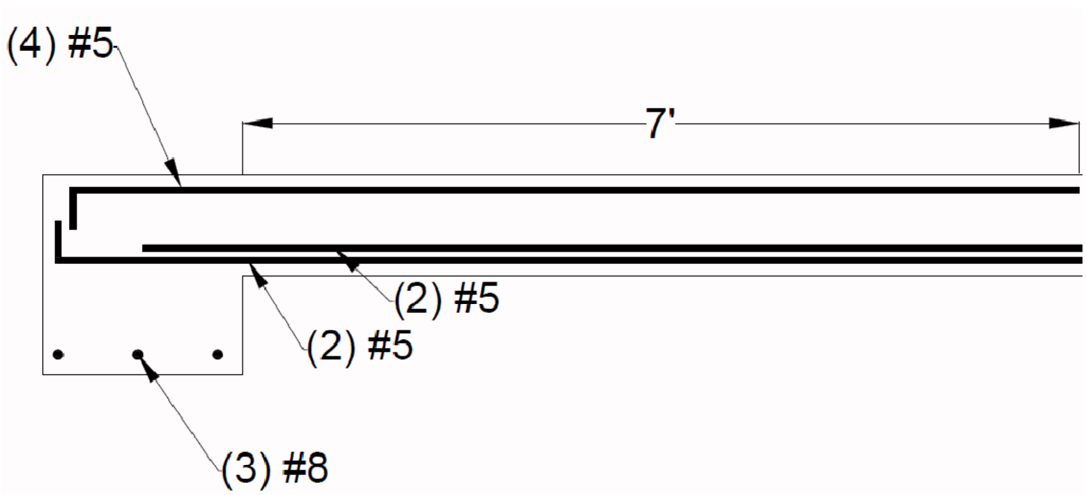


Figure 19: Area A Middle Strip

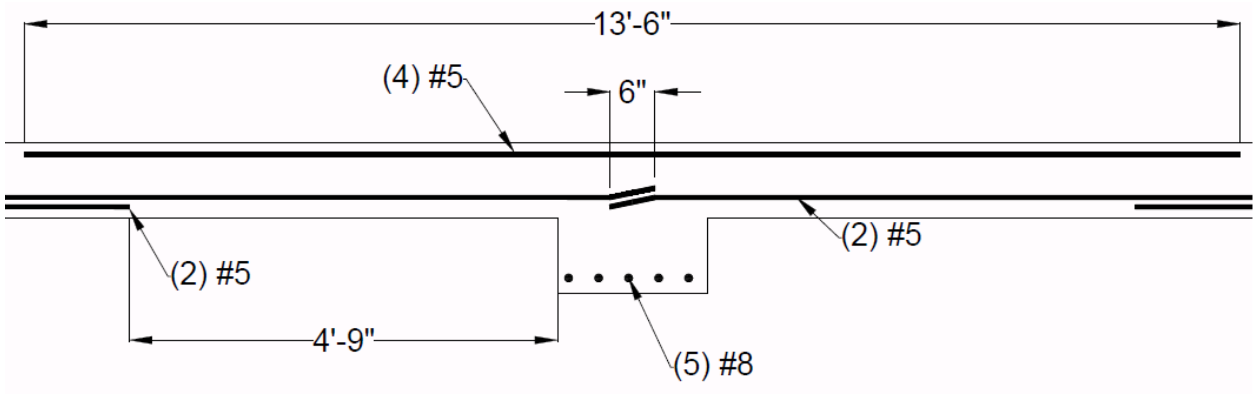


Figure 20: Area A Middle Strip

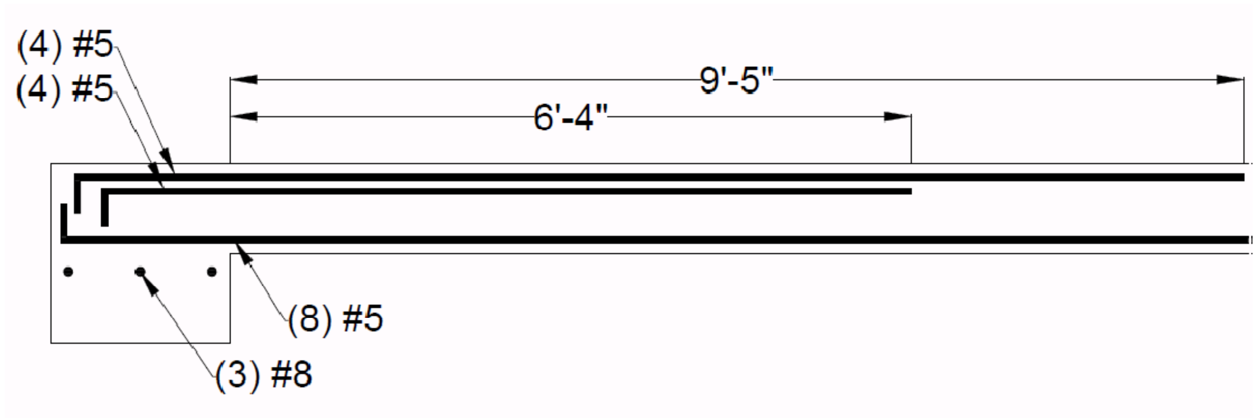


Figure 21: Area B Column Strip

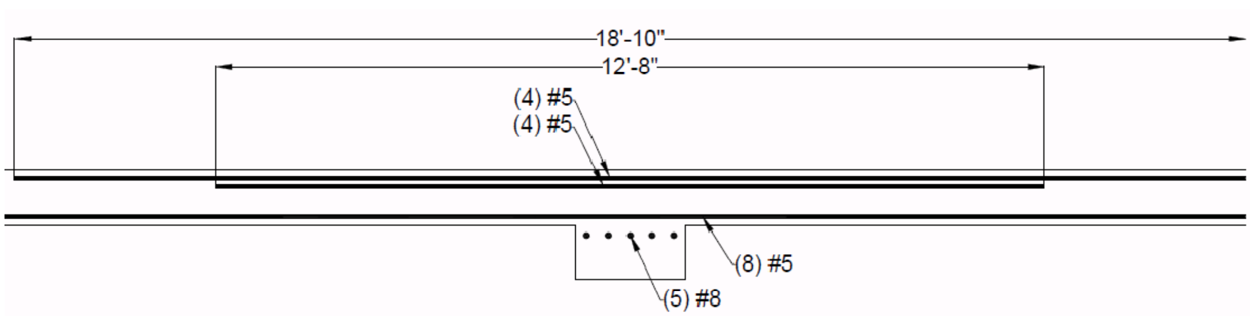


Figure 22: Area B Column Strip

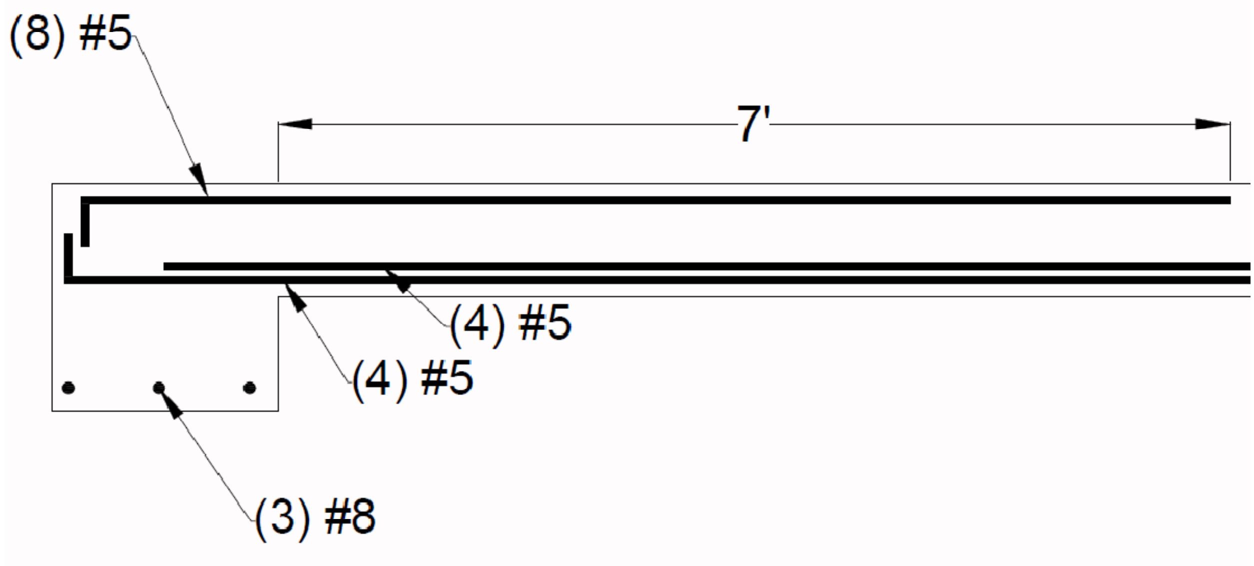


Figure 23: Area B Middle Strip

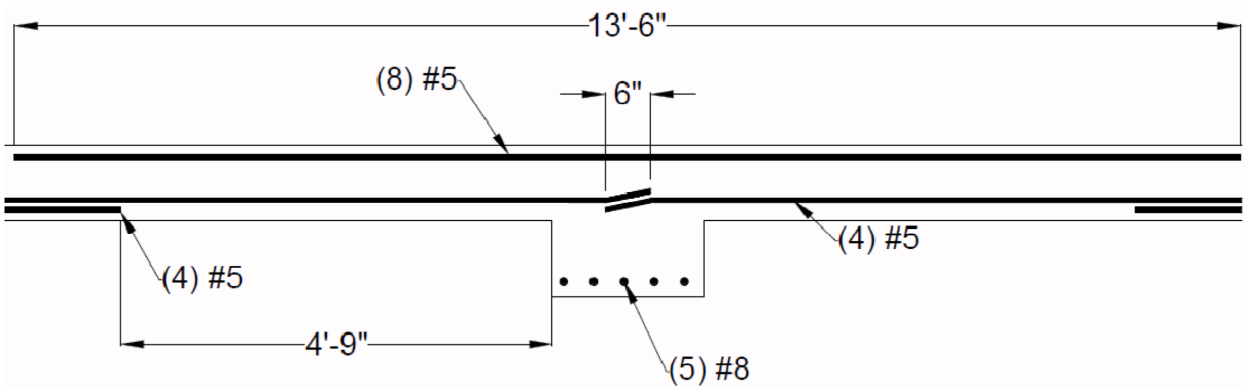


Figure 24: Area B Middle Strip

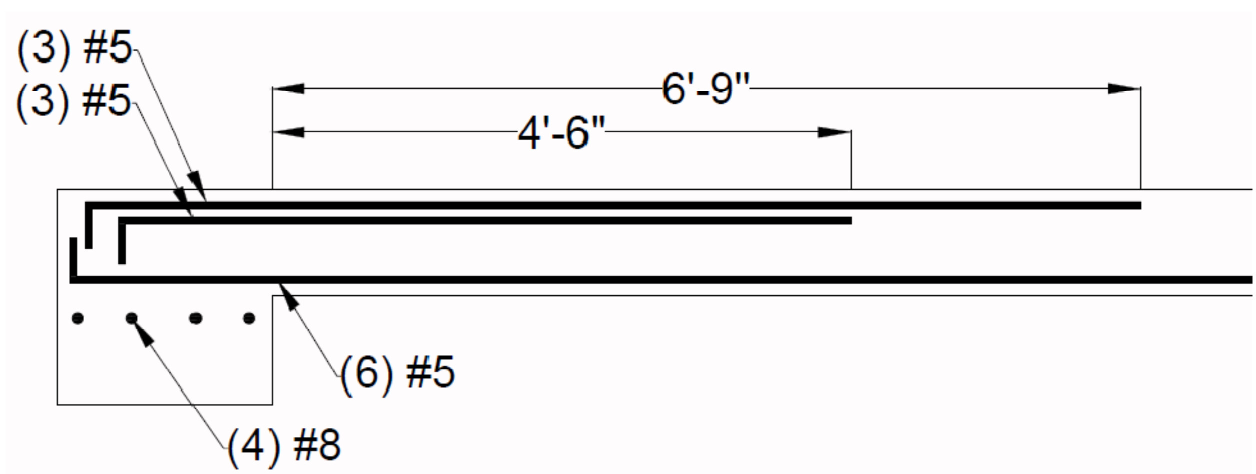


Figure 25: Area C Column Strip

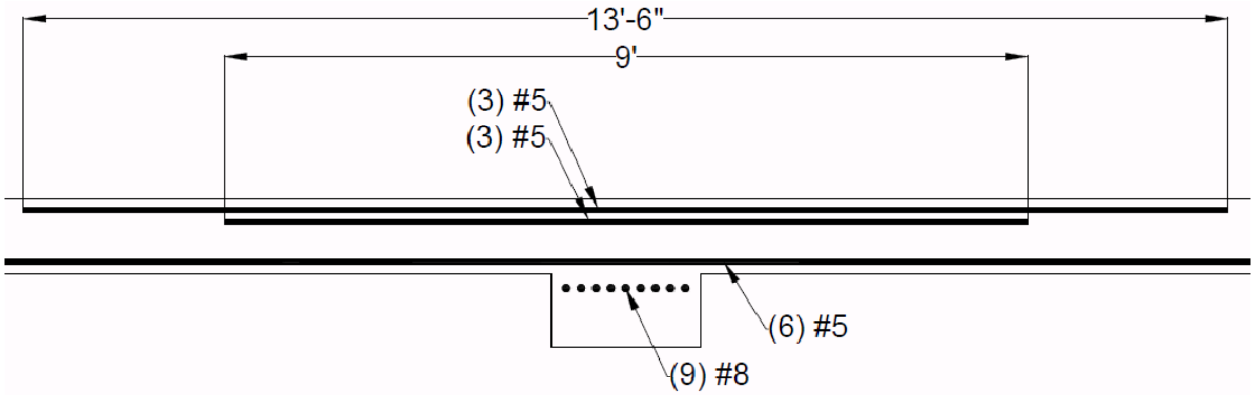


Figure 26: Area C Column Strip

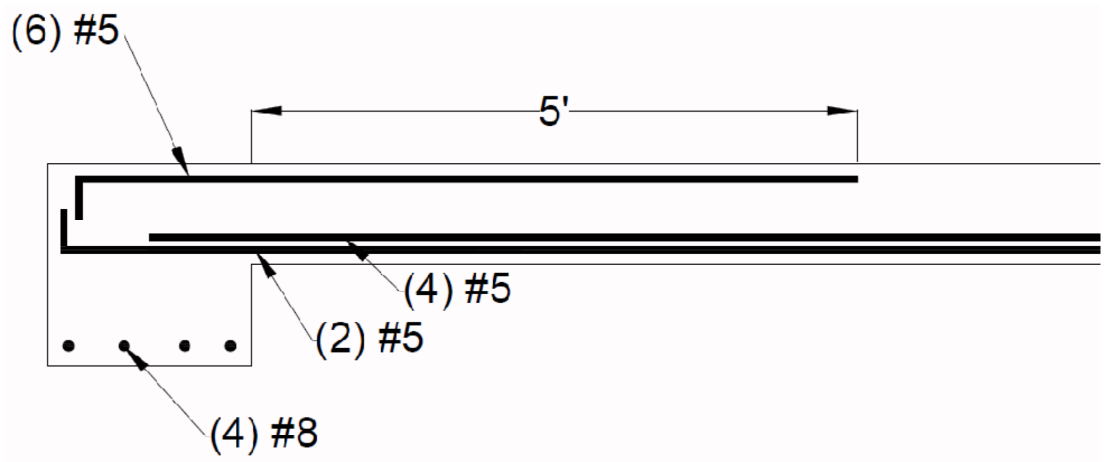


Figure 27: Area C Middle Strip

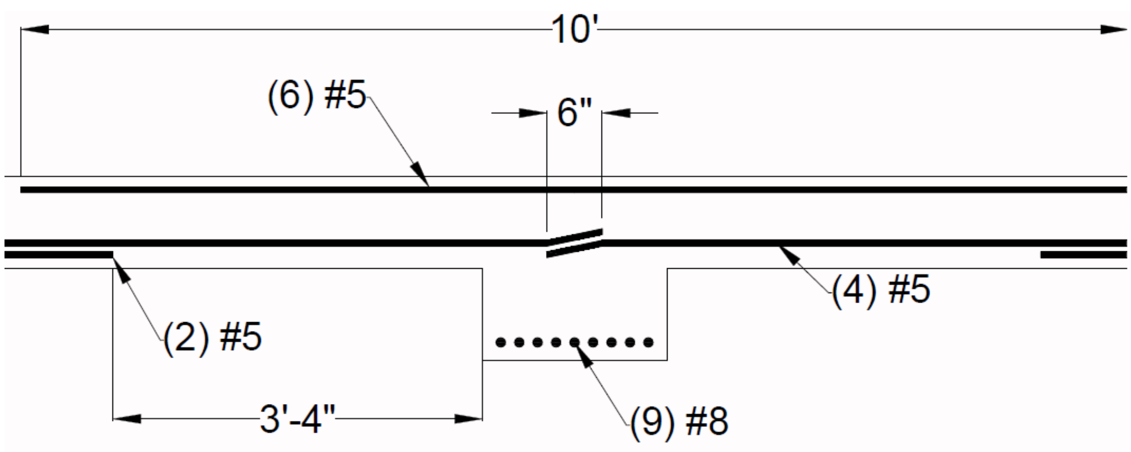


Figure 28: Area C Middle Strip

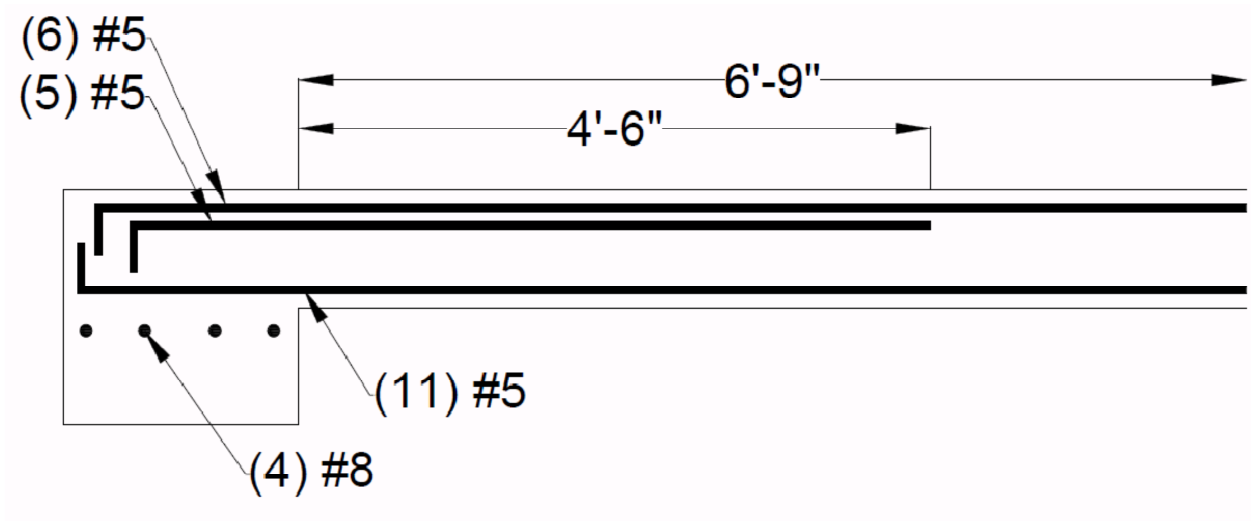


Figure 29: Area D Column Strip

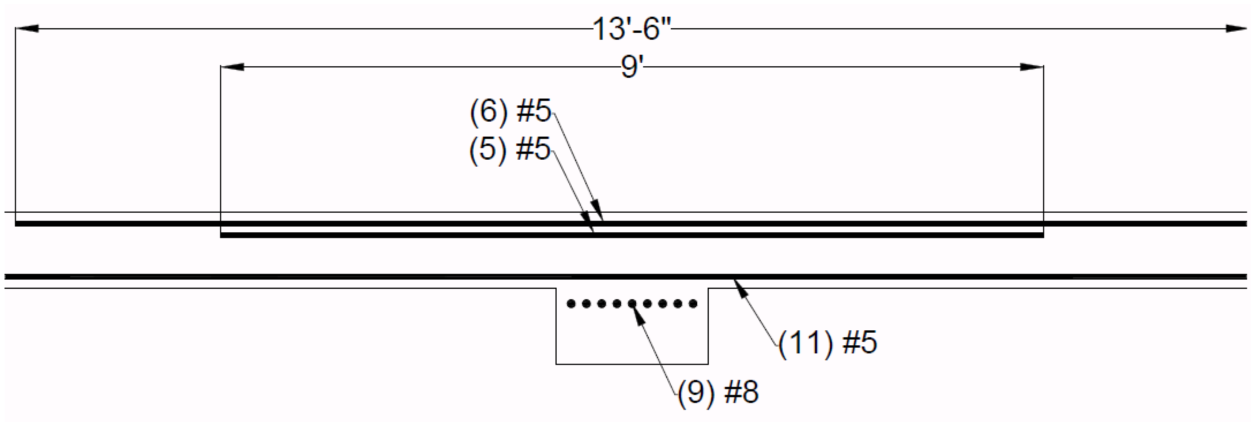


Figure 30: Area D Column Strip

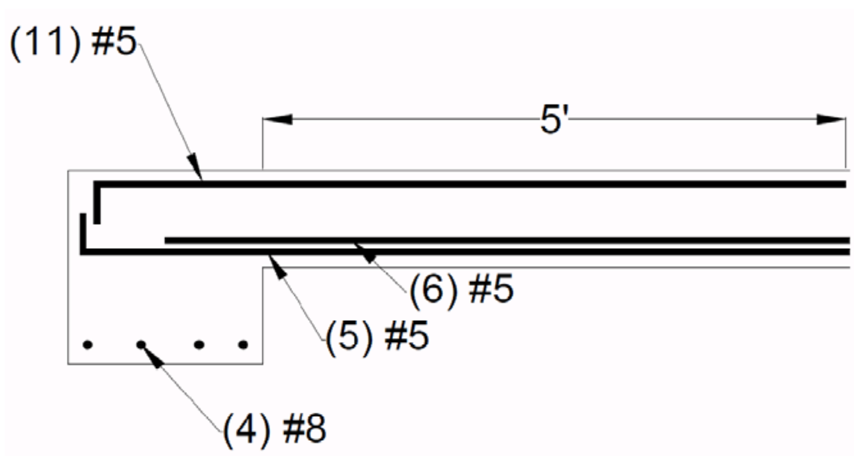


Figure 31: Area D Middle Strip

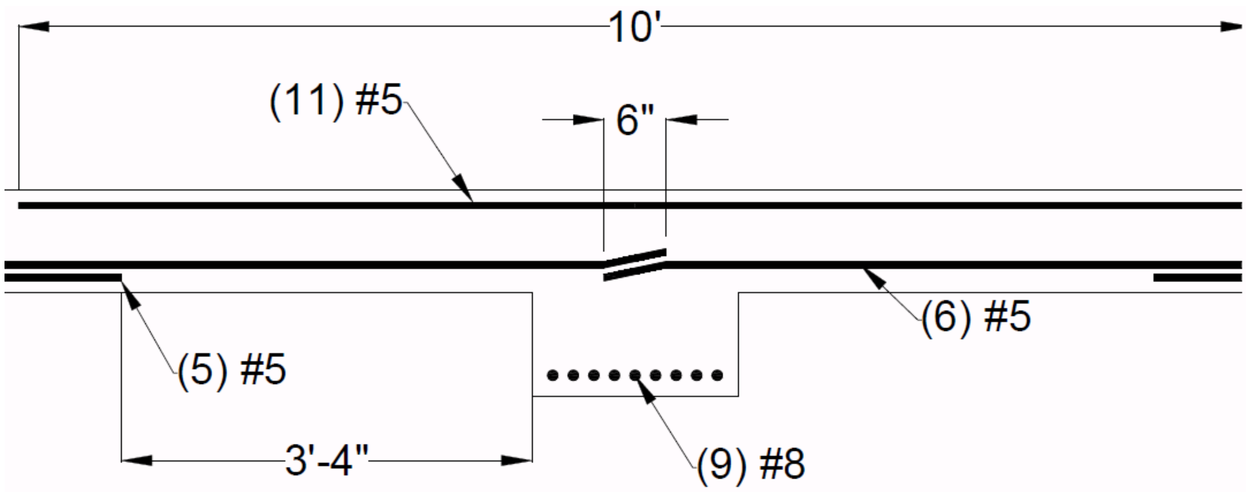


Figure 32: Area D Middle Strip

Gravity Column Redesign

In this section of the final report for the Voorhees Replacement Facility, the gravity columns supporting the helipad are designed. There are columns that support the helipad, as well as act as part of the lateral system for the building. Since these members are designed later in the report, they will be neglected in this section, leaving only the gravity members to be designed. The gravity columns that are affected by the new helipad are the columns on grid lines La-7a, La-8a, La-9a, Na-8a, Na-9a, Ma-8a, Ma-9a, and Ma-10a. The other four columns, Ma-7a, Na-7a, La-10a, and Na-10a will be designed in the lateral system design.

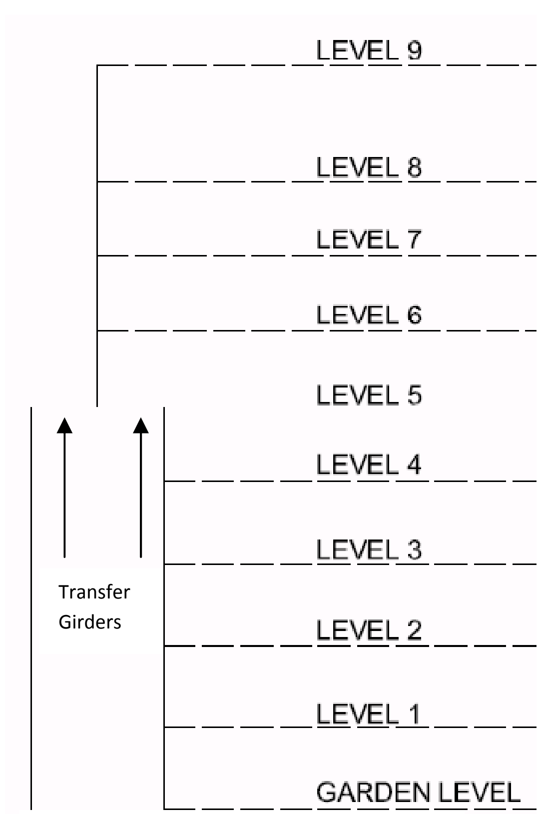


Figure 33 - Columns located in Ma Grid Line

The supporting gravity columns for the helipad are designed by hand calculations and then confirmed by a RAM model. The existing W shapes are assumed to continue up to the bottom of the pad where they would attach to a 20”x20” base plate and then to the slab. The columns are assumed to have the existing splice locations, 4 feet above levels 2, 4, 5, and 7, so column sizes will be the same in between these floors. Table 4-1 in AISC is used to find the size of all of the gravity columns. These sizes are then confirmed with the RAM design. These calculations can be seen in Appendix

C. A section of each of the designed frames can be seen in the Details section of this report.

For each of the new gravity columns, sizes are increased to account for the added loads. In a special case for columns that are located on the Ma grid line, an added member is required to account for the added load. Columns that are located on the Ma grid line use a transfer girder at level 5 to transfer load to columns on different gridlines, as seen in Figure 33. Due to the added weight of the helipad, the transfer girder required for the new load would not fit in the space allowed. Columns are added below level 5 on the Ma grid line so that the use of a transfer girder is not required. The sizes for these new columns can also be seen in the Details section of this report. The design of these new columns can be found in Appendix C.

Foundation Redesign

The foundations supporting the new gravity columns are also designed by using hand calculations and checked using RAM. The loads found in the previous section are used as the loads acting on the foundations. Values for the soil are found using the existing Geotechnical report. Values obtained from the report are as follows:

γ_{dry} = 94.39pcf γ_{wet} = 103.85pcf ω = 10.02 qa = 6ksf

These values are used to find a γ value of 103.85pcf. Also, it is assumed that a base plate size of 24"x24" is used.

When designing the foundations, certain assumptions are made for ease of construction. The first assumption is that the foundation is square. This cuts down on any mistakes that might be made by putting the foundation in the wrong direction. Also, #8 rebar is used throughout the gravity foundations in order to be consistent with sizes. A clear cover of 3" is also used according to ACI 318-05, because it is permanently exposed to the earth. The last assumption that is made is that the depth of the rebar, d, is the height from the top of the foundation to the space between the two layers of rebar. This assumption is valid because the foundation has equal length and width dimensions.

For this section of the report, only the gravity foundations are designed. All lateral foundations will be designed later in this report, in the Lateral Foundation Design section. The hand calculations for the gravity foundations can be found in Appendix D, while the results can be found in the Details section of this report.

The gravity foundations are checked against the RAM model. For a sample spot check, the foundation located at grid line La-7a is checked against the RAM model. The hand calculations found that the foundation should be 10' x 10' with a depth of 1'-8". It is also found that the foundation should use 11 #8 rebar in each direction. The RAM model found that the foundation should be 11' x 11' with a depth of 2'-6" and 12 #7 rebar. Since these two foundations are somewhat similar, the hand calculations and RAM model are assumed accurate.

Details

The following section is a summary of the gravity columns and foundations sizes. In Figures 34-42 below, the column sizes and foundation sizes are shown for the grid lines La-7a, La-8a, La-9a, Na-8a, Na-9a, Ma-8a, Ma-9a, and Ma-10a.

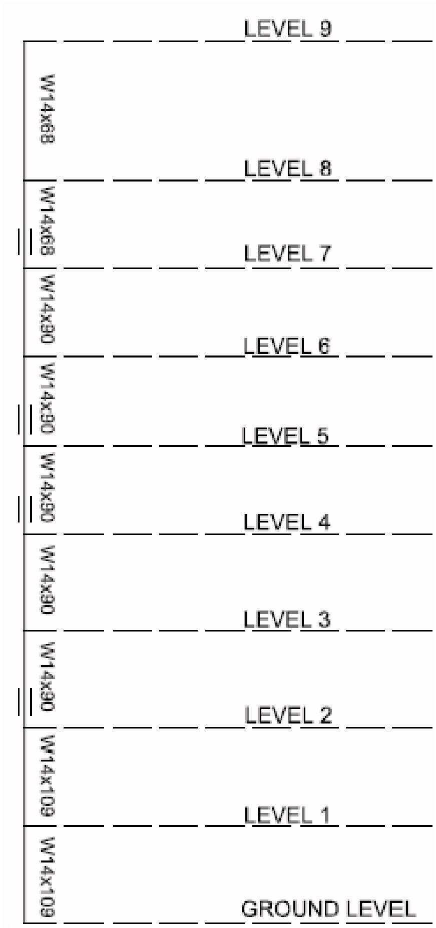


Figure 34: Grid Line La-7a Columns

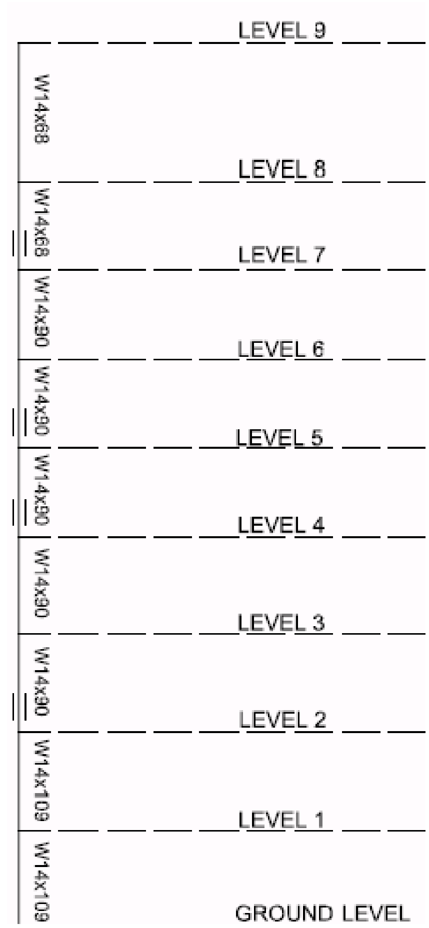


Figure 35: Grid Lines La-8a, La-9a, Na-8a, and Na-9a Columns

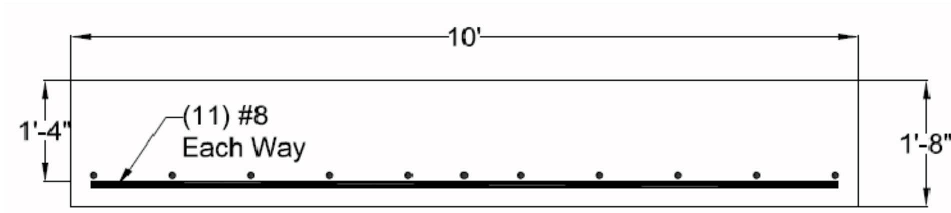


Figure 36: Grid Line La-7a Foundation

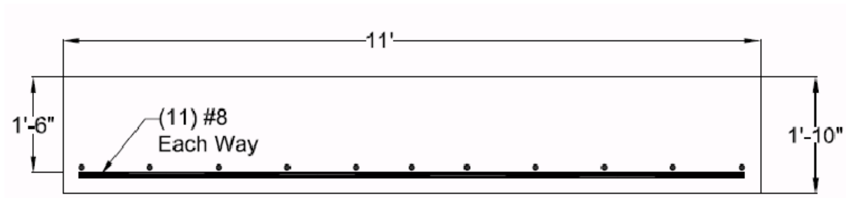


Figure 37: Grid Lines La-8a, La-9a, Na-8a, and Na-9a Foundations

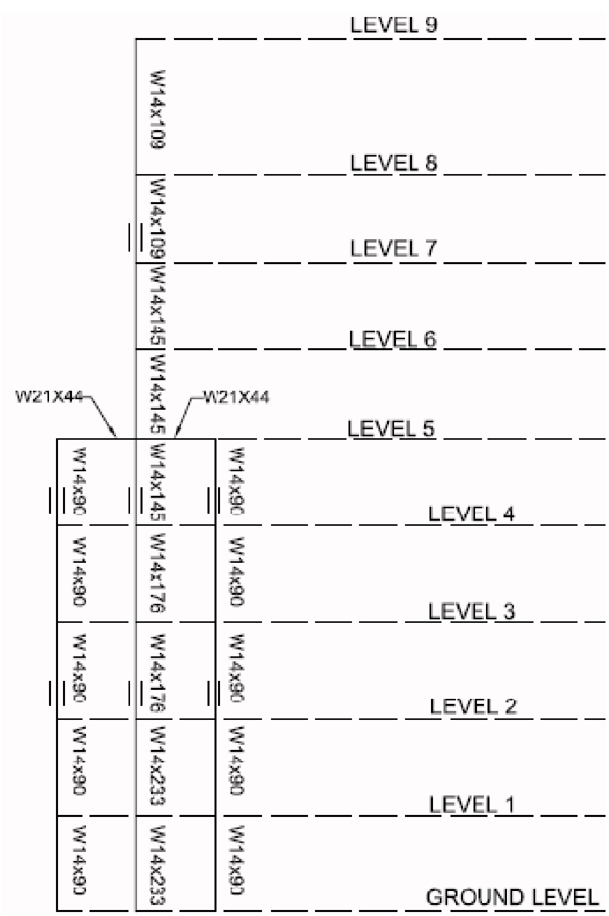


Figure 38: Grid Lines Ma-8a and Ma-9a Columns

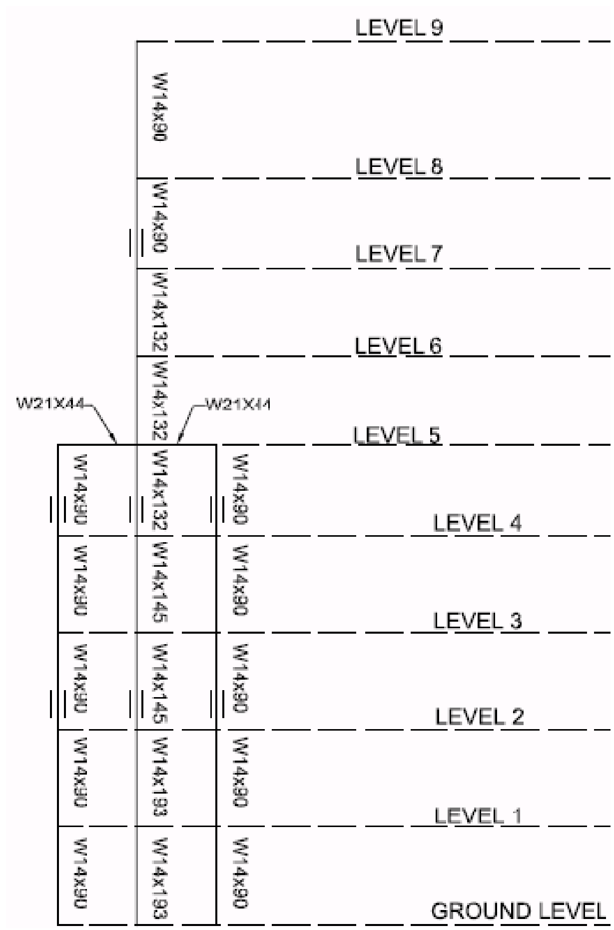


Figure 39: Grid Line Ma-10a Columns

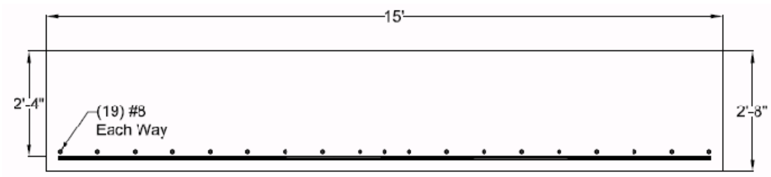


Figure 40: Grid Lines Ma-8a and Ma-9a Foundations for W14x233 Column

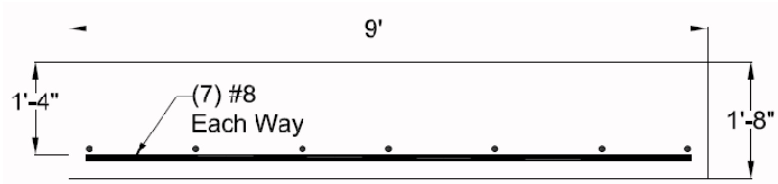


Figure 41: Grid Line Ma-10a Foundation for W14x183 Column

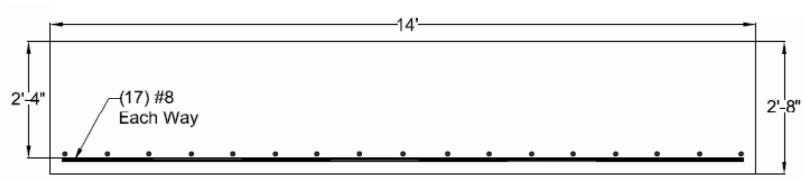


Figure 42: Grid Lines Ma-8a, Ma-9a, and Ma-10a Foundations for W14x90 Columns

Lateral System Design

Wind Loads

The wind loads used in the final report of the Voorhees Replacement Facility are the same wind loads that are calculated in the First Technical Report. For these calculations a design wind speed of 100 mph is used per Figure 6-1 in ASCE 7-05. Also, Exposure Category B and an Importance Category III are used. In Table 11 below the calculated wind forces, shear, and moments are calculated. In Figures 43 and 44 the wind loads are shown in pounds per square foot. The calculations for the wind forces can be found in Appendix E.

Level	Height Above Ground (ft)	Wind Forces					
		Load (kip)		Shear (kip)		Moment (ft-kip)	
		N-S	E-W	N-S	E-W	N-S	E-W
9	142.00	122.9	124.6	0	0	17123.7	17360.5
8	117.33	199.3	202.1	122.9	124.6	23383.9	23712.4
7	103.33	151.7	153.9	322.2	326.7	15902.5	15902.5
6	89.33	149.2	151.4	473.9	480.6	13328.0	13524.6
5	75.33	146.4	148.4	623.1	632.0	11028.3	11179.0
4	61.33	149.9	151.9	769.5	780.4	9193.4	9316.0
3	46.00	152.5	154.6	919.4	932.3	7015.0	7111.6
2	30.66	146.9	148.9	1071.9	1086.9	4504.0	4565.3
1	15.33	139.1	141.0	1218.8	1235.8	2132.4	2161.5
Total		**1357.9**	**1376.8**	**1357.9**	**1376.8**	**103611.2**	**104833.4**

Table 11 - Calculated Wind Forces

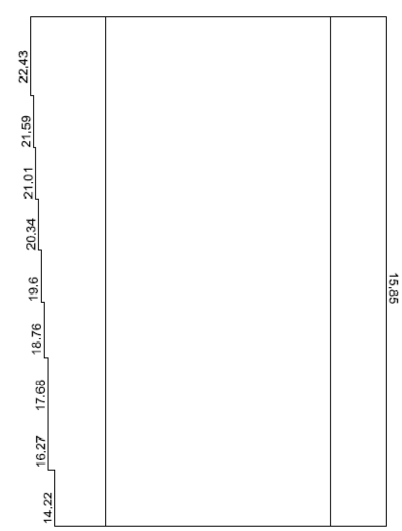


Figure 43: Wind Loads in East - West Direction (psf)

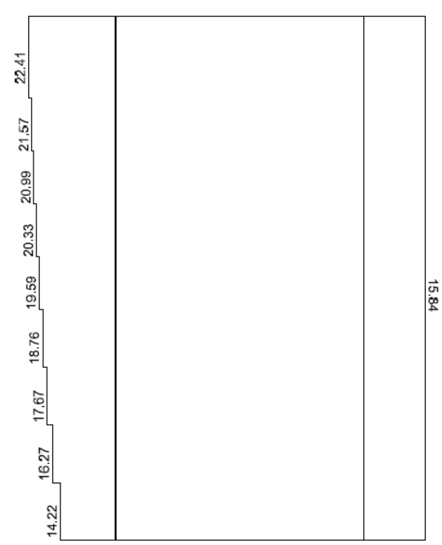


Figure 44: Wind Loads in North - South Direction (psf)

Seismic Loads

The new seismic loads are calculated using the Equivalent Lateral Force Procedure in ASCE 07-05. Some of the same assumptions that are made in Technical Report #1 are also made for this final report. One of those justifications is the use of geopiers to densify the soil. The original soil type was too sandy for the loads created by the building; in order to fix this problem geopiers, or stone piles are used to densify the soil. Because these are used, the site class is assumed to be category D. The calculations for the building's seismic loads can be found in Appendix F. In Tables 12-14 below the new and old seismic loads are shown for a comparison.

New Building B Seismic Loads							
Level	**Story Weight**	**Height**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force**	**Story Shear**	**Moment**
	(kip)	*(feet)*			*(kips)*	*(kip)*	*(foot-kip)*
R	2162.2	142	959865.819	0.229	300.410	300.410	42658.184
8	788.8	117.33	276910.905	0.066	86.665	387.075	10168.400
7	1294.5	103.33	388687.490	0.093	121.648	508.722	12569.861
6	515.75	89.33	129469.049	0.031	40.520	549.242	3619.652
5	3350.9	75.33	682074.899	0.162	213.469	762.712	16080.647
4	4709.4	61.33	744394.792	0.177	232.974	995.685	14288.274
3	5212.5	46	578413.207	0.138	181.026	1176.712	8327.210
2	4133.2	30.66	278464.711	0.066	87.151	1263.863	2672.058
1	5592.2	15.33	160619.972	0.038	50.269	1314.132	770.629
Sum	**27759.45**		**4198900.844**	**1.000**	**1314.132**		**111154.915**

Table 12 - New Seismic Loads

Old Building B N-S Seismic Loads							
Level	**Story Weight**	**Height**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force**	**Story Shear**	**Moment**
	(kip)	*(feet)*			*(kips)*	*(kip)*	*(foot-kip)*
R	1556.8	139.33	466234.47	0.164	252.125	252.125	35128.580
8	788.8	117.33	193702.12	0.068	104.748	356.873	12290.089
7	1294.5	103.33	274494.46	0.096	148.438	505.311	15338.101
6	515.75	89.33	92436.0	0.032	49.987	555.298	4465.294
5	3350.9	75.33	493241.3	0.173	266.729	822.027	20092.733
4	4709.4	61.33	546673.3	0.192	295.624	1117.651	18130.609
3	5212.5	46	434041.6	0.152	234.716	1352.367	10796.942
2	4133.2	30.66	215415.7	0.076	116.490	1468.857	3571.585
1	5592.2	15.33	130883.2	0.046	70.778	1539.635	1085.020
Sum	**27154.05**		**2847122.1**	**1.000**	**1539.635**		**120898.952**

Table 13 - Existing North - South Seismic Loads

Old Building B E-W Seismic Loads							
Level	**Story Weight**	**Height**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force**	**Story Shear**	**Moment**
	(kip)	*(feet)*			*(kips)*	*(kip)*	*(foot-kip)*
R	1556.8	139.33	466234.470	0.164	216.107	216.107	30110.211
8	788.8	117.33	193702.122	0.068	89.784	305.891	10534.362
7	1294.5	103.33	274494.458	0.096	127.233	433.124	13146.943
6	515.75	89.33	92436.003	0.032	42.846	475.969	3827.395
5	3350.9	75.33	493241.341	0.173	228.625	704.595	17222.343
4	4709.4	61.33	546673.288	0.192	253.392	957.986	15540.522
3	5212.5	46	434041.608	0.152	201.185	1159.172	9254.522
2	4133.2	30.66	215415.651	0.076	99.849	1259.020	3061.358
1	5592.2	15.33	130883.194	0.046	60.666	1319.687	930.017
Sum	**27154.05**		**2847122.135**	**1.000**	**1319.687**		**103627.673**

Table 14 - Existing East - West Seismic Loads

Because braced frames are used exclusively in the new calculations, it gives the building a new R value, thus lowering the seismic base shear in both directions. Even though the extra weight is added to the building, this is not enough to counteract the use of a new lateral system.

Load Combinations

There are seven load cases that are considered in this report. The load cases consist of seismic in the East – West direction, seismic in the North – South direction, wind case 1 in the East – West direction, wind case 1 in the North – South direction, wind case 2, wind case 3, and wind case 4. Wind case 1 consists of 100% of the wind load in one direction, while wind case 2 has 75% of the wind pressure acting in one direction with a tensional moment. Wind cases 3 and 4 are similar to 1 and 2 respectively except that they use 75% of the load in both directions at the same time.

Seven load combinations are also considered in this report. The load combinations are taken from ASCE 7-05 section 2.3 and are as follows:

1.4(D + F)
1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
1.2D + 1.6(Lr or S or R) + (L or 0.8W)
1.2D + 1.6W + L + 0.5(Lr or S or R)
1.2D + 1.0E + L + 0.2S
0.9D + 1.6W + 1.6H
0.9D + 1.0E + 1.6H

For the most part throughout this design the controlling load combination is the 1.2D + 1.6W + 1.0L + 0.5S combination. This is expected considering in most cases the new wind loads control over the new seismic loads.

Braced Frame Design

The new braced frame design is dependent on many different factors. One of the main factors that control the braced frame design, and more importantly placement, is the architecture. If a hallway or other type of room runs in between two columns a braced frame cannot be placed in that area, or it will cut the hallway or room in half and would then need to be moved so that it can stay continuous. Because this has the potential to create problems to the flow of traffic in the building, an attempt is made to keep all of the originally designed braced frames. Since moment frames are also used in the lateral system, just using the old braces, and not adding new braces was not an option. The lateral system also requires new braced frames to be designed.

Another placement factor that is considered is the symmetry of the braces. In order to provide the least amount of torsion in the building, the center of rigidity is attempted to remain as close to the center of mass as possible. This means that braces are placed on the north side of the building, requires braces to be placed on the south side of the building.

Strength also plays a part in determining the braced frame design and placement. In theory it is possible to use a very small number of braces as long as they are of adequate, large size. This report made an attempt to use smaller brace and column sizes, but use more of them to take the lateral load. Lateral column sizes are attempted to remain similar to the gravity column sizes for ease of construction. Also, the brace sizes are attempted to remain similar in size to the originally designed braced frames. Since these smaller sizes are used, more braces are required in the system.

The braced frames are also designed using a basic assumption that the only force in any of the members of the braced frame is an axial force. Due to the type of connections used to attach the braces to the columns, the only load that is transferred between them is axial. This is a crucial assumption with the braced frame since it assumes that the columns will then have no moments to resist, only axial. Hand calculations and a SAP model are used to prove this assumption in Appendix G and in Figure 45 below. A brace from the building is taken and subjected to the new user defined loads found in the Wind and Seismic sections above. The members are modeled and the moments in the strong and weak axis are released from each member. The model is run and the forces in each of the members are then recorded. These forces are then checked against the forces found in the hand calculations and found to be equal. The SAP model confirmed the original assumption that axial load is the only acting load case. Figure 46 below shows the axial forces in each member and how the load is transferred to the base.

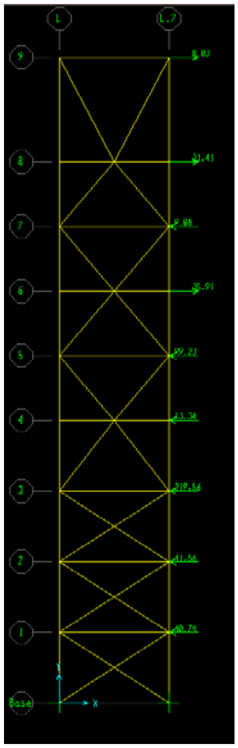


Figure 45 - SAP Model

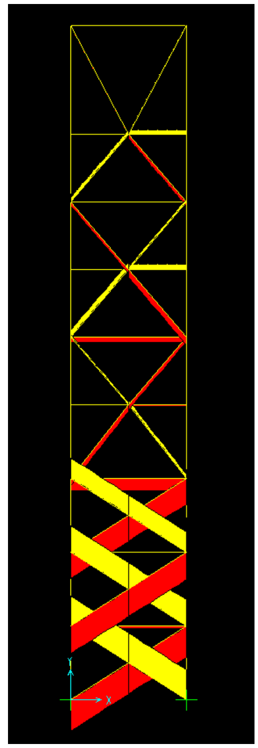


Figure 46 - SAP Model with Axial Forces Shown

RAM Model

For this report, a RAM model is created in order to design the lateral system, and check the gravity system. The model is created of the southern building only, based on the assumption that the northern and southern buildings act separately. The building is modeled as closely to the original design as possible, including minor gravity beams where specified. This is to ensure that the building has the proper dead weight included. To account for dead weight from partitions and other objects, a superimposed dead weight of 20 psf is used. An area specific live load is also added to the model to insure proper gravity member calculations. The building is split into smaller areas, and assigned the corresponding live loads; live load values can be found in Table 1 above in the Gravity Loads section. Figures 47-48 below show the RAM model used for this report.

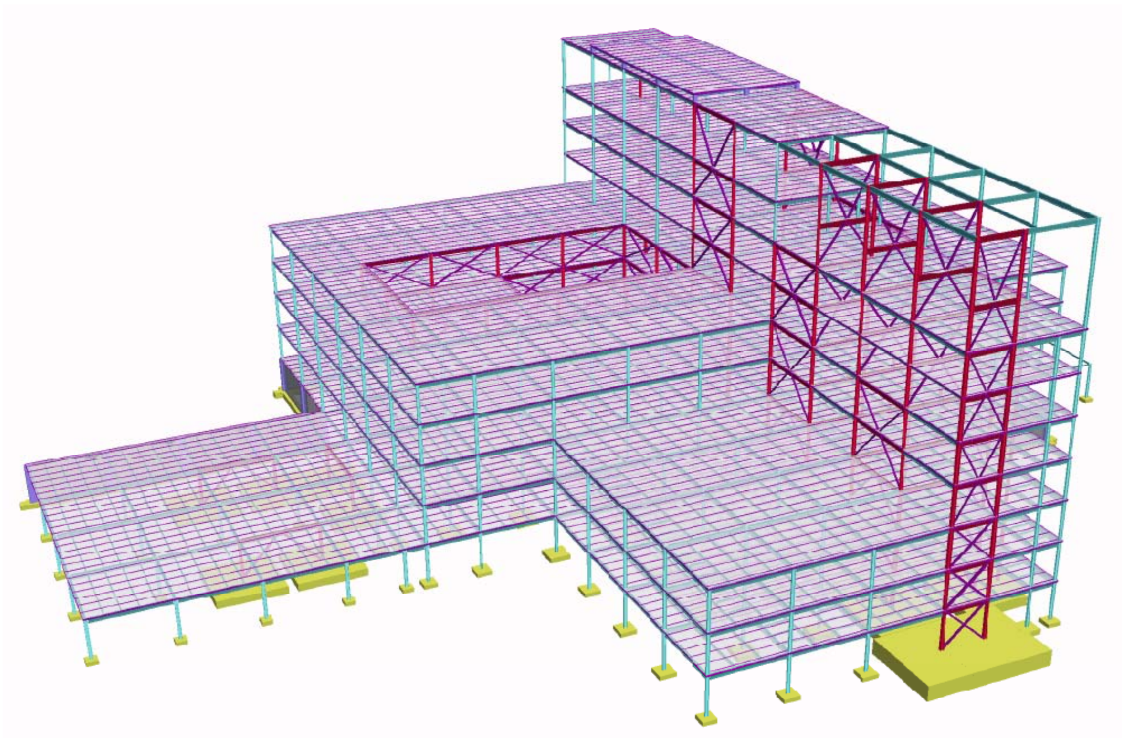


Figure 47 - RAM Model from Southwest

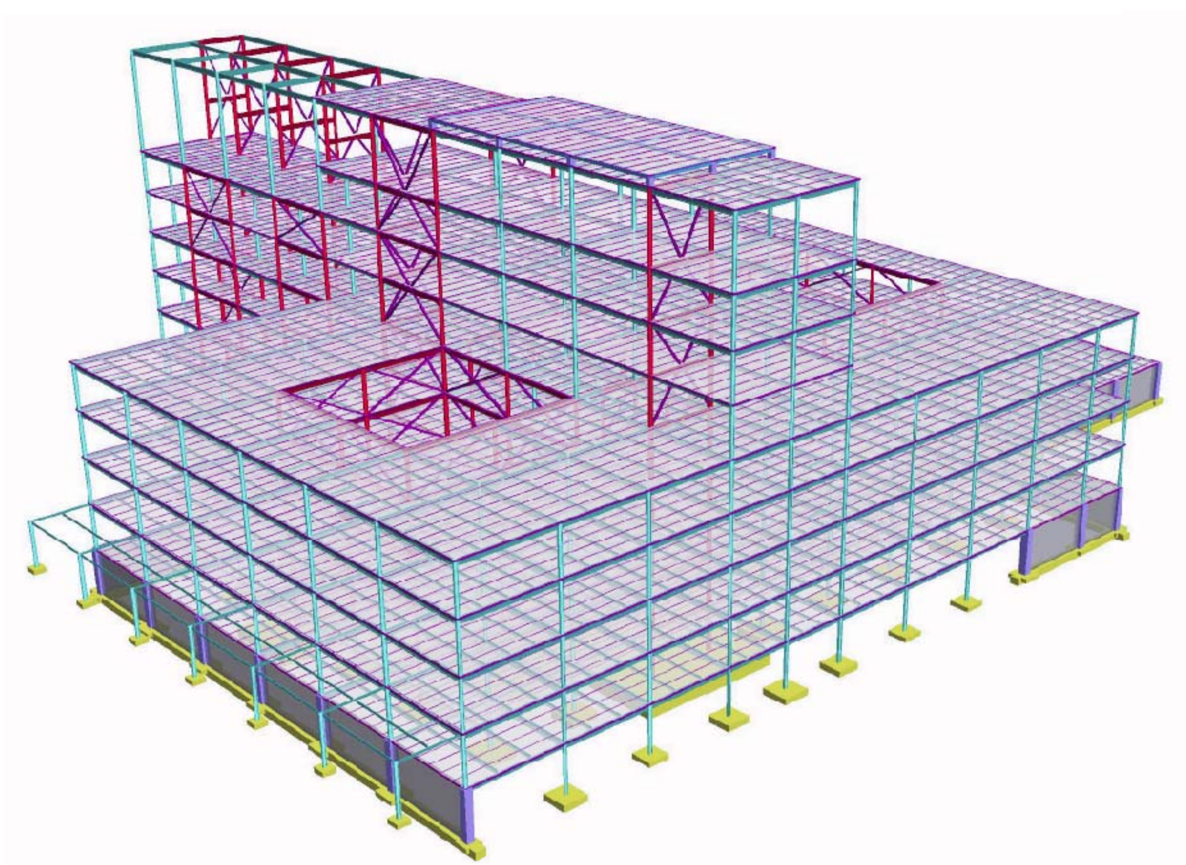


Figure 48 - RAM Model from Northeast

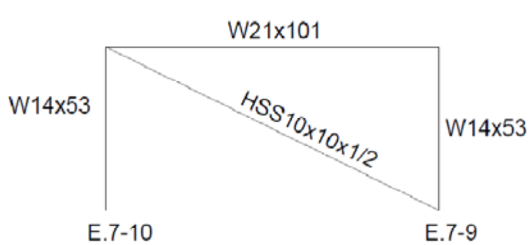
After the building is modeled, a series of designs are performed to find the beam, column, foundations, and frame sizes. The beams are designed as composite beams, using RAM Steel Beam; while the columns are all designed as W14 shapes using RAM Steel Column. The columns are kept as W14 so that the columns sizes could be kept consistent and unnecessary complicated splices could be avoided. The foundations are designed using RAM Foundation using the controlling loads found in RAM Steel Column and RAM Frame.

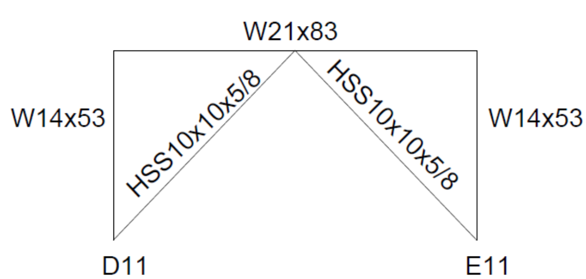
The lateral frames are designed and analyzed using RAM Frame. The frames are designed so that they consider many requirements. One of the requirements considered is the use of P-Delta effects. Another requirement for the design was to consider all loading cases and combinations. This includes all of the different wind and seismic cases discussed in Load Combinations section of this report. To ensure that the loads for wind and seismic are found correctly, wind and seismic loads from the user are used, as well as RAM calculated wind and seismic loads. Wind and seismic loads from RAM are found by the program using the same criteria used for the user calculated loads found in Appendix E and F and by using IBC 2006 and ASCE 7-05.

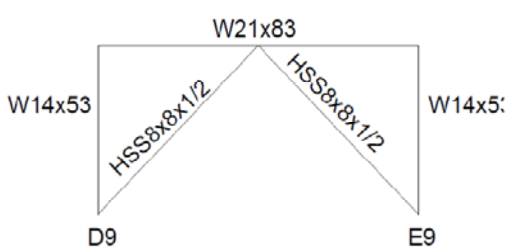
Once the lateral criteria is determined for the frames, sizes are required in order to analyze the system. W14x68's and HSS6x6x½'s are used as a starting size. Any members failing this size are increased until all of the members pass. Column sizes are increased to consistent sizes for ease of construction. The only W14 shapes used are 68, 82, 109, 120, 132, and 145. The sizes in between the W14x68, W14x82, and W14x109 are skipped for this design.

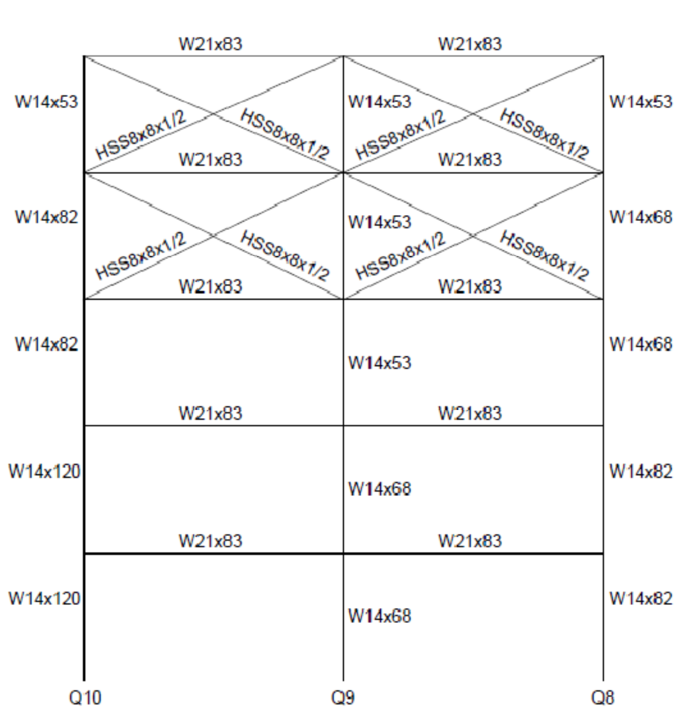
Frame Sizes and Layout

Once the analysis is complete in RAM Frame, final sizes and locations are recorded. Below are the frames found during design.








W21x83
W21x83
W14x53
HSS8x8x1/2
HSS8x8x1/2
W14x53
HSS8x8x1/2
HSS8x8x1/2
W14x53
W21x83
W21x83
W14x82
HSS8x8x1/2
HSS8x8x1/2
W14x53
HSS8x8x1/2
HSS8x8x1/2
W14x68
W21x83
W21x83
W14x82
W14x53
W14x68
W21x83
W21x83
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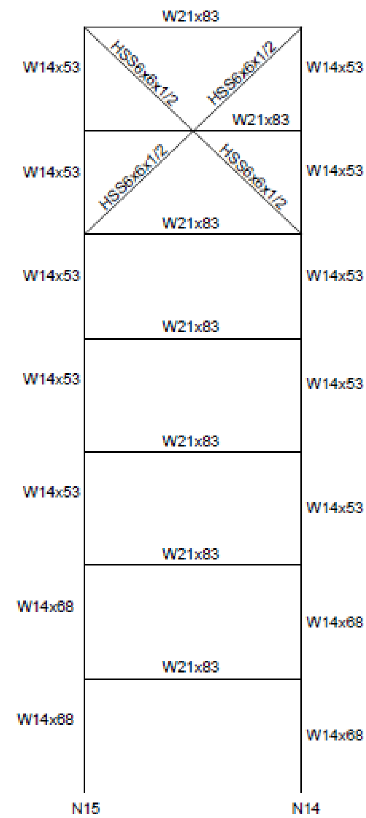
Q10
Q9
Q8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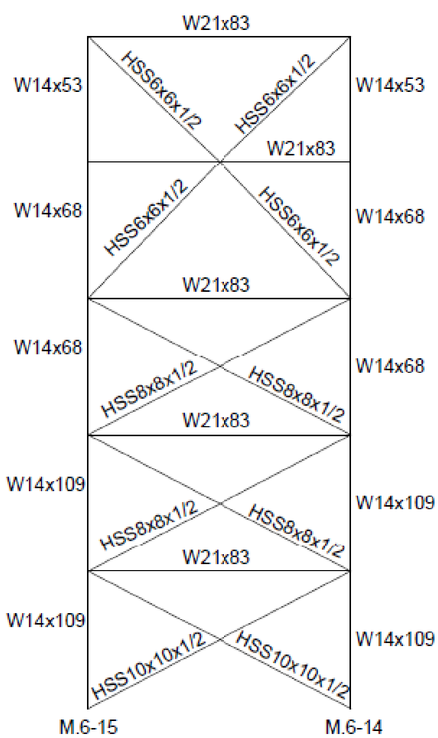
N15
N14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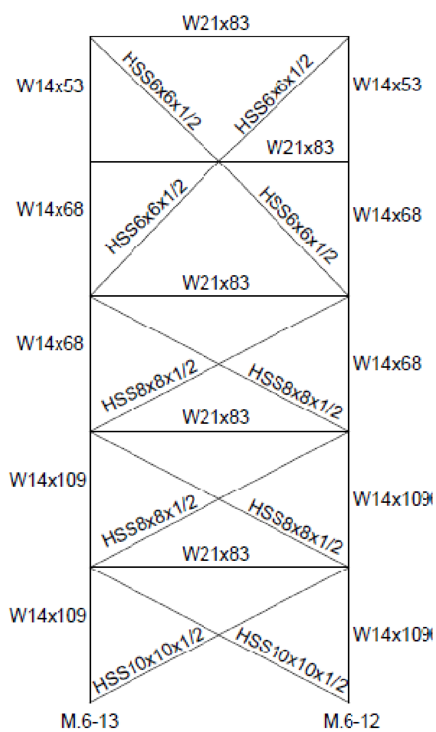
M.6-14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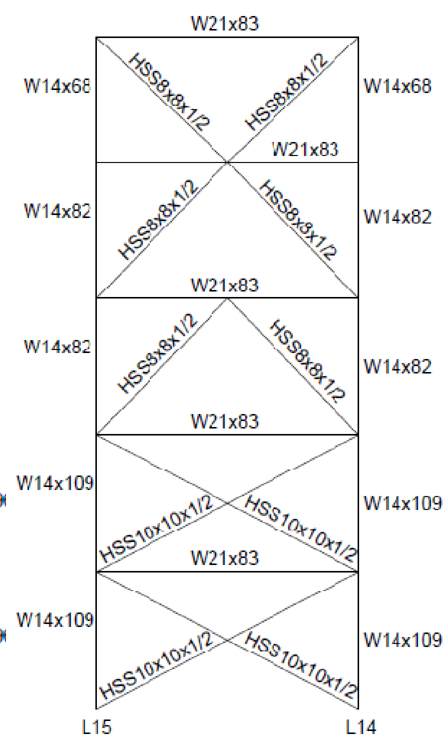
M.6-13
M.6-12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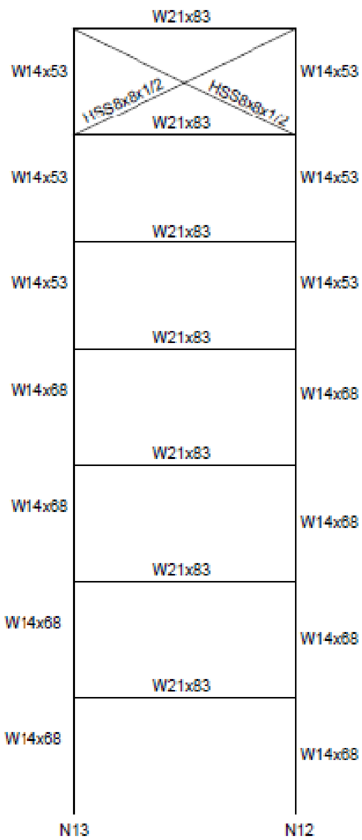
W14x109
L15
L14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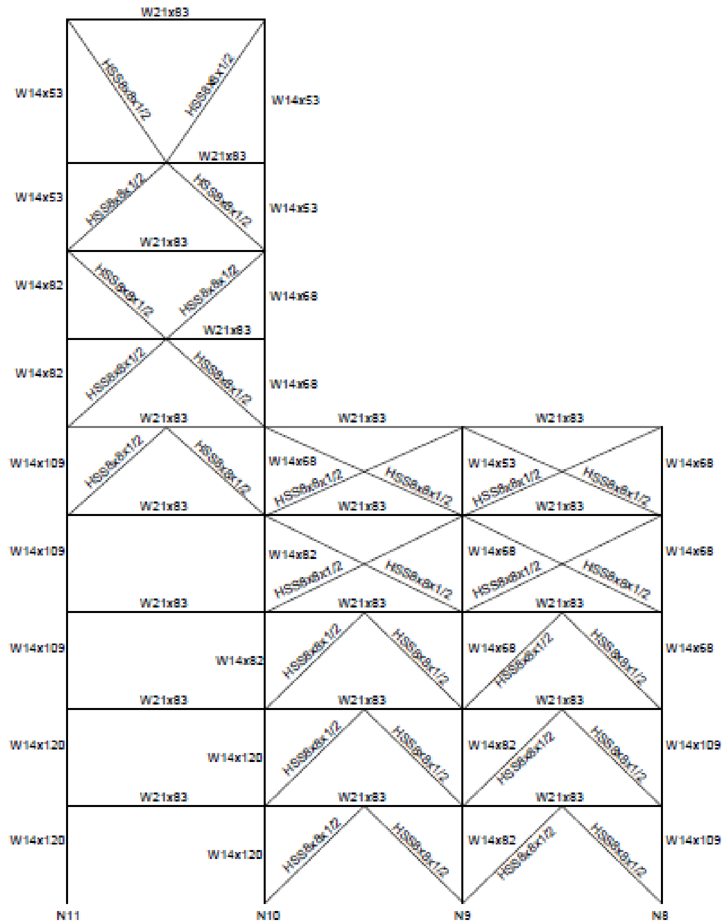
W14x68
N13
N12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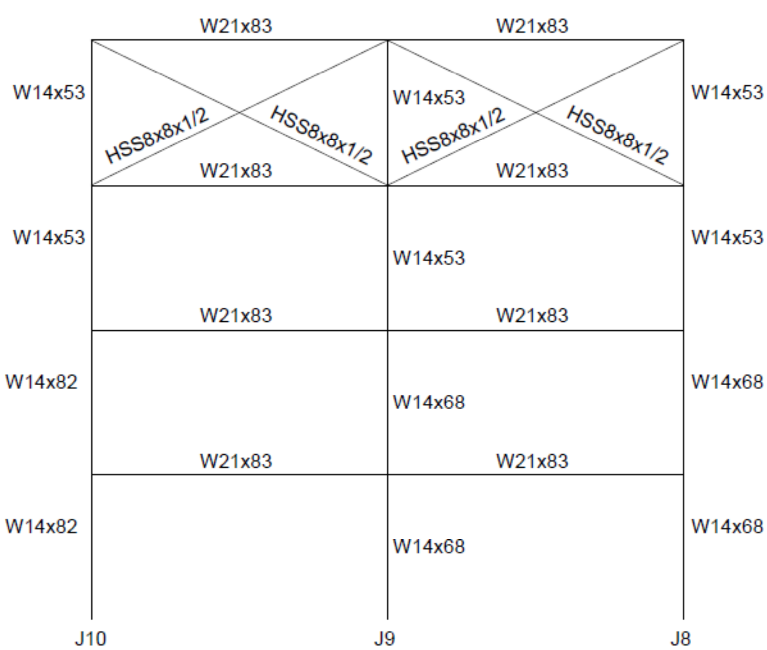
N10
N9
N8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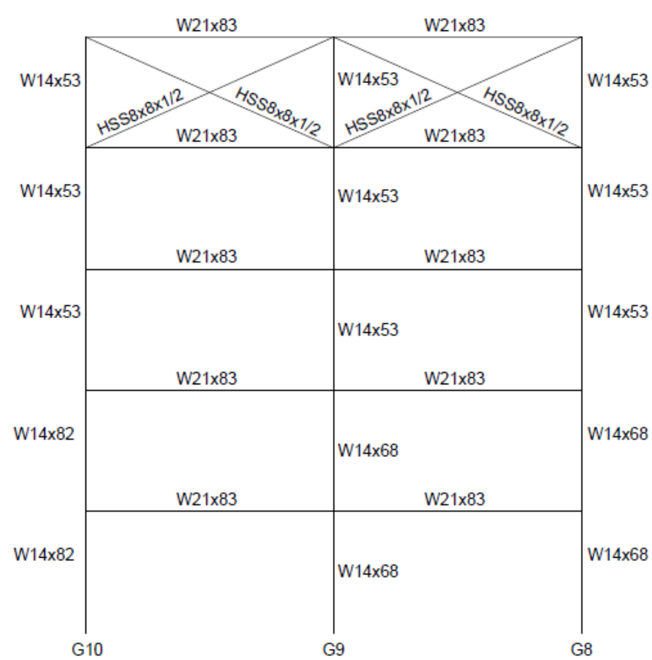
J10
J9
J8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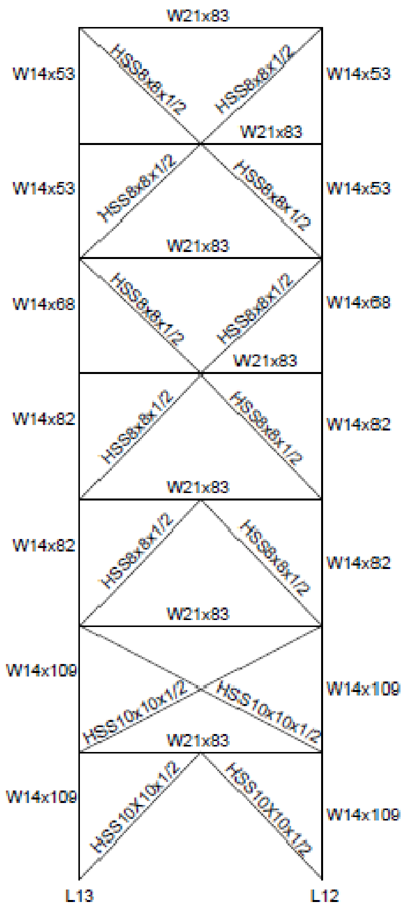
G10
G9
G8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W14x68
HSS8x8x1/2
HSS8x8x1/2
W14x68
W21x83
W14x82
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HSS8x8x1/2
W14x82
W21x83
W14x82
HSS8x8x1/2
HSS8x8x1/2
W14x82
W21x83
W14x109
HSS10x10x1/2
HSS10x10x1/2
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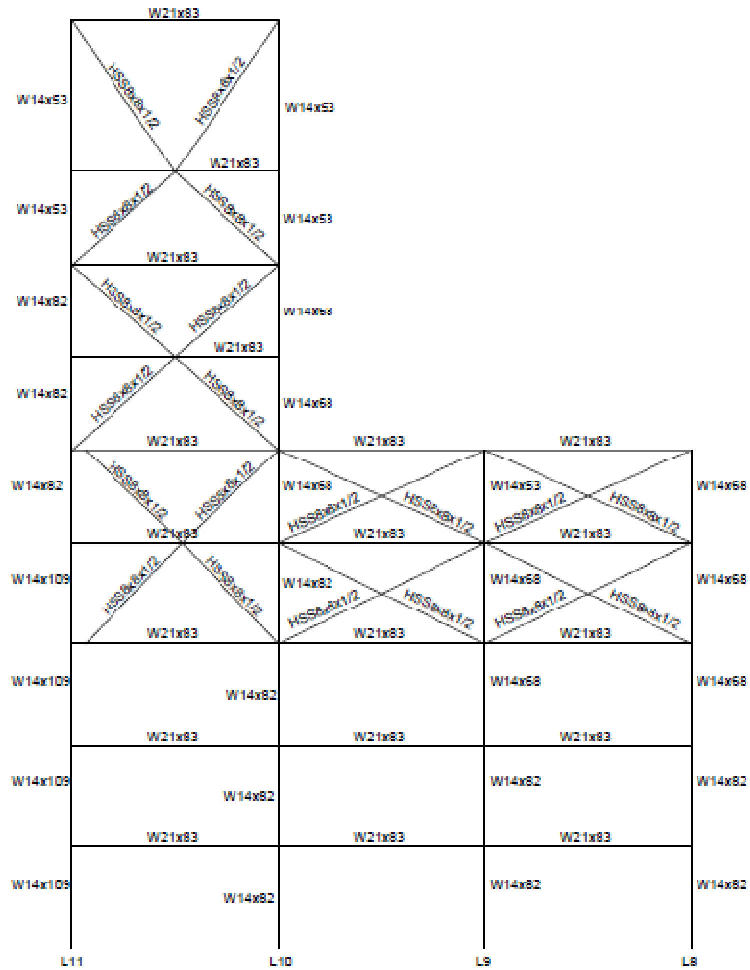
L12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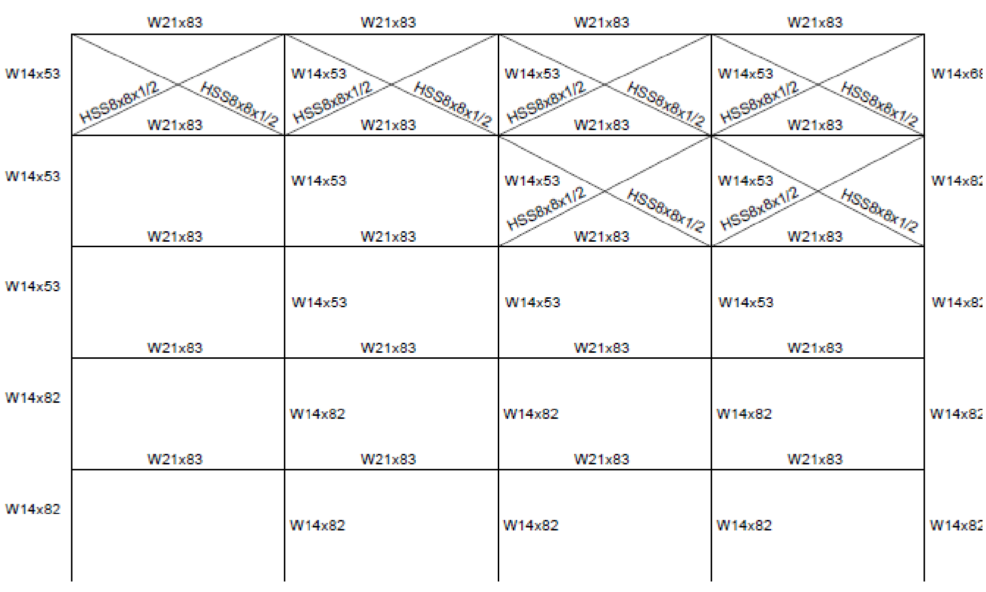
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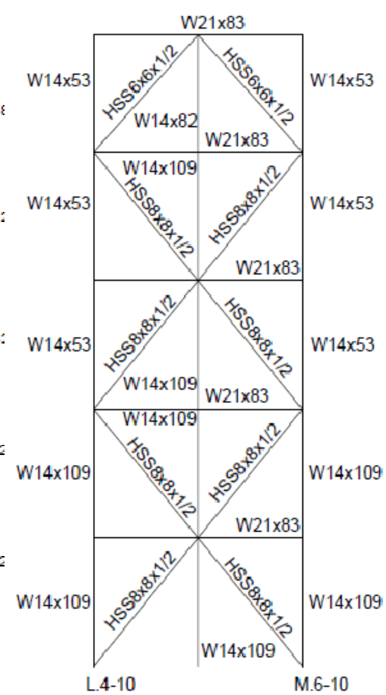
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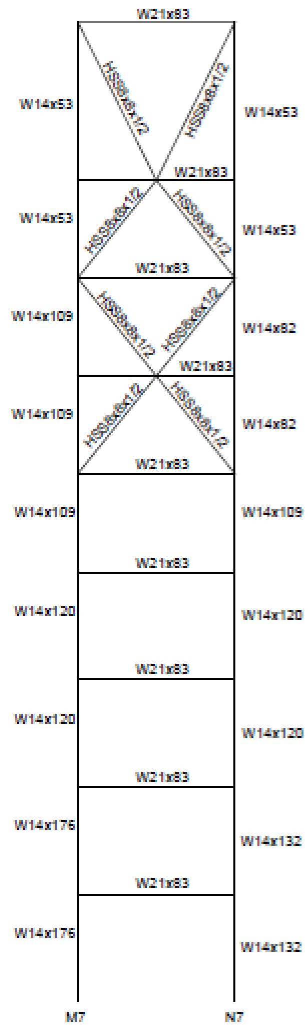
L.4-10
M.6-10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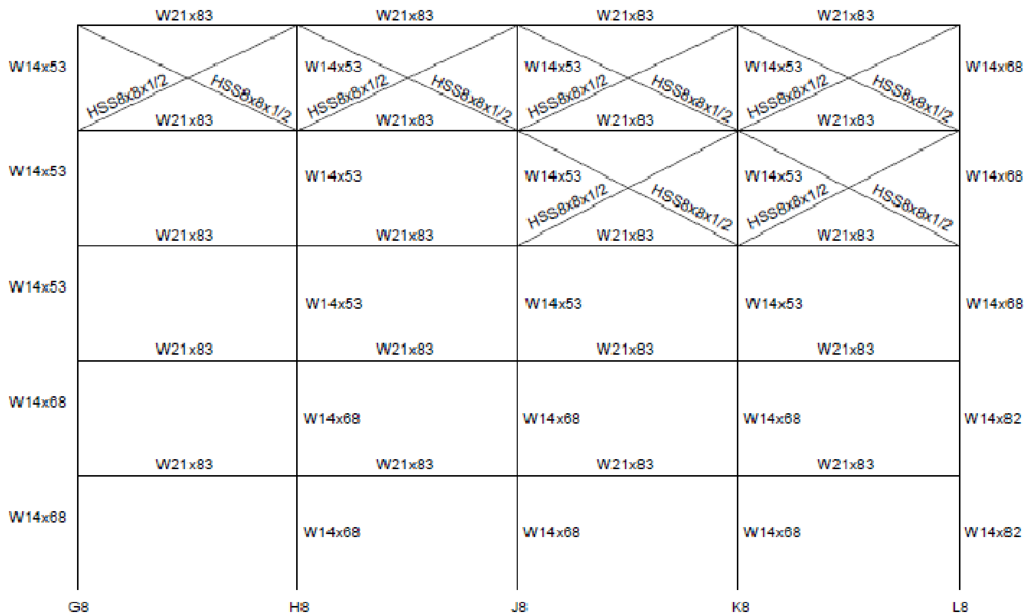
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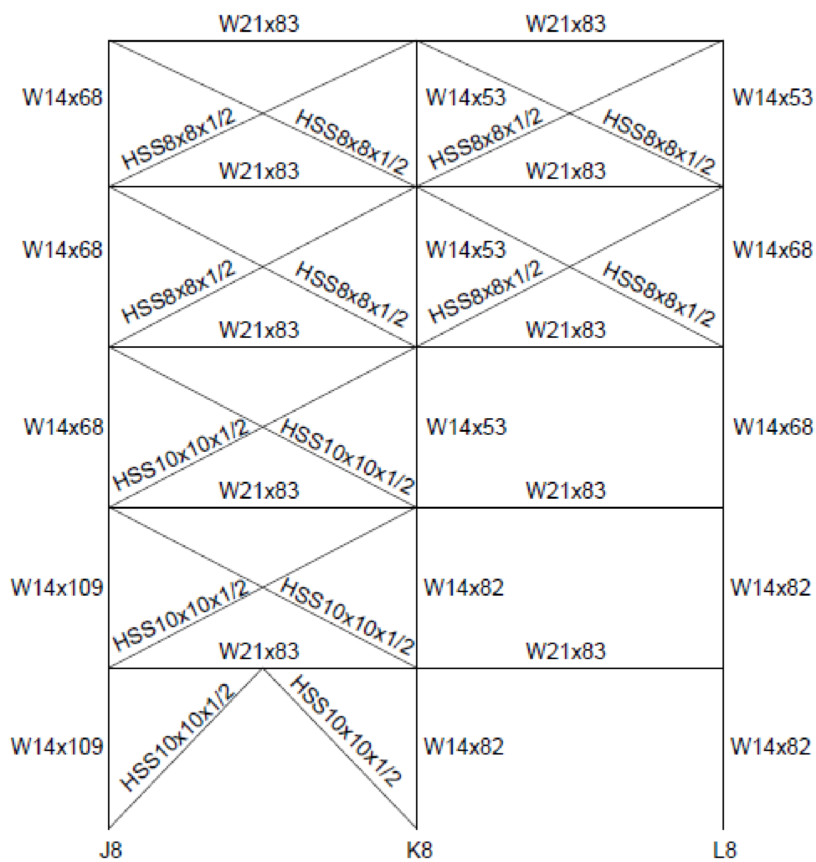
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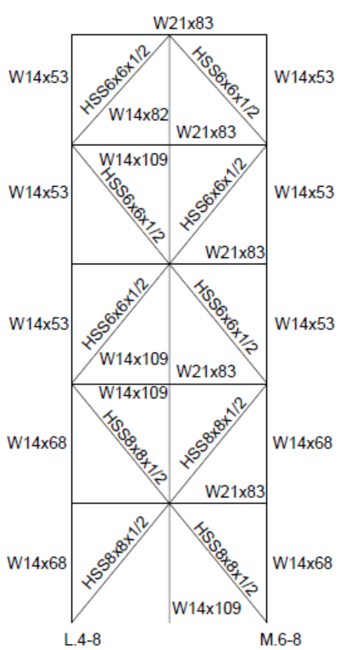
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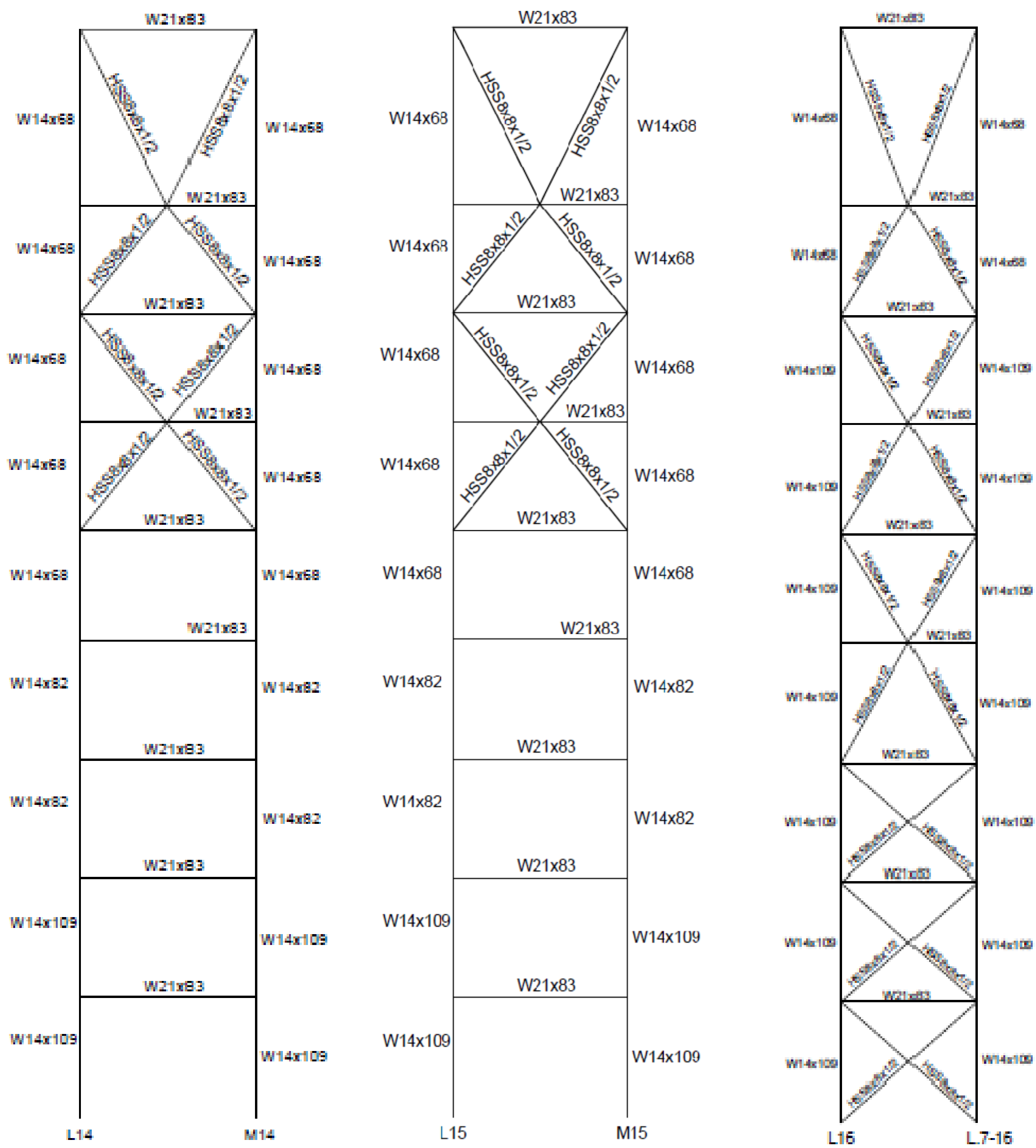
L.4-8
M.6-8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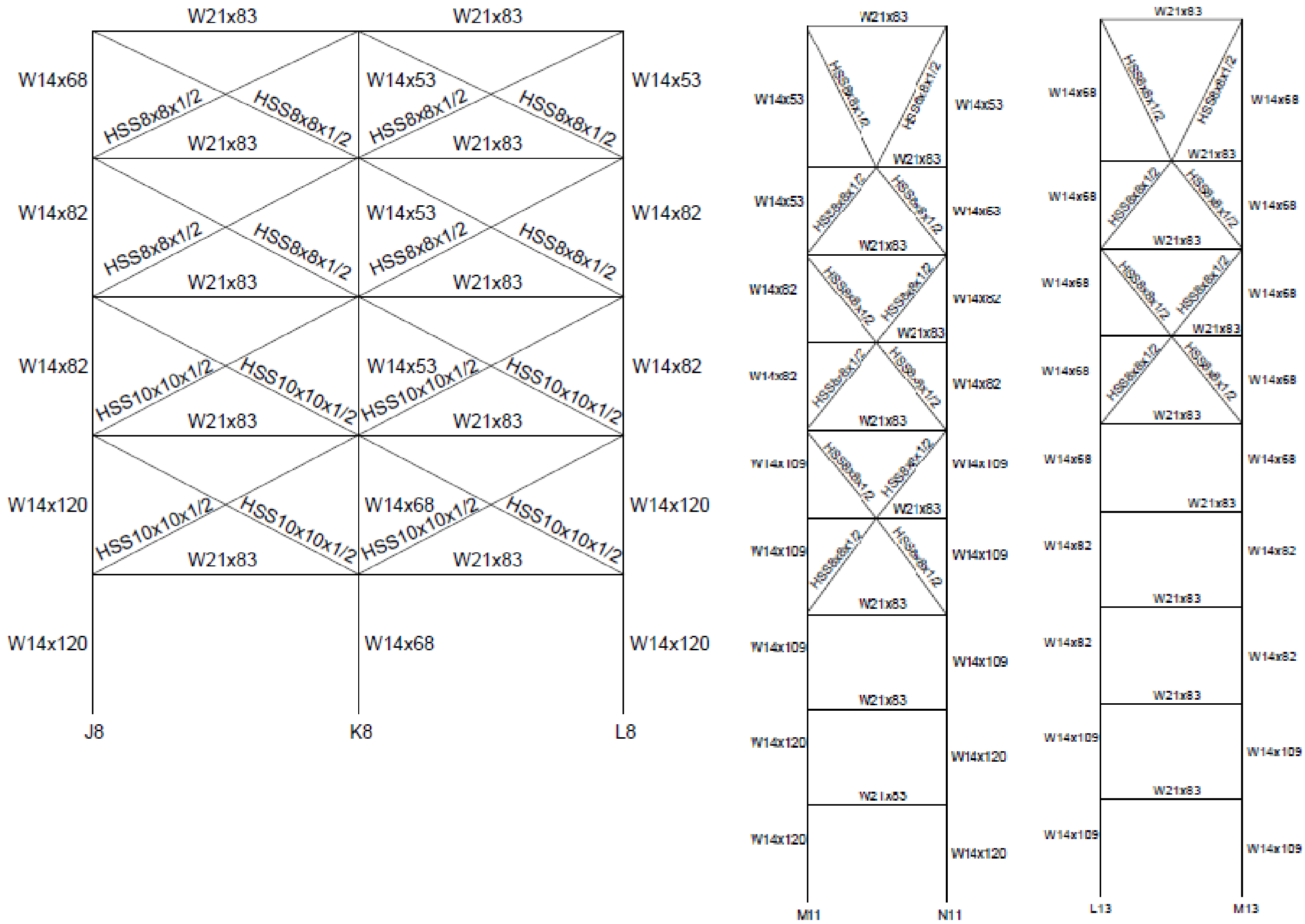
M15
L16
L7-16



The locations of all then new braced frames are able to avoid interfering with any rooms or hallways. The new braces are all able to be located in existing walls therefore not changing the layout of the building. Where the braces do cross in between a door or hallway, a ‘V’ type brace is used to avoid the open space.

Figure 49 - Windows Out Looking the Courtyard

The new braces however do make one change to the architecture of the building. In the courtyards in zones 4 and 5, ‘X’ braces span across the windows looking into the courtyard. The windows that surround the courtyards, as seen in Figure 49, will now have HSS shapes that span between bays. This is not expected to change the overall look of the rooms, or building, as it will still allow light into the area. It is assumed in this report that the addition of these braces is acceptable, as long as the steel will be painted to match the existing interior finishes.

Lateral Foundation Design

The lateral foundations are designed using RAM Foundation. The load cases and combinations used prior for the Lateral System Design are continued to be used in this section. The values for the soil are found using the existing Geotechnical report and are as follows:

γ_{dry} = 94.39pcf γ_{wet} = 103.85pcf ω = 10.02 qa = 6ksf

These values are used to find a γ value of 103.85pcf. Also, it is assumed that a base plate size of 24"x24" is used. These values are then placed into RAM and used to design the lateral foundations.

The majority of the footings used for this report are spread footings. In locations where columns are too close for individual footings, multiple columns are placed on the same footing and designed as continuous footings. In Tables 15 and 16 below the designed spread foundations' dimensions and specifications can be seen, while Tables 17-20 holds the dimensions of the continuous footings. The name of each foundation corresponds to the grid line that it is on.

Spread Footing Design Results					
Foundation	**Length Dimension** *(feet)*	**Width Dimension** *(feet)*	**Thickness** *(inches)*	**Bottom Reinforcement**	**Top Reinforcement**
Da-9a	23	23	23	(20) #7	(18) #5
Da-11a	30	30	30	(23) #7	(20) #7
Ea-9a	24	24	24	(20) #7	(21) #5
Ea-11a	30	30	30	(22) #7	(20) #7
Ga-8a	11	11	11	(10) #7	None
Ga-9a	11	11	11	(10) #7	None
Ga-10a	12	12	12	(11) #7	None
Ha-8a	11	11	11	(8) #8	None
Ha-10a	11	11	11	(8) #8	None
Ja-8a	11	11	11	(10) #7	None
Ja-9a	11	11	11	(8) #8	None
Ja-10a	11	11	11	(8) #8	None
Ka-8a	11	11	11	(8) #8	None
Ka-10a	11	11	11	(8) #8	None
La-11a	14	14	14	(17) #7	None
La-12a	30	30	30	(32) #8	20-#7
La-13a	30	30	30	(30) #8	20-#7
La-14a	30	30	30	(36) #7	20-#7
La-15a	30	30	30	(27) #8	20-#7

Table 15 - Spread Footing Design Results 1 of 2

Spread Footing Design Results					
Foundation	**Length Dimension**	**Width Dimension**	**Thickness**	**Bottom Reinforcement**	**Top Reinforcement**
	(feet)	*(feet)*	*(inches)*		
La-16a	30	30	30	(47)#7	25-#6
L.7a-16a	30	30	30	(47) #7	25-#6
Ma-7a	17	17	17	(24) #7	(13) #3
Ma-11a	15	15	15	(20) #7	None
Na-7a	23	23	23	(31) #7	(16) #5
Na-11a	14	14	14	(18) #7	None
Qa-8a	12	12	12	(13) #7	None
Qa-9a	12	12	12	(11) #7	None
Qa-10a	18	18	18	(21) #7	(16) #3

Table 16 - Spread Footing Design Results 2 of 2

Continuous Footing Dimensions			
Foundation Name	**Length Dimension (feet)**	**Width Dimension (feet)**	**Thickness (inches)**
	(feet)	*(feet)*	*(inches)*
E.7a-9a to Fa-9a	17.00	6.00	18
E.7a-10a to Fa-10a	18.00	6.00	18
La-8a to Na-8a	63.00	12.00	60
La-9a to Na-9a	54.00	12.00	30
La-10a to Na-10a	62.00	12.00	60
Ma-12a to Na-12a	51.50	11.00	60
Ma-13a to Na-13a	53.50	11.00	48
Ma-14a to Na-14a	54.50	11.00	48
Ma-15a to Na-15a	59.50	11.00	48

Table 17 - Continuous Footing Dimensions

Continuous Footing Design Results					
Foundation	**Top Longitudinal Rebar**	**Bottom Longitudinal Rebar**	**Top Transverse Rebar**	**Bottom Transverse Rebar**	**Shear Rebar**
E.7a-9a to Fa-9a	0'-4' – (5) #3 4'-12' – (12) #4 12'-17' – (12) #4	0'-8' – (5) #7 8'-17' – (10) #7	0'-17' – (1) #3	0'-6' – (10) #3 6'-11' – (6) #5 11'-17' – (10) #3	None
E.7a-10a to Fa-10a	0'-5' – (5) #3 5'-13' – (14) #4 13'-18' – (13) #4	0'-9' – (6) #7 9'-18' – (8) #8	0'-15' – (1) #3 15'-18' – None	0'-3' – (10) #3 3'-15' – (5) #5 15'-18' – (4) #5	None

Table 18 - Continuous Footing Design Results 1 of 3

Continuous Footing Design Results					
Foundation	**Top Longitudinal Rebar**	**Bottom Longitudinal Rebar**	**Top Transverse Rebar**	**Bottom Transverse Rebar**	**Shear Rebar**
La-8a to Na-8a	0'-9' – (10) #4 9'-19' – (21) #4 19'-31' – (19) #4 31'-44' – (18) #4 44'-54' – (19) #4 54'-63' – (9) #3	0'-12' – (23) #7 12'-24' – (15) #8 24'-49' – (20) #7 49'-63' – (25) #7	None	0'-5' – (11) #7 5'-13' – (18) #7 13'-15' – (6) #6 15'-23' – (18) #7 23'-27' – (9) #7 27'-36' – (15) #8 36'-40' – (9) #7 40'-48' – (18) #7 48'-50' – (6) #6 50'-58' – (18) #7 58'-63' – (11) #7	None
La-9a to Na-9a	0'-4' – (14) #3 4'-14' – (18) #4 14'-27' – (20) #4 27'-39' – (19) #4 39'-49' – (17) #4 49'-54' – (9) #3	0'-9' – (21) #5 9'-21' – (9) #4 21'-33' – (31) #5 33'-44' – (10) #4 44'-54' – (22) #5	None	0'-2' – (5) #6 2'-17' – (6) #7 17'-25' – (7) #8 25'-29' – (6) #6 29'-37' – (7) #8 37'-52' – (6) #7 52'-54' – (3) #6	None
La-10a to Na-10a	0'-8' – (9) #4 8'-18' – (22) #5 18'-31' – (15) #5 31'-43' – (22) #5 43'-53' – (26) #5 53'-62' – (9) #4	0'-12' – (23) #7 12'-32' – (14) #8 32'-62' – (23) #7	None	0'-4' – (9) #7 4'-13' – (15) #8 13'-14' – (3) #6 14'-23' – (15) #8 23'-27' – (9) #7 27'-35' – (18) #7 35'-39' – (9) #7 39'-48' – (15) #8 48'-49' – (3) #6 49'-58' – (15) #8 58'-62' – (9) #7	None
Ma-12a to Na-12a	0'-14' – (16) #4 14'-27' – (19) #4 27'-37' – (20) #4 37'-51.5' – (8) #3	0'-21'-(25) #7 21'-36'-(23) #7 36'-51.5'-(17) #8	None	0'-10'-(13) #9 10'-19'-(15) #8 19'-23'-(9) #7 23'-31'-(18) #7 31'-33-(6) #6 33'-41'-(18) #7 41'-51.5'-(23) #7	None

Table 19 - Continuous Footing Design Results 2 of 3

Continuous Footing Design Results					
Foundation	**Top Longitudinal Rebar**	**Bottom Longitudinal Rebar**	**Top Transverse Rebar**	**Bottom Transverse Rebar**	**Shear Rebar**
Ma-13a to Na-13a	0'-15' – (8) #3 15'-28' – (24) #4 28'-38' – (25) #4 38'-53.5' – (8) #3	0'-21' – (18) #8 21'-37' – (22) #7 37'-53.5'-(18) #7	0'-24' – None 24'-32' – (1) #5 32'-34' – (1) #3 34'-42' – (1) #5 42'-53.5'- None	0'-12'-(21) #7 12'-19'-(13) #7 19'-24'-(9) #7 24'-32'-(8) #9 32'-34'-(2) #9 34'-42'-(8) #9 42'-53.5'-(20) #7	None
Ma-14a to Na-14a	0'-16' – (13) #3 16'-28' – (23) #4 28'-38' – (24) #4 38'-54.5'-(11) #3	0'-22' – (18) #8 22'-38' – (22) #7 38'-54.5'-(17) #7	0'-25' – None 25'-32' – (1) #5 32'-35' – (1) #4 35'-42' – (1) #5 42'-53.5'-None	0'-12' – (21) #7 12'-20' – (14) #7 20'-25' – (9) #7 25'-32' – (7) #9 32'-35' – (5) #7 35'-42' – (7) #9 42'-54.5'-(13) #9	None
Ma-15a to Na-15a	0'-18' – (19) #3 18'-31' – (23) #4 31'-41' – (16) #5 41'-59.5'-(8) #4	0'-25' – (11) #10 25'-40' – (13) #9 40'-59.5'-(10) #9	0'-27' – None 27'-35' – (1) #5 35'-38' – (1) #4 38'-44' – (1) #5 44'-59.5'-None	0'-15' – (26) #7 15'-22' – (13) #7 22'-27' – (9) #7 27'-35' – (8) #9 35'-38' – (5) #7 38'-44' – (10) #7 44'-59.5'-(27) #7	None

Table 20 - Continuous Footing Design Results 3 of 3

It is found that the foundation sizes from RAM are justifiably large. Some of the footing sizes are as large as 30' x 30', though this is to be expected because of the large overturning moment. Since the building has a rather large overturning moment, the footing sizes must also be large to resist the uplift.

Lateral Force Distribution

The distribution of lateral forces to the braced frames is determined based on the relative stiffness of each braced frame. Floor diaphragms are assumed to be infinitely rigid, and therefore distribute lateral loads to each frame based on their stiffness. RAM Structural System uses both of these assumptions when designing the lateral members. A hand calculation of the 8th story East – West lateral system was completed in order to confirm that the correct distribution of forces is used in RAM. The hand calculations can be found in Appendix H, while the results of the calculations and RAM can be found in Table 21 below.

Lateral Force Distribution in East – West Direction of the 8th Story				
Frame	**Percentage of Shear – Hand Calc.**	**Force in Frame – Hand Calc. (kips)**	**Percentage of Shear – RAM**	**Force in Frame – RAM (kips)**
Ma-7a – Na-7a	17.62%	24.12	31.3%	42.83
Ma-11a – Na-11a	17.62%	24.12	27.04%	36.99
La-16a – L.7a-16a	11.86%	16.23	17.94%	24.54
La-15a – Ma15a	17.62%	24.12	5.55%	7.59
La-14a – Ma-14a	17.62%	24.12	6.29%	8.61
La-13a – Ma-13a	17.62%	24.12	11.88%	16.26

Table 21 - Lateral Force Distribution in E-W Direction of 8th Story

For the most part the two values are not very similar. The exact reason for this is unknown, as both methods use only the stiffness of the braces in the calculations, assuming that the columns do not take any lateral load.

Spot Checks

Spot checks are performed for the newly designed members to confirm that they did not fail. Checks are conducted on the members of a typical lateral brace. The brace chosen is located at grid lines La-16a and L.7a-16a, and can be seen below in Figure 50. A column and brace for this frame are checked with the forces found using RAM. The hand calculations for these spot checks can be seen in Appendix I.

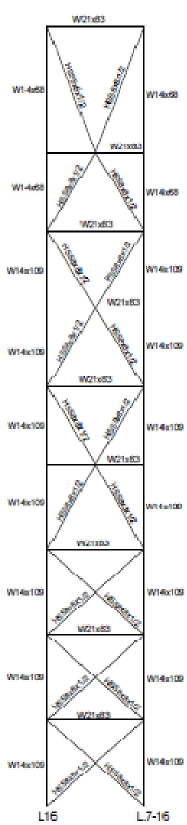


Figure 50 - Typical Frame

For the specified column a load of 753.4 kips is applied by the load combination 1.2D+1.6W+0.5L, with a length of 15'-4". From Table 4-1 in AISC, the specified W14x109 is capable of taking the load with a capacity of 1190 kips. The shape is specified even though the capacity much higher than the required load because consistency is preferred throughout the building. The next smallest size for a column throughout the building is a W14x82; whose capacity is only 698 kips, therefore a W14x109 is required.

For the specified brace a load of 210.12 kips is applied by the load combination 1.2D+1.6W+0.5L. The brace is checked by finding the required area of a brace subjected to that loading. An area of 11.86 in.^{2} is found to be needed. Since the specified shape, HSS8x8x½ has an area of 13.85 in.^{2}, the brace is found to be acceptable.

Drift

Seismic and wind drifts are computed using RAM Structural. The seismic values found in RAM are then plugged into the formula:

$$\delta = \frac{C_d \times \delta_x}{I} \times \frac{T}{T_x}$$

The story drift values found from the equations are then compared to the allowable story drift values. The allowable story drift values, or Δ_a, for the wind loads can be found by dividing the height by 400. The allowable seismic story drifts is found by multiplying the height by 0.010 per ASCE 7-05 Table 12.12-1. The new drift values are compared to the existing system's drift values in Tables 22-25 below. The calculations of the new δ can be found in Appendix J.

Controlling Wind Drift: East – West Direction												
Story	Story Height	Existing Story Drift	Story Drift	Allowable Story Drift			Existing Total Drift	Total Drift	Allowable Total Drift		
	(feet)	(inch)	(inch)	(inch)			(inch)	(inch)	(inch)		
9	140	0.011076	0.56165	≤	0.68	Acceptable	0.083184	2.32625	≤	4.2	Acceptable
8	117.33	0.015192	0.39805	≤	0.42	Acceptable	0.072108	1.7646	≤	3.52	Acceptable
7	103.33	0.01998	0.34062	≤	0.42	Acceptable	0.056916	1.36655	≤	3.1	Acceptable
6	89.33	0.011472	0.39887	≤	0.42	Acceptable	0.036936	1.02593	≤	2.68	Acceptable
5	75.33	0.005184	0.07244	≤	0.42	Acceptable	0.025464	0.62706	≤	2.26	Acceptable
4	61.33	0.001032	0.07461	≤	0.46	Acceptable	0.02028	0.55462	≤	1.84	Acceptable
3	46	0.00738	0.1708	≤	0.46	Acceptable	0.019248	0.48001	≤	1.38	Acceptable
2	30.66	0.008616	0.17387	≤	0.46	Acceptable	0.011868	0.30921	≤	0.92	Acceptable
1	15.33	0.003252	0.13534	≤	0.46	Acceptable	0.003252	0.13534	≤	0.46	Acceptable

Table 22 - Controlling Wind Drift: E-W

Controlling Wind Drift: North – South Direction											
Story	Story Height	Existing Story Drift	Story Drift	Allowable Story Drift			Existing Total Drift	Total Drift	Allowable Total Drift		
	(feet)	(inch)	(inch)	(inch)			(inch)	(inch)	(inch)		
9	140	0.025056	0.1872	≤	0.68	Acceptable	0.12828	1.31267	≤	4.2	Acceptable
8	117.33	0.046176	0.1872	≤	0.42	Acceptable	0.10322	1.12547	≤	3.52	Acceptable
7	103.33	0.026928	0.16978	≤	0.42	Acceptable	0.05705	0.93827	≤	3.1	Acceptable
6	89.33	0.017844	0.16145	≤	0.42	Acceptable	0.03012	0.76849	≤	2.68	Acceptable
5	75.33	0.000552	0.06492	≤	0.42	Acceptable	0.01228	0.60704	≤	2.26	Acceptable
4	61.33	0.0021	0.07693	≤	0.46	Acceptable	0.01172	0.54212	≤	1.84	Acceptable
3	46	0.00414	0.17606	≤	0.46	Acceptable	0.00962	0.46519	≤	1.38	Acceptable
2	30.66	0.004368	0.16246	≤	0.46	Acceptable	0.00548	0.28913	≤	0.92	Acceptable
1	15.33	0.001116	0.12667	≤	0.46	Acceptable	0.00112	0.12667	≤	0.46	Acceptable

Table 23 - Controlling Wind Drift: N-S

Controlling Seismic Drift: East – West Direction											
Story	Story Height	Existing Story Drift	Story Drift	Allowable Story Drift			Existing Total Drift	Total Drift	Allowable Total Drift		
	(feet)	*(inch)*	*(inch)*	*(inch)*			*(inch)*	*(inch)*	*(inch)*		
9	140	0.05152	0.47261	≤	2.72	Acceptable	0.3478	4.1615	≤	16.8	Acceptable
8	117.33	0.07601	0.47261	≤	1.68	Acceptable	0.2962	3.6889	≤	14.08	Acceptable
7	103.33	0.04169	0.61198	≤	1.68	Acceptable	0.2202	3.2163	≤	12.4	Acceptable
6	89.33	0.04161	0.86848	≤	1.68	Acceptable	0.1785	2.6043	≤	10.72	Acceptable
5	75.33	0.02971	0.18042	≤	1.68	Acceptable	0.1369	1.7358	≤	9.04	Acceptable
4	61.33	0.01473	0.18178	≤	1.84	Acceptable	0.1072	1.5554	≤	7.36	Acceptable
3	46	0.04161	0.50596	≤	1.84	Acceptable	0.0925	1.3736	≤	5.52	Acceptable
2	30.66	0.41852	0.50251	≤	1.84	Acceptable	0.0508	0.8677	≤	3.68	Acceptable
1	15.33	0.00903	0.36519	≤	1.84	Acceptable	0.00903	0.3651	≤	1.84	Acceptable

Table 24 - Controlling Seismic Drift: E-W

Controlling Seismic Drift: North – South Direction											
Story	Story Height	Existing Story Drift	Story Drift	Allowable Story Drift			Existing Total Drift	Total Drift	Allowable Total Drift		
	(feet)	*(inch)*	*(inch)*	*(inch)*			*(inch)*	*(inch)*	*(inch)*		
9	140	0.05722	0.3136	≤	2.72	Acceptable	0.25449	2.9279	≤	16.8	Acceptable
8	117.33	0.07446	0.3136	≤	1.68	Acceptable	0.19727	2.6143	≤	14.08	Acceptable
7	103.33	0.04169	0.3229	≤	1.68	Acceptable	0.12281	2.3006	≤	12.4	Acceptable
6	89.33	0.02377	0.2385	≤	1.68	Acceptable	0.08111	1.9776	≤	10.72	Acceptable
5	75.33	0.00370	0.1448	≤	1.68	Acceptable	0.05734	1.7391	≤	9.04	Acceptable
4	61.33	0.00979	0.2175	≤	1.84	Acceptable	0.05369	1.5942	≤	7.36	Acceptable
3	46	0.02014	0.5436	≤	1.84	Acceptable	0.04384	1.3767	≤	5.52	Acceptable
2	30.66	0.01923	0.4837	≤	1.84	Acceptable	0.02369	0.8330	≤	3.68	Acceptable
1	15.33	0.00445	0.3492	≤	1.84	Acceptable	0.00445	0.3492	≤	1.84	Acceptable

Table 25 - Controlling Seismic Drift N-S

As displayed in the tables above, all of the new drift values are acceptable per serviceability requirements and code requirements. All the new drift values are much larger than the original design. This shows that the new design that eliminates moment frames is less stiff than the original moment frame building and will deflect more. Even though the new drift values are larger than the original, the new design is acceptable because they are still below the code requirements.

Center of Mass and Rigidity

The center of mass, or COM, and the center of rigidity, or COR, for the newly designed building are found using RAM Frame. The values in Table 26 below show the East – West and North – South components of the center of rigidity and the center of mass for each level. In

Table 27 below that, the values for the existing building are seen; these values are found in the Third Technical Report of the Voorhees Replacement Facility. The origin for both the newly calculated values and the existing values is the southwest corner of the grid line.

Newly Calculated RAM Values for Center of Mass and Center of Rigidity				
Level	E-W Center of Mass	N-S Center of Mass	E-W Center of Rigidity	N-S Center of Rigidity
	(feet)	*(feet)*	*(feet)*	*(feet)*
9	277.15	156.03	274.98	129.05
8	277.10	210.90	275.49	142.35
7	277.21	151.06	275.57	154.97
6	277.16	150.04	274.92	170.68
5	259.22	191.89	274.19	183.98
4	253.08	193.30	275.89	182.18
3	248.81	175.08	276.93	177.34
2	248.58	174.60	274.01	176.13
1	216.95	180.33	264.71	169.02

Table 26 - New Center of Mass and Center of Rigidity

Existing ETABS Values for Center of Mass and Center of Rigidity				
Level	E-W Center of Mass	N-S Center of Mass	E-W Center of Rigidity	N-S Center of Rigidity
	(feet)	*(feet)*	*(feet)*	*(feet)*
9	306.4	158.6	309.5	148.2
8	307.1	81.9	306.5	86.2
7	307.0	154.0	306.5	174.6
6	318.1	232.1	317.9	236.7
5	287.6	114.4	306.4	174.4
4	278.0	113.4	303.9	164.7
3	278.5	130.8	302.2	152.0
2	285.7	129.3	300.6	154.3
1	245.5	126.3	275.5	125.1

Table 27 - Existing Center of Mass and Center of Rigidity

The difference between the new center of mass and the new center of rigidity appear to be similar in some areas, while others vary by a large amount. For the East – West direction, the values in the first 5 floors are very different, differing by 30 of more feet. For the floors above the 5th in the East – West direction, the values are fairly similar, only differing by a couple of feet. For the North – South direction, the values are fairly similar only differing by 10 or so feet each floor except the 8th. The 8th floor varies a large amount because the mechanical room spans from the 7th floor to the roof level, skipping the 8th in sections, thus decreasing that level's total mass and changing the location of the center of mass.

The difference in the center of mass and the center of rigidity of the existing system appear to be minimal in East – West direction. The difference in the North – South direction however, appears to be more significant than the East – West direction, however the values are still small.

When comparing the new system's COM and COR to the existing system's COM And COR, the values vary by a large amount. The first difference is that both systems center of mass varies from one another. This is expected to change since the new system is now supporting a helipad located on the top of the building. Another possible reason for the change is that the two buildings are modeled in different programs, starting from scratch each time it was modeled. Since the RAM model is created with more detail than the ETABS model, which did not include any gravity members, the RAM model is assumed to be more correct.

The two centers of rigidity also varied in each system. This is also expected to change since the existing lateral system included moment frames on the outer edge of the building. By changing the system to a lateral frame, frames are not placed in the exact locations of the old moment frames, thus a change in the center of rigidity is expected. The differences in the COM and COR for each system however are fairly similar between the new and old lateral system. For the East – West direction, both systems are similar, with minimal differences in the COM and COR for level 6 and above, while level 5 and below differ by large amounts. For the North – South direction however there are differences between the two systems. While the new system's COM and COR only vary by 10 or so feet, the old system's COM and COR vary by as much as 50 or more feet for story 5 and below. This is ideal for the system since the lower differences create less torsion in the building, reducing the forces acting on the lateral system.

Torsion

Since the center of mass and the center of Rigidity differ by a somewhat large amount, it is to be expected that the building's torsion will be relatively big. The total building torsion consists of two parts, M_t, which is the inherent torsional moment. This is found by multiplying the distance between the center of mass and the center of rigidity by the story shear. The inherent torsional moment is then added to the accidental torsional moment, M_{ta}. M_{ta} is found by multiplying the story shear by 5% of the building width at a specific level. The inherent torsional moment is added to the accidental torsional moment to find the total torsional moment, M. The torsion values for the new and existing systems can be seen below in Table 28 and 29.

New Building Torsion						
Story	**North – South Building Torsion**			**East – West Building Torsion**		
	M_t	**M_{ta}**	**M_{total}**	**M_t**	**M_{ta}**	**M_{total}**
	(kip-ft)	*(kip-ft)*	*(kip-ft)*	*(kip-ft)*	*(kip-ft)*	*(kip-ft)*
9	427.75	443.5	871.3	5399.5	3055.3	8454.8
8	515.76	720.8	1236.6	22358.9	4979.5	27338.4
7	400.57	549.6	950.17	977.9	3818.5	4796.4
6	538.9	541.3	1080.2	5033.9	3723.3	8757.2
5	3477.1	3169.5	6646.6	1841.1	3553.4	5394.5
4	5536.7	3305.2	8841.9	2752.8	3779.2	6532
3	5026.7	3522.8	8549.5	581.9	3931.1	4513
2	5850.7	3496.6	9347.3	371.2	3703.7	4074.8
1	10609.9	4322.7	14932.6	2624.9	3543.2	6168.1
Sum			**52456.1**			**76028.3**

Table 28 - New Building Torsion

Existing Building Torsion						
Story	**North – South Building Torsion**			**East – West Building Torsion**		
	M_t	**M_{ta}**	**M_{total}**	**M_t**	**M_{ta}**	**M_{total}**
	(kip-ft)	*(kip-ft)*	*(kip-ft)*	*(kip-ft)*	*(kip-ft)*	*(kip-ft)*
9	781.59	567.2813	1348.869	1295.8	2552.244	3848.084
8	62.849	235.683	298.5318	869.03	4139.715	5008.745
7	74.219	333.9855	408.2045	3170.3	3152.411	6322.751
6	9.9974	112.4708	122.4682	696.44	3101.202	3797.642
5	5014.5	3636.316	8650.822	8904.0	3039.751	11943.75
4	7656.7	4030.242	11686.9	7792.5	3111.444	10903.91
3	5562.8	3199.883	8762.652	3277.5	3166.749	6444.269
2	1735.7	1702.676	3438.377	3722.5	3049.993	6772.493
1	2123.3	1080.532	3203.872	169.2	2888.174	3057.374
Sum			**37920.7**			**58099.02**

Table 29 - Existing Building Torsion

It is seen in the above tables that the torsion in the new design is more than the original design. This is to be expected due to the bigger differences in the COM and COR in the new design. This is a potential problem when designing the new system because the large torsional moments can add shear forces to the frames increasing their load.

Overturning Moments

An analysis is performed to determine the overturning moments cause by the controlling forces. The new overturning moments are then compared to the existing overturning moments that are found in Technical Report #3. The results can be seen in Table 30 and 31 below.

Level	Height Above Ground (ft)	New Overturning Moments					
		Load (kip)		Shear (kip)		Overturning Moment (ft-kip)	
		N-S	E-W	N-S	E-W	N-S	E-W
9	140.00	197.12	200.13	197.12	200.13	27596.8	28018.2
8	117.33	320.35	326.12	517.47	523.30	37586.7	38263.7
7	103.33	244.25	250.12	761.72	776.42	25238.4	25844.9
6	89.33	240.58	243.89	1002.31	1020.44	21491.0	21786.7
5	75.33	232.27	232.76	1235.08	1253.20	17496.9	17533.8
4	61.33	242.73	247.55	1477.81	1500.74	14886.6	15182.2
3	46.00	258.71	257.50	1736.52	1758.25	11900.7	11845.0
2	30.66	239.49	242.60	1976.00	2000.85	7342.8	7438.1
1	15.33	222.15	232.09	2198.16	2232.94	3405.6	3557.9
Total		**2198.16**	**2232.94**	**2198.16**	**2232.94**	**166945.5**	**169470.5**

Table 30 - New Overturning Moments

Level	Height Above Ground (ft)	Existing Overturning Moments					
		Load (kip)		Shear (kip)		Overturning Moment (ft-kip)	
		N-S	E-W	N-S	E-W	N-S	E-W
9	140.00	252.125	124.6	252.125	124.6	35128.580	17360.5
8	117.33	104.748	202.1	356.873	326.7	12290.089	23712.4
7	103.33	148.438	153.9	505.311	480.6	15338.101	15902.5
6	89.33	49.987	151.4	555.298	632.0	4465.294	13524.6
5	75.33	266.729	148.4	822.027	780.4	20092.733	11179.0
4	61.33	295.624	151.9	1117.651	932.3	18130.609	9316.0
3	46.00	234.716	154.6	1352.367	1086.9	10796.942	7111.6
2	30.66	116.490	148.9	1468.857	1235.8	3571.585	4565.3
1	15.33	70.778	141.0	1539.635	1376.8	1085.020	2161.5
Total		**1539.635**	**1376.8**	**1539.635**	**1376.8**	**120898**	**104833**

Table 31 - Existing Overturning Moments

The results for the new overturning moments are found to be larger than the previous overturning moments by a significant amount. This is not to be expected because the new seismic forces are smaller than existing forces. A possible reason for this is the inclusion of wind cases in this report. By adding wind cases, larger forces are added to the building creating more building shear, and thus creating more overturning moment. These new large overturning moment values will have to be taken into account when designing the new foundations in the Lateral Foundation Design section of this report.

Comparison of Existing Lateral System vs. New System

The new design, using only braced frames for the lateral system, and the existing design, using a mix of moment frames and braced frames, are compared in this section purely to understand the feasibility of eliminating moment frames. The two systems are compared on many different factors including seismic loads, member sizes, foundation changes, architectural changes, drift, torsion, overturning moment, constructability, and cost.

Because the new system is able to use just braced frames instead of a combination of the two, the seismic base shear is lower for the new design. The new R value that is used for the seismic calculations is able to reduce the seismic load in both directions, North – South and East – West. In the North – South direction it is able to reduce the loads by almost 200 kips, while in the East – West it is able to reduce it an almost negligible 5 kips. This helps in reducing the forces in the building, allowing the wind to control.

The new design also helps in decreasing member sizes of the previous design. Because the existing design uses moment frames, the beam members have to be increased to take added moment forces, with members reaching typical sizes of W24x176 and W36x150. With the new design, since there is no moment in the beams or girders the members can be reduced to a W21x63 or smaller. Also, column sizes can be reduced since they do not take moment loads either. Again, due to the moments that are created in the moment frame, columns can be as large as a W14x311. With the new design column sizes can be greatly reduced with maximum sizes reaching a W14x176. The down side to this new design is the addition of the HSS braces. Because the new lateral system uses braces to resist lateral load, extra HSS members must be used, thus possibly offsetting the savings created by smaller members.

Foundation changes will also have to be made for the new design. In general the foundations for the new design have increased due to the added overturning moment. Because there are fewer lateral foundations throughout the building, there is more overturning moment applied to each of them. This will force the members to increase in size, in some cases as much as 10’ in each direction.

The added braces also added architectural challenges for the new design. Because a brace spans the between two columns, any hallways or rooms that are in that area will be cut off and need to be redesigned so the building’s floor plan still works. For all cases of new frames throughout this building, interfering with a hallway or room was avoided. This is not possible however for the windows looking into the courtyard. In order to resist the torsional moment, braces are needed in the area surrounding the two courtyards. ‘X’ type braces are inserted into those areas to assist with lateral movement and to add to the architecture. By making these braces exposed it continues to allow users to see out into the courtyard, but also allows them to see the structural system. This change to the architecture will need to be studied further to ensure that it will look architecturally pleasing.

Drift is also a factor that is considered when comparing the two systems. The new system is found to have much higher drift values for both the wind and seismic. The new drift values are found to be almost 10 times more than the exiting values. Although the drift values are higher for the new system, they are still all acceptable per serviceability and code requirements.

The torsional and overturning moments are also considered in the comparison of the two systems. For the new building because the COM and COR have a much larger difference, it is expected that the torsion values will be larger than the existing values. This found to be true and adds to the forces that the lateral system must resist. The overturning forces are also larger than the previously calculated values for the existing system. Since the forces are larger in the new system it will affect the sizes of the new foundations because they will have to resist this overturning moment. This will force the foundations to be larger in size than the originally designed foundations.

The final considerations when comparing the two systems are the constructability and cost. Moment frames can require much more welding than the newly designed connections in the braced frames. Since this is the case, time and money can be saved by installing the brace frames. Installing the new system will also not require the use of stiffener plates on any of the connections to stiffen the columns or beams, again saving time and money.

Connection Design

Typical frame connections are designed for this report to fulfill the MAE requirement. In this section four connections are designed. All four connections deal with the newly designed lateral system. The first two connections deal with a brace to column/beam connection. The third connection deals with a brace to beam connection, while the fourth deals with a brace/brace connection. Each of the connections is designed to limit the amount of welding wherever possible. The locations of each connection can be seen in Figure 51, while the calculations for each connection can be seen in Appendix K. Below in Figures 52-55 each connection is detailed.

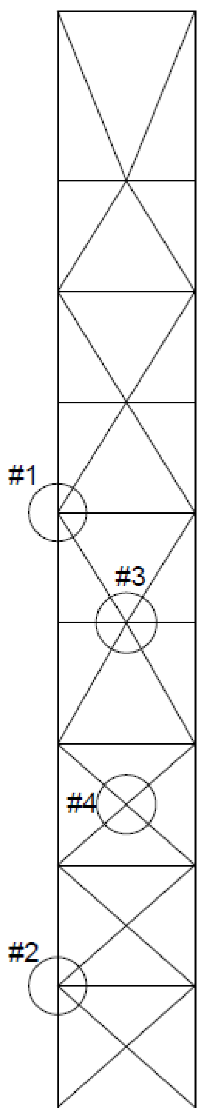


Figure 51 - Location of Connection Designs

Typical Connection #1

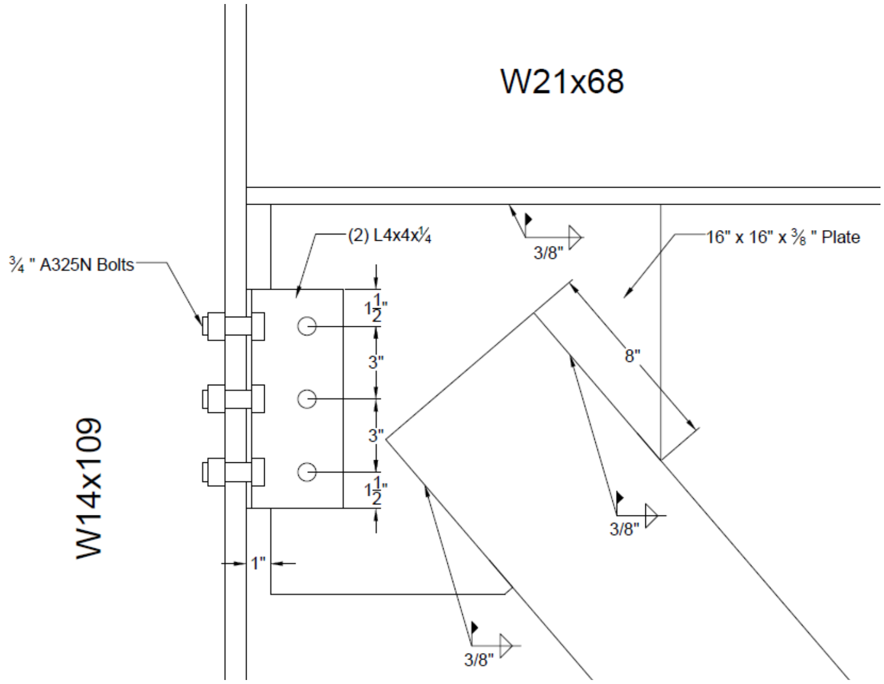


Figure 52 - Brace to Column Connection

Typical Connection #2

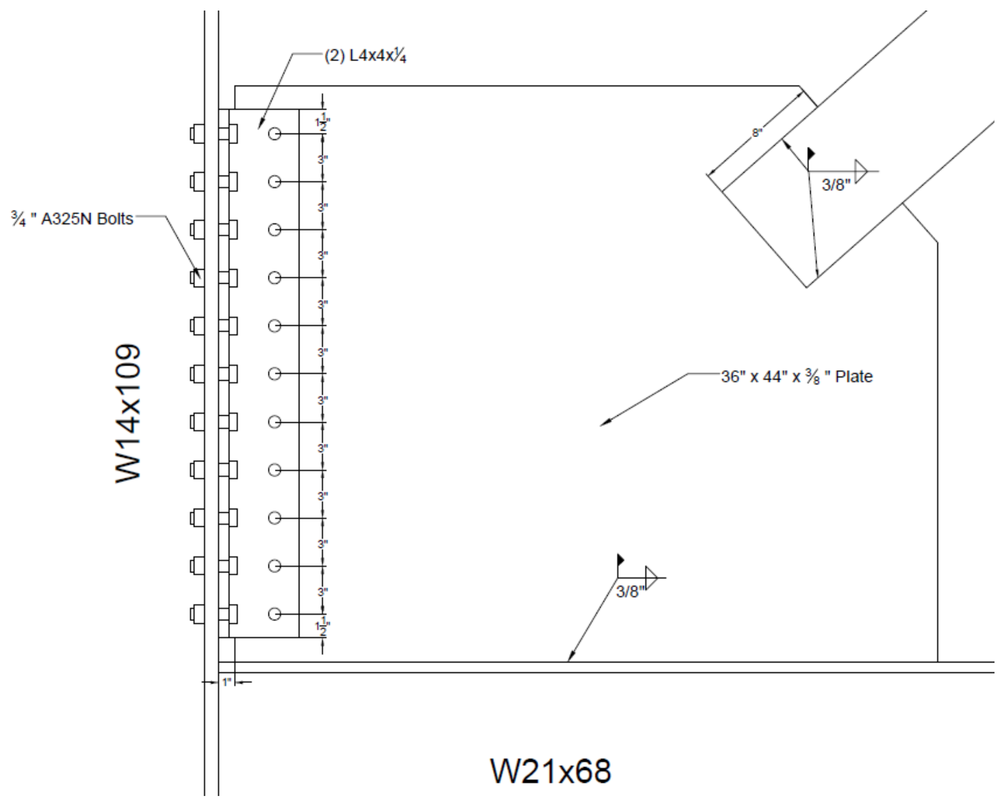


Figure 53 - Brace to Column Connection

Typical Connection #3

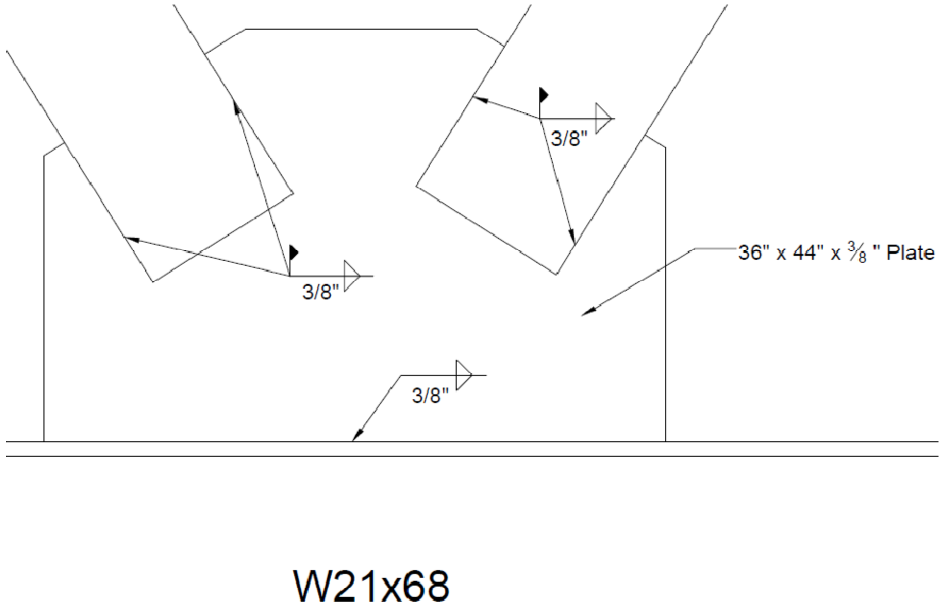


Figure 54 - Brace to Beam Connection

Typical Connection #4

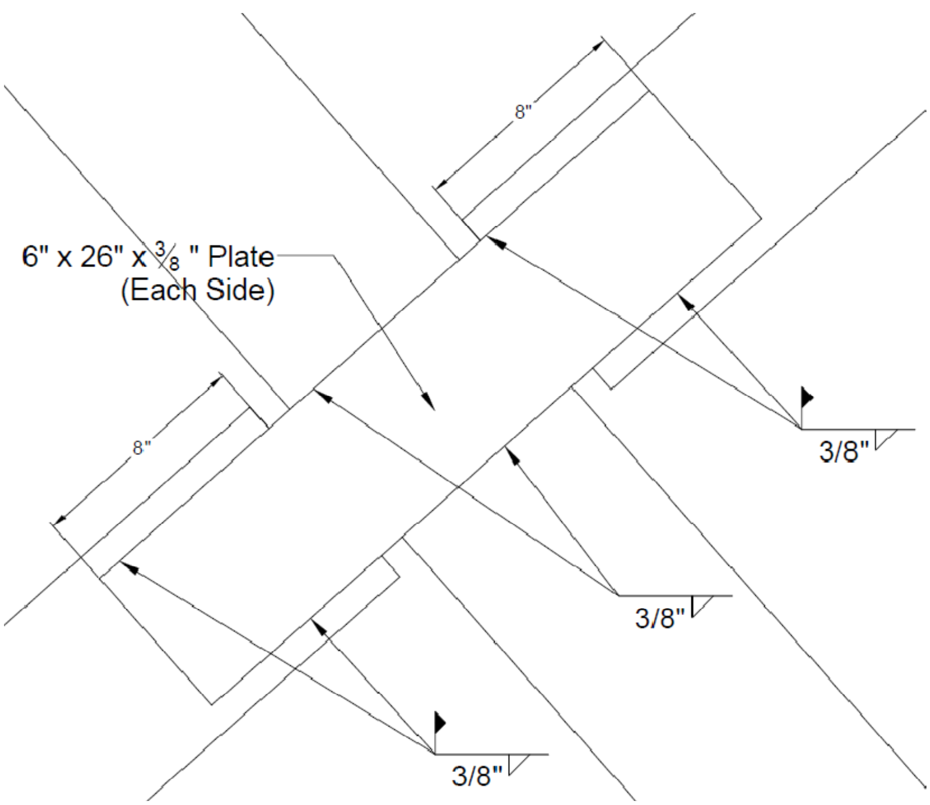


Figure 55 - Brace to Brace Connection

Electrical Analysis

Due to the relocation of the helipad, the systems supporting the pad must also be moved. According to the FAA lights are needed to help guide any approaching helicopters to the helipad. With the relocation of the helipad, the lights must also be relocated and tied into a circuit breaker. The luminaires used for the helipad lighting include the following:

- ZG1 – (14) – Helipad Omnidirectional Marker Lights – 40W – 120V
- ZG2 – (1) – Lighted Windsock and Rigid Frame – 464W – 120V
- ZG3 – (4) – Helipad Red Obstruction Light – 116W – 120V

The luminaires ZG1 are designed to be recessed in the concrete pad as in Figure 56. Each of the luminaires, ZG1, ZG2, and ZG3, are all placed on separate circuits to avoid voltage drop. All three circuits use a 60^{o}, 14 AWG copper wire in ½" metal conduit. The wire types are found using two assumptions. The first assumption is that the lights are continuous loads and require a reduction factor. Also, it is assumed that the temperatures that the wires will be subjected to will reach 106^{o} – 113^{o}, because the lights are located on the roof of the structure. The three wires also checked to ensure that the Voltage Drop is below 3%. For the circuit containing the luminaires ZG1, the Voltage Drop, or VD is found to be 2.49%, while the VD in the other two wires is 2.07%. The calculations for these wires can be found in Appendix L.

Figure 56 - Recessed Light in Helipad

The three circuits are all designed to tie into the Panel LEQPH7A1 using 20A breakers. The Panel LEQPH7A1 is located in the 8th floor mechanical space. A breakdown of the panel can be seen below in Figure 57. The highlighted portions of the panel are the new circuits added.

PANEL LEQPH7A1

SURFACE MOUNT: X	100 AMPS BUS	100A MAIN CIRCUIT BREAKER	NOTES:	
FLUSH MOUNT	208 VOLTS L-L	SUB FEED LUGS	4,000	AIC RATING
	120 VOLTS L-N	MAIN LUGS ONLY		DOUBLE PANEL
	3 PHASE	DOUBLE LUGS		DOOR-IN-DOOR CONSTRUCTION
	4 WIRE			INTEGRAL TVSS
				ISOLATED GROUND BUS

LOAD DESCRIPTION	ROOM OR AREA	VA PER PHASE A	VA PER PHASE B	VA PER PHASE C	CODE	BREAKER AMP	BREAKER POLE	CCT No	PHASE A	PHASE B	PHASE C	CCT No	BREAKER AMP	BREAKER POLE	CODE	VA PER PHASE A	VA PER PHASE B	VA PER PHASE C	LOAD DESCRIPTION	ROOM OR AREA
RECEPTACLES		360			3	20	1	1	X			2	20	1	2	696			LIGHTING	
RECEPTACLES			360		3	20	1	3		X		4	20	1	2		696		LIGHTING	
RECEPTACLES				360	3	20	1	5			X	6	20	1	2			800	LIGHTING	
AHU-1A LIGHTING		1,000			2	20	1	7	X			8	20	1	2	1,000			AHU-2B LIGHTING	
AHU-1B LIGHTING			1,000		2	20	1	9		X		10	20	1	2		1,000		AHU-3A LIGHTING	
AHU-2A LIGHTING				1,000	2	20	1	11			X	12	20	1	2			1,000	AHU-3B LIGHTING	
AHU-2C LIGHTING		1,000			2	20	1	13	X			14	20	1	2	1,000			AHU-3E LIGHTING	
AHU-3D LIGHTING			1,000		2	20	1	15		X		16	20	1	2		1,000		AHU-3F LIGHTING	
RECEPTACLES				360	3	20	1	17			X	18	20	1	3			360	RECEPTACLES	
HEAT TRACE		1,500			5	20	1	19	X			20	20	1	3	360			RECEPTACLES	
HEAT TRACE			1,500		5	20	1	21		X		22	20	1	3		540		RECEPTACLES	
HEAT TRACE				1,500	5	20	1	23			X	24	20	1	7			670	M221	CB-2
HEAT TRACE		1,500			5	20	1	25	X			26	20	1	6	670			M218	CB-10
NEW LIGHTS - ZG1	HELIPAD		560		N/A	20	1	27		X		28	20	1			464		NEW LIGHTS - ZG3	HELIPAD
NEW LIGHTS - ZG2	HELIPAD			464	N/A	20	1	29			X	30	20	1					SPARE	
SPARE						20	1	31	X			32	20	1					SPARE	
SPARE						20	1	33		X		34	20	1					SPARE	
SPARE						20	1	35			X	36	20	1					SPARE	
SPARE						20	1	37	X			38	20	1					SPARE	
SPARE						20	1	39		X		40	20	1					SPARE	
SPARE						20	1	41			X	42	20	1					SPARE	
SUBTOTAL CONNECTED VA/PHASE		5,360	4,420	3,684												3,726	3,700	2,830	SUBTOTAL CONNECTED VA/PHASE	

	A	B	C	VA	A	CODE SUMMARY	TUB VA	PANEL VA	A	B	C	VA	A	
CONNECTED AMPS/PHASE (THIS TUB)	76	64	55			1 NONCONTIN.	-	-	76	64	55			CONNECTED AMPS/PHASE (ENTIRE PANEL)
TOTAL CONNECTED (THIS TUB)				23,720	65	2 LIGHTS	16,380	16,380				23,720	65	TOTAL CONNECTED (ENTIRE PANEL)
						3 RECEPTACLES	2,700	2,700						
CODE CONNECTED AMPS/PHASE (THIS TUB)	92	72	61			4 CONTINUOUS	-	-	92	72	61			CODE CONNECTED AMPS/PHASE (ENTIRE PANEL)
TOTAL CODE CONNECTED (THIS TUB)				26,948	75	5 HEATING	7,500	7,500				26,948	75	TOTAL CODE CONNECTED (ENTIRE PANEL)
						6 MOTORS	670	670						
						7 LARGEST MOTOR	838	838						

Figure 57 - Updated Panel

Acoustical Study

Since relocating the helicopter to the roof of the building will move it closer to the building, an acoustical check is required to ensure that the residents of the building will not be bothered by the increased noise. The façade that is closest to the new helipad site is checked and compared to similar, but alternative, façades. The closest area to the new helipad is a patient room located in Zone 3 of the Northern Building. The façades checked for this area are ¼" glass, which is the existing façade, ¼" laminated glass and 3/16" glass with a 2" airspace, ¼" laminated glass and 3/16" glass with a 4" airspace, and (2) ¼" laminated glass with ½" airspace. Each façade has a Transmission Loss of 31, 48, 44, and 40 respectively.

Since it is assumed that the helicopter will be located on the helipad when it is the loudest, a distance of 142 feet is used in the calculations. It is also assumed that the noise a helicopter of the size used for this report can make is 105 dB. After performing the noise reduction calculations, found in Appendix M, the noise levels are recorded and are shown in Table 32.

dB's recorded in Patient's Room due to Helicopter		
Type of façade	**TL**	**dB Recorded in Room**
¼" Glass	31	47.3
¼" Laminated Glass and 3/16" Glass with a 2" Airspace	48	34.3
¼" Laminated Glass and 3/16" Glass with a 4" Airspace	44	30.3
(2) ¼" Laminated Glass with ½" Airspace	40	38.3

Table 32 - dB's in Patient's Room

It is found that the existing noise in the patient's room is too high. 47.3 dB is roughly the same amount of noise as the average home or office. Since the rooms are meant for overnight patients, this would be too loud and would result in complaints from the users. It is determined that the ¼" laminated glass and 3/16" glass with a 2" airspace would be ideal for this room. This façade reduces the noise in the room to 34.3 dB. This is roughly the equivalent of a quiet office or conversation which would be ideal for sleeping patients.

Conclusions

The Voorhees Replacement Hospital is a new hospital replacing the current Voorhees hospital due to its inability to expand and be renovated. The new building is 9 stories tall, approximately 140 feet tall. It consists of two parts, a main bed tower, and a services building.

In the final report the current building's systems is be explained in depth. An in depth description of the mechanical, electrical, lighting, telecommunication, and structural systems is also given. The architecture of the building is explained as well as some of the building's sustainable features.

Descriptions of the proposed changes to the building are then given. An overview look at the potential benefits of the changes is given, and the process of making the changes is explained.

The current helipad is looked at and the possibility of moving it from the parking area to the top of the building is explored. This report looks into possible problems and requirements that a helipad located on the roof will require. A location is then determined based on the requirements found. In order to move it from its current location, the helipad must first be redesigned as a two-way slab with beams spanning from column to column. The beams supporting the slab are then designed as concrete beams. Due to the added weight on the roof, the gravity columns that support the new helipad are adjusted for the added weight. Also, since weight is added to the columns, the foundations are redesigned for the added load. These new foundations are then checked against a RAM model to insure they are accurate.

In order to eliminate moment frames throughout the building, the lateral system is also changed. The lateral system is changed in the Southern Building from a combination of braced frames and moment frames, to a system that only uses braced frames. Since changing the lateral system will also change the seismic forces, these forces are redesigned. A RAM model is then created to assist with the design of the new system. This model assists in the design and analysis of the new building.

Because new braces are added to the building, a study of their new location is performed to insure that the architecture is unchanged. The location of these new braces is then plugged into the RAM model and a design is performed. The location and sizes of the new braces is then reported. Since the forces on the foundations will also change, the foundations are redesigned. The new foundations are redesigned and the sizes are reported.

The new system is also analyzed for a number of different factors. The analysis looks at the new building's drift, center of mass and center of rigidity, torsion, overturning moment, and constructability. These factors are then compared to the old system's values. It is found that the new systems' seismic forces are reduced due to the new R value. It is also found that by eliminating moment frames, time and money can be saved with the new system, although the new system will have a larger drift, torsion, and overturning moment when compared to the old system.

Due to the new type of lateral system a design of the connections for the new braces is performed. Two brace to column connections, a brace to beam connection, and a brace to brace connection are designed and reported in this report.

Because of the new location of the helipad, an electrical breadth and an acoustical breadth is performed. The electrical study determines new wire sizes for lights located on the new helipad. These wires are also tied into an existing circuit breaker. The acoustical study looks at the potential sound entering patients' rooms. The study found that a new façade type is necessary in order to eliminate sound entering the space.

Bibliography

Bill Davis. Personal Interview. 1/11/10.

Das,, Braja M.. Principles of Foundation Engineering. Stamford, CT: Cengage Learning, 2007.

"Decibel Levels." Cool math Lessons - Decibel Levels. Online. 2010 <http://www.coolmath .com/decibels1.htm> 5 April 2010.

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

Federal Aviation Administration. "Advisory Circular." 105_5390_2b. Online. 9/30/04 <http://www.faa.gov/airports/resources/advisory_cirulars/media/105-5390-2b/150_5390_2b.pdf> 5 April 2010.

Geschwindner, Louis F.. Unified Design of Steel Structures. Hoboken, NJ: John Wiley & Sons Inc., 2008.

"Heliport Systems, Inc.." Heliport Systems, Inc. - Design, Manufacture, Construction. Online. 2000 <http://www.heliport.com/index.htm> 5 April 2010.

Jeff Harris. Personal Interview. 9/18/09.

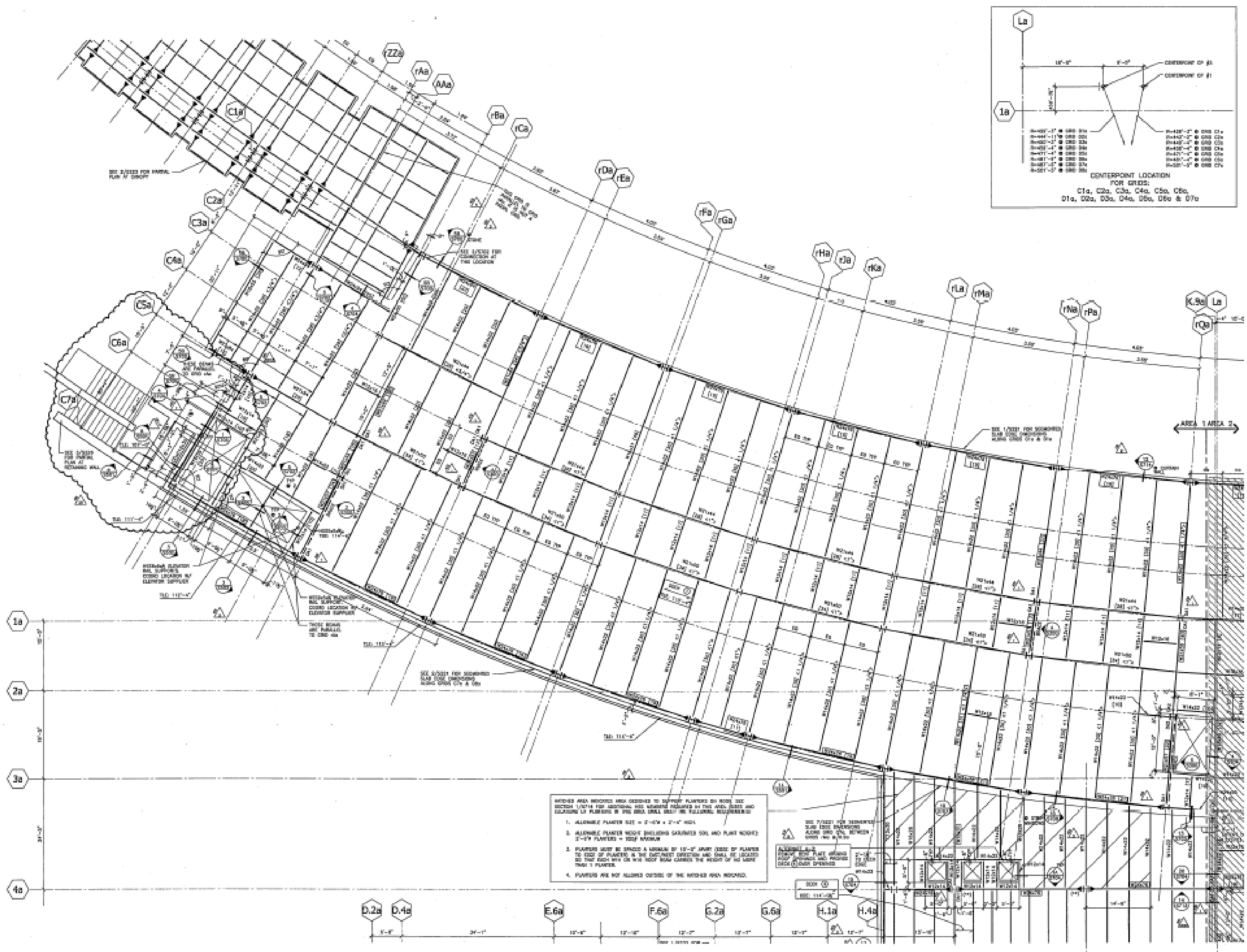
Mark Debrauske. Personal Interview. 9/9/09.

Mike Torine. Personal Interview. 9/14/09.

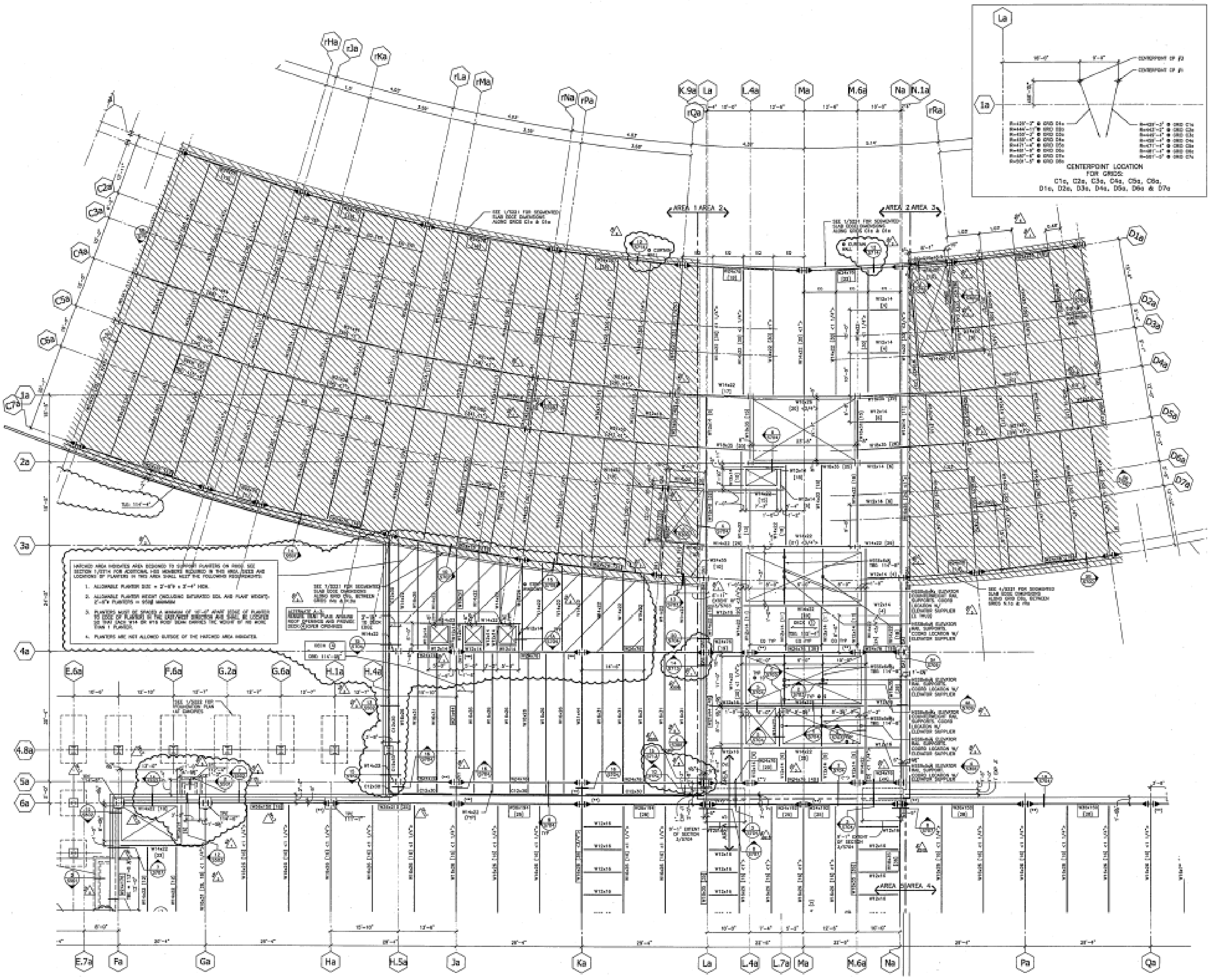
Paul Gruettner. Personal Interview. 9/19/09.

"Stop the Helipad." Noise and Low Flying Helicopters - Stop the Helipad at the SF General Hosptial. Online. April 05, 2010 <http://stophelipad.com/noise.shtml> 5 April 2010.

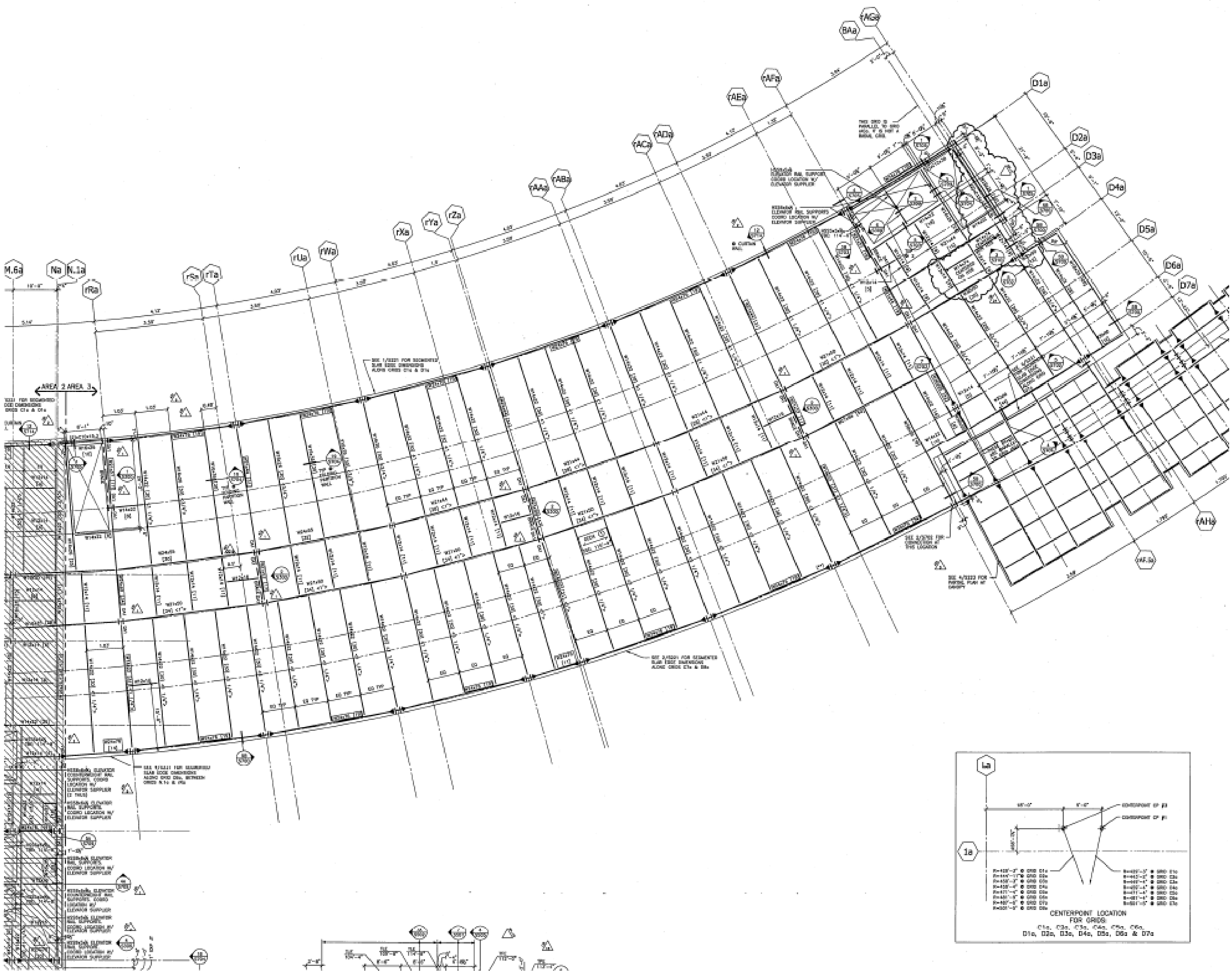
Appendix A: Typical Floor Plans



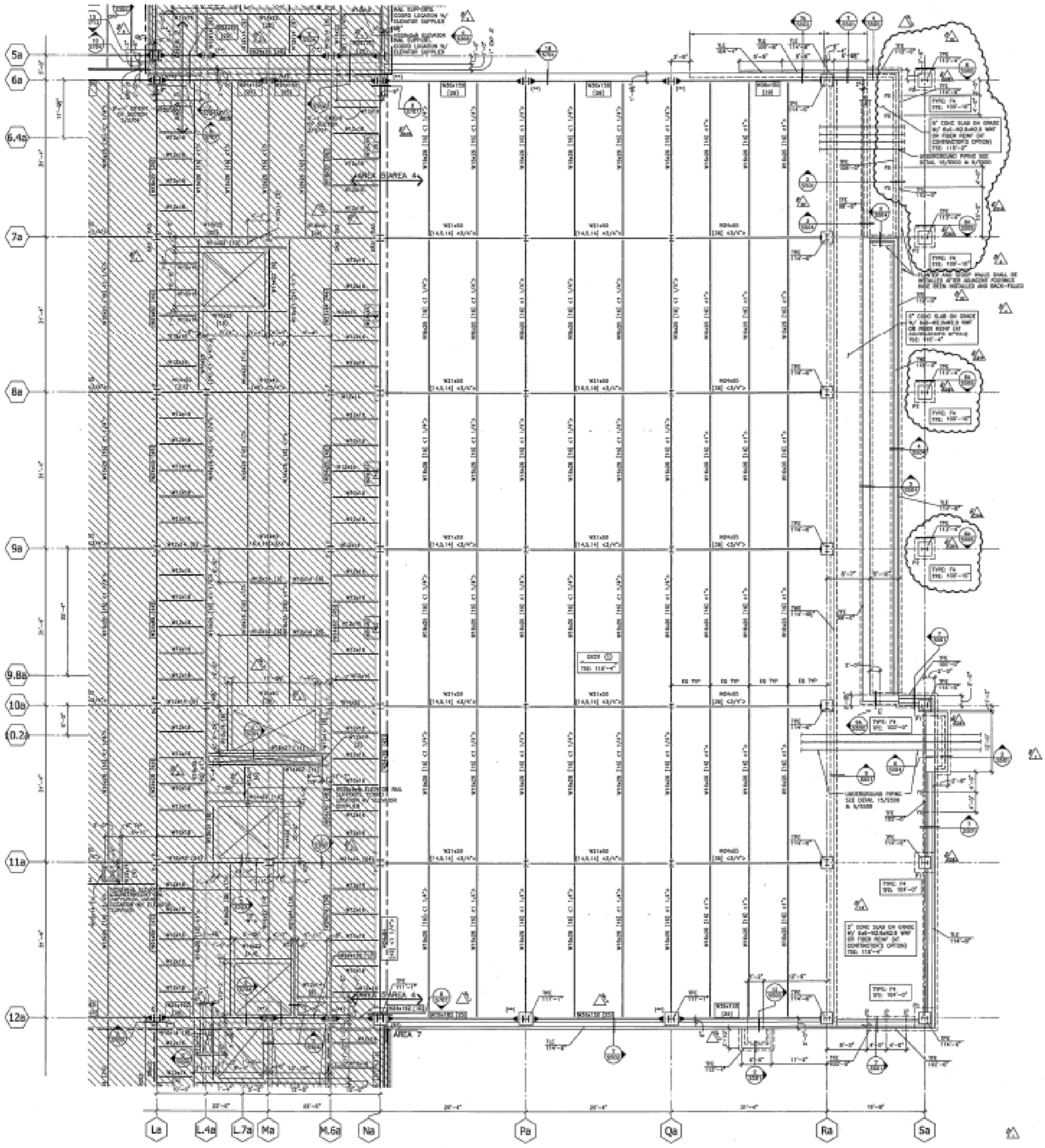
Level 1, Zone 1 Framing Plan



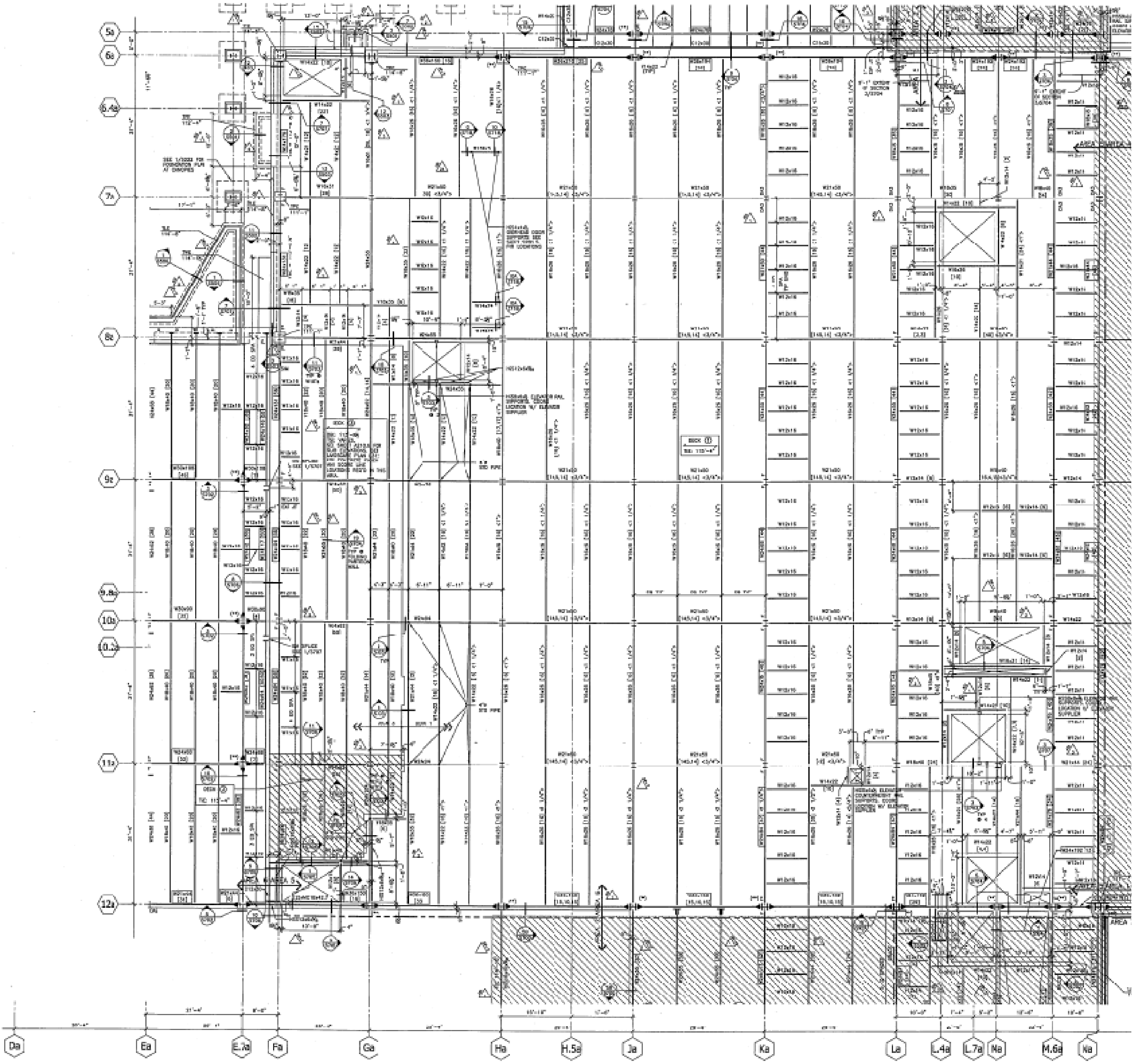
Level 1, Zone 2 Framing Plan



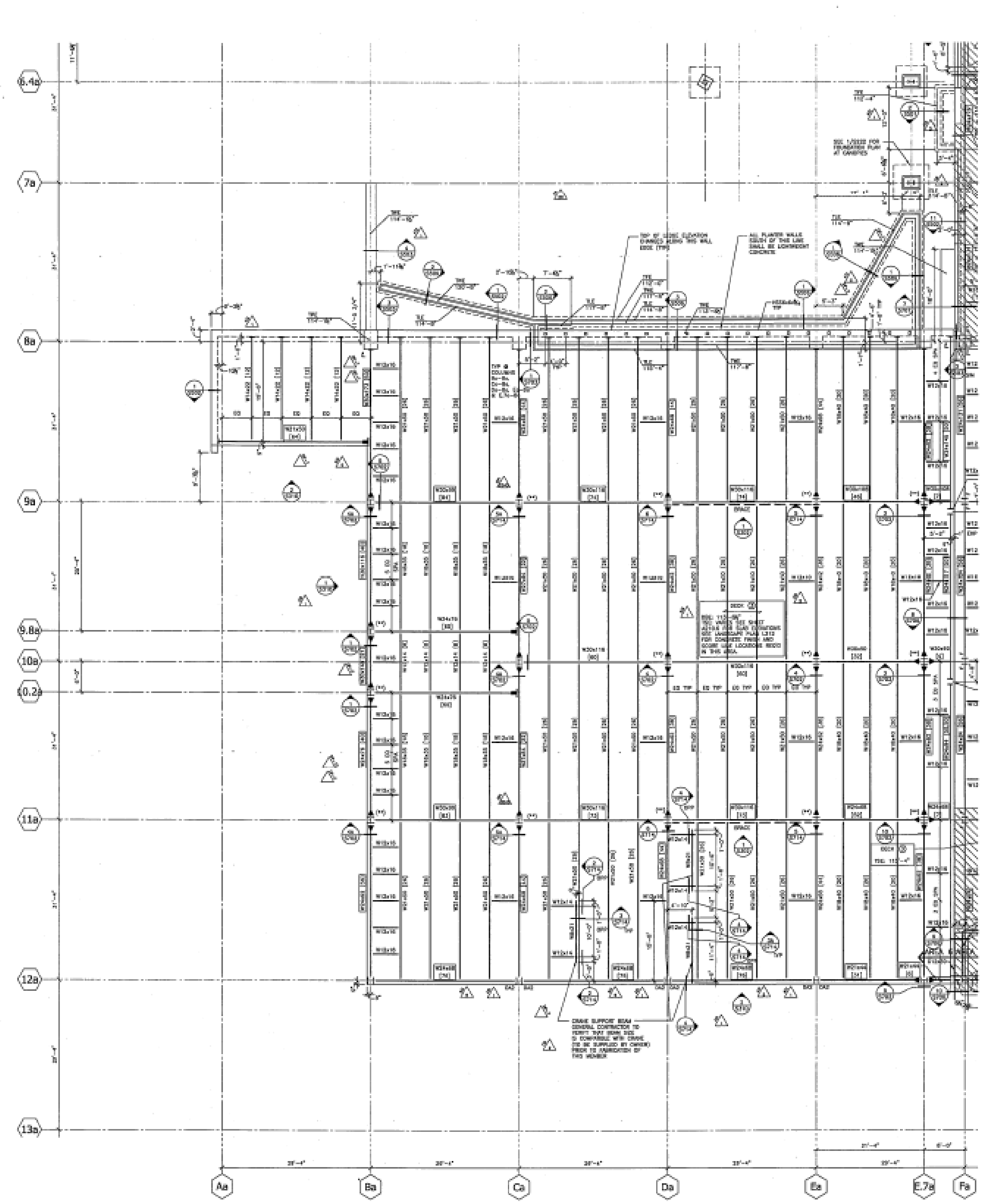
Level 1, Zone 3 Framing Plan



Level 1, Zone 4 Framing Plan



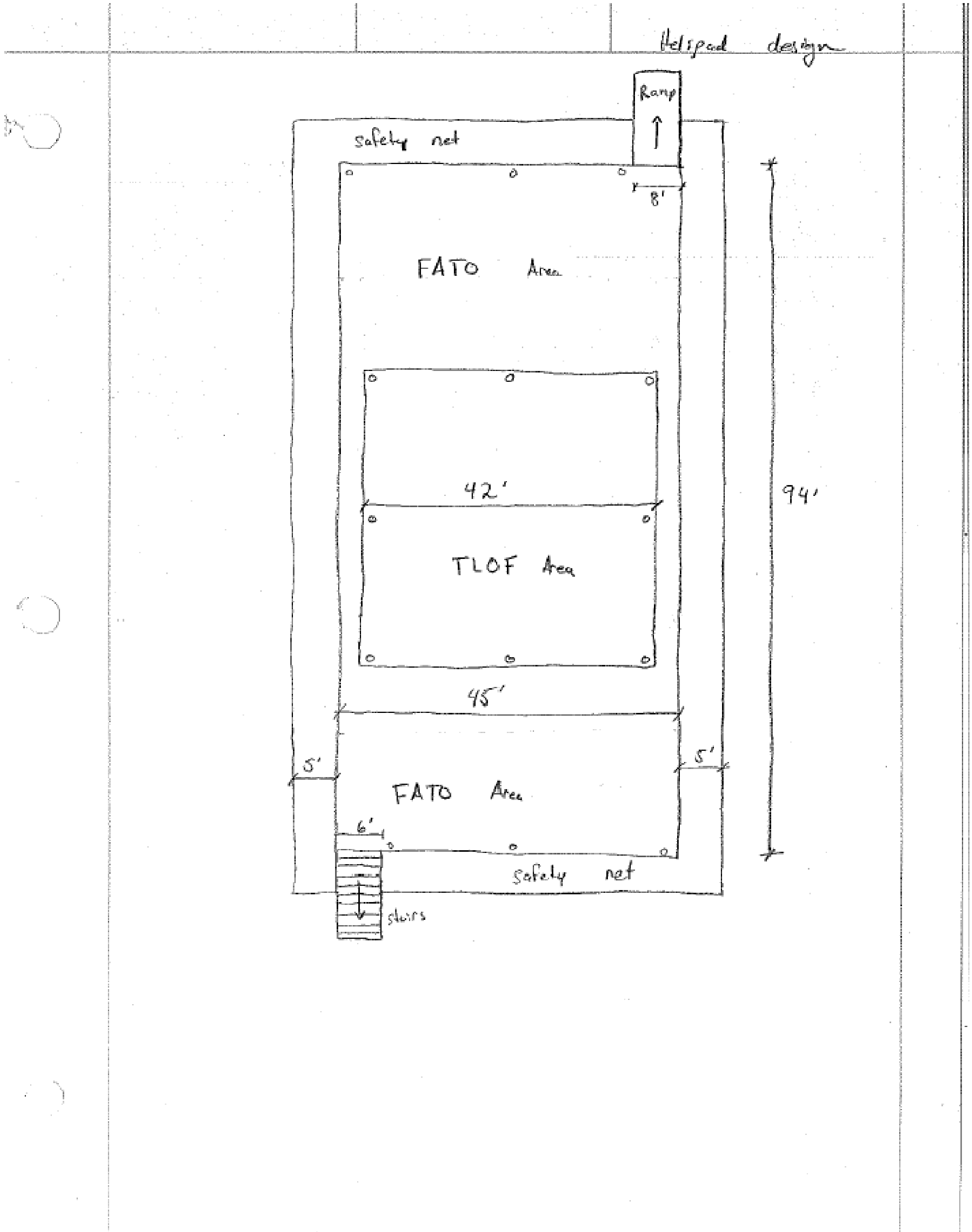
Level 1, Zone 5 Framing Plan



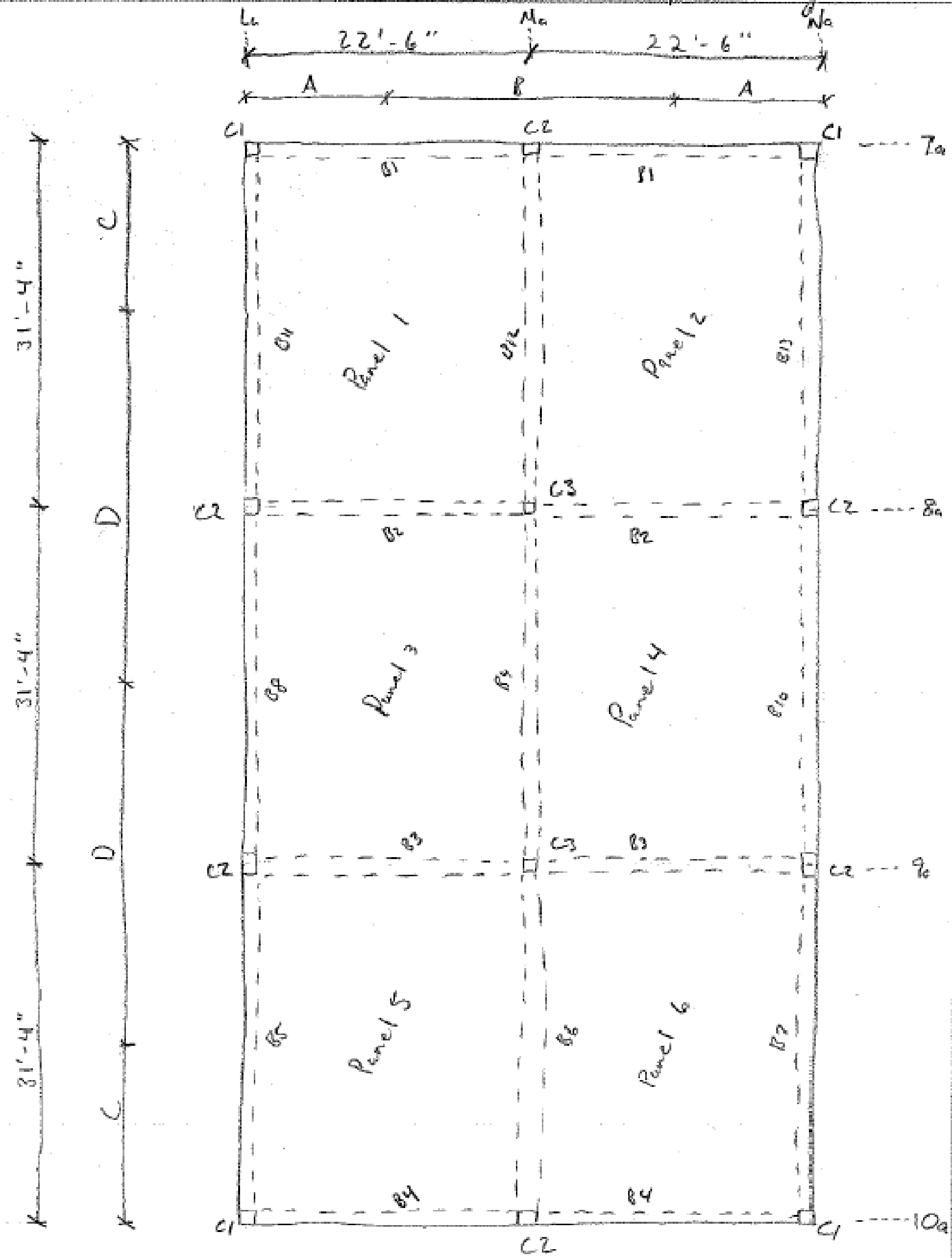
Level 1, Zone 6 Framing Plan

Level 1, Zone 7 Framing Plan

Appendix B: Helipad Slab Calculations



helipad design 1a



columns are 20" × 20"
$f'_c = 4$ ksi
$f_y = 60$ ksi

Live load = 100 psf
Snow load = 25 psf
$DL = 12^k(1.5) = 18^k / (42)^2 = 10.2$ psf $= 11$ psf $+ \frac{10}{12}(150)$
$= 136$ psf

$w_u = 1.2D + 1.6L + .5S = 1.2(136) + 1.6(100) + .5(25) = 335.7$ psf

2a

thickness: $t_{min} = \dfrac{L_n(.8 + f_y/200000)}{36 + 9\beta}$

$\beta = \dfrac{22.5 - (20/12)}{31.33 - (20/12)} = .702$

or

$\dfrac{31.33 - (20/12)}{22.5 - (20/12)} = 1.423$

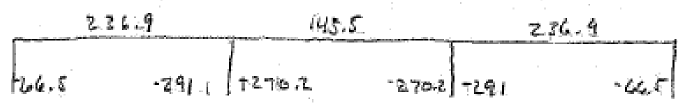
$t_{min} = \dfrac{20.833(.8 + 60000/200000)}{36 + 9(.702)} = 6.5''$

$= \dfrac{29.66(.8 + 60000/200000)}{36 + 9(1.423)} = 8.023''$

use 10" slab

Moments

A

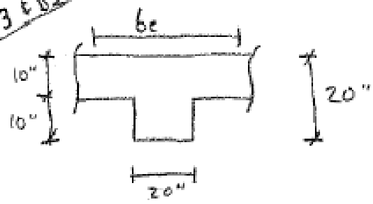


ft-k

$M_o = 1/8(.335)(11.25)(31.33 - 20/12)^2 = 415.8$ ft-k

$M^-_{int} = .7$ $M^+ = .57$ $M^-_{ext} = .16$ $M_o^- = .65$ $M_o^+ = .35$

B3 & B2



$b_e = b_w + 2h_w = 20 + 2(10) = 40'' \leq b_w + 8t$
$= 20 + 8(10) = 100''$

$\dfrac{b_e}{b_w} = \dfrac{40}{20} = 2$ $\dfrac{t}{h} = \dfrac{10}{20} = .5$

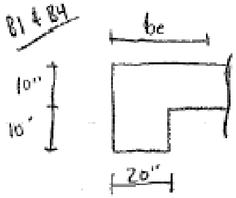
From 16.4.1 → k = 1.375

$I_b = \dfrac{k b_w h^3}{12} = \dfrac{1.375(20)(20)^3}{12} = 18333.3$

$I_s = \dfrac{L_2 t^3}{12} = \dfrac{(31.33)(10)^3(12)}{12} = 31330$

$\alpha = \dfrac{18333.3}{31330} = .585$

B1 & B4



$b_e = b_w + h_w = 20 + (10) = 30'' \leq b_w + 4t = 60''$

$\dfrac{b_e}{b_w} = \dfrac{30}{20} = 1.5$ $\dfrac{t}{h} = \dfrac{10}{20} = .5$

k = 1.1875 through interpolation

$I_b = 1.1875\left(\dfrac{20(20)^3}{12}\right) = 15833.3$

$I_s = \dfrac{(31.33/2)(10)^3(12)}{12} = 15665$

$\alpha = \dfrac{15833.3}{15665} = 1.011$

39

B11, B13, B8, B10, B5, B7

$I_b = 15833.3$

$I_s = 11250$

$\alpha = 1.407$

B12, B9, B6

$I_b = 18333.3$

$I_s = 22500$

$\alpha = .8148$

Panel 1: $\alpha_m = \dfrac{1.011 + .585 + 1.407 + .8148}{4} = .95$

Panel 2: $\alpha_m = .95$

Panel 3: $\alpha_m = .848$

Panel 4: $\alpha_m = .848$ Beams are medium stiffness

Panel 5: $\alpha_m = .95$

Panel 6: $\alpha_m = .95$

$$t_{min} = \frac{L_n(.8 + F_y/200000)}{36 + 5\beta(\alpha_m - .2)} = \frac{29.66(.8 + 60000/200000)}{36 + 5(1.423)(.848 - .2)} = 9.64'' < 10''$$

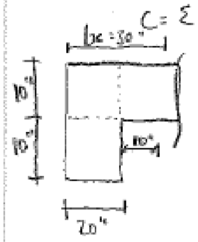
$t = 10''$ still ok ✓

$L_n = 31.33 - 20/12 = 29.66'$ $S_n = 22.5 - 20/12 = 20.833'$

$\beta = \dfrac{29.66}{20.833} = 1.424$

For edge beam in long direction:

$C = \Sigma(1 - .63\, x/y)(x^3 y/3)$



$C = [(1 - .63(20/20))((20)^3(20)/3)] + [(1 - .63(10/10))((10)^3(10)/3)] = \underline{20966.67}$

or

$C = [(1 - .63(20/10))((20)^3(10)/3)] + [(1 - .63(10/30))((10)^3(30)/3)] = 966.67$

4a

Transverse Distribution of Longitudinal Moments:

a) Negative Moment @ face of support

$\frac{L_2}{L_1} = \frac{22.5}{31.33} = .7182$

$\alpha_1 = 1.011$

$\beta_t = \frac{C}{2I_s} = \frac{20966.67}{2(11250)} = .93$

$\frac{\alpha L_2}{L_1} = .726 < 1.0$

$I_s = \frac{(22.5/2)(12)(10)^3}{12} = 11250$

l_2/l_1	.5	.718	1.0
$\beta = 0$	100	100	100
$\beta = .93$		92.3	
$\beta \geq 2.5$	82.5	79.23	75

-66.5: .923 to CS = -61.47; .85 to beam = -52.2; .15 to cs slab = -9.21
.077 to MS = -5.12

b) Negative Moment @ exterior face of 1st support

$\frac{\alpha L_2}{L_1} = (.585)(.7182) = .42$

l_2/l_1	.5	.718	1.0
	81.3	78.6	75

-291: .786 to CS = -228.7; .85 to beam = -194.4; .15 to CS slab = -34.3
.214 to MS = -62.3

c) Negative Moment @ interior face of 1st support

-270.2: .786 to CS = -212.4; .85 to beam = -180.5; .15 to CS slab = -31.9
.214 to MS = -57.8

d) Positive Moments

	.5	.718	1
.0	60	60	60
.573		73.44	
1	90	83.46	75

5a

236.9 < .734 to CS = 173.9 < .85 to beam = 147.8 ; .15 to CS slab = 26.1
236.9 < .266 to MS = 63

145.5 < .734 to CS = 106.8 < .85 to beam = 90.8 ; .15 to CS slab = 16.02
145.5 < .266 to MS = 38.7

Summary:

Frame A

Total Moment	-66.5	236.9	-291	-270.2	145.5
M_{Beam}	-52.5	147.8	-144.4	-180.5	90.8
M_{cs} slab	-9.21	26.1	-34.3	-31.9	16.02
M_{MS} slab	-5.121	63	-62.3	-57.8	38.7

Reinforcement design of CS slab:

Item #	Description	Ext. Span M^-_{Ext}	M^+	M^-_{int}	Int. Span M^-_o	M^+_o
1	$M_{U slab}$	-9.21	26.1	-34.3	-31.9	16.02
2	CS width, b	67.5"	67.5	67.5"	67.5"	67.5"
3	Effective depth, d $d = 3" - .5(5/8) - 1/4$ clear cover; Assume #5	6.0625"	6.06"	6.06"	6.06"	6.06"
4	$\frac{M_u \times 12}{b}$	-1.64	4.64	-6.1	-5.67	2.85
5	$M_n = \frac{M_u}{\phi = .9}$	-1.82	5.16	-6.78	-6.3	3.16
6	$R = \frac{M_n (12000)}{bd^2}$	8.8	24.94	32.77	30.48	15.31
7	ρ_{req} $R = \rho f_y (1 - .59\rho\, f_y/f'_c)$ $0 = -531000\rho^2 + 60000\rho - R$ $x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$	.0001469	.0004172	.0005489	.0005103	.0002557
8	$As_{req} = \rho bd$	.06	.171	.225	.209	.105
9	$As_{min} = .0018\, bt$	1.22	1.22	1.22	1.22	1.22

6a

Item	Description	M_{ext}^-	M^+	M_{int}^-	M_o^-	M_o^+
10	#bars = $A_s/.31$	3.9 = 4	4	4	4	4
11	N_{min} = $b/2t$	3.4 = 4	4	4	4	4

* see excel for remaining reinforcement designs

check shear in slab:

$w_u = .335$ kcf $d = 10 - 3 - \frac{1}{2}(.625) = 6.6875''$

$V_u = .335\,(1')(11.25' - \frac{20/12}{2} - \frac{6.69}{12}) = 3.3^k$

$\phi V_c = \phi 2\sqrt{f'_c}\,bd = .75(2)\sqrt{4000}\,(12)(6.6875) = 7.613^k > 3.3^k \therefore ok\checkmark$

Moments

B

472.5	290.12	472.5
-132.6 / -580	-538.9 / -538.9	-580 / -132

$M_o = \frac{1}{8}(.335)(22.5)(31.33 - \frac{20}{12})^2 = 829.3$

$M_{int}^- = .7$ $M^+ = .57$ $M_{ext}^- = .16$ $M_o^- = .65$ $M_o^+ = .35$

Negative Moments @ face of support

-132.6 < .96 to CS = -127.3 < .85 to beam = -108.2 ; .15 to slab = -19.1
.039 to MS = -19.1

Negative Moment @ exterior face of 1st support

-580 < .786 to CS = -455.9 < .85 to beam = -387.5 ; .15 to slab = -68.4
.214 to MS = -124.12

Negative Moment @ interior face of 1st support

-538.9 < .786 to CS = -423.6 < .85 to beam = -360 ; .15 to slab = -63.6
.214 to MS = -115.3

Reinforcement design of MS slab: Section A

Item #	Description	Exterior Span M_{ext}^-	Exterior Span M^+	Interior Span M_{int}^-	Interior Span M_{int}^-	Interior Span M^+
1	M_u	-5.12	63.00	-62.30	-57.80	38.70
2	b	67.50	67.50	67.50	67.50	67.50
3	d	6.0625	6.0625	6.0625	6.0625	6.0625
4	$(M_u * 12)/b$	-0.91	11.20	-11.08	-10.28	6.88
5	$M_n = M_u/\phi$	-1.01	12.44	-12.31	-11.42	7.64
6	$R = (M_n * 12000)/(bd^2)$	4.89	60.19	59.52	55.23	36.98
7	ρ_{req}	0.0000816	0.0010123	0.0010009	0.0009280	0.0006197
8	$A_s = \rho bd$	0.033	0.414	0.410	0.380	0.254
9	$A_{s\,min} = 0.0018bt$	1.22	1.22	1.22	1.22	1.22
10	# of bars, N	4	4	4	4	4
11	$N_{min} = b/2t$	4	4	4	4	4
12	**Bars Needed**	**4**	**4**	**4**	**4**	**4**

	t =	10	in
Size of bars:	#	5	
Bar Diameter	=	0.625	in
Bar Area	=	0.31	in
Clear Cover	=	3	in
d_{bottom}	=	6.6875	in
f_y	=	60000	psi
f'_c	=	4000	psi

7q

Positive Moments

472.5 { .734 to CS = 346.8 { .85 to beam = 294.8 ; .15 to slab = 52 } ; .266 to MS = 125.7 }

290.2 { .734 to CS = 213 { .85 to beam = 181.1 ; .15 to slab = 32 } ; .266 to MS = 77.2 }

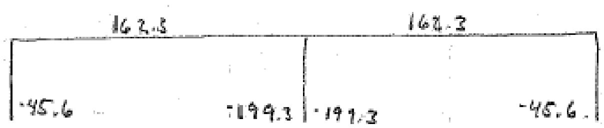
Summary:

	Frame B				
Total M	-132.6	472.5	-580	-538.9	290.2
M_{beam}	-108.2	294.8	-387.5	-366	181.1
M_{cs} slab	-19.1	52	-68.4	-63.6	32
M_{ms} slab	-19.1	125.7	-124.12	-115.3	77.2

see excel for reinforcement designs

Moments:

C



$M_o = 1/8(.335)(\frac{31.33}{2})(22.5 - {}^{24}/_{12})^2 = 284.7$

$M^-_{int} = .7$ $M^+ = .57$ $M^-_{ext} = .16$

Negative Moment @ face:

$\frac{L_2}{L_1} = 1.392$ $\alpha = 1.407$ $\frac{\alpha L_2}{L_1} = 1.959$

$I_s = 15665$

$\beta = \frac{C}{2 I_s} = \frac{20966.67}{2(15665)} = .669$

-45.6 { .902 to CS = -41.1 { .85 to beam = -34.9 ; .15 to CS = -6.2 } ; .0984 to MS = -4.5 }

Reinforcement design of CS slab: Section B

Item #	Description	Exterior Span M_{ext}^-	Exterior Span M^+	M_{int}^-	Interior Span M_{int}^-	Interior Span M^+
1	M_u	-19.10	52.00	-68.40	-63.60	32.00
2	b	135.00	135.00	135.00	135.00	135.00
3	d	6.06	6.06	6.06	6.06	6.06
4	$(M_u * 12)/b$	-1.70	4.62	-6.08	-5.65	2.84
5	$M_n = M_u/\phi$	-1.88642	5.14	-6.76	-6.28	3.16
6	$R = (M_n * 12000)/(bd^2)$	4.56	12.42	16.34	15.19	7.64
7	ρ_{req}	0.0000761	0.0002074	0.0002730	0.0002538	0.0001275
8	$A_s = \rho bd$	0.062	0.170	0.223	0.208	0.104
9	$A_{s\,min} = 0.0018bt$	2.43	2.43	2.43	2.43	2.43
10	# of bars, N	8	8	8	8	8
11	$N_{min} = b/2t$	7	7	7	7	7
12	**Bars Needed**	**8**	**8**	**8**	**8**	**8**

t =		10 in
Size of bars:	#	5
Bar Diameter =		0.625 in
Bar Area =		0.31 in
Clear Cover =		3 in
d =		6.0625 in
f_y =		60000 psi
f'_c =		4000 psi

Reinforcement design of MS slab:

Item #	Description	Exterior Span M_{ext}^-	Exterior Span M^+	M_{int}^-	Interior Span M_{int}^-	Interior Span M^+
1	M_u	-19.10	125.70	-124.12	-115.30	77.20
2	b	67.50	67.50	67.50	67.50	67.50
3	d	6.06	6.06	6.06	6.06	6.06
4	$(M_u * 12)/b$	-3.40	22.35	-22.07	-20.50	13.72
5	$M_n = M_u/\phi$	-3.77	24.83	-24.52	-22.78	15.25
6	$R = (M_n * 12000)/(bd^2)$	18.25	120.10	118.59	110.16	73.76
7	ρ_{req}	0.0003050	0.0020384	0.0020124	0.0018669	0.0012430
8	$A_s = \rho bd$	0.125	0.834	0.823	0.764	0.509
9	$A_{s\,min} = 0.0018bt$	1.22	1.22	1.22	1.22	1.22
10	# of bars, N	4	4	4	4	4
11	$N_{min} = b/2t$	4	4	4	4	4
12	**Bars Needed**	**4**	**4**	**4**	**4**	**4**

84

Negative Moment (int)

$\alpha = .848$ $\frac{\alpha L_2}{L_1} = 1.13$

$-199.3 \begin{cases} .75 \text{ to } CS = -149.5 \begin{cases} .85 \text{ to beam} = -127.1 \\ .15 \text{ to slab} = -22.4 \end{cases} \\ .25 \text{ to } MS = -49.8 \end{cases}$

Positive Moment

$162.3 \begin{cases} .632 \text{ to } CS = 102.6 \begin{cases} .85 \text{ to beam} = 87.2 \\ .15 \text{ to slab} = 15.4 \end{cases} \\ .367 \text{ to } MS = 59.6 \end{cases}$

Summary

	Frame C		
Total M	-45.6	162.3	-199.3
M_{beam}	-34.9	87.2	-127.1
M_{cs}	-6.2	15.4	-22.4
M_{ms}	-4.5	59.6	-49.8

see excel for reinforcement designs

Moments

D

324.6		324.6	
-91.1	-398.6	-398.6	-91.1

$M_o = \frac{1}{8}(.335)(31.33)(22.5 - {}^{20}/_{12})^2 = 569.4$

Negative Moment @ face:

$I_s = 31330$ $\beta = .3346$

$-91.1 \begin{cases} .951 \text{ to } CS = -86.6 \begin{cases} .85 \text{ to beam} = -73.6 \\ .15 \text{ to slab} = -13 \end{cases} \\ .0491 \text{ to } MS = -4.5 \end{cases}$

t =	10 in
Size of bars: #	5
Bar Diameter =	0.625 in
Bar Area =	0.31 in
Clear Cover =	3 in
d =	6.6875 in
f_y =	60000 psi
f'_c =	4000 psi

Reinforcement design of CS slab: Section C

Item #	Description	M_{ext}^-	M^+	M_{int}^-
1	M_u	-6.20	15.40	-22.40
2	b	94.00	94.00	94.00
3	d	6.69	6.69	6.69
4	$(M_u * 12)/b$	-0.79	1.97	-2.86
5	$M_n = M_u/\phi$	-0.88	2.18	-3.18
6	$R = (M_n * 12000)/(bd^2)$	2.51	6.24	9.07
7	ρ_{req}	0.0000419	0.0001040	0.0001514
8	$A_s = \rho bd$	0.026	0.065	0.095
9	$A_{s\,min} = 0.0018bt$	1.69	1.69	1.69
10	# of bars, N	6	6	6
11	$N_{min} = b/2t$	5	5	5
12	**Bars Needed**	**6**	**6**	**6**

Reinforcement design of MS slab:

Item #	Description	M_{ext}^-	M^+	M_{int}^-
1	M_u	-4.50	59.60	-49.80
2	b	94.00	94.00	94.00
3	d	6.69	6.69	6.69
4	$(M_u * 12)/b$	-0.57	7.61	-6.36
5	$M_n = M_u/\phi$	-0.64	8.45	-7.06
6	$R = (M_n * 12000)/(bd^2)$	1.82	24.13	20.16
7	ρ_{req}	0.0000304	0.0004036	0.0003371
8	$A_s = \rho bd$	0.019	0.254	0.212
9	$A_{s\,min} = 0.0018bt$	1.69	1.69	1.69
10	# of bars, N	6	6	6
11	$N_{min} = b/2t$	5	5	5
12	**Bars Needed**	**6**	**6**	**6**

9a

Negative Moment (int)

-398.6 < .75 to CS = -299 < .85 to beam = -254.1 ; .15 to slab = -44.9
.25 to MS = -99.7

Positive Moment:

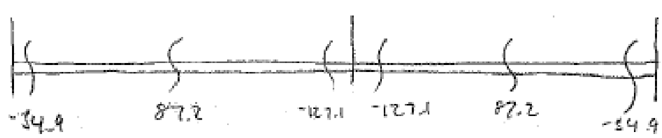
324.6 < .632 to CS = 205.1 < .85 to beam = 174.3 ; .15 to slab = 30.8
.367 to MS = 119.1

Summary

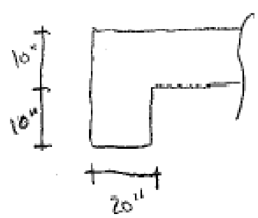
	Frame D		
Total M	-91.1	+324.6	-398.6
M_{beam}	-73.6	174.3	-254.1
M_{cs}	-13	30.8	-44.9
M_{ms}	-4.5	119.1	-99.7

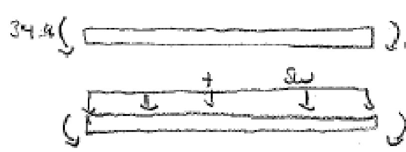
see excel for reinforcement design

check beam strength:



Beam 1/4 in area C





$SW = \frac{10(20)}{144}(150)$

$= 208.3 psf$

$w_u = \frac{1.2(.208)(22.5 - \frac{20}{12})^2}{16}$

$= 6.77$ k-ft

Total $M_u = 127.1 + 6.77 = 133.9$ k-ft

$d = 20 - 3 = 17"$

$R_n = \frac{M_u}{\phi b_w d^2} = 133.9 / (.9)(20)(17)^2 \quad (12000) = 308.8$ ft-k

Reinforcement design of CS slab: Section D

Item #	Description	M_{ext}^-	M^+	M_{int}^-
1	M_u	-73.60	174.30	-254.10
2	b	188.00	188.00	188.00
3	d	6.69	6.69	6.69
4	$(M_u * 12)/b$	-4.70	11.13	-16.22
5	$M_n = M_u/\phi$	-5.22	12.36	-18.02
6	$R = (M_n * 12000)/(bd^2)$	7.45	17.64	25.72
7	ρ_{req}	0.0001243	0.0002948	0.0004303
8	$A_s = \rho bd$	0.156	0.371	0.541
9	$A_{s\,min} = 0.0018bt$	3.38	3.38	3.38
10	# of bars, N	11	11	11
11	$N_{min} = b/2t$	10	10	10
12	**Bars Needed**	11	11	11

	t =	10 in
Size of bars:	#	5
Bar Diameter	=	0.625 in
Bar Area	=	0.31 in
Clear Cover	=	3 in
d	=	6.6875 in
f_y	=	60000 psi
f'_c	=	4000 psi

Reinforcement design of MS slab:

Item #	Description	Exterior Span		
		M_{ext}^-	M^+	M_{int}^-
1	M_u	-4.50	119.10	-99.70
2	b	94.00	94.00	94.00
3	d	6.69	6.69	6.69
4	$(M_u * 12)/b$	-0.57	15.20	-12.73
5	$M_n = M_u/\phi$	-0.64	16.89	-14.14
6	$R = (M_n * 12000)/(bd^2)$	1.82	48.22	40.37
7	ρ_{req}	0.0000304	0.0008095	0.0006768
8	$A_s = \rho bd$	0.019	0.509	0.425
9	$A_{s\,min} = 0.0018bt$	1.69	1.69	1.69
10	# of bars, N	6	6	6
11	$N_{min} = b/2t$	5	5	5
12	**Bars Needed**	6	6	6

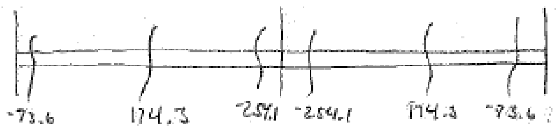
10a

$308.8 = 60000\rho\,(1 - .59\rho\,(60000/4000))$

$\rho_{req} = .00541$

$A_s = \rho b d = 1.839\,in^2$

use (3) #8



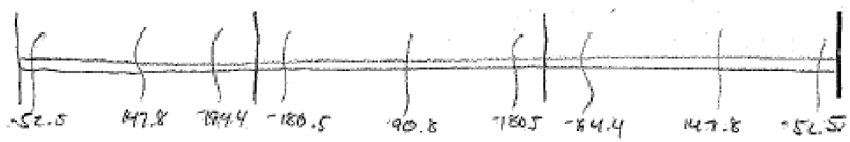
Beams 2 & 3 area D

$M_u = 254 + 6.77 = 260.77$

$R = 260.77(12000) / .9(20)(17)^2 = 601.5$

$\rho_{req} = .0111$

$A_{sreq} = 3.78$ use (5) #8



Beams 5, 7, 8, 10, 11, & 13 in Area A

$SW = \frac{10(20)}{144}(150) = 208.3\,psf$

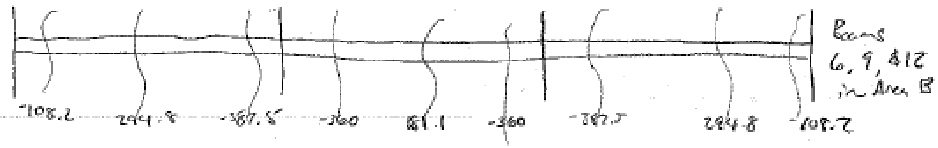
Total $M_u = 194.4 + 21.96 = 216.4$

$w = \frac{1.2(208)(31.33 - 20/12)^2}{10} = 21.96\,k\text{-}ft$

$R_n = \frac{216.4(12000)}{.9(20)(17)^2} = 499.1$ $\rho_{req} = .00904$

$A_{sreq} = 3.07$ use (4) #8

11a



$M_u = 387.5 + 21.96 = 409.5$

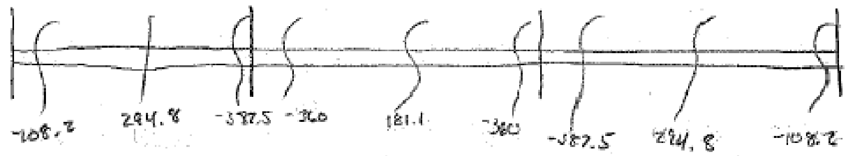
$R_n = 944.5$ $A = 6.43$ use (9) #8

$\rho = .0189$

Per ACI 318-08 11.4.6.1, No shear reinforcement required for $h < 10''$

$h = 10''$ so no reinforcement required.

11a



Beams 6, 9, & 12 in Area B

$M_u = 387.5 + 21.96 = 409.5$

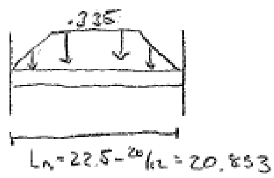
$R_n = 409.5\,(12000) / .9(20)(17)^2 = 944.5$

$\rho = .0189$

$A_s = 6.43$ use (9) #8

Check Shear in Beams

B2 & 3



$w_L L = .335(31.33) = 10.5$ klf

$w_u = 10.5(9) = 94.5^K$

shear $= \frac{1}{2} w_u = \frac{1}{2}(94.5) = 47.21$ due to D+L

Shear SW $= \dfrac{1.2(.208)(20.833)}{2} = 2.6^K$

$V_u = 1.15(47.21) + 1.15(2.6) + 254.1 - 72.8 / 20.833 = 65.9^K$

$\phi V_n = .75(3.5)\sqrt{4000}(20)(17) = 56.44^K < V_u$ ∴ shear reinforcement required

use (2) #4 @ 8.5"

B1 & 4

$w_L L = .335(31.33)(.5) = 5.25$ klf

Shear $= 23.6^K$

$w_u = 47.2^K$

$V_u = 1.15(23.6) + 1.15(2.6) + 127.1 - 34.9 / 20.833 = 34.4 < \phi V_n = 56.4^K$

12a

B5, 7, 8, 10, 11, & 13

$w_u L = .335(22.5).5 = 3.77\text{ kip}$

$w_u = 3.77(20) = 75.38$

$V_u = .5(75.38) = 37.7^k$ $\qquad V_{usw} = \frac{1.2(.208)(31.33)}{2} = 3.9^k$

$V_{uT} = 1.15(37.7) + 1.15(3.9) + {}^{194.7-52.5}/(31.33-20/12) = 52.6^k < \phi V_n = 56.4^k$

B6, 9, & 12

$w_u L = 7.54$

$w_u = 150.75^k$ $\qquad V_u = 75.4$

$V_{uT} = 1.15(75.4) + 1.15(3.9) + {}^{387.5-108}/_{29.66} = 100.6 \quad < \phi V_n$ ∴ requires shear reinforcement

Design: $8\sqrt{4000}(20)(17) = 172^k$

$V_s = \frac{V_u}{\phi} - V_c = \frac{100.6}{.9} - 43 = 68.7^k$

$V_s \le 4\sqrt{f'_c}\, b_w d = 86^k \therefore s_{max} = d/2 = 17/2 = 8.5''$

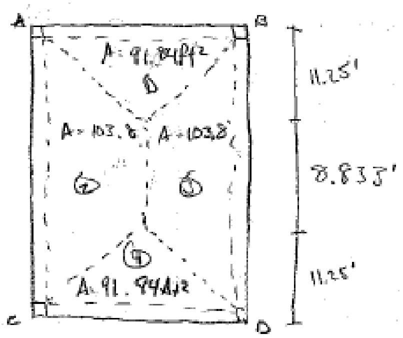
$A_{v,min} = \begin{cases} .75\sqrt{4000}(20)(8.5)/60000 = .134 \\ 50(8.5)(20)/60000 = .142 \leftarrow * \end{cases}$

use (2) #4 @ 8.5''

Appendix C: Helipad Column Calculations

13q

Load on each column



$w_u = 185.7\,psf$

$SW = \left(\frac{10}{12}\right)(150) = 125\,psf$

$SW_{bm} = \left(\frac{20}{12}\right)(150) = 250\,psf$

Area 1 = $(.5)(11.25 - 20/12)(185.7 + 125) = 1480.8\,plf$ on beam

beam: $(20/12)(185.7 + 250) = 726.2\,plf$

on B or A: $\frac{1}{2}(1480.8 + 726.2)(22.5 - 40/12) = 21.2^K$

Area 2: $[(2)\frac{1}{2}(11.25 - 20/12)(185.7 + 125)] + [(11.25 - 20/12)(185.7 + 125)]$
$= 5955.1\,plf$

beam: $(20/12)(185.7 + 250) = 726.2\,plf$

on A or C: $\frac{1}{2}(5955.1 + 726.2)(31.33 - 40/12) = 93.5^K$

load on each column: $21.2 + 93.5 = 114.7^K$

Load on C1: 114.7^K

Load on C2: 229.4^K

Load on C3: 458.8^K

14a

columns (gravity):

L7:
L8:
L9:
N8:
N9:

W14x61 (a)
W14x74 (b)
W14x74 (c)
W14x74 (d)
W14x90 (e)

M8
M9
M10

W14x61
W14x90
W24x250 W14x90
W14x90
W14x90

— roof
— level 8
— level 7
— level 6
— level 5
— level 4
— level 3
— level 2
— level 1
— Ground

21'-5.5"
14'-0"
14'-0"
14'-0"
14'-0"
15'-4"
15'-4"
18'-4"
15'-4"

M7:
N7: Design in lateral
L10: section
N10:

L7/ Assume 25 psf DL

Level	Load	Takedown - DL/column
9	114.7k	114.7
8	52.9k	167.6
7	52.9k	220.5
6	52.9k	273.4
5	52.9k	326.3
4	44.1k	370.4
3	44.1k	414.5
2	44.1k	458.6
1	44.1k	502.7

P_u

Level 9 & 8
L = 31'-6"
K = 1
P = 167.6k

use W14x68

P_u

Level 7 & 6

L: 28'
K = 1.0
P = 326.3 + (1.2)(31.5)(68)
= 328.87
use W14x90

Level 5
K = 1.0
P = 376
L = 14'
use W14x90

Level 4 & 3
L = 30.66'
K = 1.0
P = 421.6

use W14x90

level 1 & 2 K = 1.0
P = 513.1k L = 34'-8"
use W14x99

15a

L8/L9/N8/N9:

Level	Load	Takedown - DL
9	229.4k	229.4
8	52.9	282.3
7	52.9	335.2
6	52.9	388.1
5	52.9	441
4	44.1	485.1
3	44.1	529.2
2	44.1	573.3
1	44.1	617.4

Level	Size
9/8	W14x90
7/6	W14x90
5	W14x90
4/3	W14x90
2/1	W14x120

M8/M9:

Level	Load	Takedown - DL
9	458.8	458.8
8	162.13	620.9
7	162.13	783.1
6	162.13	945.2
5	see below	555.7
4	66.9	622.6
3	66.9	689.5
2	66.9	756.4
1	66.9	823.3

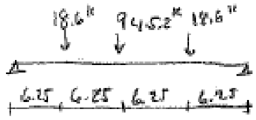
on column from beam = ½ DL above + ½[1.2(25)(22.5)(25)+1.6(25)(22.5)(100)]
= ½ DL + 52.4

DL on column = ½ DL above + 52.4 + 29.7 = 83.1

Level	L	Size
9/8	31'-6"	W14x109
7/6/5	32'-0"	W14x145
5/4	10'	W14x90
4/3	30'-8"	W14x109
2/1	34'-8"	W14x145

16a

Beam design for M8|M9:



$M = \frac{(945.2)(25)}{4} + (18.6)(6.25) = 6023.75$ ft-K

$V = P + \frac{P}{2} = 945.2 + \frac{18.6}{2} = 954.5^k$

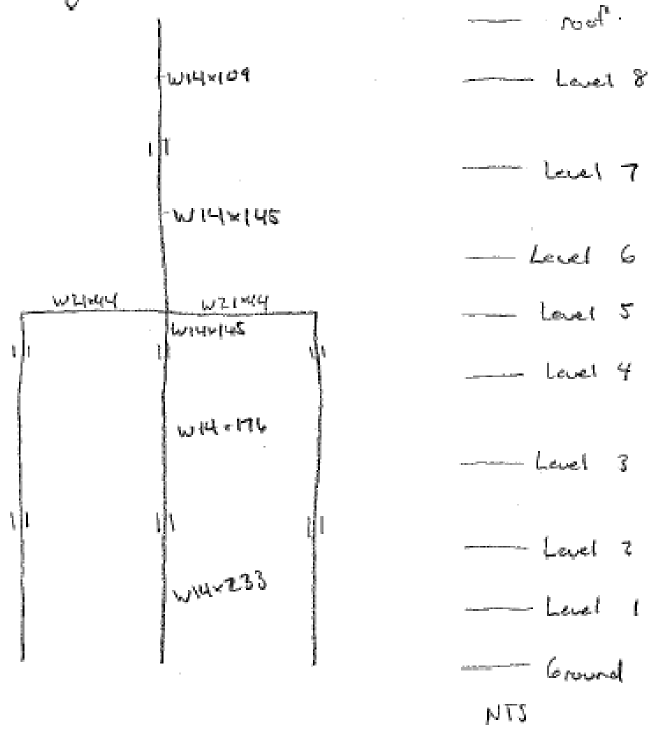
Assume $a = 1''$ $Y_2 = 6''$

$b_{eff} = S = L/4 = 25/4\,(12) = 75''$

Try W40x235 $\phi M_n = 6240$

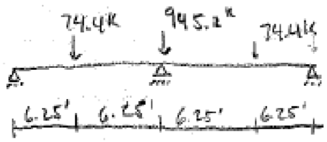
$a = \frac{2510}{.85(3.5)(75)} = 11.25'' \geq 1''$ assumption not ok

Load is too large, introduce a new column



17a

Beam design for H8/M9:



design 2 seperate beams

$M_u = \frac{PL}{4} = \frac{74.4(12.5)}{4} = 232.5$ ft-k

$V = \frac{74.4}{2} = 37.2^k$

Assume $a=1$ $Y_2 = 6.5 - \frac{1}{2} = 6$

$b_{eff} = 3.125'(12) = 37.5"$

Try W21×44 PNA=7 $\phi M_n = 528$ ft-k

Must use W21 b/c beams frame into it.

$a = \frac{162}{.85(3.5)(37.5)} = 1.45 > 1"$ ∴ wrong assumption

assume $a = 2"$ $Y_2 = 6.5 - 1 = 5.5$ use W21×44 PNA=7 $\phi M_n = 522$ ft-k

$a = 1.45 < 2"$ ∴ ok assumption

$\#$ studs $= \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{2(162)}{21} = 15.4 = 16$ studs

Level	Load	Takedown - DL
9	458.8	458.8
8	162.13	620.9
7	162.13	783.1
6	162.13	945.2
5	74.4	1019.6
4	74.4	1094
3	74.4	1168.4
2	74.4	1242.8
1	74.4	1317.2

18a

Level	L	Size
9/8	31'-6"	W14x109
7/6/5	32'-0"	W14x145
5/4	10'	W14x145
4/3	30'-8"	W14x176
2/1	34'-8"	W14x233

see diagram on page 16a

Level	Load	Takedown
5	95.5	95.5
4	95.5	190
3	95.5	286.5
2	95.5	382
1	95.5	477.5

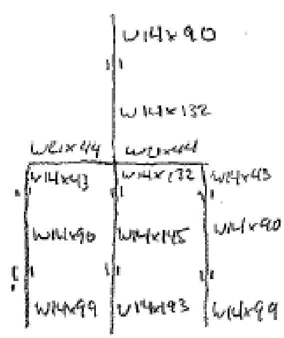
Level	L	Size
5/4	10'	W14x43
4/3	30'-8"	W14x90
2/1	34'-8"	W14x99

M10:

Level	Load	Takedown
9	229.4	229.4
8	162.13	391.5
7	162.13	553.7
6	162.13	715.79
5	74.4	790.2
4	74.4	864.6
3	74.4	940
2	74.4	1013.4
1	74.4	1087.8

Level	L	Size
9/8	31'-6"	W14x90
7/6/5	32'-0"	W14x132
5/4	10'	W14x132
4/3	30'-8"	W14x145
2/1	34'-8"	W14x193

side beams are the same as M8 M9



Appendix D: Helipad Foundation Calculations

19a

Design Foundations:

Column	Load
L7	516.53^k
L8	621.6^k
L9	621.6^k
N8	621.6^k
N9	621.6^k
$M8_c$	1325.3^k
$M9_c$	1325.3^k
$M10_c$	1094.5^k
$M8_{LR}$	480.9^k
$M9_{LR}$	480.9^k
$M10_{LR}$	480.9^k

Soil: $\gamma_{dry} = 94.39$ pcf
$\gamma_{wet} = 103.85$ pcf
$w = 10.02$
$q_a = 6$ ksf
piers/columns: 24" x 24"

$\gamma_{dry} = \frac{\gamma}{1+w}$

$94.39 = \frac{\gamma}{1+.1002}$
$\gamma = 103.85$ pcf

L7:

$P = 516.53^k$

$\sigma = 516.53/A$
$A = 86.1\ ft^2 = b^2$
$b = 9.3\ ft = 10\ ft$

L8/L9/N8/N9:

$P = 621.6^k$
$A = 103.6\ ft^2$
$b = 11\ ft$

$M8_c$, $M9_c$:

$P = 1325.3^k$
$A = 220.9\ ft^2$
$b = 15\ ft$

$M10_c$

$P = 1094.5^k$
$A = 182.4\ ft^2$
$b = 14\ ft$

$M8_{LR}$, $M9_{LR}$, $M10_{LR}$:

$P = 480.9^k$
$A = 80.15\ ft^2$
$b = 9\ ft$

20a

L7

$d^2[v_c + q/4] + d[v_c + q/2]w = q/4[BL - w^2]$ $\quad v_c = \phi V_c = \phi(4)\sqrt{f'_c}$

$d^2[199.7 + \frac{5.17(1000)}{(12)^2(4)}] + d[189.7 + \frac{5.17(1000)}{(12)^2(2)}](24) = \frac{5.17(1000)}{(4)(12)^2}[(120)^2 - 24^2] = .75(4)\sqrt{4000}$
$= 189.7$ psi

$198.7d^2 + 4983.6d - 124080 = 0$ $\quad q = 516.5/(10)^2 = 5.165$ ksf

$d \geq 15.4"$ Assume #8

$h = d + d_b + 3 = 15.4 + 1 + 3 = 19.4"$ use 20" let $d = 20 - 3 - 1 = 16"$

$b_o = (24 + 16)4 = 160"$

$v_c = .75\left(\frac{40(16)}{160} + 2\right)\sqrt{4000} = 284.6$ psi

$M_u = \frac{qL^2}{2} = \frac{5.165(4)^2}{2} = 41.32$ k-ft $\quad a = \frac{A_s f_y}{.85f'_c b} = \frac{A_s(60)}{.85(4)(12)} = 1.47A_s$

$41.32(12) = .9A_s(60)(16 - 1.47A_s/2)$

$495.84 = 864 A_s - 39.69 A_s^2$

$A_s = .589$ in²/ft use #8 @ 14" o.c. $A_s = .68$

$\rho = \frac{A_s}{bh} = \frac{.68}{(12)(20)} = .00283 > .0018$ ∴ good ✓

$\varepsilon_s = \frac{.003}{c}(d-c) = \frac{.003}{1.176}(16 - 1.176)$ $\quad c = \frac{1.47(.68)}{.85} = 1.176$

$= .0378 > .005$ ∴ $\phi = .9$ ∴ ok ✓

use (10) #8 each way

$\phi B_n = .65(.85)(4)(24)^2 = 1273^k > 516.53^k$

21a

L8/L9/N8/N9: $P = 621.6^k$ 11'×11' $q = 5.14$

$$d^2\left[189.7 + \frac{5.14(1000)}{(12)^2(4)}\right] + d\left[189.7 + \frac{5.14(1000)}{(12)^2(2)}\right](24) = \frac{5.17(1000)}{4(12)^2}\left[(132)^2 - (24)^2\right]$$

$$198.6d^2 + 4981.13d - 150345 = 0$$

$$d = 17.69''$$

$h = 17.69 + 3 + 1 = 21.69$ use 22" $\quad d = 22 - 3 - 1 = 18''$

$$b_o = (24 + 18) = 42''(4) = 168''$$

$$V_c = .75\left(\frac{40(18)}{168} + 2\right)\sqrt{4000} = 298.2\ psi$$

$$M_u = \frac{(5.14)(4.5)^2}{2} = 52.04\ ^{k\text{-}ft}$$

$$52.04(12) = .9A_s(60)(18 - {}^{1.47A_s}/_2)$$

$$624.48 = 972A_s - 39.7A_s^2$$

$A_s = .66$ use #8@14" o.c. $A_s = .677$

$\rho = \frac{A_s}{bh} = \frac{.68}{(12)(22)} = .00257 > .0018$ ∴ good ✓

$\varepsilon_s = \frac{.003}{1.176}(18 - 1.176) = .043 > .005$ ∴ ok ✓ $\quad c = \frac{1.47(.677)}{.85} = 1.176$

use (10) #8 each way

$\phi B_n = .65(.85)(4)(24)^2 = 1273^k > 621.6^k$ ∴ ok ✓

22a

$M8_c, M9_c$: $P = 1325.3^k$ use 30"×50" pier
$\varepsilon = 5.89$
15×15'

$$d^2\left[189.7 + \frac{5.89(1000)}{(12)^2(4)}\right] + d\left[189.7 + \frac{5.89(1000)}{(12)^2(2)}\right][30] = \frac{5.89(1000)}{(12)^2(4)}\left[(180)^2 - (30)^2\right]$$

$$199.93d^2 + 6304.5d = 322109.4$$

$$d = 27.4''$$

$h = 27.4 + 3 + 1 = 31.4''$ use 32" $d = 32 - 3 - 1 = 28''$

$$b_o = (32 + 28)4 = 240''$$

$$v_c = .75\left(\frac{40(28)}{240} + 2\right)\sqrt{4000} = 316.23\,psi$$

$$M_u = \frac{(5.89)(6.25)^2}{2} = 113.3^{k\text{-}ft} \qquad a = 1.47A_s$$

$$113.3(12) = .9A_s\,60(28 - 1.47A_s/2)$$

$$-39.7A_s^2 + 1512A_s - 1359.6 = 0$$

$A_s = .921$ use #8 @ 10" O.C $A_s = .96$

$$\rho = \frac{.96}{(12)(32)} = .0025 > .0018 \therefore ok ✓$$

$$\varepsilon_s = \frac{.003}{1.66}(28 - 1.66) = .0476 > .005 \quad \phi = .9 \therefore ok ✓$$

use (17) #8 each way

$$\phi P_n = .65(.85)(4)(30)^2 = 1989^k > 1325.3^k$$

23a

M_{10c}: $P = 1094.5^k$ $24'' \times 24''$ pier
$q = 5.6$ ksf
$14' \times 14'$

$$d^2\left[189.7 + \frac{5.6(1000)}{4(12)^2}\right] + d\left[189.7 + 5.6(1000)/2(12)^2\right][24] = \frac{5.6(1000)}{4(12)^2}\left[168^2 - (24)^2\right]$$

$$199.4d^2 + 5019.5d = 268800$$

$$d = 26.227''$$

$h = 26.23 + 3 + 1 = 30.23''$ use $32''$ $d = 32 - 3 - 1 = 28''$

$$b_o = (28 + 32)4 = 240''$$

$$V_c = .75\left(\frac{40(28)}{240} + 2\right)\sqrt{4000} = 316.23\,psi$$

$$M_u = \frac{5.6(6)^2}{2} = 100.8$$

$$(100.8)(12) = .9A_s(60)(28 - 1.47A_s/2)$$

$$1209.6 = 1512A_s - 39.7A_s^2$$

$A_s = 0.8175$ use #8 @ 10" O.C. $A_s = .96$

$\rho = \frac{.96}{12(32)} = .0025 > .0018$ ∴ ok ✓

$\varepsilon_s = \frac{.003}{1.66}(28 - 1.66) = .0476 > .005$ $\phi = .9$ ∴ ok ✓

use (18) #8 each way

$\phi P_n = .65(.85)(4)(24)^2 = 1273^k > 1094.5^k$ ∴ ok ✓

24a

M8LR, M9LR, M10LR: $P = 480.9^K$ 24" x 24" pier
$q = 5.94$ ksf
9' x 9'

$$d^2\left[189.7 + \frac{5.9(1000)}{4(12)^2}\right] + d\left[189.7 + \frac{5.9(1000)}{2(12)^2}\right](24) = \frac{5.9(1000)}{4(12)^2}\left[108^2 - 24^2\right]$$

$$199.9d^2 + 5044.5d = 113575$$

$$d = 14.35"$$

$h = 14.35 + 3 + 1 = 18.35"$ use 20" $d = 20 - 3 - 1 = 16"$

$b_o = (20+16)4 = 144"$

$$v_c = .75\left(\frac{40(16)}{144} + 2\right)\sqrt{4000} = 305.7 \text{ psi}$$

$$M_u = \frac{5.9(3.5)^2}{2} = 36.14 \text{ ft-k}$$

$$(36.14)(12) = .9A_s(60)(16 - 1.47A_s/2)$$

$$433.68 = 864A_s - 39.69A_s^2$$

$A_s = .5141$ use #8 @ 18" O.C. $A_s = .5267$

$$\rho = \frac{.53}{(12)(20)} = .00221 > .0018 \therefore ok ✓$$

$$\varepsilon_s = \frac{.003}{.911}(16 - .911) = .0497 > .005 \therefore \phi = .9 \therefore ok ✓$$

use (6) #8 each way

$$\phi B_n = .65(.85)(4)(24)^2 = 1273^K \geq 480.9^K$$

Appendix E: Wind Calculations

Paul Stewart | Virtua Health Hospital | Tech #1 | Wind Loads | 4

According to ASCE 07-05:

V = 100 mph per Fig 6-1

K_d = .85 per Table 6-4

I = 1.15 per Table 6-1

Exposure Category: B per 6.5.6

6.5.7.1: all points not met

→ K_{zt} = 1.0

β = .01

N-S:
Area A = Areas 1-3: B = 74.166', h = 139'-4"
Area B = Areas 4-7: B = 295.875', h = 139'-4"

E-W:
Areas A: B = 242.4', h = 139'-4"
Areas B: B = 292', h = 139'-4"

Story heights = 15'-4"

roof mean height = 139.33 ft

For exposure type B per Table 6-2:

α = 7.0
z_g = 1200'

$\hat{a}$ = 1/7
$\hat{b}$ = 0.84
$\bar{\alpha}$ = 1/4.0
$\bar{b}$ = 0.45
c = 0.30
ℓ = 320'
$\bar{\epsilon}$ = 1/3.0

·Find K_z per Table 6-3:

$K_z = 2.01(z/z_g)^{2/\alpha}$

Level	Height (ft)	K_z
0	0	0
1	15.33	0.578
2	30.66	0.705
3	46	0.792
4	61.33	0.859
5	75.33	0.911
6	89.33	0.957
7	103.33	0.998
8	117.33	1.034
9	139.33	1.086

$K_h = 2.01(h/z_g)^{2/\alpha} = 1.086$

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· Find velocity pressure:

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

Height	q_z
0	0
15.33	14.463
30.66	17.642
46	19.819
61.33	21.496
75.33	22.797
89.33	23.948
103.33	24.974
117.33	25.875
139.33	27.176

$q_h = .00256\, K_h K_{zt} K_d V^2 I = .00256(1.036)(1)(.85)(100)^2(1.15) = 27.176$

Gust Effects Factors:

$n_1 = 22.2 / H^{0.8} = 22.2 / (139.33)^{.8} = .4276$ per C6.5.8

$n_1 < 1$ ∴ building is flexible per 6.2

$g_Q = g_v = 3.4$

$g_r = \sqrt{2\ln(3600 n_1)} + \dfrac{.577}{\sqrt{2\ln(3600 n_1)}} = 3.982$ Eq 6-9

$\bar{z} = .6h = .6(139.33) = 83.598$

$I_{\bar{z}} = c\left(\dfrac{33}{\bar{z}}\right)^{1/6} = .30\left(\dfrac{33}{83.6}\right)^{1/6} = .2569$ Eq 6-5

$L_{\bar{z}} = \ell\left(\dfrac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\dfrac{83.6}{33}\right)^{1/3} = 436.23$ Eq 6-7

$Q = \sqrt{\dfrac{1}{1+.63\left(\dfrac{B+h}{L_{\bar{z}}}\right)^{.63}}}$ Eq 6-6

$Q_{A\,N\text{-}S} = 0.845$

$Q_{B\,N\text{-}S} = 0.783$

$Q_{A\,E\text{-}W} = 0.796$

$Q_{B\,E\text{-}W} = 0.784$

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$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 83.265$$ Eq 6-14

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = 2.24$$ Eq 6-12

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = 0.083$$ Eq 6-11

$$\eta_1 = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = 3.29$$

$$R_h = \frac{1}{\eta_1} - \frac{1}{2\eta_1^2}\left(1-e^{-2\eta_1}\right) = .2578$$

$$\eta_2 = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} =$$

$\eta_{2A\,N\text{-}S} = 1.752$ $\quad$ $\eta_{2A\,E\text{-}W} = 5.724$

$\eta_{2B\,N\text{-}S} = 6.989$ $\quad$ $\eta_{2B\,E\text{-}W} = 6.897$

$$R_B = \frac{1}{\eta_2} - \frac{1}{2\eta_2^2}\left(1-e^{-2\eta_2}\right) =$$

$R_{BA\,N\text{-}S} = .4127$ $\quad$ $R_{BA\,E\text{-}W} = .1594$

$R_{BB\,N\text{-}S} = .1328$ $\quad$ $R_{BB\,E\text{-}W} = .1345$

$$\eta_3 = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} =$$

$\eta_{3A\,NS} = 19.165$ $\quad$ $\eta_{3A\,EW} = 5.865$

$\eta_{3B\,NS} = 23.093$ $\quad$ $\eta_{3B\,EW} = 23.399$

$$R_L = \frac{1}{\eta_3} - \frac{1}{2\eta_3^2}\left(1-e^{-2\eta_3}\right) =$$

$R_{LA\,NS} = .0508$ $\quad$ $R_{LA\,EW} = .1559$

$R_{LB\,NS} = .0423$ $\quad$ $R_{LB\,EW} = .0418$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$$ Eq 6-10

$R_{A\,NS} = .6994$ $\quad$ $R_{A\,EW} = .4536$

$R_{B\,NS} = .3953$ $\quad$ $R_{B\,EW} = .3977$

$$G_f = .925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$ Eq 6-8

$G_{FA\,NS} = 1.0227$ $\quad$ $G_{FA\,EW} = .8237$

$G_{FB\,NS} = .8059$ $\quad$ $G_{FB\,EW} = .8065$

The building is enclosed with no parapet, & is not a low-rise building.

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From 6-6

Cp:

$GC_{pi} = +/-0.18$

Windward Wall: .8

Leeward Walls: A-NS: -.2388
B-NS: -.5
A-EW: -.5
B-EW: -.5

Side walls: -.7

$p_z = q_z G_f C_p - q_h(GC_{pi})$ windward Eq 6-19

$p_h = q_h G_f C_p - q_h(GC_{pi})$ leeward Eq 6-19

Area 1, 2, 3 N-S

height	Floor	p_z	p_h
0	0	0	11.529
15.33	1	16.725	
30.66	2	19.326	
46	3	21.107	
61.33	4	22.479	
75.33	5	23.543	
89.33	6	24.485	
103.33	7	25.324	
117.33	8	26.062	
129.33	9	27.126	

Area 4, 5, 6, 7 NS

Floor	p_z	p_h
0	0	15.842
1	14.216	
2	16.266	
3	17.669	
4	18.751	
5	19.589	
6	20.331	
7	20.993	
8	21.574	
9	22.413	

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Area 1, 2, 3 E-W

Floor	Pz	Ph
0	0	16.084
1	14.422	
2	16.517	
3	17.952	
4	19.057	
5	19.914	
6	20.672	
7	21.349	
8	21.942	
9	22.800	

Area 4, 5, 6, 7 E-W

Floor	Pz	Ph
0	0	15.85
1	14.223	
2	16.274	
3	17.679	
4	18.761	
5	19.600	
6	20.343	
7	21.005	
8	21.586	
9	22.426	

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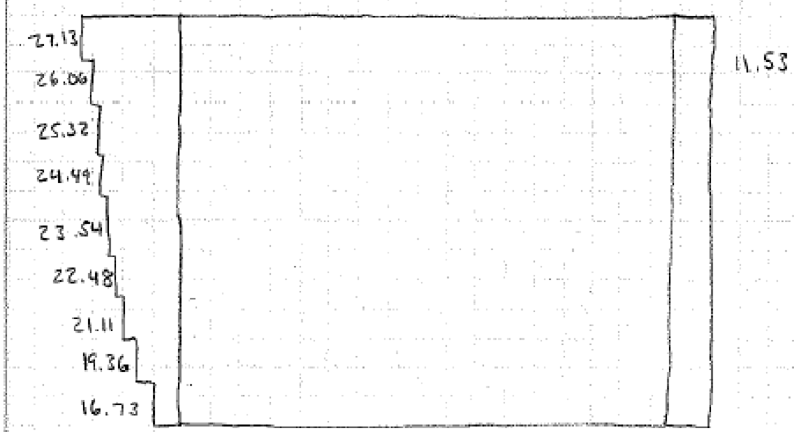
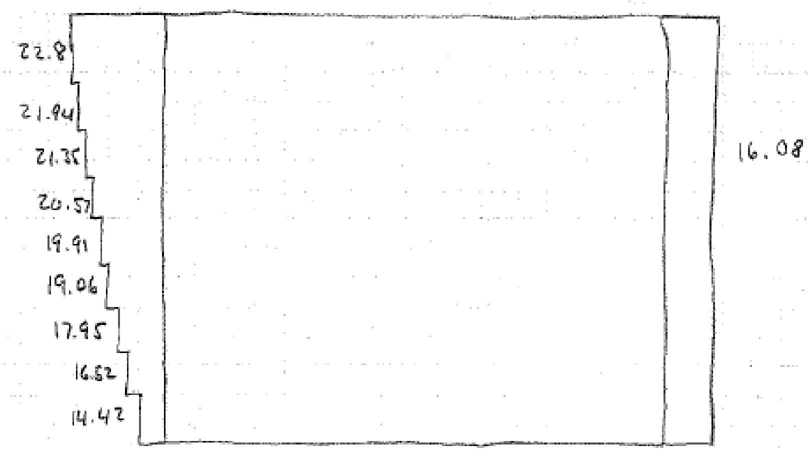


Area 1,2,3 N-S



Area 1,2,3. E-W

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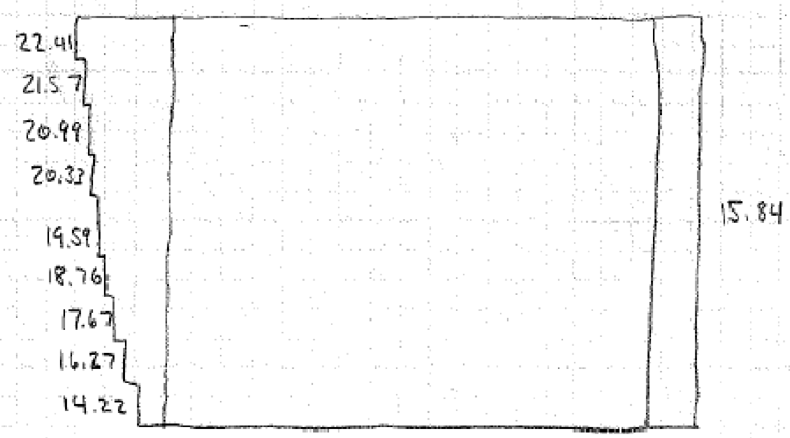
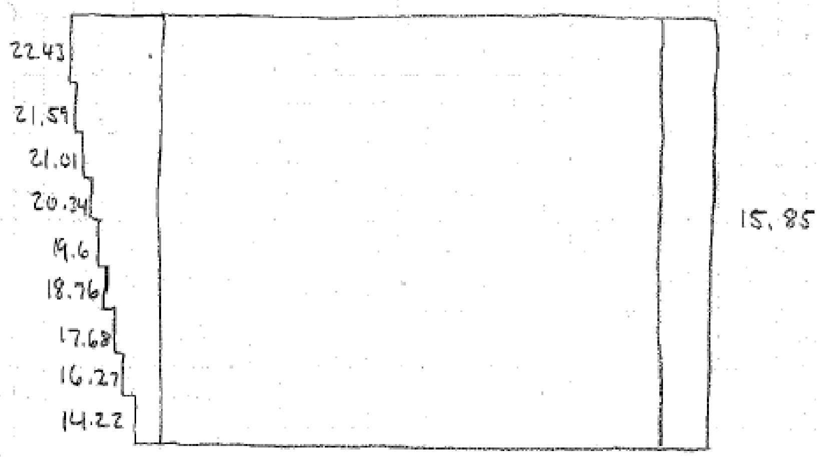
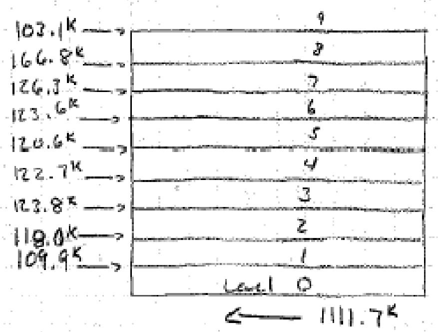


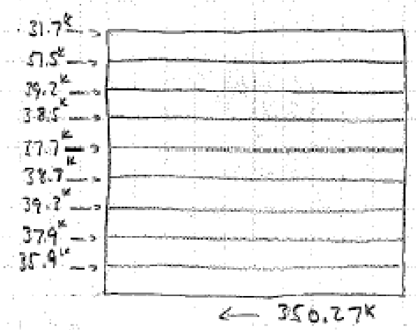
Areas 4,5,6,7 N-S



Areas 4,5,6,7 E-W

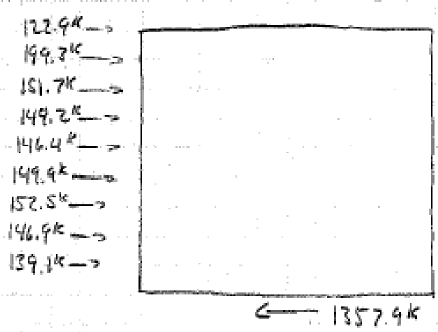
Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (8) | 11

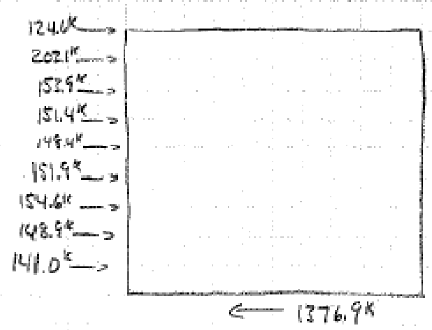




Areas 1,2,3 N-S

Areas 1,2,3 E-W





Areas 4,5,6,7 N-S

Areas 4,5,6,7 E-W

$$M_{1\text{-}3\,NS} = 103.1(139.33) + 166.8(117.33) + 126.3(103.33) + 123.6(89.33) + 120.6 \times 75.33 + 122.7(61.33) + 123.8(46) + 118.0(30.66) + 109.9(15.33)$$

$$= 85603.77 \text{ ft-k}$$

Appendix F: Seismic Calculations

Determine New Seismic Loads | 16

Southern Building:

Location: Voorhees, NJ (Latitude: 39.84° Longitude: -74.93°)

Soil Classification: D

Occupancy: Medical

Material: Structural Steel

Structural System: Braced frames

1) a) $S_S = 0.249$, $S_1 = 0.057$ } According to USGS Ground Motion Parameter Calculator

b) Site Class D

c) $S_{MS} = F_a S_S = (1.6)(.249) = .3984$

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

$S_{M1} = F_v S_1 = (2.4)(.057) = .1368$

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

d) $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.3984) = .2656$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(.1368) = .0912$

2) a) $S_1 \geq 0.04$ $S_S > 0.15$ can not be assigned to Design Category A

b) Occupancy Category is IV because it is a hospital with emergency treatment facilities

c) $S_1 < 0.75$ not assigned to SDC E or F

d) SDC = C from Tables 11.6-1 & 11.6-2

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Flowchart 6.6: Diaphragms are rigid

Occupancy Category: IV

R = 3.25 for ordinary steel concentrically braced frames for both directions

I = 1.5

$T_a = C_t h_n^x = .02(142)^{.75} = .9623$

$T_L = 6 > T_a$

$C_s = \frac{S_{D1}}{T(R/I)} \leq \frac{S_{DS}}{(R/I)}$

$\Rightarrow \frac{.0912}{.9623(3.25/1.5)} \leq \frac{.2656}{(3.25/1.5)} \Rightarrow .043756 \quad .1226$

$C_s = .04374$

Weights from Tech #1

Level	W	W w/ helipad (see below)
9	1556.8	2162.2
8	788.8	788.8
7	1294.5	1294.5
6	515.75	515.75
5	3350.9	3350.9
4	4709.4	4709.4
3	5212.5	5212.5
2	4133.2	4133.2
1	5592.2	5592.2
Sum	27154.1	27759.5

Weight of helipad = (10/12")(150 pcf) = 125 psf
Area of helipad = 3(31.33)(45) = 4229.6 ft²
Beam weight = (10/12)(20/12)(6)(31.33)(150) = 39162.5 lb
= (10/12)(20/12)(4)(45)(150) = 37500 lb

Total weight of HP = 39162.5 + 37500 + (125)(3)(31.33)(45)

= 605.356 K

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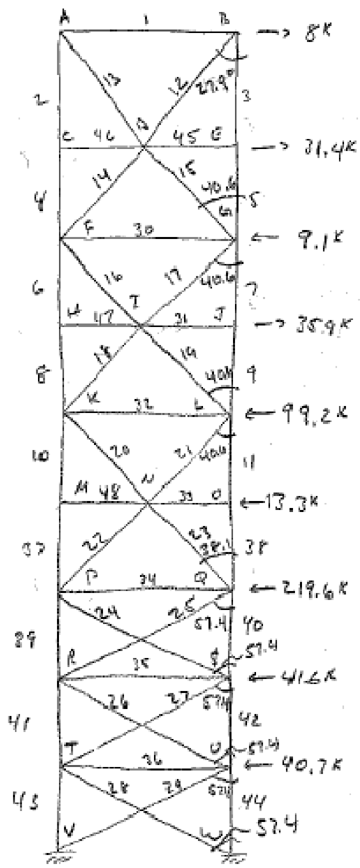
$V = C_s W = .04375(27759.5) = 1314.13^k$

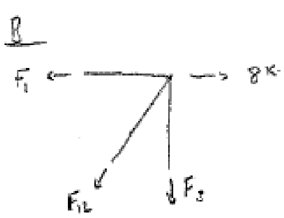
$k = .75 + .5T = .75 + .5(.9623) = 1.23$

see excel sheet for story shear distribution

Story	Lateral Force (K)	Story Shear (K)
9	300.4	300.4
8	86.67	387.08
7	121.65	508.72
6.5	40.52	549.24
5	213.47	762.71
4	232.97	995.69
3	181.03	1176.71
2	87.15	1263.86
1	50.27	1314.13
Sum	1314.132	

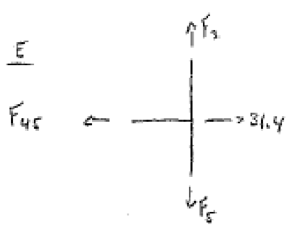
Appendix G: Free Body Diagram





$\Sigma F_x = 8 - F_1 - \sin 27.9 F_{12} = 0$

$\Sigma F_y = -F_3 - \cos 27.9 F_{12} = 0$



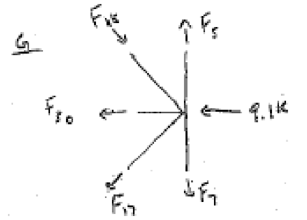
$\Sigma F_x = 31.4 - F_{45} = 0$

$F_{45} = 31.4$

$\Sigma F_y = F_3 - F_5 = 0$

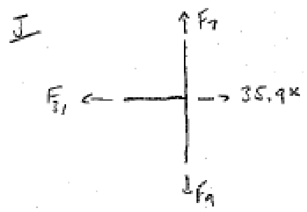
$F_3 = F_5$

2



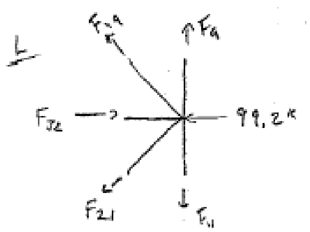
ΣF_x: $-9.1 + \sin 40.6 F_{15} - \sin 40.6 F_{17} - F_{30} = 0$

ΣF_y: $F_5 - \cos 40.6 F_{15} - \cos 40.6 F_{17} - F_7 = 0$



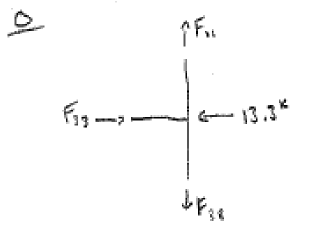
ΣF_x: $F_{31} = 35.9^k$

$\Sigma F_y = F_7 = F_5$



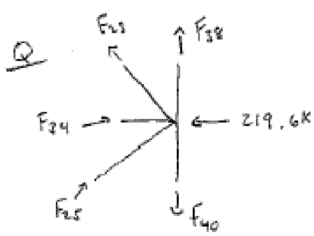
$\Sigma F_x = -99.2 + F_{32} - \sin 40.6 F_{19} - \sin 40.6 F_{21} = 0$

$\Sigma F_y = F_9 + \cos 40.6 F_{19} - F_{11} - \cos 40.6 F_{21} = 0$



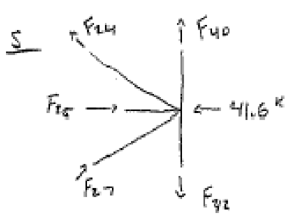
$\Sigma F_x = 13.3 = F_{33}$

$\Sigma F_y = F_{11} = F_{38}$



$\Sigma F_x = -219.6 + F_{34} - \sin 38.1 F_{23} + \sin 57.4 F_{25} = 0$

ΣF_y: $F_{38} + \cos 38.1 F_{23} - F_{40} + \cos 57.4 F_{25} = 0$

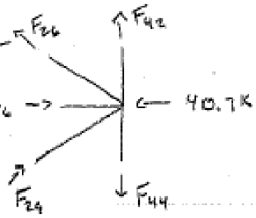


$\Sigma F_x = -41.6 - \sin 57.4 F_{24} + F_{35} + \sin 57.4 F_{27} = 0$

ΣF_y: $F_{40} + \cos 57.4 F_{24} - F_{42} + \cos 57.4 F_{27} = 0$

3

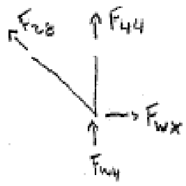
U



$\Sigma F_x = -40.7 - \sin 57.4 F_{26} + F_{36} + \sin 57.4 F_{24} = 0$

$\Sigma F_y = F_{42} + \cos 57.4 F_{26} + \cos 57.4 F_{24} - F_{44} = 0$

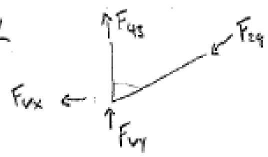
W



$\Sigma F_x = F_{wx} = \sin 57.4 F_{28}$

$\Sigma F_y = F_{44} + F_{wy} + \cos 57.4 F_{28} = 0$

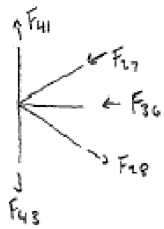
V



$\Sigma F_x = F_{vx} = \sin 57.4 F_{29}$

$\Sigma F_y = F_{43} + F_{vy} - \cos 57.4 F_{29} = 0$

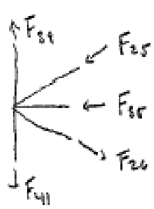
I



$\Sigma F_x = \sin 57.4 F_{28} - \sin 57.4 F_{27} - F_{36} = 0$

$\Sigma F_y = F_{41} - \cos 57.4 F_{27} - \cos 57.4 F_{28} - F_{43} = 0$

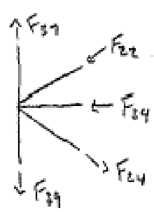
R



$\Sigma F_x = -\sin 57.4 F_{25} - F_{35} + \sin 57.4 F_{26} = 0$

$\Sigma F_y = F_{39} - \cos 57.4 F_{25} - \cos 57.4 F_{26} - F_{41} = 0$

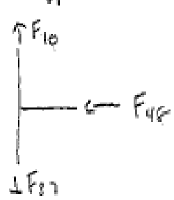
P



$\Sigma F_x = -\sin 38.1 F_{22} - F_{34} + \sin 57.4 F_{24} = 0$

$\Sigma F_y = F_{37} - \cos 38.1 F_{22} - \cos 57.4 F_{24} - F_{39} = 0$

M

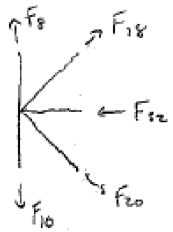


$F_{48} = 0$

$\Sigma F_y = F_{10} = F_{37}$

4

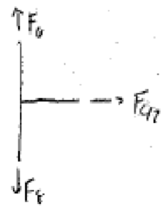
K



$\Sigma F_x = \sin 40.6 F_{18} + \sin 40.6 F_{20} - F_{22} = 0$

$\Sigma F_y : F_8 + \cos 40.6 F_{18} - \cos 40.6 F_{20} - F_{10} = 0$

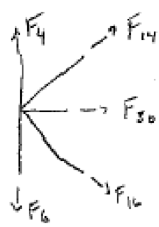
H



$F_{47} = 0$

$\Sigma F_y : \quad F_6 = F_8$

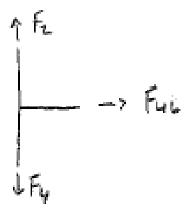
F



$\Sigma F_x = \sin 40.6 F_{14} + F_{30} + \sin 40.6 F_{16} = 0$

$\Sigma F_y : F_4 + \cos 40.6 F_{14} - F_6 - \cos 40.6 F_{16} = 0$

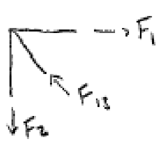
C



$F_{46} = 0$

$F_2 = F_4$

A



$\Sigma F_x : \quad F_1 = \sin 27.9 F_{13}$

$\Sigma F_y : \quad F_2 = \cos 27.9 F_{13}$

SYSTEM

$$\Sigma M_w = 40.7(15.33) + 41.6(30.66) + 219.6(46.00) + 13.3(61.33) + 99.2(75.33) + 35.9(89.33) + 9.1(103.33) - 31.4(117.33) - 8(140) - V_y(24) = 0$$

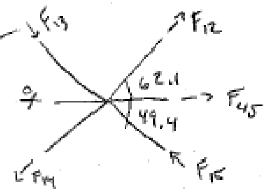
$$24 V_y = 13218.6$$

$$V_y = 550.78 \qquad W_y = -550.78$$

$$\Sigma F_x = W_x - V_x = 348.1$$

5

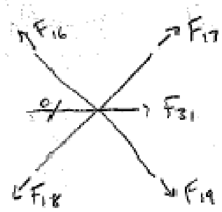
D



$\Sigma F_x = F_{12}\cos 62.1 + F_{13}\cos 62.1 + F_{45} - F_{15}\cos 49.4 - F_{14}\cos 49.4 = 0$

$\Sigma F_y = F_{12}\sin 62.1 + F_{15}\sin 49.4 - F_{13}\sin 62.1 - F_{14}\sin 49.4 = 0$

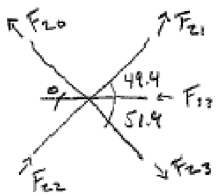
I



$\Sigma F_x = F_{17}\cos 49.4 + F_{19}\cos 49.4 - F_{16}\cos 49.4 - F_{18}\cos 49.4 + F_{31} = 0$

$\Sigma F_y = F_{17}\sin 49.4 + F_{16}\sin 49.4 - F_{18}\sin 49.4 - F_{19}\sin 49.4 = 0$

N



$\Sigma F_x = F_{21}\cos 49.4 + F_{23}\cos 51.9 + F_{22}\cos 51.9 - F_{20}\cos 49.4 - F_{33} = 0$

$\Sigma F_y = F_{21}\sin 49.4 + F_{20}\sin 49.4 + F_{22}\sin 51.9 - F_{23}\sin 51.9 = 0$

6

$F_1 = 4.00^k$ T
$F_2 = 7.578^k$ T
$F_3 = 7.636^k$ C
$F_4 = 7.578^k$ T
$F_5 = 7.636^k$ C
$F_6 = 51.303^k$ T
$F_7 = 45.501^k$ C
$F_8 = 51.303^k$ T
$F_9 = 45.501^k$ C
$F_{10} = 75.965^k$ T
$F_{11} = 59.498^k$ C
$F_{12} = 8.64^k$ T
$F_{13} = 8.57^k$ C
$F_{14} = 30.31^k$ T
$F_{15} = 30.23^k$ C
$F_{16} = 27.28^k$ C
$F_{17} = 19.64^k$ T
$F_{18} = 46.96^k$ T
$F_{19} = 54.60^k$ C
$F_{20} = 14.48^k$ T
$F_{21} = 36.17^k$ C
$F_{22} = 48.54^k$ C
$F_{23} = 27.63^k$ T

$F_{24} = 137.55^k$ T
$F_{25} = 177.15^k$ C
$F_{26} = 178.58^k$ T
$F_{27} = 185.71^k$ C
$F_{28} = 203.462^k$ T
$F_{29} = 210.0^k$ C
$F_{30} = 1.994^k$ C
$F_{31} = 15.58^k$ T
$F_{32} = 19.99^k$ C
$F_{33} = 13.79^k$ C
$F_{34} = 86.37^k$ C
$F_{35} = 1.37^k$ C
$F_{36} = 4.53^k$ C
$F_{37} = 36.26^k$ C
$F_{38} = 59.49^k$ C
$F_{39} = 75.96^k$ T
$F_{40} = 57.58^k$ T
$F_{41} = 227.8^k$ C
$F_{42} = 231.64^k$ T
$F_{43} = 437.72^k$ C
$F_{44} = 440.84^k$ T
$F_{45} = 31.37^k$ T
$F_{46} = 0$

$F_{47} = 0$
$F_{48} = 0$

V_x $176.8^k \rightarrow$
$V_y = 550.78^k \uparrow$

$W_x = 171.36^k \rightarrow$
$W_y = 550.78^k \downarrow$

Appendix H: Lateral Force Distribution

Lateral Force Dist. 1c

Story 9

$\Delta_{xx} = 5.89'' - 3.88 = 2.01''$
$\Delta_{yy} =$ N/A
$h = 22.66' = 272''$

$P_{9i} = 199.36^k$

Frames in x: M7-N7, M11-N11, L16-L.7 16, L15-M15, L14-M14, L13-M13

M7:

$$K = \frac{AE}{L}\cos^2\theta = \frac{(2)(13.5)(29000)}{\underbrace{\sqrt{(272)^2+(135)^2}}_{303.65}}\left(\frac{135}{\sqrt{272^2+(135)^2}}\right)^2 = 509.7\ ^k/in$$

M11:

$$K = \frac{(2)(13.5)(29000)}{\sqrt{272^2+270^2}}\left(\frac{270}{\sqrt{272^2+270^2}}\right)^2 = 509.7\ ^k/in$$

L16:

$$K = \frac{(2)(13.5)(29000)}{\underbrace{\sqrt{104^2+272^2}}_{291.2}}\left(\frac{104}{\sqrt{104^2+272^2}}\right)^2 = 342.95\ ^k/in$$

L15

$$K = \frac{2(13.5)(29000)}{\sqrt{(272)^2+(135)^2}}\left(\frac{135}{\sqrt{272^2+135^2}}\right)^2 = 509.7$$

L14 $K = 509.7$ L15 $K = 509.7$

$\Sigma K = 2795.9\ ^k/in$

%P in Brace:
M7 = 17.63%
M11 = 17.63%
L16 = 11.86%
L15 = 17.63%
L14 = 17.63%
L13 = 17.63%

P in Brace
M7: 24.12^k
M11: 24.12^k
L16: 16.23^k
L15: 24.12^k
L14: 24.12^k
L13: 24.12^k

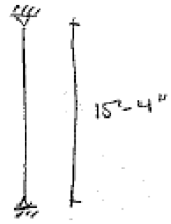
Appendix I: Spot Checks

Column Check 2c.1

$P_r = 753.4$

$K = 1.0$

$KL = 15.33'$



From Table 4-1 in AISC: W14x109 is ok. $\phi P_n = 1190^k > P_r \therefore$ ok ✓

Brace Check 2c.2

$\phi P_r = 210.12^k$ $L_b = 17.33'$ $L_r = 23.14'$ $\theta = 41.5°$

$$\beta_{br} = 1/\phi \, (8P_r/L_b) = 1/.75 \, (8(210.12)/17.33) = 129.3$$

$$\Rightarrow \frac{A_{br} E}{L_r} \cos^2\theta = 129.3$$

$$A_{br} = \frac{(129.3)(23.14)}{(29000)(\cos^2 41.5)} = .184$$

$$P_{brq} = 210.12$$

$$A_{br} = \frac{210.12}{.9(46)} = 5.08$$

$$A_{br} = \frac{210.12(23.14/17.33)}{.9(46)} = 6.78$$

$$A_{req} = 5.08 + 6.78 = 11.86 \ in^2$$

$$A_{HSS8\times8\times1/2} = 13.85\,in^2 \geq A_{req} \therefore ok \checkmark$$

Appendix J: Drift

Drift 3c

Drift = $0.010 h_{sx}$ $\delta_x = 3.17781''$

Drift = $\frac{C_d \delta_{xe}}{I}\left(\frac{T}{T_x}\right) = \frac{(3.25)(3.17781)}{1.5}\left(\frac{.9623}{.6283}\right) = 10.545''$

Max $\Delta = .010(140')(12\text{in}/1\text{ft}) = 16.8'' >$ Drift $\therefore$ ok ✓

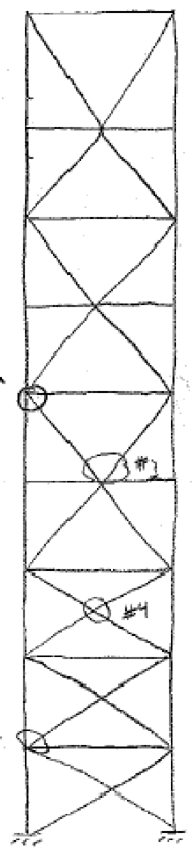
$\delta_y = 2.27774''$

Drift = $\frac{(3.25)(2.27774)}{1.5}\left(\frac{.9623}{.6283}\right) = 7.56'' \le$ Max $\Delta = 16.8''$

See excel

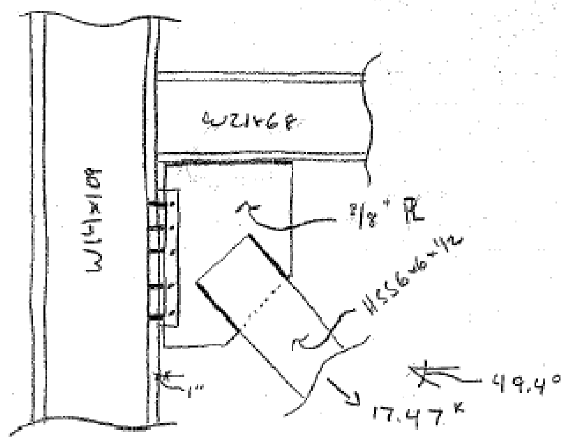
Appendix K: Connections

Connection Design 1d



Design the circled connections

1)



Try 3/8" E70xx weld

$L_{weld} \geq w$ $w = 6"$ let $L_{weld} = 6"$

Weld rupture: $\emptyset R_n = 1.392(6)(6)(2) = 200.45^K > P_u$

base metal: $\emptyset R_n = 2(.75)(.6)(58)(3/8)(6) = 117.45^K > P_u$

5/16" weld 6" long is ok ✓

Equilibrium Model Equation for Plate:

$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$

$e_c = 7.15"$
$e_b = 10.55"$
$\theta = 40.6°$
$\tan\theta = .857$

$\alpha - \beta(.857) = 1.892$ let $\alpha = \beta = x$

$x - .857x = 1.892$

$.143x = 1.892$

$x = 13.2307$ or $\alpha = 7$, $\beta = 6$ $\approx .857$

2d

$\alpha = 7 = 1" + \frac{1}{2} R$ $\beta = 4"$

$R = 12"$ $r = \sqrt{(e_c+\alpha)^2 + (e_b+\beta)^2} = \sqrt{(7.15+16)^2 + (10.55+4)^2}$

Limit states: $= 19.61$

a) Gusset - Beam
- Gusset - Beam weld
- Beam Web Yield
- Beam Web Crippling

b) Gusset - Column
- Bolt Shear
- Bolt Bearing & Tear out on Angles
- Bolt Bearing & Tear out on column web
- Shear Yield of Angles
- Shear Rupture of Angles
- Block Shear of Angles
- Bolt Bearing & Tearout on Gussett Plate

c) HSS Shape
- Tension Yield
- Tension Rupture

$V_c = \frac{\beta}{r} P_u = \frac{4}{19.6}(17.47) = 3.56^k$ $V_B = \frac{e_b}{r} P_u = \frac{10.55}{19.6}(17.47) = 9.4^k$

$H_c = \frac{e_c}{r} P_u = \frac{7.15}{19.6}(17.47) = 6.37^k$ $H_c = \frac{\alpha}{r} P_u = \frac{16}{19.6}(17.47) = 5.35^k$

a) Gusset - Beam

$V_b = 5.35 + 12.85 = 18.2\ k$

try 3/8" weld both sides

Weld: $\phi R_n = 1.392(1.5)(3)(12)(2) = 125.3^k >> V_b$ ✓

R: $\phi R_n = 2(.75)(.6)(58)(3/8)(12) = 195.75^k >> V_b$ ✓

b) Gussett - column

$V_u = 3.56 + 21.57 = 25.13^k$ try (3) 3/4" ϕ 325 N bolts
$T_u = 6.37$

i) Shear Stress in bolts

$f_v = \frac{V_u}{(\#bolts)(A_b)} = \frac{28.95}{(3)(.442)} = 21.83\ ksi$

ii) Tensile stress in bolts

$r_{ut} = \frac{3.98}{29.8} = .13356$
$\uparrow \phi r_{nt}$

3d

iii) Determine if there's Prying: try (2) $L\ 4 \times 4 \times 1/4 \times 9''$

$P = 3''$

$b = 2.5 - 5/32 - 3/8 = 1.969''$

$r_{ut} = .13356 < \dfrac{.9(58)(3)(1/4)^2}{4(1.969)} = 1.24$ ∴ no prying ✓

iv) Bolt Shear

$\phi_v r_n = 3(31.8) = 95.4^k > 25.13^k$ ∴ ok ✓

v) Bolt Bearing & Tearout on Angles

Bearing: $\phi r_n = .75(2.4)(3/4)(58)(1/4)2 = 39.15$

Tearout on edge: $\phi r_n = .75(1.2)(1.5)(58)(1/4)2 = 39.15^k$

Tearout in middle: $\phi r_n = .75(1.2)(3)(58)(1/4)2 = 78.3^k$

$\phi R_n = 39.15 + 39.15 + 39.15 = 117.45^k > 25.13^k$ ∴ ok ✓

vi) Bolt Bearing & Tearout on column will not control

vii) Shear Yield of Angles:

$\phi R_n = \phi F_y A_g = .9(36)(1.94)(2) = 125.712^k > 25.13^k$ ∴ ok ✓

viii) Shear Rupture of Angles: * controls connection

$\phi R_n = \phi F_u A_e$
$= (.75)(58)(.8733)$
$= 37.98^k > 25.13^k$ ∴ ok ✓

$A_e = U A_n$
$A_e = .82[(3.88) - 2(7/8)](1/4)(2)$
$= .8733$

$U = 1 - \bar{x}/L$
$= 1 - 1.08/6$
$= .82 > .6$ ∴ ok

ix) Block Shear of Angles:

$\phi R_n = \phi[.6 F_u A_{nv} + U_{bs} F_u A_{nt}] \leq \phi[.6 F_y A_{gv} + U_{bs} F_u A_{nt}]$

$A_g = 7.25(1/4)(2) = 3.625\ in^2$

$A_{nt} = 2(1/4)(2 - 1/2(7/8)) = .78125$

$A_{nv} = 3.625 - (2.5)(2)(1/4)(7/8) = 1.094$

$\phi R_n = .75[(.6)(58)(1.094) + 1(58)(.78125)] \leq (.75[.6(36)(3.625) + (58)(.78125)]$

$= 62.54 \leq 92.71$ $\phi R_n > 25.13^k$

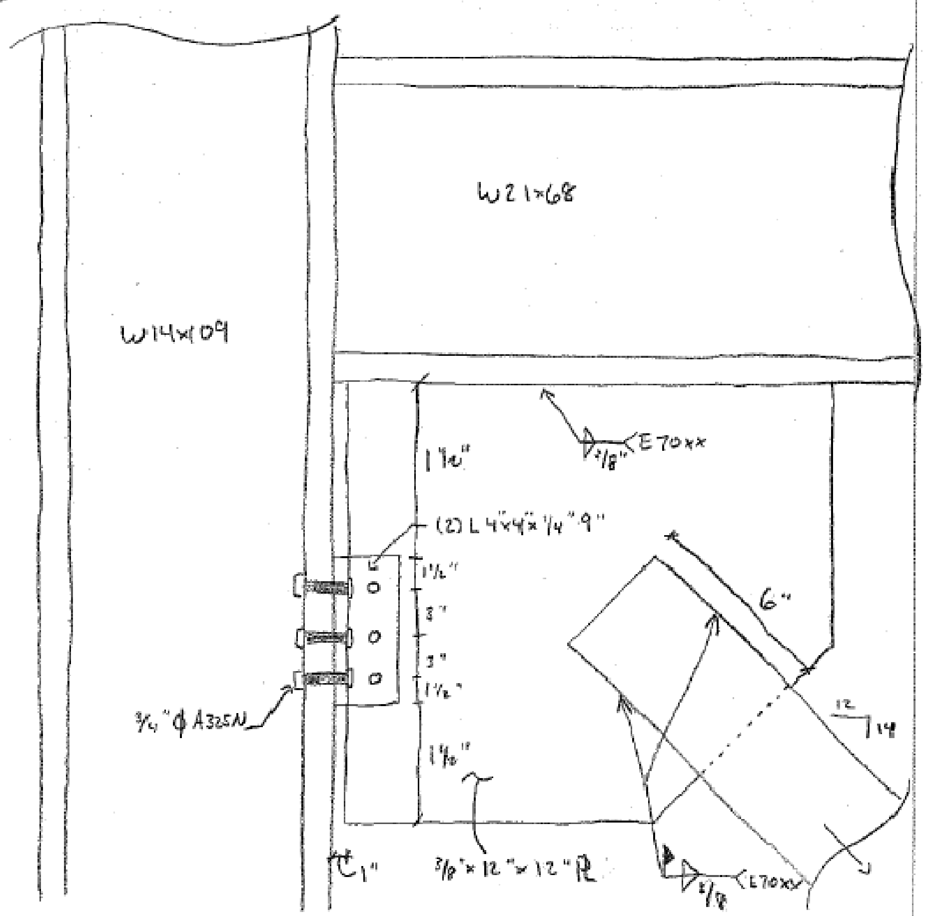
controls

4d

x) Bolt Bearing & Tearout on Gusset Plate does not control

Gusset Size = + 4.5 1.5 = 20.75" = "

NTS



c) HSS Tension Rupture:

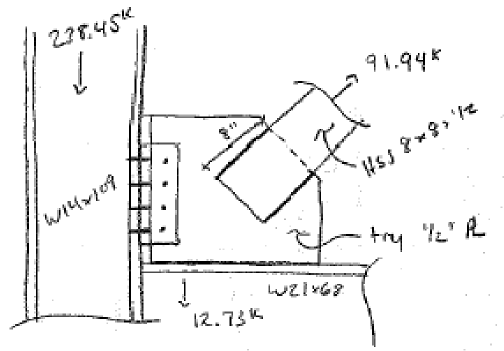
$\phi R_n = .75(58)A_e$ $\quad A_e = UA_n$ $\quad A_n = 9.74 - (2)(1/2)(3/8)$

$= .75(58)(5.85)$ $\quad A_e = 5.85$ $\quad = 9.365\,in^2$ $\quad U = 1 - 2.25/6$

$= 254.6 > 17.5^k \therefore ok \checkmark$ $\quad = .625$

5d

2)



a) Weld Rupture: $\phi R_n = 1.392(6)(8)(4) = 267.26^K \geq 91.94^K \therefore ok$ ✓ (Try 3/8")

b) Base Metal: $\phi R_n = 2(.75)(.6)(58)(1/2)(8) = 208.8^K \geq 91.94^K \therefore ok$

c) Equilibrium Model Equation for Plate

$e_c = 7.15"$
$e_b = 10.55"$
$\theta = 32.57°$

$\tan\theta = .638$

$\frac{e_c + \alpha}{e_b + \beta} = .638$ let $\alpha = 23"$ & $\beta = 36"$

$\frac{7.15 + \ }{10.55 + \ } = .637 \approx .638 \therefore ok$ ✓

$r = \sqrt{(7.15+23)^2 + (10.55+36)^2} = 55.46"$

$V_c = \frac{\beta}{r} P_u = \frac{9}{23.02}(91.94) = 35.96$ $V_B = \frac{10.55}{23.02}(91.94) = 42.14$

$H_c = \frac{(7.15)}{23.02}(91.94) = 28.56$ $H_B = \frac{\ 5}{23.02}(91.94) = 19.97$

$\alpha = 5 = 1" + 1/2\, PL$
$PL = 8"$

d) Gusset - Beam: $V_u = 42.14 + 12.73 = 54.87^K$

use 7/8" weld both sides

i) Weld: $\phi R_n = 1.392(1.5)(6)(44)(2) = 551.23^K \gg 54.87^K \therefore ok$ ✓

Plate Rupture: $\phi R_n = 2(.75)(.6)(58)(1/2)(44) = 861.3^K \gg 54.87^K \therefore ok$ ✓

e) Gusset - Column:

$V_c = 35.96 + 238.45 = 274.41^K$

$T_c = H_c = 28.56^K$

try (10) 3/4" ϕ A350N bolts

6d

i) Shear Stress in Bolts:

$f_v = \frac{274.41}{(10)(.442)} = 62.08$ ksi

ii) Tensile stress in bolts:

$r_{ut} = \frac{28.56^k}{29.8} = .958$

iii) Determine if prying occurs: try (2) L 4"x4"x 1/4"

$p = 3"$

$b = 2.5 - 1/8 - 3/8 = 2"$

$r_{ut} = .958 \leq \frac{.9(58)(3)(1/4)^2}{4(2)} = 1.223$ ∴ ok no prying ✓

iv) Bolt Shear:

$\phi R_v = 10(31.8) = 318^k > 274.4^k$ ∴ ok ✓

v) Bolt Bearing & Tearout on Gusset PL:

Bearing: $\phi r_n = .75(2.4)(3/4)(58)(3/8) = 29.36^k$

Tearout on edge: $.75(1.2)(1.5)(58)(1/4)2 = 39.15^k$

Tearout in middle: $.75(1.2)(3)(58)(1/4)(2) = 78.3^k$

$\phi R_n = 29.36(10) = 293.6^k > 274.4^k$ ∴ ok ✓

vi) Bearing & tearout will not control for column flange

vii) Shear Yield of Angles:

$\phi R_n = .9(36)(1.94)(2) = 125.7^k < 274.4^k$ ∴ no good ✗

$= .9(36)(4.61)(2) = 298.73^k > 274.4^k$ ∴ ok ✓ try (2) L 4"x 4"x 5/8

viii) Shear Rupture of Angles:

$U = 1 - 1.22/30 = .959$ $A_n = [4.61)2 - [2(7/8)(5/8)] = 8.126$

$A_e = 7.79$ $\phi R_n = .75(58)(7.79) = 338.9^k > 274.4$ ∴ ok ✓

71

ix) Block shear of Angles:

$A_g = (28.875)(5/8)(2) = 36.09\,in^2$ $\quad A_{nt} = 2(5/8)(2 - 1/2(7/8)) = 1.953$

$A_{nv} = 36.09 - (9.5)(2)(5/8)(7/8) = 25.7\,in^2$

$\phi R_n = .75[.6(58)(25.7) + (58)(1.953)] \leq .75[(.6)(36)(36.09) + 58(1.953)]$

$= 755.73 \leq 669.6 \geq 274.4^k \therefore ok$ ✓

controls

x) Bolt Shear & Tearout on Angles does not control

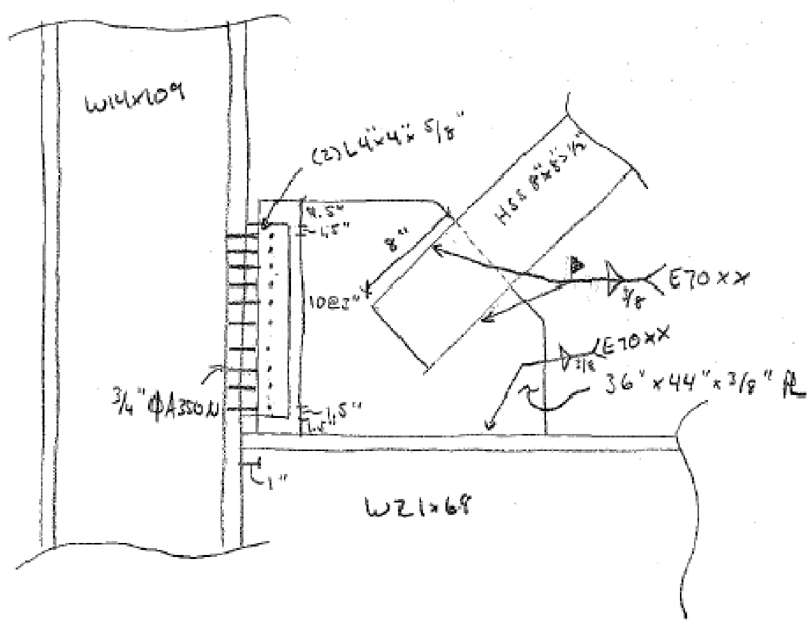
f) HSS Tension Rupture:

$U = 1 - 3/8 = .625$ $\quad A_n = 13.5 - 2(1/2)(3/8) = 13.125$

$A_e = .625(13.125) = 8.2$

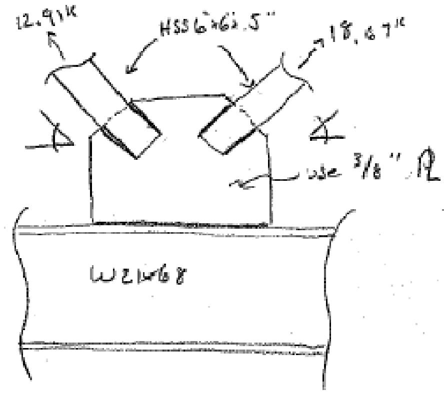
$\phi R_n = .75(58)(8.2) = 356.8^k > 91.94^k \therefore ok$ ✓

g) Gussett size:



8d

3)



a) Check Weld in HSS members: Assume 7/8" E70xx Weld:

$\phi R_n = 1.369(6)(6)(4) = 197.136^K > 18.67^K > 12.91^K$ ∴ ok ✓

b) Base metal:

$\phi R_n = 2(.75)(.6)(58)(3/8)(6) = 117.45^K > 18.67^K > 12.91^K$

c) HSS Tension Rupture:

$U = 1 - 2.25/6 = .625$ $A_n = 9.365$ $A_e = 5.85$

$\phi R_n = .75(58)(5.85) = 254.6^K > 18.67^K > 12.91^K$

d) Web Yielding:

$P_u = .55 + \sin(49.4)18.67 + \sin(49.4)12.91 = 24.53^K$

e) Gusset - Beam: use 7/8" weld & 18" x 18" x 3/8" PL

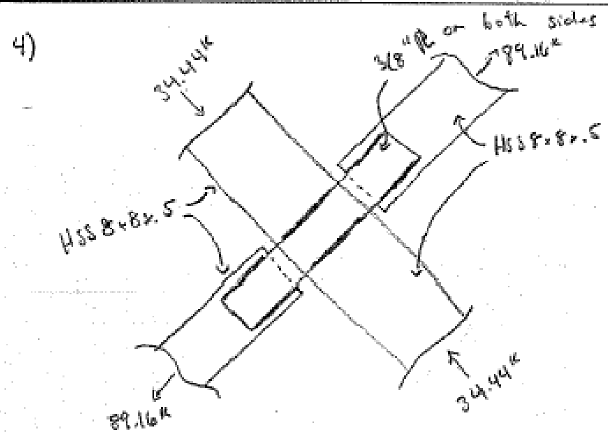
$\phi R_n = 1.392(1.5)(6)(18")(2) = 451 > 24.53^K$ ∴ ok ✓

PL: $2(.75)(.6)(58)(3/8)(18) = 352.4^K > 24.53^K$ ∴ ok ✓

Connection will not need stiffener plate.

9d

4)



use 3/8" E70XX welds

a) Plate rupture:

$\phi R_n = .75 F_u A_e = .75(58)(6)(3/8)(2) = 195.75^k > 89.16^k$ ∴ ok ✓

b) Plate yield:

$\phi R_n = .9 F_y A_g = .9(36)(6)(3/8)(2) = 145.8^k > 89.16^k$ ∴ ok ✓

c) weld rupture:

$\phi R_n = 1.392(6)(8)(4) = 267.3^k > 89.16^k$ ∴ ok ✓

d) base metal:

$\phi R_n = 2(.75)(.6)(58)(3/8)(8) = 156.6^k > 89.16^k$ ∴ ok ✓

Appendix L: Electrical

Electric 1e

ZG1 – (14) 40W, 120V – 1   use circuit LEQPH7A1

ZG2 – (4) 116W, 120V – 2   Distance to circuit ≈ 146'

ZG3 – (1) 464W, 120V – 3

A_1 = .33 A
A_2 = .9667 A
A_3 = 3.866 A

Put ZG1 on one circuit & ZG2, ZG3 on seperate circuit

continuous loads   temp (106°F – 118°F)

Total A_1 = .333(14) = 4.662 (1.25) = 5.83 A (1/.71) = 8.21 A
A_2 = 7.733 = 7.733 (1.25) = 9.6661 A (1/.71) = 13.61 A

use 60°C 14 AWG copper wire in ½" metal conduit

Voltage Drop:

Assume PF = 95%   VD = 1.833

VD_1 = 4.662(146)(1.833)(2)/1000 = 2.49% < 3% ok ✓
VD_2 = 7.733(146)(1.833)(2)/1000 = 4.138 > 3% not ok

Split A_2 & A_3

VD_2 = 2.069% < 3% ∴ ok

VD_3 = 2.069% < 3% ∴ ok

size Panel → see excel

Appendix M: Acoustical

Acoustics 1f

Helicopter produces 105 dB

Glazing on back side of bed tower has an STC = 30

	TL	STC
1/4" glass	31	31
1/4" laminated + 3/16 glass		
w/ 4" Airspace	48	48
w/ 2" Airspace	44	45
(2) 1/4" laminated w/ 1/2" Air space	40	42

$NR = TL + 10\log a_2/s$

Assume $10\log a_2/s = 0$
$TL = 31$ @ 500 Hz

$NR = 31$ dB

L_1:

$$\frac{I_1}{I_2} = \left(\frac{d_2}{d_1}\right)^2$$

I_1: $L_1 = 10\log \frac{I_1}{I_0}$ $d_1 = 2$ m

$$105 = 10\log \frac{I_1}{10^{-12}}$$

$I_1 = .0316$

M → P 9 → 6

8 → 3

$$d_2 = \sqrt{(51.66)^2 + (122)^2 + (50.66)^2} = 141.846' = 43.23\,m$$

$$\frac{.0316}{I_2} = \left(\frac{43.23}{2}\right)^2$$

$I_2 = 6.764e\text{-}5$

$L_2 = 10\log \frac{6.764e\text{-}5}{10^{-12}} = 78.3$ dB ≈ sound of a minibike

$NR = L_2 - L_3$

2f

$31 = 78.3 - L_2$

$L_2 = 47.3$ dB ≈ Average Home / office noise

would prefer 30 dB due to night patients

with (2) 1/4" laminated glass w/ 1/4" air space:

$L_3 = 78.3 - 40 = 38.3$ dB

with 1/4" laminated & 3/16" glass w/ 2" airspace:

$L_2 = 78.3 - 44 = 34.3$ dB

with 4" Airspace:

$L_2 = 78.3 - 48 = 30.3$ dB

use the 1/4" laminated glass & 3/16" glass w/ 2" airspace

dB in room = 34.3 dB is a quiet office / conversation