Final Thesis Report

University of Maryland College Park Dorm Building 7
College Park, MD

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The Pennsylvania State University
Department of Architectural Engineering
Senior Thesis 2008-2009

University of Maryland College Park Dorm Building 7

College Park, Maryland

Ryan L. Solnosky Structural Option

Design & Construction Team

- Owner: Capstone Development
- Architect: Design Collective
- Structural Engineer: Hope Furrer Associates Inc.
- MEP Engineer: Burette, Koehler, Murphy & Associates
- Construction Manager: Whiting-Turner Contracting

General Information

- Location: 6801 Preinkert Drive College Park, MD 20742
- Occupancy: Residential
- Size: 133,000 square feet
- Height: 9 stories with a Maximum height of 94 feet
- Construction Dates: July 2008-Janurary 2010
- Building Cost: $23.5 million
- Delivery Method: Design-Bid-Build with a CM-at-Risk

Architecture

- An unsymmetrical U-shape with a central courtyard for gathering
- Houses 370 bedrooms, study lounges, seminar spaces and resident life offices
- Designed to reach a LEED Gold rating
- Brick veneer & light gage stud cavity wall with cast stone accents
- EPDM fully adhered roof system bonded to rigid insulation

Structural Design

- The lower 2 stories of reinforced concrete columns, beams, and shear walls
- The upper 7 stories are comprised of light gage bearing walls to support joists
- 3" thick concrete slab on Hambro open web joists with a depth of 16"
- Light-gage shearwalls on the upper 7 floors.
- Auger cast grout piles 16" in diameter with a 65 ton allowable load capacity

Mechanical Design

- 1500CFM cooling/heating split system closet type units for apartments
- Apartments are provided by natural ventilation through operable windows
- 2800CFM Rooftop Units
- 9000CFM Fans for stairwell Pressurization

Electrical/Lighting Design

- The service voltage will be 480/277-volt, 3-phase, 4-wire, and 60 hertz
- A separate 208/120-volt, 3-phase, 4-wire feeder to supply the residential rooms
- A main distribution switchboard (SWBD) rated at 2500 amperes
- A diesel emergency generator will supply backup to the emergency systems
- 2'x2' 277V, 32 watt parabolic fluorescent fixtures with electronic ballasts

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Executive Summary

The University of Maryland College Park Dorm Building 7 (Building 7) is the final stage of the south campus master plan at the University of Maryland. Building 7 is an eight story residential dorm in the shape of an unsymmetrical-U that compliments the adjacent two existing dorm buildings in architectural styles with its shape and material usage. This eight story-133,000 square feet residential building, houses 370 bedrooms, study lounges, seminar spaces and resident life offices. The layout of each floor is such that all of the rooms have an exterior view of the surrounding campus with a central corridor running the length of the building. The roof level houses the mechanical equipment along with the elevator and stair towers. Building 7 is also in the process of achieving a LEED Gold rating.

This report includes a seismic analysis of the Building 7 which the location was moved to San Diego, California which has a high seismic activity. San Diego was chosen based on its seismic activity and also because the San Diego Region has a University, The University of California at San Diego, since this building is a dorm this location makes it a good choice if USD would ever want a new dorm.

Building 7 was redesigned from the original Hambro Composite Joists and bearing walls with light gage shear walls to a more standard and reliable structural steel system. Structural steel was chosen for back in Technical Report 2 it was determined to be the most efficient for the cost. A new bay layout and also the locations of the new Special Concentric braced Frame had to be determined. A double loaded corridor was determined to be the best bay layout and the redesign was able to reduce the number of lateral frames as compared to the original (16 before to 10 at the end). Lateral connections were looked and were designed to meet the seismic requirements.

The AISC Steel Construction Manual, 13th Edition and Steel Seismic Design Manual were used as a basis for all of the structural steel designs. A Ram Structural Model was created to help with the analysis and the design of both the gravity and the lateral systems. Preliminary hand calculations and spot checks were performed to verify the computers results to ensure the design was valid. ASCE 7-05 was used to determine the required seismic loads and conditions along with all the other loading and general requirements. Advanced computer modeling along with connections were looked at for the MAE requirement.

Two breadth studies were conducted; the first was a green roof study. A green roof was designed to bring and add to the Green Standard and make the building more efficient. A water collection was also designed for both locations so that the roof runoff can be used to help reduce the water consumed by the sanitary system. The second breadth study was an acoustic study to see the impacts of changing the structural system to steel. It was determined that the new system is acceptable and recommendations were made to make the space more efficient at reducing sound leaks throughout.

Acknowledgements

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* Nick Mansperger

Burdette, Koehler, Murphy & Associates

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* Yefim Rubin

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* Parents
* Simon Miller
* Kim McKitish
* Joe Wilcher

Building Overview

The University of Maryland College Park Dorm Building 7 (Building 7) is the final stage of the south campus master plan at the University of Maryland. Building 7 is the corner stone of the south campus entrance for all to take part of as they approach the campus. Building 7 is an eight story residential dorm in the shape of an unsymmetrical-U that compliments the adjacent two existing dorm buildings in architectural styles with its shape and material usage.

Architecture

This eight story-133,000 square feet residential building, houses 370 bedrooms, study lounges, seminar spaces and resident life offices. The average floor to floor height is 10 feet on each floor with an average floor area of 12,000-15,500 square feet per floor, depending on shifts in the vertical plane. The layout of each floor is such that all of the rooms have an exterior view of the surrounding campus with a central corridor running the length of the building. The roof level houses the mechanical equipment along with the elevator and stair towers.

The façade and building envelope is comprised of light gage studs with a brick masonry veneer exterior around the entire building. There is rigid insulation on the exterior of the studs between the veneer with a 1.5 inch air cavity. The walls are filled with batt insulation and covered in drywall.

The windows are fixed casement aluminum windows with cast stone sills to accent them. In the regions where the wall sections are pulled away from the primary facade, the wall system is composed of composite metal panel and cast stone veneer panels. The roof system is an EPDM classification which is a fully adhered system comprised of a waterproof membrane that is bonded to rigid insulation by mechanical and chemical means with appropriate flashing at the base of the parapets and where the brick meets the top of the parapet.

Mechanical System

Building 7's mechanical system is for a residential space requirements with small areas using office requirements where needed. The corridors of Building 7 utilize two rooftop packaged heat pumps that supply heating cooling and ventilation to the corridors. Apartments and community areas utilize split system closet type heat pump units that provide heating and cooling only. Ventilation to these areas is not mechanically supplied but instead there is natural ventilation by the means of operable windows. The reason for this is to help gain LEED points. The exterior walls were also carefully designed to limit the amount of heat loss and gain through them, to better control the inside environment.

The mechanical heating and cooling units are all located on the roof level of Building 7 and there are 101 units on a concrete curb. The split system heat pumps have a range of 500-1500 CFM depending on the space they support. The packaged rooftop units on the north side supply 1680CFM while the south side units supply 2800 CFM. The stairwell pressurization fans for fire emergencies produce 9000 CFM for each stairwell and are run on the fire alarm system.

Electrical System

Building 7's electrical system is powered by PEPCO and they design as well as install the primary underground cables to the pad mounted transformer. The secondary cables to the building distribution system will be handled by the utility company. The service voltage will be 480/277-volt, 3-phase, 4-wire, and 60 hertz. The main distribution switchboard (SWBD) is rated at 2500 amperes, 480/277V, 3-phase, and 4-wire. This switchboard will include a manually operated insulated case stationary main circuit breaker with an adjustable solid state trip unit.

The distribution system will stem from the SWBD with feeders to panels on each floor. A separate 208/120-volt, 3-phase, 4-wire feeder will provide power to the residential distribution panel on each floor. There is not residential sub-metering for the individual loads in each living unit. A 208/120-volt, 1 phase, 3 wire load center will be located within each living unit and will be dedicated to all the electrical loads within the associated unit.

Construction Management

The construction of Building 7 started on July 21, 2008 and is expected to be finished in January 2010. The construction manager for the project is Whiting-Turner Contracting; they are taking on the role of CM at Risk. The total cost of the project is at $23.5 million with and estimated structural system cost of 3.98 million at the current time. Due to the size of the site, the construction team was permitted to set-up their trailer complex nearby on an existing parking lot. This area provides more space for field offices and a staging site. A Tower crane will most likely be employed as it would avoid any coordination and traffic maintenance around the site. No other details can be given at this time due to the early stages of construction.

Lighting System

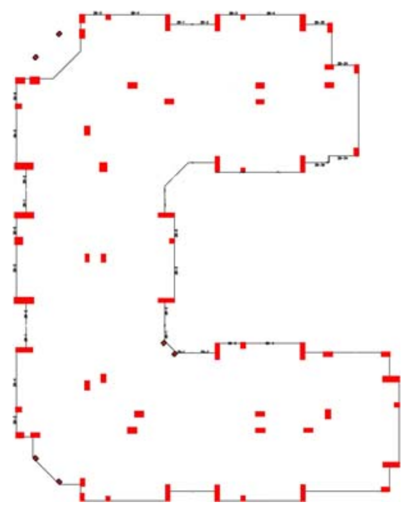
The lighting system primarily uses fluorescent lighting fixtures throughout the building. The corridors are lighted by 2x2 277V parabolic fluorescent fixture with electronic ballast with a 32 watt lamp. The seminar room uses the same style fixture except it is a 2x4 and has a dimmer ballast. The apartment units are comprised of 8" compact fluorescent downlights with electronic ballasts in the common living areas and surface mounted fluorescent with a contoured acrylic diffuser, both of these fixtures run on 120V. The entrance lobby is accented with 8" fluorescent downlight wallwashers and 8" recessed fluorescent fixtures.

Existing Structural Systems Conditions

Foundation

The foundation system is composed of reinforced concrete grade beams 24"x30" with 3#8's on the top and bottom with number #4 stirrups placed every 14". The deep foundation portion is auger cast grout piles 16" in diameter. These piles are to be 65' below elevation and are to be able to carry at 65 ton allowable load capacity. The pile configurations range from 2-4 piles per cap. The slab on grade for the foundation is 4" thick normal weight concrete reinforced with 6x6-1.4xW1.4 welded wire fabric. All foundation concrete is 4ksi except for the SOG which is 3.5 ksi. Due to the site's soil conditions it was necessary that the differential settlement over the entire building was limited, because of this the allowable soil bearing capacity was held to 500 psf.

Column and Bearing Wall Systems



The concrete columns support the lower two floors of Building 7. They arranged to form a typical bay of 15'x20'. These columns are gravity bearing only due to the type of lateral system in the building. The typical size of the columns range from 18x14 to 64x14 with the reinforcing ranging in each from 4#9's to 10#9's for vertical bars with #4 stirrups spaced at 14" O.C.. The concrete compressive strength for the columns is 6 ksi.

The bearing walls in Building 7 support the upper 6 floors and run along the outside perimeter of the building as well as along the corridors. The typical spans for the floor joists are 20'. Dealing with the concerns that the joists may not line up with the studs causing the header to buckle, this problem was solved by placing a distribution tube across the tops of all bearing walls. These walls are also to be designed by the contractor who is given general criteria to follow along with a loading diagram for all the different bearing walls. The general criteria are: a maximum stud spacing of 16" O.C., a minimum G90 galvanized coating, and have a minimum 16 gage thickness.

Roof System

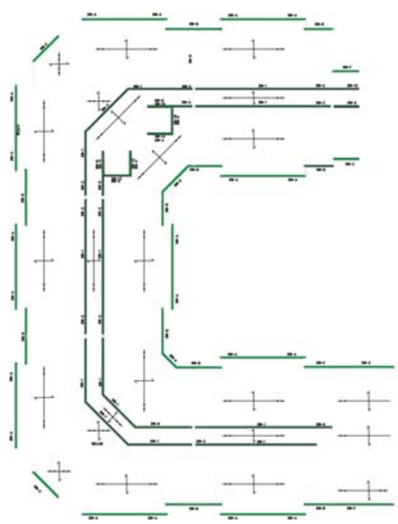
The roof system is made of the same Hambro Composite Floor System bearing on light gage walls. This Hambro Composite Floor System is also to be designed by the contractor instead of the Engineer just as the other floors are to be designed. Here are the criteria for the roof: overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep with a 3" thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric. The mechanical unit weights are listed and are placed close to the corridors for they are formed by the bearing walls. The elevator towers and stair towers are made of the same light gage studs.

Floor Systems

Lower 2 Floors

The lower two floors are made of reinforced concrete beams spanning between the columns. The intermediate members between these beams are made up of the Hambro Composite Floor System, which includes the steel joists and the slab system. The concrete beams range from 16x36 to 18x18 to 24x36 with the reinforcing ranging in each from 3#5's to 6#10's for longitudinal bars with #4 stirrups spaced from 8" to 16" O.C.

The Hambro Composite Floor System in Building 7 is not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed criteria that the contractor must follow. The following is the criteria: are overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep, the slab on top is to be 5" thick reinforced with 6x6-W4.0xW4.0 welded wire fabric.

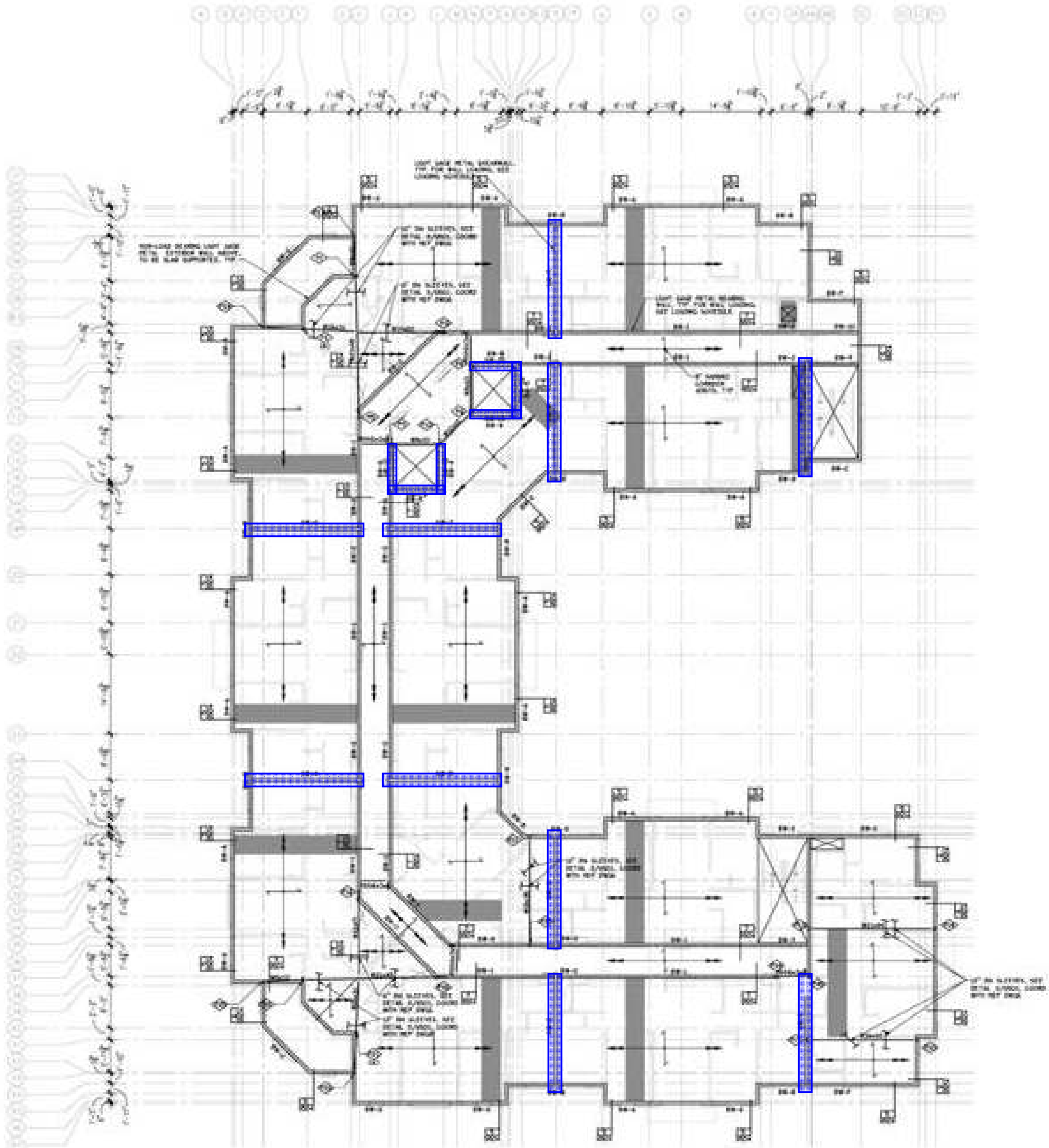


Upper 6 Floors

The floor system is made of the same Hambro Floor System but instead of them bearing on concrete girders they bear on light-gage stud bearing walls. This Hambro Floor System is also to be designed by the contractor instead of the Engineer. Here are the criteria for these 7 stories: overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep with a 3" thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric.

Lateral Systems

The primary lateral system for Building 7 is shear walls. On each floor there are 16 shear walls spanning both directions of the building, 9 in the north-south direction and 7 in the east-west direction. The lower two stories shear walls are 10" thick reinforced concrete with 10#5's on each end for flexure and for shear reinforcement there is #5@12" each way, each face. All concrete shear walls are 6 ksi normal weight concrete. The upper floors shear walls are to be light gage studs with maximum stud spacing of 16" O.C. they are also have a minimum G90 galvanized coating and have a minimum gage of 16 for the studs while the tracks are permitted to have a 20 gage minimum. There is to be bridging at 4' spacing throughout the shear walls. Since these are light gage it was determined that steel strapping was needed and is being provided in an X pattern connecting to the farthest opposite ends. The light-gage shear walls not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed loading diagrams of each load and the type of load on every shear wall.



Structural Depth Study

Problem Statement and Solution

Problem Statement

The present design of Building 7 utilizes primarily propriety systems as the structure for as much as the materials will allow. These systems, while they may be cheaper require long lead times while also the general contractor to is required to design them. The propriety systems used involve many miscellaneous metal supports for the lack of strength in the light-gage studs. From investigating these systems in the past technical reports noise, vibration, and fire proofing issues arise that makes these system less desirable.

These systems prevent the multiple floors from being build before the inner walls are placed (due to so many bearing walls). This issue increases the construction time. The low floor to floor heights (10' with an 8' ceiling) are an issue for placing the structural elements and along with the MEP systems within the floor cavity. Finally the current system as 16 shear walls due to light-gage cannot take a large amount of shear. Also the location of Building 7 is in a region where the Seismic Design Category is A, which allows for a simplified approach. This category almost entirely eliminates the need for seismic design and checking for the resulting forces are so much smaller than wind.

Problem Solution

In an effort to address the issues stated above a redesign of the structural system is being proposed in steel. This redesign will include both a gravity system and a lateral system. The gravity system will take a look at two systems composite steel and composite castellated beams. An initial study will be made to see which is best when looking at the thin ceiling cavity to allow for more space for the MEP systems. After the better choice has been determined that system will be further developed and used throughout the rest of the structural redesign. All steel gravity framing will be designed to conform to AISC Manual of Steel Construction, 13th Edition.

In regarding the lateral design and also the seismic issue it was decided that moving the building to a high seismic zone located somewhere, to be determined, in California will take place. The lateral system will be redesigned after a study of the different types of lateral systems that are acceptable in high zones and their benefits and shortcoming will be considered. After a system is selected an optimum layout with hopefully fewer elements in plan can be resolved. It should also be noted that since the material has been changed to steel and that the site is being moved a represented new geo-technical report will be use and for this reason a detailed study on the foundations can not be addressed in the given time but will be looked at in general overall aspects of the new steel system.

Structural Goals

- Design an overall structure made of steel and has limited use of propriety systems.
- Design a gravity system that does not require a change in the building height while still being acceptable.
- Move the location of Building 7 to a high seismic to better understand and work with seismic requirements in detail.
- Pick a single lateral system that will work for the new location and design it while trying to optimize it.

Design Codes and General Criteria

Design Codes & Guides

1. AISC Unified Manual 13th Edition
2. ASCE 7-05
3. International Building Code (IBC) 2006
4. AISC Seismic Design Manual
5. Steel Design Guide 19: Fire Resistance of Structural Steel Framing
6. Vulcraft Steel Roof and Deck Catalog

Deflection Criteria

Typical live load deflections limited to: L/360
Typical total deflections limited to: L/240
Typical construction load deflections limited to: L/360

Load Combinations

Listed here are the load combinations that are being considered when generating the computer model and analyzing the gravity system. Some of these combinations are acceptable in the lateral redesign but also special modified load combinations per ASCE 7-05 and AISC 341-05 are to be used and are listed in the respected lateral portion of this report due to the site being Seismic Design Category D. All of these combinations are based on LRFD design method.

- $1.4(D + F)$
- $1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$
- $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
- $1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$
- $1.2D + 1.0E + L + 0.2S$
- $0.9D + 1.6W + 1.6H$
- $0.9D + 1.0E + 1.6H$

Building Loads

Gravity Loads

Live Loads

The live loads used in this study and the report regarding Building 7 were calculated in accordance with IBC 2006 which references ASCE 7-05, Chapter 6. In the event that ASCE did not list loads needed a close equivalent was chosen to meet that particular space or condition. The table listed below summarizes the lives loads used.

Live Loads			
Occupancy	Design Load	Code Required Loads	
		Load	Code
Corridors	100 psf	100 psf	ASCE 7
Offices	50 psf	50 psf	ASCE 7
Seminar Room	100 psf	40 psf	ASCE 7
Mechanical Room	125 psf	125 psf	ASCE 7
Partition	20 psf	-	-
Roof	100 psf	100 psf	ASCE 7
Dormitory Rooms	40 psf	40 psf	ASCE 7
Lobby	100 psf	100 psf	ASCE 7
Staris and exit ways	100 psf	100 psf	ASCE 7

Dead Loads

The dead loads used in the study and the report regarding Building 7 were determined by referencing various standards and textbooks to find the corresponding values for their weights. Approximate values were assumed when ranges were listed depending on how dense the layouts were and the author's personal preference as well as considering the life history and usage of the building.

Dead Loads		
Roof Dead Load	Material	Design Weight
	Green Roof	50 psf wet
	Structural Members	15 psf
	Floor Slab	46 psf
	M/E/P	5 psf
	Ceiling Finishes	5 psf
	Total Dead Load	121 psf
Typ. Floor Dead Load	Material	Design Weight
	Structural Members	15 psf
	Floor Slab	46 psf
	M/E/P	5 psf
	Ceiling Finishes	5 psf
	Total Dead Load	71 psf

Lateral Loads

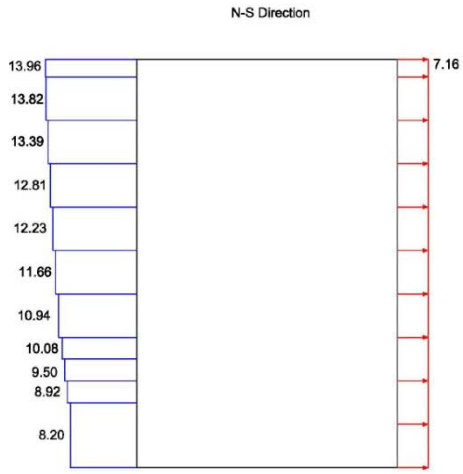
Wind Loads

All wind loads were calculated in accordance with ASCE 7-05, Chapter 6. The analytical method 2 was used to examine lateral wind loads in the North/South direction as well as the East/West direction. Also due to the irregular shape of the building it was necessary to look at the most critical orthogonal for it could possibly control, this was taken into consideration during the modeling process and a Ram Structural System load case was auto generated so to quicken this process of finding the critical angle.

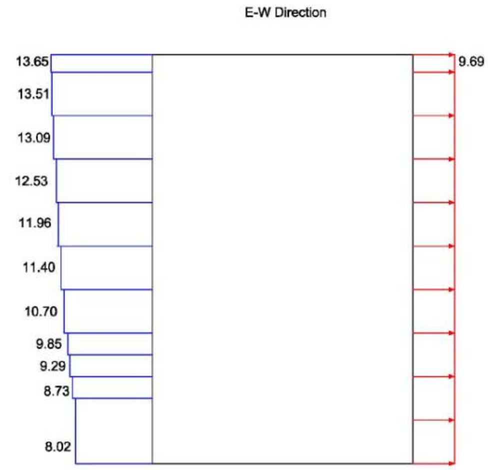
Building 7 is categorized as Exposure B due to its urban setting and location in San Diego, CA. The basic wind speed was found to be 85 mph per Figure 6-1 in ASCE 7. The building is not quite a square relative to the four directions, with the N/S direction (169'-8") slightly longer than the E/W direction (133'-6"). When inputting the wind forces in the computer model the wind loading cases dictated in Chapter 6 and illustrated in Figure 6-9 was done. All four of the load cases were inputted.

Wind pressure step diagrams were drawn of the final forces acting on the building. Also story forces and story shears were calculated by hand to compare to the computer models calculation. Based on reviewing the model it was determined that the same values were calculating it making its wind calculations valid. These diagrams can be found on the next page while the calculations and wind criteria can be found in Appendix A.

Wind Pressures



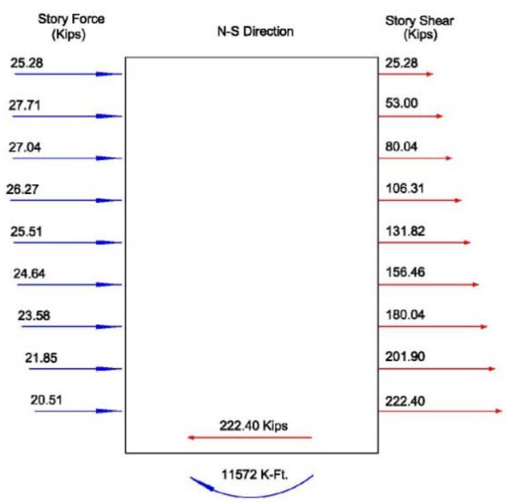
Wind Pressure Distribution in the North-South Direction



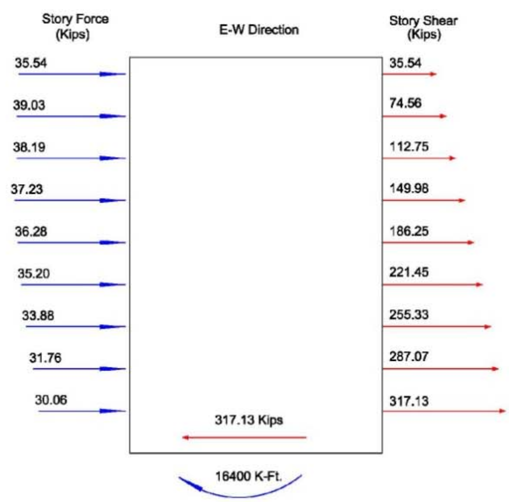
Wind Pressure Distribution in the East-West Direction

All Values on Wind Pressure Step Diagrams are in pounds per square foot (psf). The Blue indicates windward and the red indicate leeward pressures.

Wind Story Forces and Story Shears



Story Force and Shear in the North-South Direction



Story Force and Shear in the East-West Direction

Seismic Loading

The seismic loads were calculated in accordance with ASCE 7-05, Chapter 12 and referencing Chapter 22. Since moving the building to San Diego it was clearly seen that the original Simplified method for Seismic Design Category A was not going to be valid, this resulted in a more rigorous method of calculating the seismic forces. After looking at the represented geotechnical report for the San Diego Region, it was concluded that the building site is very stiff to hard clays at the ground level with bedrock at the deep foundation level, resulting in a Site Class B. The new site location was determined to be Seismic Design Category D. to determine the lateral forces and also the allowed procedure that can be used in Seismic Design Category D many conditions had to be met and considered. The rest of this section of the report describes the findings of the conditions as well as the lateral forced used for the design.

Structural Irregularities

Section 12.3 of the ASCE 7-05 code determines and dictates the limitations for diaphragm flexibilities and also determines what a structural irregularity is on the horizontal and the vertical planes of the building. Table 12.6-1 gives the permitted analytical procedures for each design class along with the limitations due to a structural irregularity.

Horizontal structural irregularities were determined according to Section 12.3.2.1. The descriptions of the horizontal irregularities are listed in Table 12.3-1. The following summary table below represents each irregularity type and its regard to Building 7.

Horizontal Structural Irregularities			
Type	Irregularity	Comment	Status
1a	Torsional	After Modeling structure it can been concluded that this irregularity does not exist.	Good
2	Reentrant Corner	This irregularity does exist due to the U-Shape of the plans but ELFP is allowed, a 25% force increase for the connections between the diaphragm and the vertical elements are required.	Not Met
3	Diaphragm Discontinuity	Irregularity does not exist by inspection of the drawings.	Good
4	Out of plane Offsets	No vertical element out of plane offsets exists by inspection of the drawing.	Good
5	Non Parallel System	All lateral force resisting systems are parallel to the orthogonal axes.	Good

Vertical structural irregularities determined according to Section 12.3.2.2. The descriptions of the vertical irregularities are listed in Table 12.3-2. The following summary table below represents each irregularity type and its regard to Building 7.

Vertical Structural Irregularities			
Type	Irregularity	Comment	Status
1a	Stiffness-Soft Story	Members are larger going down the building, plus calculations were done to prove it in a later section.	Good
2	Weight (Mass)	Was carefully looked at due to the green roof and Mechanical Units but roof was approx. 400 kips under limit.	Good
3	Vertical Geometric	Plans show same geometry the height of the building.	Good
4	In-Plane Discontinuity of Vertical Lateral Force Resisting Element	No discontinuity exists by inspection of the drawings.	Good
5a, b	Discontinuity in Lateral Strength	Members are upsized going down the building resulting in a higher strength.	Good

After looking at the structure and the limiting factors that govern the analytical procedure determined by Section 12.6, it was found that the structure is considered regular with only one irregularity in which the diaphragm connections need a 25% increase in their force (ELFP permits this) and since T<3.5Ts then it is permitted to use The Equivalent Lateral Force Analysis. This procedure will be for simplicity reasons and lack of experience regarding modal analysis.

Loading Direction and Redundancy

When looking at the possible permitted direction to load the structure the provisions of Section 12.5 needs to be followed. Since the horizontal structural irregularity 5 does not exist and the new design has no individual column taking seismic forces from both orthogonal directions, then this section permits the design seismic forces to be applied independently in each of the two orthogonal directions. Also the orthogonal interaction effects are also permitted to be neglected.

Redundancy was checked in accordance with Section 12.3.4. After inspecting the new lateral force resisting system which has is a total of 6 braces in the N-S Direction and a total of 4 braces in the E-W direction. It can be concluded that both of the criteria cannot be met in this section, which are no more than one frame takes 33% of the shear when one is removed and that there must be a minimum of 2 frames in each direction along the perimeter. It is evident that there is no perimeter framing in the new design due to trying to sticking with one single system to lower cost and complexity of the structure while not disturbing the exterior façade look. Since this is not met a redundancy factor of 1.3 was used in the lateral design and analysis.

Lateral System Criteria (Special Concentric Braced Frames)

The lateral system being looked at for the lateral redesign of Building 7 is the Special Concentric Braced Frame, which the reason for this lateral system is discussed in detail in the Lateral Study section. To compute exact values for S_s and S_1 the United States Geological Survey's software under NEHRP design provisions was used. The table below summarizes the seismic criteria and its associated values along with what part of the relevant code was sued for determining it. Refer to Appendix B for more detailed spreadsheets and along with the Cs calculations.

Seismic Criteria for SCBF Design		
Criteria	Value	Code Reference
Occupancy Category	II	Table 1.1
Importance Factor	1.000	Table 11.5-1
Seismic Category	D	ASCE 7-05 Section 11.6
Site Class	C	Geotechnical Report
Spectral Acceleration for Short Periods (Ss)	1.572	www.usgs.org
Spectral Acceleration for 1 Second Periods (S1)	0.617	www.usgs.org
Site Coefficient, Fa	1.000	ASCE 7-05 Table 11.4-1
Site Coefficient, Fv	1.300	ASCE 7-05 Table 11.4-2
Seismic Design Category	D	ASCE 7-05 Table 11.6-1,2
R Factor	6.000	ASCE 7-05 Table 12.2-1 # B3
S_{MS}	1.572	ASCE 7-05 Equation 11.4-1
S_{M1}	0.802	ASCE 7-05 Equation 11.4-2
S_{DS}	1.048	ASCE 7-05 Equation 11.4-3
S_{D1}	0.535	ASCE 7-05 Equation 11.4-3
Deflection Amplification Cd	5.00	ASCE 7-05 Table 12.2-1 # B3
Overstrength Factor	2.00	ASCE 7-05 Table 12.2-1 # B3

Vertical Force Distribution for SCBF

The vertical distribution of the seismic based shear is determined by ASCE7-05 Section 12.8.3 the force at any given floor is based on a distribution factor times the total design base shear. After the Cs factor was determined by Section 12.8.1.1 and the overall weight of the building was calculated the seismic base shear was determined the distribution factor was calculated and the tables below represent the vertical forces. The reason for the different forces in each direction was due to an adjustment in the north south direction when the actual period was inputted after the structure was initially designed. After this the new forces were placed in the model again and final designs were worked out.

Vertical Force Distribution E-W Direction					
Floor	Height (Ft.)	Weight (Kips)	Cvx	Fx (kips)	Story Shear
Roof	90	2145.00	0.24	398.08	398.08
8	80	1700.00	0.17	280.44	678.52
7	70	1700.00	0.15	245.39	923.91
6	60	1700.00	0.13	210.33	1134.24
5	50	1700.00	0.11	175.28	1309.51
4	40	1700.00	0.08	140.22	1449.73
3	30	1700.00	0.06	105.17	1554.90
2	20	1700.00	0.04	70.11	1625.01
1	10	1700.00	0.02	35.06	1660.06
Total Weight		15745	kips		

Seismic Base Shear	1660.06	kips
Overturning Moment	107,339.65	kip-ft

Vertical Force Distribution N-S Direction					
floor	Height (Ft.)	Weight (Kips)	Cvx	Fx (kips)	Story Shear
Roof	90	2145.00	0.24	467.42	467.42
8	80	1700.00	0.17	329.29	796.70
7	70	1700.00	0.15	288.12	1084.83
6	60	1700.00	0.13	246.96	1331.79
5	50	1700.00	0.11	205.80	1537.59
4	40	1700.00	0.08	164.64	1702.24
3	30	1700.00	0.06	123.48	1825.72
2	20	1700.00	0.04	82.32	1908.04
1	10	1700.00	0.02	41.16	1949.20
Total Weight		15745	kips		

Seismic Base Shear	1949.20	kips
Overturning Moment	126,035.22	kip-ft

Gravity Study and Redesign

Gravity System Considerations

When the decision to go to a steel structure was made several possible types were thought of and they are: non-composite steel, composite steel and finally a castellated beam and girder system. A disadvantage of a steel structural system is the added depth of the members and this reason was the critical reason why non-composite steel was first eliminated due it having deeper members. To avoid interference with MEP systems it was key to keep the overall depth of the structure to a minimum or to allow for adequate room for the said systems. Though castellated beams are deeper than composite steel members they have natural holes in them due to the design. It should also be noted that the overall building height was not changed so as to not impact the architecture of the exterior and the building height limitations this was another reason for the two remaining systems.

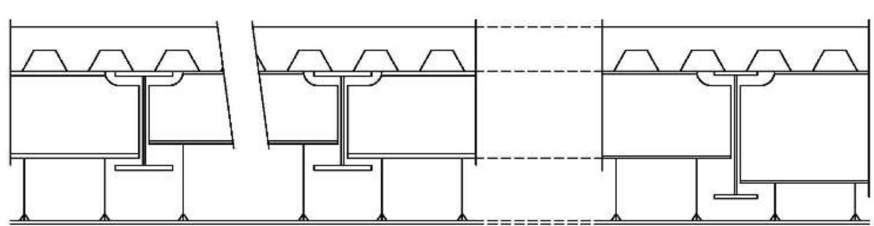
The specification and range of the holes in size were looked at and compared to the difference in thickness of the two remaining systems. It was concluded that the hole size was not large enough to run a decent size round duct, the composite floor would be better suited for rectangular ductwork and also you have more flexibility of turns in MEP systems but also a wider range of MEP shapes that will work. So in conclusion from looking briefly looking at composite steel and castellated, composite steel is the better chose and will be used for the rest of the structural design.

Bay and Column Grid Layout

Since the original structure of Building 7 was Hambro joists on light gage stud bearing walls and the lateral system was light-gage stud bearing walls and reinforced concrete it was critical and obvious that the existing layout of a column/wall grid was not going to work, so a new grid needed to be created. A typical bay was also created as best that could be for repetition and ease of construction throughout the floor plan.

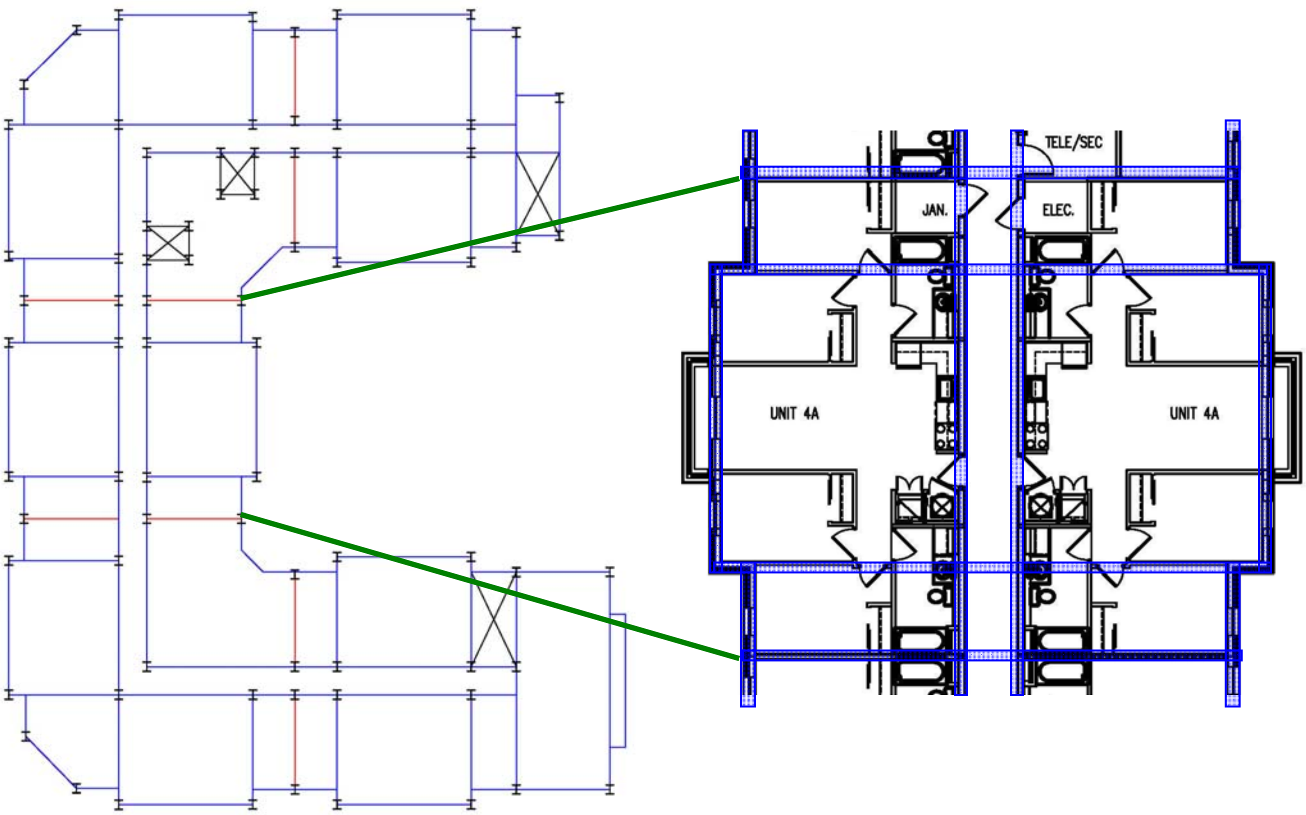
Due to there being a corridor in the middle of the building a simple two bay layout with equal bay sizes was not going to work alone. After looking at the plans more carefully a primary factor in determining the bay sizes was the corridor walls. These walls provide a stopping place for the typical bay. Two possible solutions came from this: an unequal size two bay layout which one is wider or a double loaded corridor with two equal bays on each size. A simple design was looked at for both to see how deep the systems may become. The diagram below shows the depth of the two different layouts in the early stage.

Corridor



No Corridor

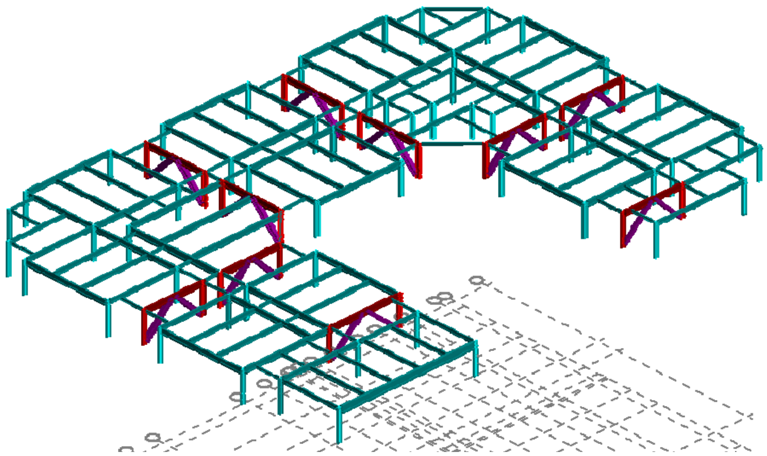
Taking into considerations this relationship it was determined that a double loaded corridor was the best choice so that the bays on each side would be the same but also the connecting beams would be smaller resulting in more ceiling cavity space for the larger MEP components. In the figures below you can see the new grid and the typical bay without infill beams.



Beam, Girder and Slab Design

From the developed bay layout the composite metal deck that forms the slabs would need to span perpendicular to the infill beams, which is a maximum of 9’-6” typical. The composite metal deck was chosen from the Vulcraft Steel Roof and Deck Catalog. It was determined that a 3VL21 composite steel deck was selected, with a light-weight concrete slab. This type was chosen to ensure a two-hour fire rating for the slab without requiring the use of fireproofing of the deck and also to help lower the sound transmission from one side of the slab to the other. This resulting design gives a total slab thickness of 6.25” with 3.25” of concrete above the top of the deck.

When the gravity loads were applied to typical beams it was decided to have the beams act compositely so to have the concrete deck assist in the shear and making the sections smaller. Shear studs sizes and quantity was determined by the provisions listed in 13th Edition of the Steel Manual. The load case that controlled in all of the gravity framing was 1.2D + 1.6L except for the roof which used 1.2D + 1.6Lr + L. The figure below shows a 3-D representation of the gravity system along with the lateral braces on a typical floor.



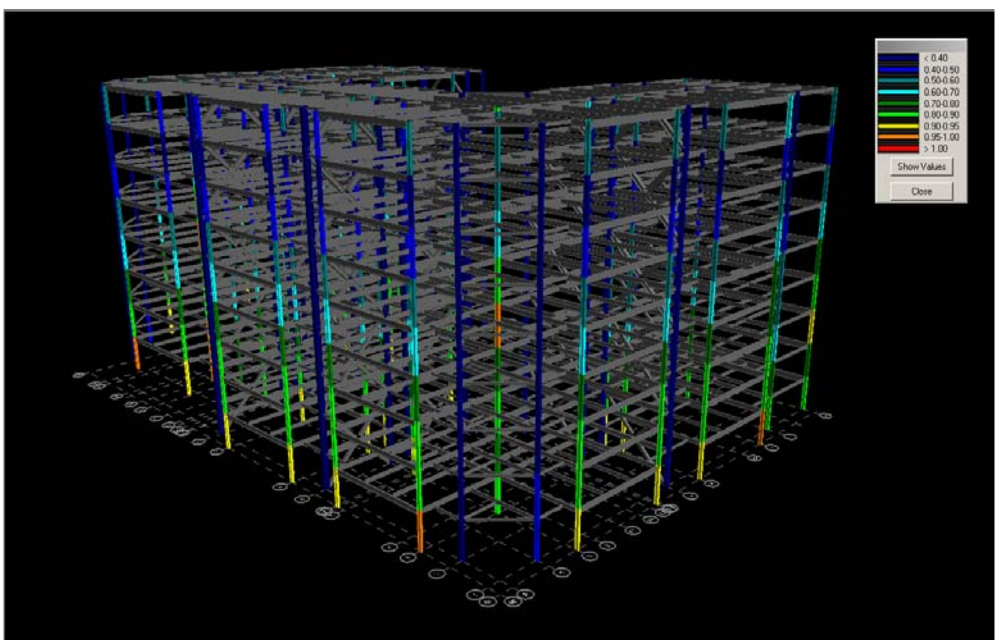
After a size was selected for strength using Table 3-19 (composite W-shapes) in the manual during the preliminary stage, a Ram Model was then built so to optimize the gravity system at each level. Deflection of the beams was considered in the design process during the construction loading, much deflection would lead to the addition of extra concrete to the slab, and during the long term life of the structure

To create a more efficient design repetition is an important factor. By using fewer different size beams and girders can cut down on material costs and reduces the amount of coordination necessary in the field while reducing the chance of a mistake being made during construction. Member sizes were coordinated such that beams and girders in similar bays and in similar locations on different floors were made same size while sticking to a limited number of sizes for the large areas. The final gravity design based on the new bay layout resulted in light W14 sizes for girders on the second through eighth floors while the roof had W18's as girders, typically. The infill beams were light W12 sizes on the second through eight floors while the roof had W14 sizes. The calculations for these designs can be found in Appendix C.

Column Design

The gravity loads in the building are carried by the slab and deck then to the beams which carry the load to the girders, in turn goes to the columns. The columns then take this load to the ground, through the building's foundation. This is the typical load path used when designing the gravity columns. The tributary area was found on each floor for a given column and the total axial load was determined after reducing the live load according to ASCE 7-05 Section 4.8 and 4.9. All the columns were designed for the axial load and gravity induced moments determined and were designed according to 13th Edition of the Steel Manual.

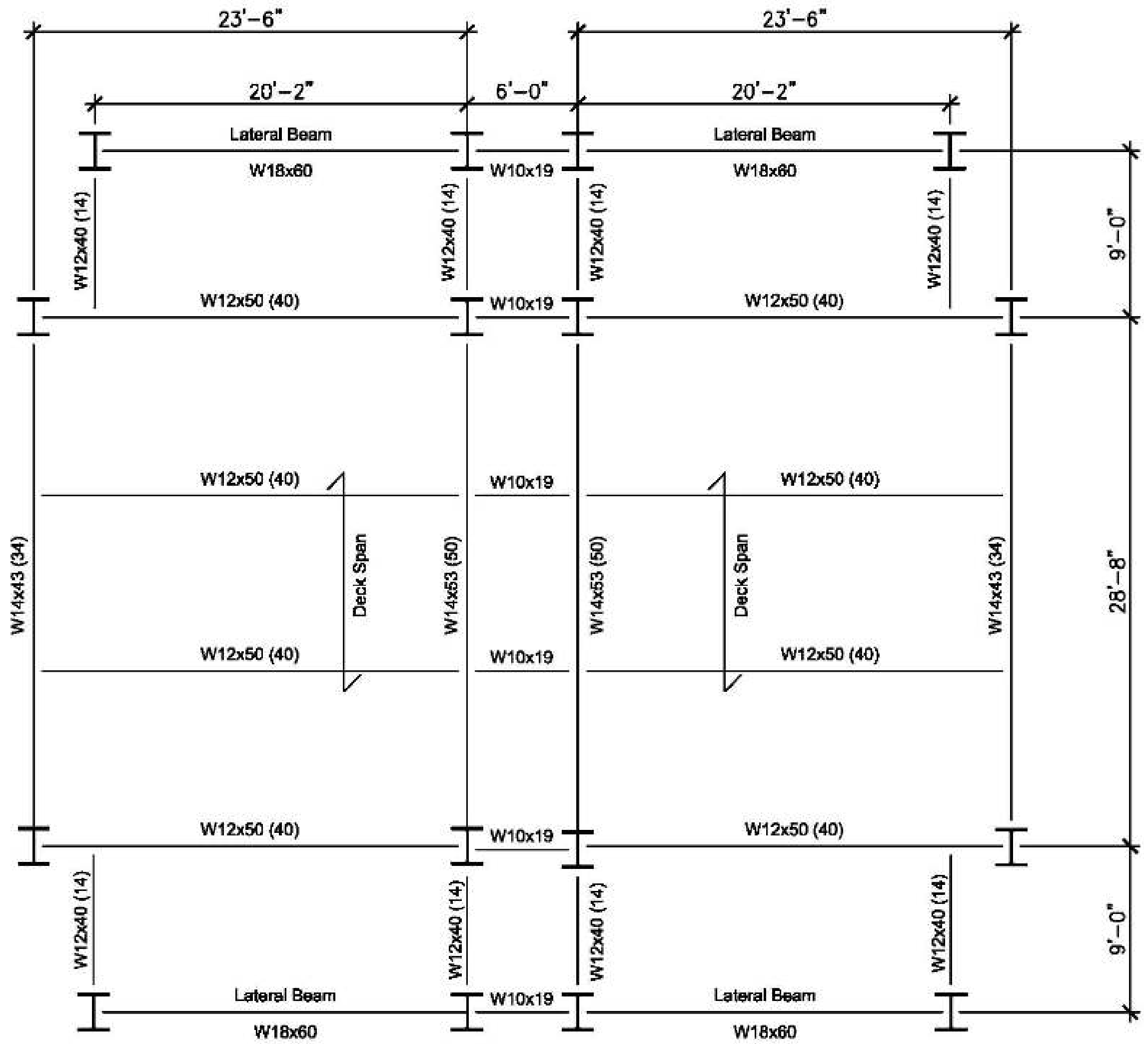
After a size was selected for strength using Table 4-1 (axial compression) and Table 6-1 (combined axial and bending) in the manual during the preliminary stage then the Ram Model was used so to optimize the columns at each level and limit the different sizes of each column. To minimize architectural impact of the columns on the new grid all of the columns were designed to be no larger than then W12 sizes. Also the splicing of the columns was considered for construction reasons. The resulting design was to splice the column at every second floor while the first floor started with a two story column due to the odd number of floors. As with the beam design adjustments were made to increase repetition the column sizes so to cut down on the number of different sections. The resulting adjustments gave a total of 6 different column sizes used throughout the building for the gravity loading only; these are all in the W12.



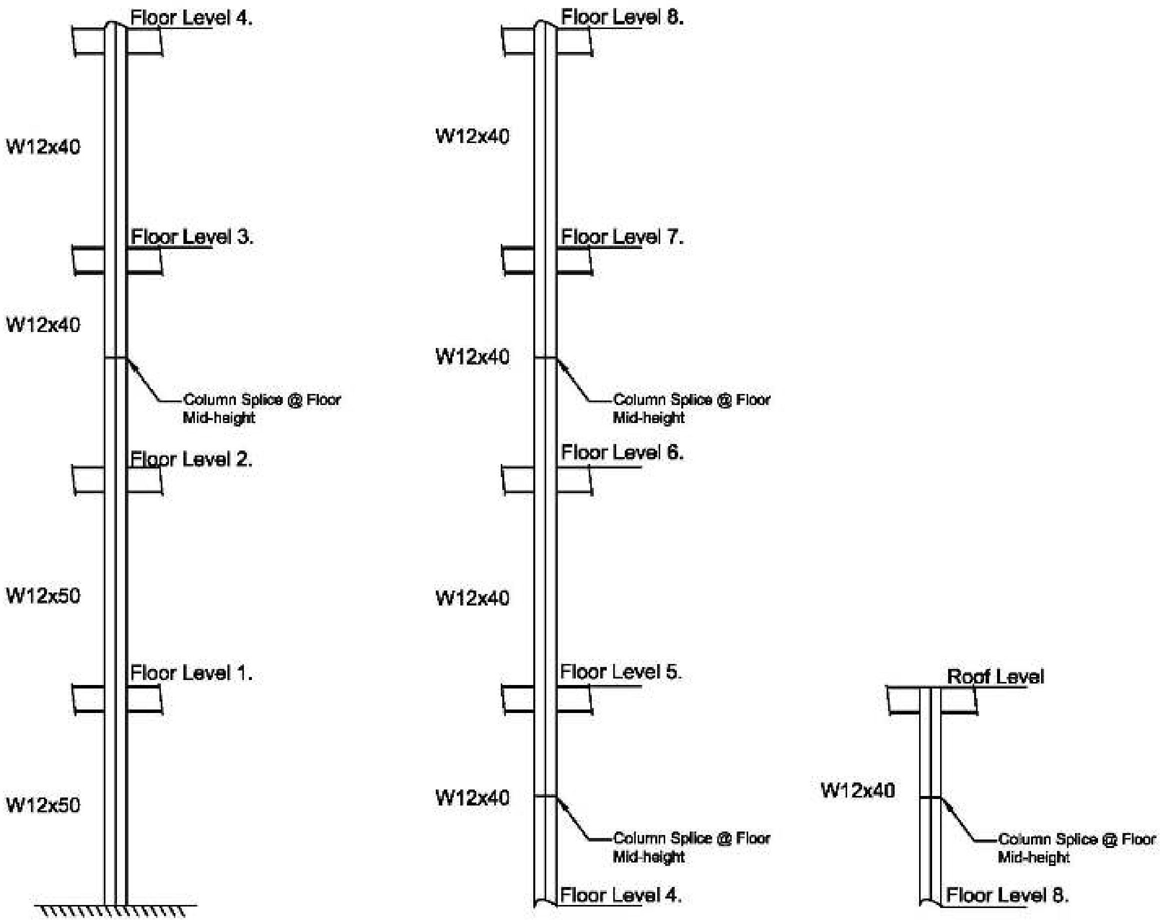
Final Gravity Layouts

This section of the report contains the final designs of a typical column row from floor to roof and also a typical bay design across the entire width of the building. The details of these designs are listed in the drawing while the thought process was listed in past sections. The details include member sizes, orientations, studs, splice locations and dimensions. Please refer to Appendix C for a complete layout of the three different typical floor plans and details regarding the column designs.

Typical Bay Layout



Typical Column Layout

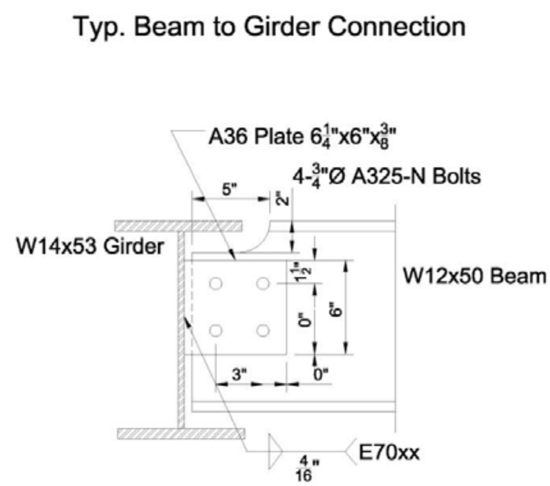


Gravity Connection Designs and Details

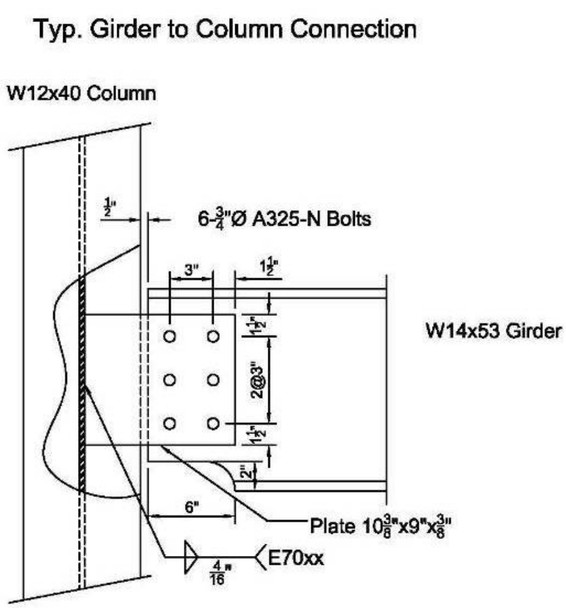
When looking at the gravity design during the design phase connection issues were considered and designs were adjusted to make connections simpler. This primarily relates having the girders deeper than the beams so that a double cope and significant reduction in the beam section did not occur. To make construction easier and faster single shear tab connections were chosen for the beam to girder connection. For the girder to column connection an extended single shear tab connection was chosen. These two connections were looked at in the typical bay and results, reasons for these connections, along with a sketch of the final typical connection is below. Refer to Appendix C for sample calculations of these designs.

Beam to Girder Connection

The reason for selecting the single shear tab is so that the beams don't need to be lowered between two angles or plates and risk damaging those items, this allows for bringing the beams in from the side. The plate also allows for all plates to be welded in the shop also the issue of not having to deal with bolting issues with the beam on the other side of the girder. Based on the forces used to design this connection, overall design resulted in a reasonable connection.



Girder to Column Connection



The reason for selecting the extended single shear tab is so that the beams don't need to be lowered between the column flanges and making the bolting process smoother, this allows for bringing the beams in from the side. A bottom cope was used though not absolutely needed just in case the construction doesn't allow for side placement for this is safer when placing with a crane. The connection again allows for all plates to be welded in the shop thus saving on at field welding. Based on the forces used to design this connection, overall design resulted in a reasonable connection.

Lateral Study and Redesign

Considered Lateral Systems for Building 7

Of all the different possible lateral systems most valid for high seismic regions, four were chosen and looked at to see their benefits and also disadvantages .the four that were chosen to be looked at from with a non-design format are: Special Concentric Braced Frames, Special Moment Frames, Special Plate Shear Walls and Buckling Restrained Braced Frames. It should be noticed that only steel systems were considered. The reasons for this was to keep the entire or as much as possible of the structure as steel, keep the different trades to a minimum and also for simplicity of construction coordination with trade issues. Listed below are the summaries of the finding and at the end of this section will list the chosen lateral system to actually be designed.

Special Concentric Braced Frames

Special Concentric Braced Frames (SCBF's) were looked at due to their simplicity of design and they are one of the most commonly used. They can have many different configurations and this can be beneficial to try and work around the architectural limits. SCBF's though tend to have inherent problems due to the vastly different compression and tension capacities of the braces. Also the size of the gusset plate can get rather large when the forces are high due to how the connection must behave. Finally even though they are man configurations the braces tend to get in the way unless a clear area in the plan is available for them.

Special Moment Frames

Special Moment Frames (SMF's) were considered as a possible alternative for they allow for a very open floor plan and have a limited impact on the structure. SMF's also have a minimum number of members which will affect the cost and overall volume of steel in relative terms. The best place for these in Building 7 to use their benefits would be around the perimeter. The down side to SMF's is that there are a limited number of approved connections, drift can be a major issue in controlling, and also the beams could be large and could occupy more than the ceiling cavity.

Special Plate Shear Walls

Special Plate Shear Walls (SPSW's) were looked at for they tend to be very thin and can be placed between walls easily without affecting the overall thickness of the wall. SPSW's were created when the gusset plates on SCBF's tend to get large and almost touch. The shear walls can be either un-stiffened or stiffened with extra plates. On a disadvantage standpoint many configurations of openings and stiffener configurations have not been tested and pose design issues especially in modeling for right now this issue can only be solved with a true finite modeling.

Buckling Restrained Braced Frames

Buckling Restrained Braced Frames (BRBF's) are a relatively new yet promising solution. This system has near equal tension and compression capacities which eliminate post-bucking load imbalances commonly found in other braced frames. Depending on the style of brace it can behave as true pinned-pined members. The number of braces can be reduced due to the capacity is almost equal in tension and compression, possible single brace per story per frame. BRBF's are not a proprietary system but their configurations and details of the assembly and in some cases the connections are subject to US patent laws and recreation is limited to the holding companies. They tend to cost more than standard HSS or W-shapes. The last disadvantage is that the design of BRBF's involves some complexities in modeling and also in managing drift control in modeling.

Initial Lateral System Decision

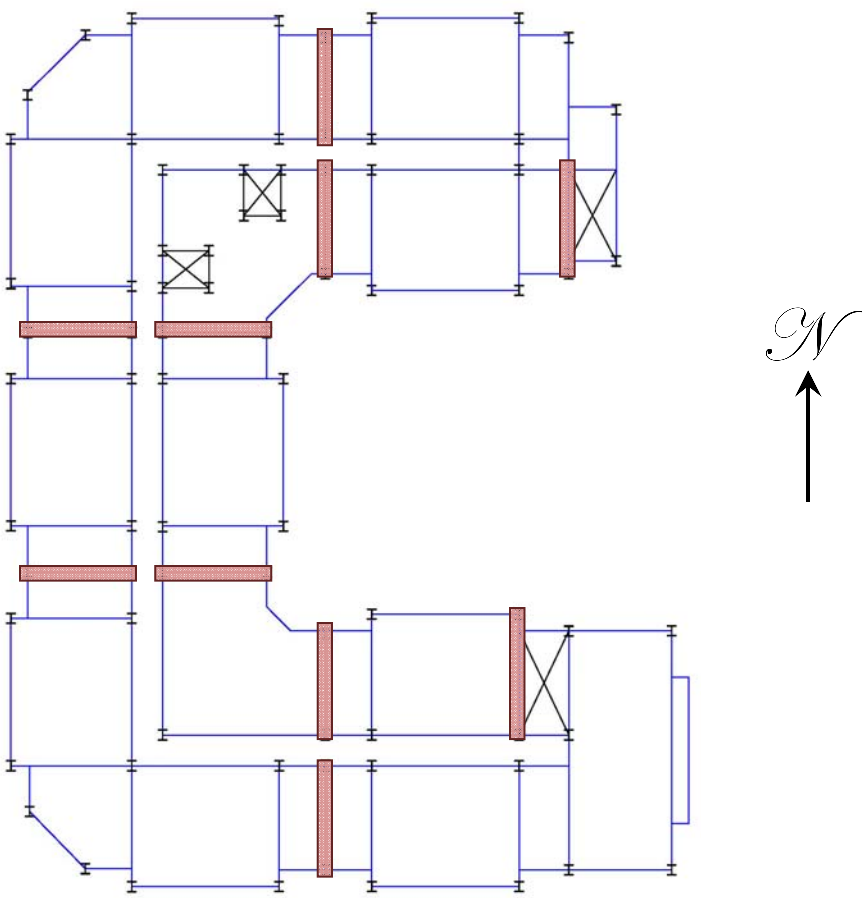
After reviewing the different listed systems with their benefits and drawbacks it was determined that the best system for the new lateral system is the Special Steel Concentric Brace Frame. The reasons for choosing this system are:

1. Most commonly used and a good place to start with the multiple bracing systems
2. Initial available wall space for the frames to fit in so the lateral system does not interrupt the architecture
3. Better at controlling drift when compared to Special Moment Frames
4. Multiple styles of bracing configurations to choose from
5. Multiple ways to design and detail the seismic connections
6. Doesn't require specialty or complex software to model like Special Plate Shear Walls

This system of Special Steel Concentric Brace frames will be worked with and designed in detail for the new lateral system of Building 7. The overall design as well as the process for locating, designing, and detailing this system is described in detail in the next few sections of this report.

Layout and Location of Lateral Elements

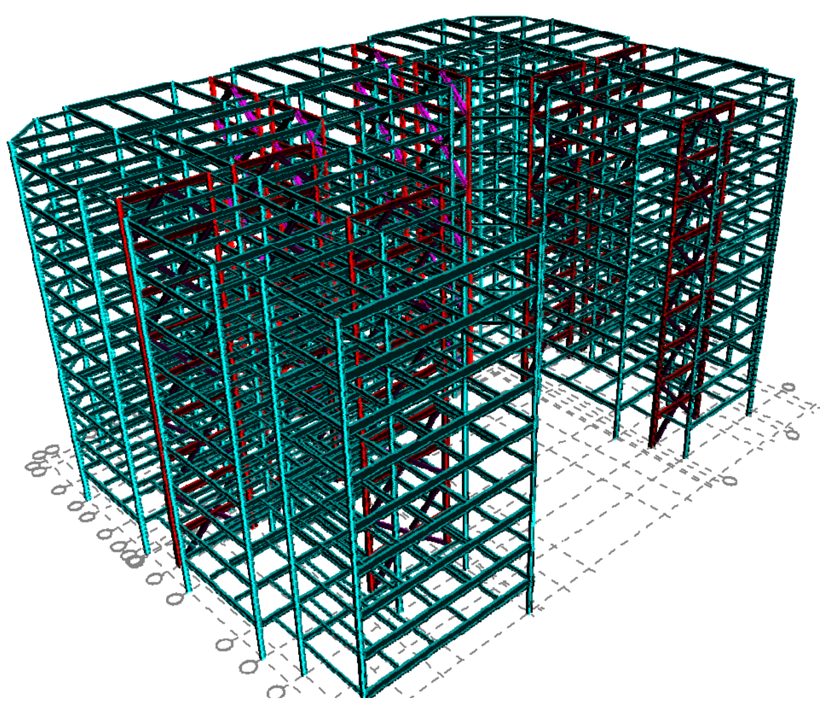
The initial placement of where the lateral system would be was based on the same location as where the original ones were at with the exception of that the original lateral system was also around the elevator core. The small size of the elevator shafts do not allow for an effective braced frame to be placed there for the average width of one of those would be only 10' max. The east west direction had more limited space to place these due to the architecture of the building when compared to the west side so it is likely that those frames will be larger since there are less of them. It can also be seen that the original plan of including only one lateral system is being kept and a dual system is not being considered. The boxed areas in red indicate the location of the new lateral frames.



Modeling Assumptions and Considerations

Once all the necessary seismic provisions were taken into account and seismic loads were determined, a lateral analysis and design was done using a combination of Ram Structural System and Etabs, depending on what was being looked at determined the modeling software was used. Etabs was used primarily used to verify and check torsion effects and the building periods. The following modeling assumptions and requirements were taken into account for both programs.

* The Main Lateral Resisting System was only modeled in the case of ETABS but for Ram Structural Steel the entire structure was model, for Ram was used to optimize the gravity system.
* A Rigid Diaphragm was modeled at every floor with the lateral load being assigned to the diaphragm.
* Lateral forces were applied to the center of mass along with a calculated moment due to accidental torsion.
* The mass of the structure was assigned to a Null Shell Property at each floor. This gives us an approximate period from the modal analysis.
* The Proper Load Combinations were generated and used in accordance to all relevant codes.
* The Braces of the SCBF’s were assumed to be pinned at each end.
* The Structure was assigned a fixed support at the base for all gravity columns
* The lateral Columns were modeled with fixed bases to help with drift slightly also fixity is not hard to accomplish.
* P-Delta effects were automatically taken into account in the model and ASCE7-05 Conditions for modeling P-Delta effects were considered.



Lateral SCBF Design

To design the new lateral system of Building 7 the new building lateral loads listed in the previous sections were taken and placed into the computer model Ram Structural Frame and also at times Etabs was used to verify the accidental torsion components as the design changed. The lateral forces applied to the building's diaphragm at the center of mass. Initial sizes were chosen so to allow the program to perform its analysis, these sizes were chosen based on what final design might be but due to early stages it was more of a hypothetical guess.

Since moving the building to a high seismic zone and the fact that the seismic loading clearly controls additional special load cases per AISC 341-05 and ASCE7-05 were used along with the standard combinations. Listed here are a few of the primary special required load combinations of the many overall load combinations that were inputted in the model.

* $1.4(D + F)$
* $1.2(D+F+T) + 1.6(L+H) + 0.5(Lr \text{ or } S \text{ or } R)$
* $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
* $1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$
* $(1.2 + 0.2SDS)D + \rho QE + L + 0.2S$
* $0.9D + 1.6W + 1.6H$
* $(0.9 - 0.2SDS)D + \rho QE + 1.6H$

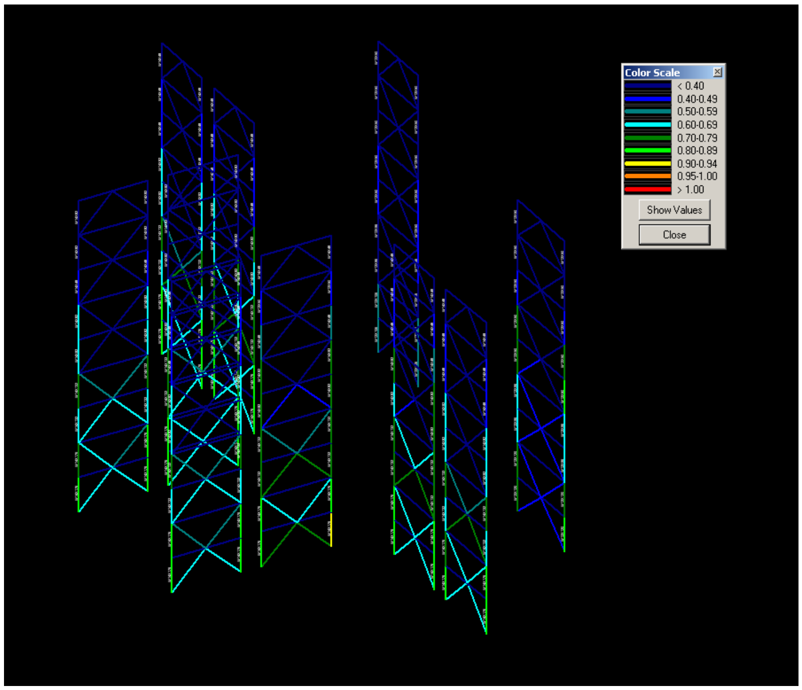
After all advanced modeling criteria were placed into the model for both the MAE integrated requirements and also according to the code. The analysis was performed and a design check was made on the initial members. The members that failed under loading were increased in size and the model analysis was repeated and checked once again. This process was repeated till all members were designed sufficiently for strength. The capacity of each member was taken to between 60-75% if design strength for all lateral members.

Once all of the members were satisfactory for strength requirements drift issues were looked and calculated. It was determined that the lower four floors met the seismic drift requirement but the upper levels were exceeding the drift limit. Since drift was over the allowable it was necessary to try and bring down the drift on the upper floors. Three options were considered in increasing the member sizes to control the drift, they are:

* Increase beam sizes
* Increase column sizes
* Increase brace sizes

All three of these options were tried but the one that had the most significant effect was increasing the brace sizes for they are the key component of the lateral SCBF system. Column sizes were also played with slightly also if just the drift needed to be lowered slightly. Model iterations were completed and rerun after each change of member sizes so to make ensure the possible change COM and COR were taken into account.

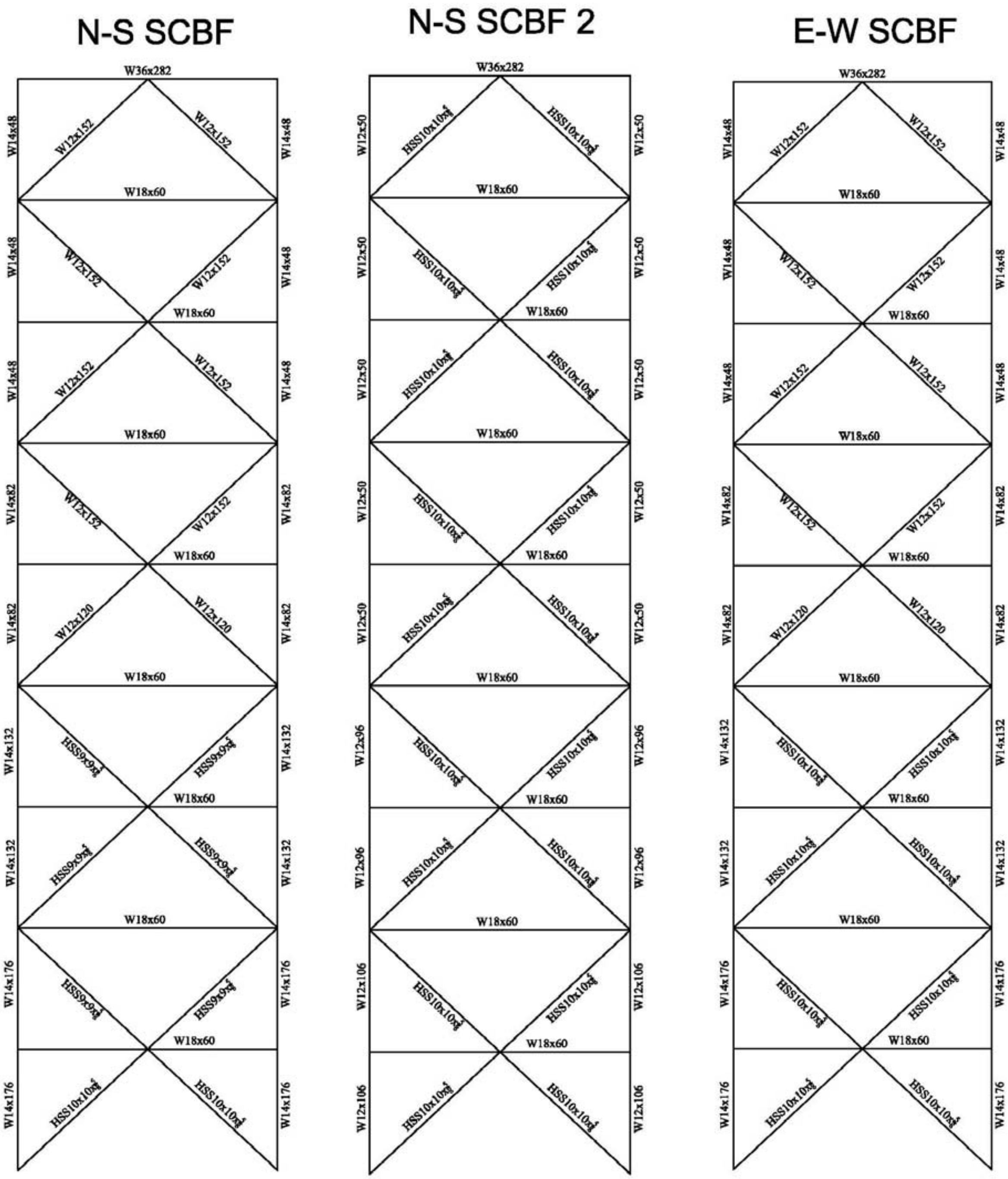
In the end it was clear that the upper five stories were controlled by drift for the capacity of the member was only between 15-25% of its design strength. It can be seen in the figure below that the brighter the colored members are the closer they are to their design strength and an indication where drift was the deciding design factor. This result is reasonable for the lower floors are typically stiffer and have less load acting on them to contribute to drift. Where at the upper floors are less stiff.



As the members were being designed special requirements were followed per AISC 341-05 and ASCE7-05 so to ensure that the lateral system behaves ductility and has hysteric damping. All members were limited so that local buckling requirements were acceptable per AISC 341-05. Table 1-2 and Table 1-4b was used to verify all columns and braces in the SCBF design met this requirement.

The member sizes in the frames were considered and limited were at all possible so to fit within the barrier walls between the apartments. The vertical members of the frames were all selected to be either W12 or W14 shapes, each frame was limited to one W range so that the splicing of the columns were easy for the interior flange to flange dimension is the same. Rectangular hollow structural steel sections were used as the diagonal bracing members so the bending strength was the same about both axes and thus the brace could buckle equally about either direction during an earthquake. The sizes of these members range from an 8” to 10 inch wide with a nominal thickness of 5/8 inch or an equivalent W-shape with the same cross-sectional area was used. Three sample elevations are shown on the next page to represent a typical SCBF in both the N-S and E-W direction. Please refer to Appendix D for more detailed frames and also calculations.

Typical SCBF Lateral Layouts

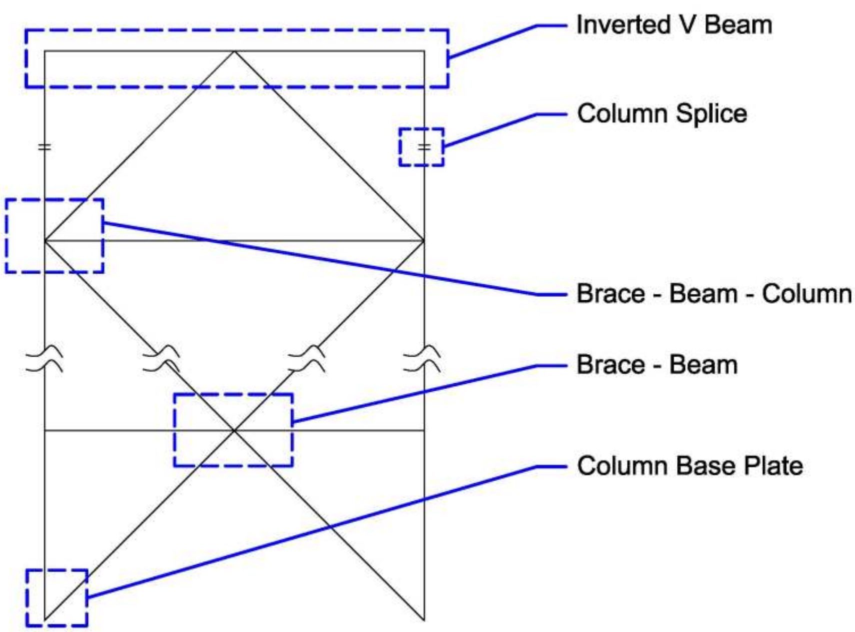


Lateral Detailing

Since SCBF's have an R=5 it is import and critical, not to mention required to do seismic detailing if the lateral system. Seismic detailing of steel SCBF is controlled per IBC 2006 and also AISC 341-05 Specification. The sections of the newly designed lateral system, SCBF, were chosen for they have the larger effect on the overall behavior of the system. The areas that were looked at were:

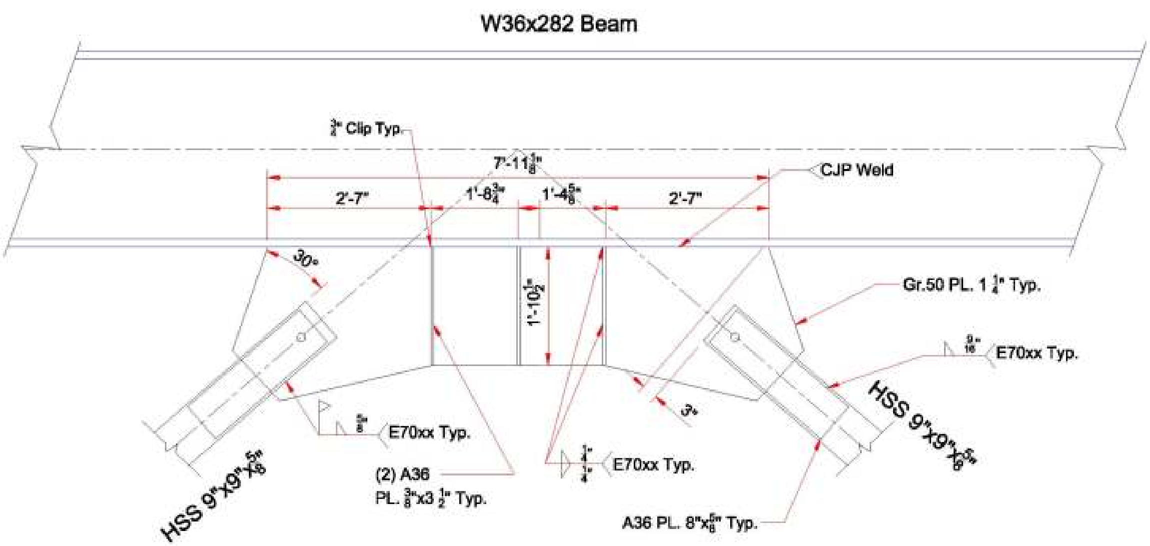
* Brace to beam connection
* Column splice
* Inverted V beam
* Brace to beam to column connection
* The column to foundation

The codes were followed resulting in a represented typical connection and member detailing of each. The resulted of the studied areas are listed in this section and the region where each was designed at and typically located can be seen in the figure below.



Inverted V Beam

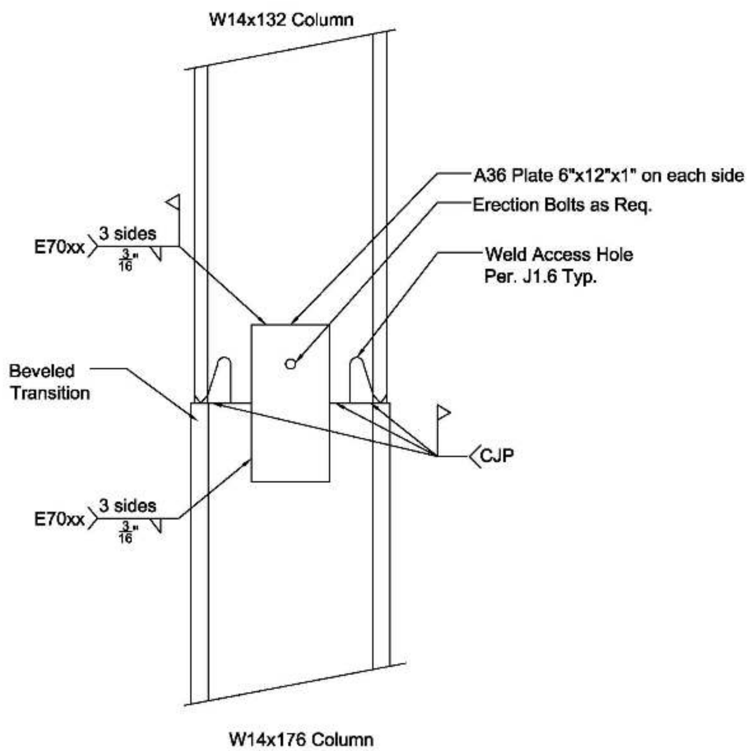
The inverted V beam and brace configuration at the top of each frame was designed similarity to the lower brace to beam connections with the exception that the beam at this level had additional requirements. The beam was designed as continuous and as if the bracing is not there for gravity loads. Also it was designed to take the vertical unbalanced load from 100% of the tension expected yield strength and 30% of the compression brace nominal strength as per AISC 341-05 Section 13.4. Finally the beam top and bottom flanges were braced where the intersection of the braces met.



It should be noted that the seismic steel code from 2002 allowed the exception of the top beam on a braced frame where a V configuration ends in the middle of the beam on the roof from taking the unbalanced load. The new 2005 code does not permit this and was needed to be considered.

Column Splice

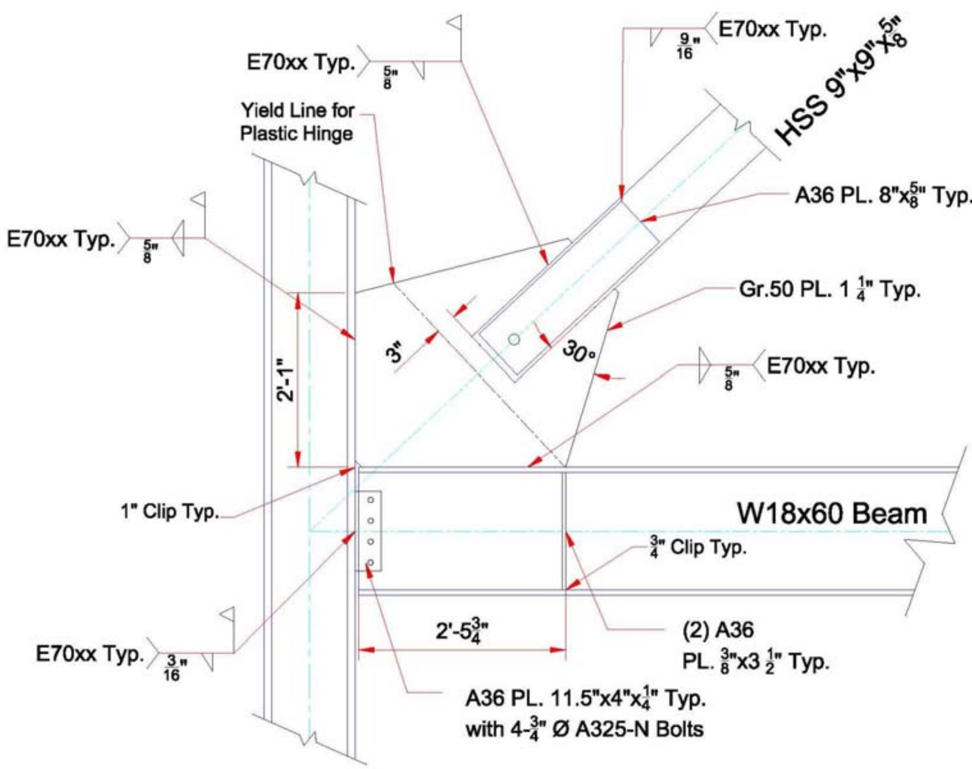
AISC 341-05 Section 13.5 requires all splices be located in the middle third of the column clear height so to prevent a story mechanism. This section also requires that the splice shear strength can carry the strength of the lesser two column shapes in shear. These requirements were implemented in the column splice design, to which a plate on each side was added to carry the shear demand and a CJP was used to carry the flexural capacity of the members. Also all column splices were located at the mid-height of the clear column.



Brace-Beam-Column Connection

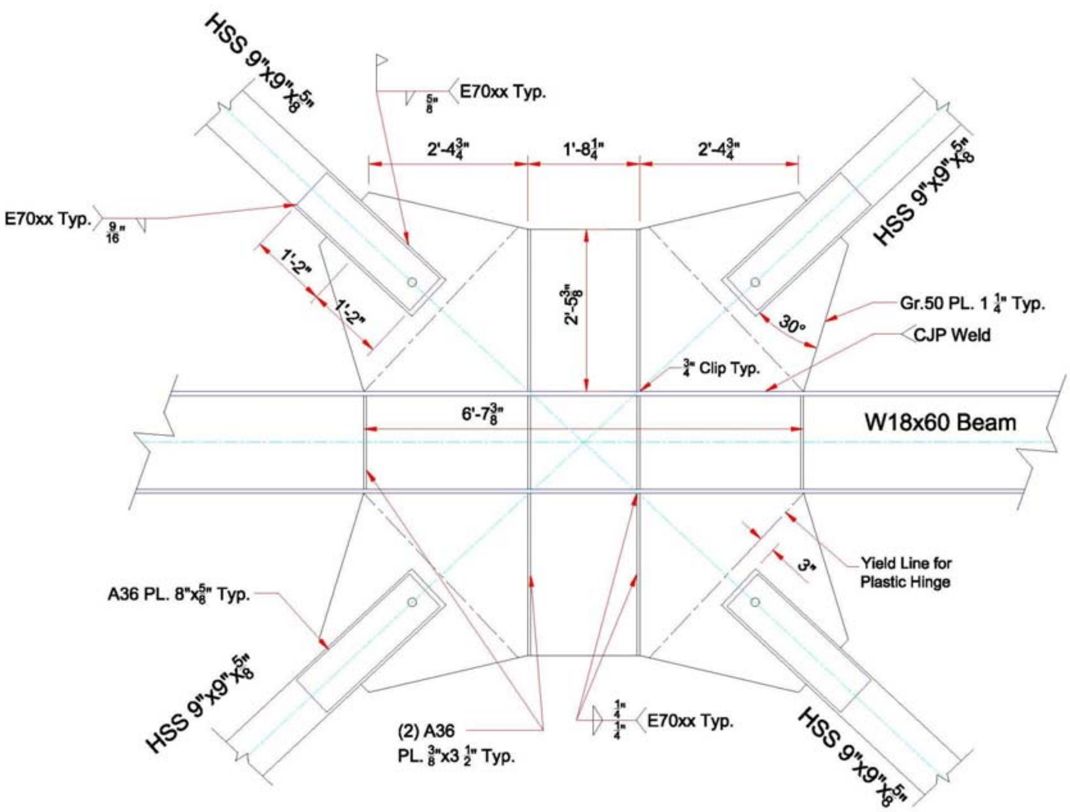
When this connection was being designed the Uniform Force Method was used to determine the length and height of the gusset plate so that there were no moments on the three connection interfaces. The connection along with the gusset plate was designed to meet AISC 341-05 Section 13. The gusset plate is designed so to develop a plastic hinge and buckle out of plane, it also has to be able to take the expected capacity of the brace in tension and also in compression. Buckling limitations were checked and followed so that the plate behaves correctly under severe loading. A Grade 50 steel plate was used to help cut down on the overall size and thickness of the plate which is allowed in AISC 341.

The gusset plate was designed so that the plate was welded to the beam in the shop for this approach is simpler for infield setting. The beam to column connection part was designed as a simple shear table for the moment was low but also because the relative lengths of the gusset plates would take and resist the moment. The brace had to be reinforced around the connection interface due to the slot taking away from the cross-section and also an issue with shear lag. The drawing shows the details of the design; the brace connection would be repeated on the lower half of the beam but was omitted for clarity of the connection.



Brace - Beam Connection

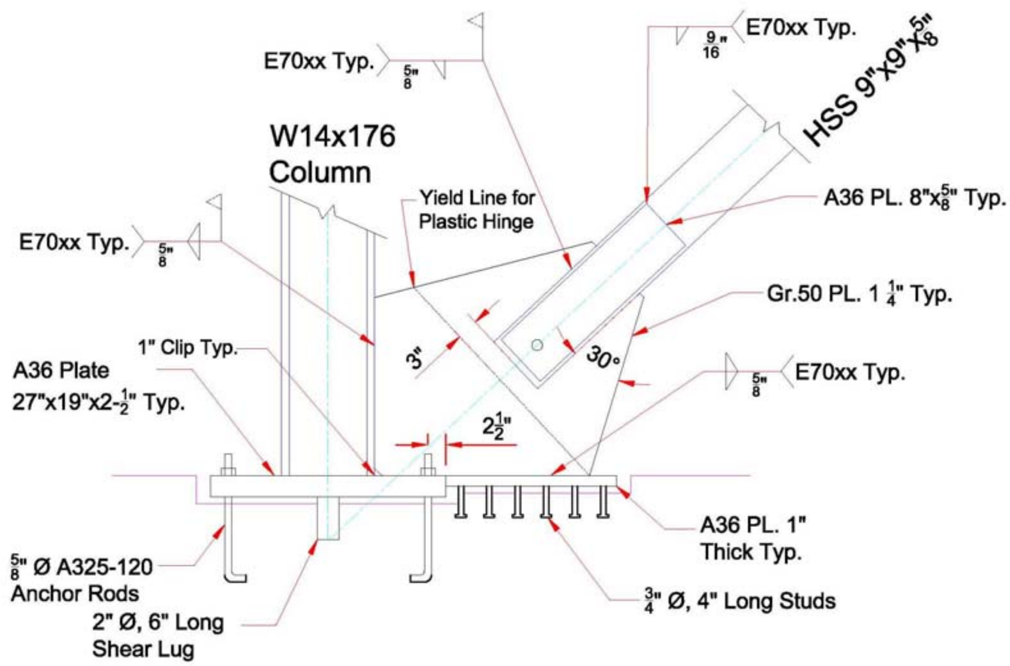
The gusset plate was designed to meet AISC 341-05 Section 13. The gusset plate is designed so to develop a plastic hinge and buckle out of plane, the gusset plate has to be able to take the expected capacity of the brace in tension and also in compression. Buckling limitations were checked and followed so that the plate behaves correctly under severe loading. A CJP was used so to cut down on the length and size of weld needed along the beam. The brace had to be reinforced around the connection interface due to the slot taking away from the cross-section and also an issue with shear lag. Finally the stiffener plates were required so that the brace didn't buckling in the center but allow the brace to yield below where the brace ends. The drawing shows the details of the design.



Column Base Plate

To obtain a fixed end at the column's base of the foundation in the lateral system the base plate and anchors need to be able to resist moment. This can be achieved by using a thick base plate that won't bend. The column is then CJP welded to the plate to endure it is rigid and sufficiently strong. To ensure a rigid action the plate had to have a larger moment of inertia (I) than the column, to achieve this anchor rods were placed at the outer edge of the plate to keep the I larger. These bolts were than determined that a through nut would be best so to be able to place them in during the foundation pouring.

A space was needed so to be able to grout the plate to the foundation as one, also a shear was placed on the base plate to take the shear so the anchors didn't have to carry it all. As it can be seen in the drawing it was necessary to add a plate for the gusset plate t rest on and act as it should. Also since the ground floor is inhabited the base plate could need to be "counter sunk" into the foundation so that it does not interfere with the architecture and floor space. For this Connection and detail Ram Base Plate was used to design the elements based on the correct corresponding material properties and the forces inputted.



Seismic Drift

Drift is a serviceability issue and should be limited as much as possible while staying within reason. The allowable seismic story drift was calculated using ASCE 7 Chapter 12. Story drift for each floor is calculated per equation 12.8-15 and allowable story drift is per equation 12.12-1. Drift was looked at and limited so to stay within the allowable. Drift was a controlling factor in the lateral system and iterations were done with the design so to get acceptable values as code dictates.

The deflection values were taken from Ram Structural System at the center of mass at each floor which is permitted by ASCE 7-05 Section 12.8.6. The change in deflection from one story to another was obtained to track any possible jumps and compare these changes against the allowable values. After examining the deflections and working on limiting it the final design it can clearly be stated that a steady increase of deflection going up the building. No story in either direction fails in meeting the allowable drift. The tables below show sample calculations of the drift.

Drift and Displacement Calculations for SCBF N-S Direction							
Story	Height (Ft.)	hsx (ft.)	Story Displacement (in)	δxe (in)	δx (in)	Δa (in)	Final Results
Roof	10	10	3.236	0.494	2.257	2.400	Good
8	10	10	2.742	0.487	2.223	2.400	Good
7	10	10	2.255	0.441	2.016	2.400	Good
6	10	10	1.814	0.433	1.977	2.400	Good
5	10	10	1.381	0.364	1.663	2.400	Good
4	10	10	1.017	0.388	1.774	2.400	Good
3	10	10	0.629	0.275	1.256	2.400	Good
2	10	10	0.354	0.257	1.175	2.400	Good
1	10	10	0.097	0.097	0.441	2.400	Good

Drift and Displacement Calculations for SCBF E-W Direction							
Story	Height (Ft.)	hsx (ft.)	Story Displacement (in)	δxe (in)	δx (in)	Δa (in)	Final Results
Roof	10	10	3.402	0.483	2.316	2.400	Good
8	10	10	2.919	0.441	2.115	2.400	Good
7	10	10	2.478	0.487	2.335	2.400	Good
6	10	10	1.991	0.478	2.291	2.400	Good
5	10	10	1.514	0.398	1.907	2.400	Good
4	10	10	1.116	0.422	2.022	2.400	Good
3	10	10	0.694	0.299	1.432	2.400	Good
2	10	10	0.396	0.280	1.341	2.400	Good
1	10	10	0.116	0.116	0.557	2.400	Good

Soft Story

To ensure that there are no soft stories which would result in vertical irregularity, calculations were computed to verify if any exist within the new system. The tables below verify that the final design does not have a soft story issue and therefore ELFP is still valid.

Soft Story Check for SCBF N-S Direction						
Story	Story Drift	Drift Ratio	0.7x the Story Drift Ratio	0.8x the Story Drift Ratio	Avg. Story Drift Ratio of Next 3 Stories	Soft Story Issue
Roof	0.494	0.0494	0.0346	0.0395	--	No
8	0.487	0.0487	0.0341	0.0389	--	No
7	0.441	0.0441	0.0309	0.0353	--	No
6	0.433	0.0433	0.0303	0.0346	0.0474	No
5	0.364	0.0364	0.0255	0.0291	0.0454	No
4	0.388	0.0388	0.0272	0.0311	0.0413	No
3	0.275	0.0275	0.0193	0.0220	0.0395	No
2	0.257	0.0257	0.0180	0.0206	0.0343	No
1	0.097	0.0097	0.0068	0.0077	0.0307	No

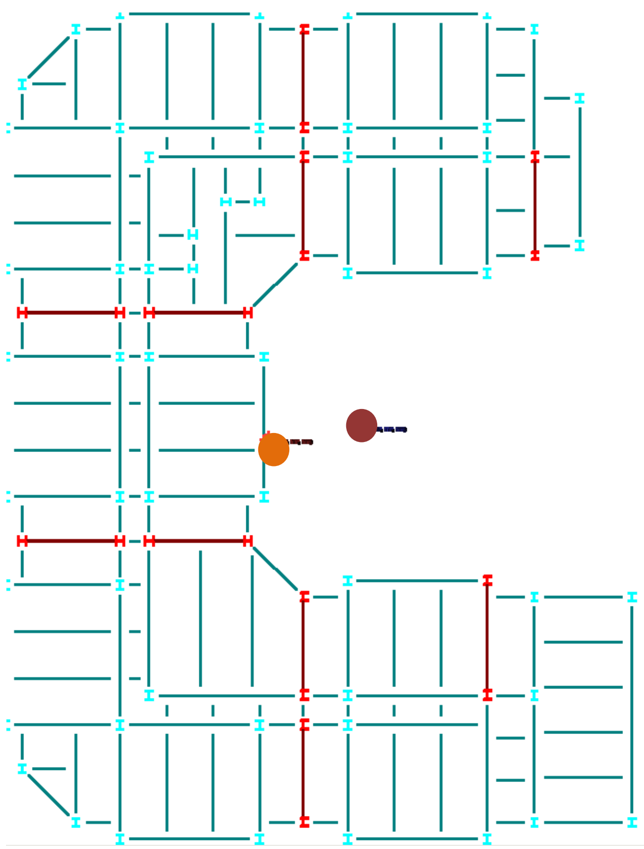
Soft Story Check for SCBF E-W Direction						
Story	Story Drift	Drift Ratio	0.7x the Story Drift Ratio	0.8x the Story Drift Ratio	Avg. Story Drift Ratio of Next 3 Stories	Soft Story Issue
Roof	0.483	0.0483	0.0338	0.0386	--	No
8	0.441	0.0441	0.0309	0.0353	--	No
7	0.487	0.0487	0.0341	0.0390	--	No
6	0.478	0.0478	0.0334	0.0382	0.0470	No
5	0.398	0.0398	0.0278	0.0318	0.0469	No
4	0.422	0.0422	0.0295	0.0337	0.0454	No
3	0.299	0.0299	0.0209	0.0239	0.0432	No
2	0.280	0.0280	0.0196	0.0224	0.0373	No
1	0.116	0.0116	0.0081	0.0093	0.0333	No

Torsion Effects

Center of Mass and Rigidity

For each diaphragm the center of mass (COM) and center of rigidity (COR) were calculated so that the exact location of the resultant story force was is located. These two points on the diaphragm determine how much eccentricity there will be, which in turn will cause a torsional moment on each floor. A sample calculation was performed on a typical upper level floor plan. The figure below shows the location of both. The orange dot is the COM and the red dot is the COR.

These locations seem valid for more of the mass is on the west side while the mass is near equal on the north and south side. The stiffness in the east-west direction is equal for the four frames all have the same stiffness. In the north-south direction there are more frames which shift the COR in their direction which is what is happening.



Same calculations of these values were done and are very close to the Ram's. These numbers were used for the torsion calculations listed in the next segment. The values are almost the same for each floor with respect to each other due mostly to each floor being relatively the same layout and plan area except for the first floor. Values to these can be found in Appendix D.

Inherent Torsion

When the center of mass and the center of rigidity are not located in the same exact spot then there is inherent torsion acting on the diaphragm which is carried to the lateral system. ASCE 7-05 Section 12.8.4.1 was followed when looking at this issue and it was determined that the building had a rigid diaphragm. The Y direction had a close COM and COR resulting in a small torsional moment but the X direction had a larger torsional moment due to the COM and COR being farther apart. An analysis was performed to determine the torsional shear on each story caused by wind forces.

Accidental Torsion

Since Building 7 is now in a Seismic Design Category D it is required to consider accidental torsion now. ASCE 7-05 Section 12.8.4.2 was followed to calculate the accidental torsion at each story on the structure when a 5% eccentricity is created in each direction individually from the center of mass. When calculating the Accidental torsion an amplification factor needs to be multiplied to this value. ASCE 7-05 Section 12.8.4.3 was followed and determined, in the cases where it was less than one a value of 1.0 was used and a max of 3.0 could be used. It was determined though when looking at the data and performing the calculations that the amplification factor for the structure was all under 1.0. Values to these calculations can be found in Appendix D.

Integrated MAE Work

To meet the requirements of the integrated class with advanced topics covered in graduate level classes, several key areas and sections of material were utilized in this structural study and redesign. All of the results and details can be found throughout this report in the previous sections. The reason why this was not grouped separately was because it was more relevant to place those sections with the corresponding section of the report it deals with. Listed in this section is a brief overview of what was used for the MAE required integration.

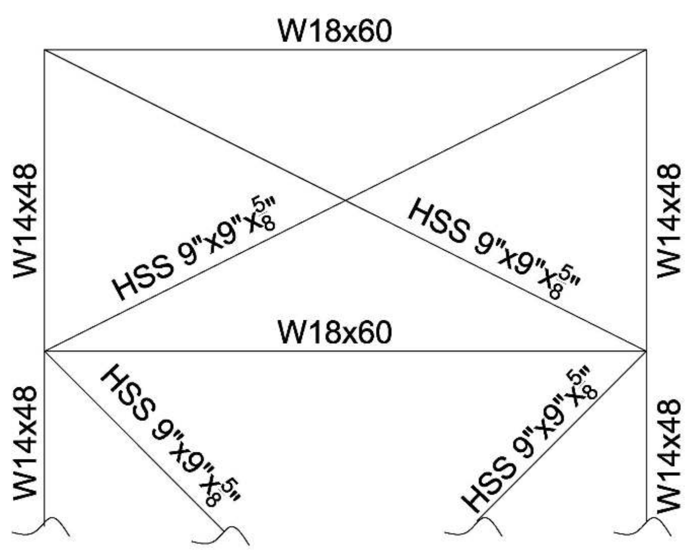
- Advanced computer modeling techniques were used so to obtain more accurate results and include more variables when the corresponding codes required it.
- The study and design of both gravity and lateral connections were looked at for the new structural steel system.
- By moving the building to a high seismic zone a more in-depth lateral analysis and special criteria had to be determined when designing the MLFRS.

Structural Alternatives

Top Story Brace Configuration

When the top stories of the Braced frames were laid out it was noticed that an inverted-V was going to be present. Since this configuration lead to massive beams as it is shown in the Seismic detailing section of the report it was realized that this configuration at the top isn't the most optimum for the beam itself is deeper than the total thickness of the ceiling cavity. An alternate configuration was considered and the alternate is a single story X configuration so that the members did not frame into the mid-span of the beam creating a requirement for the unbalanced load in the brace members. This alternate configuration allowed for the same style of gusset connects along the column beam interface as before on the lower levels.

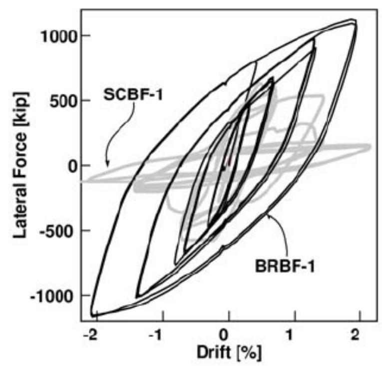
It was determined through rerunning the model with the new configuration that the sizes of the members were able to be reduced to HSS9"x9"x5/8". This is mainly due to their being more brace on that level to take the shear. The top beam was able to stay at a W18x60. The connections were not looked at in detail but since the member sizes went done it can be said that the gusset plate sizes went down as well. This new configuration has a small positive contribution to drift for it reduces it by a small amount but isn't a large contributor overall. There is a reduction in the price of this configuration for the top beam is smaller and the larger massive connection at the top is replaced by four small connections. The figure shows the sizes of the members for this new configuration.



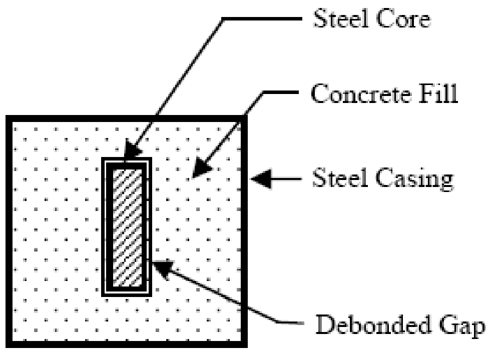
Alternative Brace Frame Type

Since the gusset plates are getting thick and are requiring large welds along with the braces needing to be reinforced at the connection interface there could be a more efficient approach to the lateral system can be considered. Based in the information listed at the beginning of the lateral Study the next best alternative would be to look at the Buckling Restrained Braced Frames. With BRBF brace members can act as a true pinned connection with the Stare Seismic LLC brand, which uses a pin connecting it to the gusset plate. The gusset plates themselves tend to be smaller for they tend to have lower forces because of the R value BRBF being a 7 or 8 depending on if the beam column connection takes moment.

Even thought BRBF's are not a proprietary system their configurations and details of the assembly and in some cases the connections are subject to US patent laws and recreation is limited to the holding companies. This does not exact fit with the structural goals of eliminating the propriety systems but the benefits especially for building 7 would be worth it. The BRBF brace members tend to be more expensive then the standard HSS or W-shapes SCBF but the connection terms and labor related to making the SCBF connections would be cheaper. The last disadvantage is that the design of BRBF's involves some complexities in modeling and also in managing drift control in modeling.



The pictures and figures below show the standard details of BRBF systems and it can be seen when comparing them to the seismic connections that were design for this thesis in the lateral detailing of this report.



Fire Proofing

Structural steel may be very durable but when it comes in contact with high heat from a fire its strength can rapidly decrease and speed up the failure. The original building was bearing walls and Hambro Joists, these systems did not require fireproofing in large volumes but instead had complex UL rated assemblies had had to be met to ensure the correct fire rating for the different sections.

When the gravity system was designed the deck was chosen and designed such that the decking did not require spray on fireproofing. This decision can and will save an enormous about of fireproofing for not having to cover the decking cuts down on the overall area. Even thought he deck doesn't need fireproofing the steel beams, girders and columns do require it. Upon looking into the different types of fireproofing it was concluded that a cementitious plaster based fire proofing with a combination of Portland cement and lightweight aggregates, vermiculite and perlite. Typically the columns get a much larger thickness for they are more critical. It can be seen in the table below the overall cost of the fireproofing for the gravity system when the deck is sprayed and when it is not. The overall savings is $1.14 when the deck is designed to have no spray of fire proofing. Note these numbers also include the materials costs for the steel, concrete and deck, though the prices of these materials were the same cost as noted in RS Means.

Level	Area	SOFP for All	SOFP Except Deck
Roof	14750	$ 378,337.50	$ 356,950.00
Floor 8	14750	$ 310,487.50	$ 294,262.50
Floor 7	14750	$ 310,487.50	$ 294,262.50
Floor 6	14750	$ 310,487.50	$ 294,262.50
Floor 5	14750	$ 310,487.50	$ 294,262.50
Floor 4	14750	$ 310,487.50	$ 294,262.50
Floor 3	14750	$ 310,487.50	$ 294,262.50
Floor 2	14750	$ 310,487.50	$ 294,262.50
Floor 1	12621	$ 265,672.05	$ 251,788.95
	Total Cost	$ 2,817,422.05	$ 2,668,576.45
	Savings Total	$ 148,845.60	
	Saving per SF	$ 1.14	

The lateral system would also need to be sprayed for it is all steel. A direct comparison can not be made for everything needs to be sprayed and the original was an assembly wall. Still though the lateral system would be costly to fireproof due to the large members and also that the gusset plates are very large resulting in more area to protect.

Foundation Implications

Due to the change in location and also the change in the structural system it can clearly be seen that the foundation system would be affected. The original system had bearing walls and shear walls which resulted in more strip footings than spread footings. The new system has more columns which will mean that more footings are required, however since the new site has a better soil conditions less piles may be needed. This system was not looked into detail for the depth of this thesis but certain areas were looked at in a more general aspect to see how things would be affected.

Code Requirements

Due to the change in location to a site with a Seismic Design Category D, special requirements need to be met with the new location. ASCE 7-05 Chapter 19, Section 19.2 needs to be followed so to incorporate the soil-structure interaction of the foundations can be considered and modeled. The foundation requirements for SDC D as controlled by ASCE 7-05 Section 12.13.6 which tells that the foundation must be tied to the piles and caps along with the correct procedure to design the different styles of foundations. Section 12.1.5 tells that the foundation must be designed to accommodate the dynamic ground motion, structure movement, the shifting of the soil creating stresses on the soil, and also energy dissipations requirements.

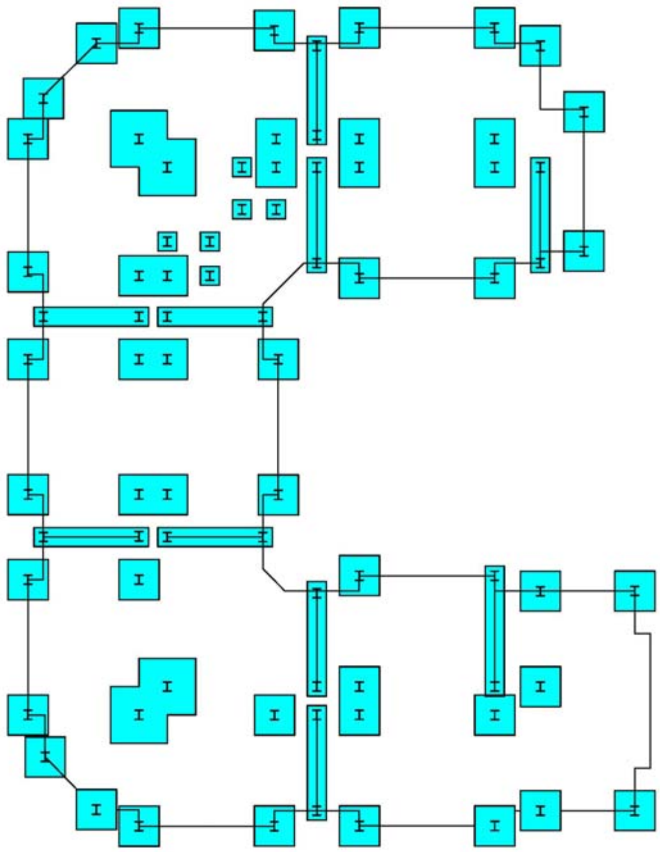
It is because of these requirements in Chapter 12 and 19 that the foundation design was not looked at in details and very simplified assumptions were performed in the next segment of this report for the code requirements of the foundation system is beyond the scope of this thesis study.

Schematic Design of the Foundation

In this schematic design of the foundation only the gravity loads were considered for the complexity of the seismic lateral provisions related to SDC D was beyond the scope. The each of the columns would require a separate foundation except with the columns on each side of the corridor for since they are close a shared foundation would be best for them. Gravity loads were l taken from the Ram Structural Model and from here a foundation area was determined based on a soil bearing capacity of 4000 psi with this information an area was determined for the foundations.

For the foundations under the lateral system exact sizes were not calculated but it is safe to say that strip footing would be present from one side of the frame to the other. Beneath these on the ends would be the caisson grouping with the cap connecting it to the footing. This is the same configuration that is currently being used but since the forces are higher it very well could make this foundation larger especially on the uplift design.

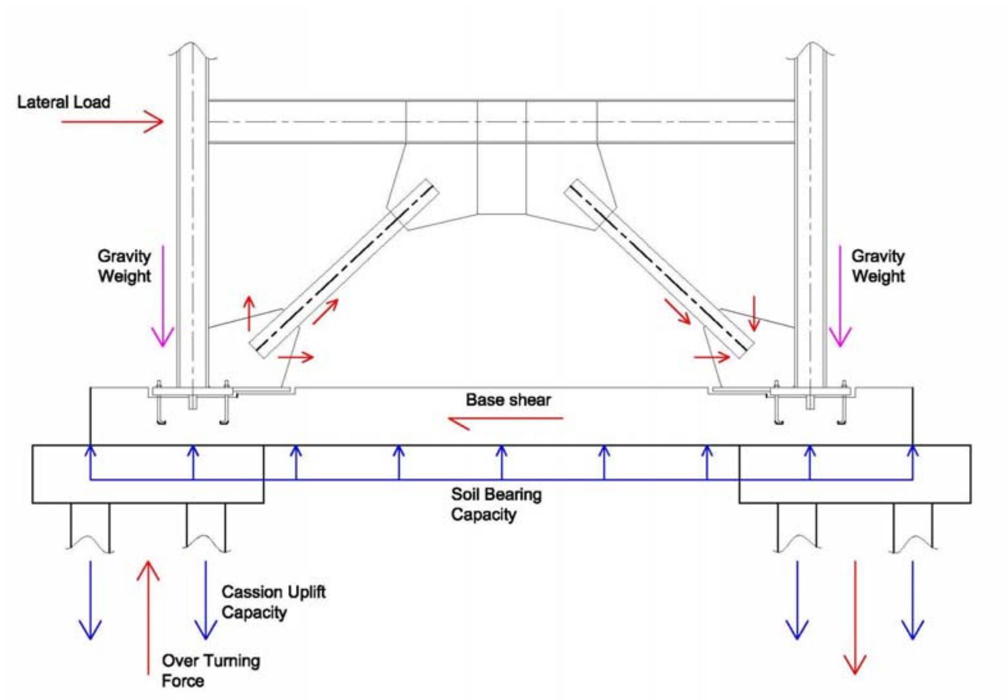
When looking at the new schematic foundation plan on this page as compared to the original found in Appendix G it is clear to see that the new steel system requires more and larger foundations also it can been seen that some of the foundations are very close to touching and in two cases they are overlapping creating a complex shape. The table below also shows the sample calculations made for the gravity schematic design.



	Load (Kips)	Soil Bearing (ksf)	Area needed (sf)	Sizes (ft.)
Typical Exterior Column	355.00	5.56	63.90	8
Typical Interior Column	324.00	5.56	58.32	8
Corridor bend Column	600.00	5.56	108.00	10

Acting Forces on the Foundation and Overturning

Overturning issues in foundations arise when the forces on the lateral elements are greater than the weight that the lateral element. Also the soil bearing capacity has an effect on overturning by how much load it can take before a strength failure or a bearing failure occurs. When the lateral moments and axial forces are not balanced out by the weight and soil capacity, then the foundation wants to start and tip over inside the ground. One end tends to lift up while the other often likes to sink into the soil. Since seismic foundation design was not considered and the complexity of the structure it is hard to tell if the new foundation will have overturning issues. However the figure below shows how the forces are interacting with each other at the foundation level. The red loads are trying to force the foundation to rotate as the blue loads are trying to resist the movement.



Cost Considerations

When looking at the structure of a building the cost of the structure plays a major impact in the overall selection of a system but also helps set the price of the building's total cost. The original structure cost $2.93 million when Building 7 was located at University of Maryland. Since the building was moved to San Diego now a direct comparison of just cost for changing the structural system to a steel system is hard and not necessarily a good representation.

High seismic regions have a tendency of having a high structure cost due to the lateral complexity of the connections and also the mass of the members. Also west coast practices have different common in-field techniques from east coast especially regarding welding. Welding in the west coast is more regulated and all welds are required to be inspected. This will result in higher costs in labor. All these factors and many not mentioned are a large contributor to the overall cost. Because of these implications it is clear and reasonable to state that west coast building are more costly to design and build, the structure alone is more costly and when designing in the west coast a different way of thinking is needed when designing for structural costs increase and more of the overall budget will be in this part of the building as compared to the east coast.

Additionally there are other designs a structural engineer must perform that will increase the cost. These include but are not limited to: mechanical system attachments, non-structural architectural components and walls systems-both interior and exterior.

Structural Design Summary

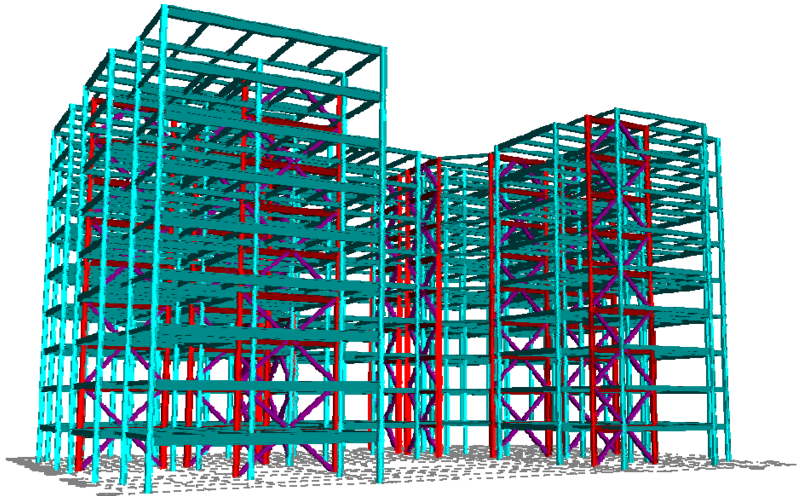
After completing the structural depth study of redesigning the entire system to steel and moving Building 7 to San Diego, CA to look at the effects and implications of a high seismic zone it was determined that the structural goals were met. It was clear that a structural design in steel can be developed to carry the loads required by code, without a large impact on other aspects of the building. This while the building height did not need to be changed in order to fit the gravity beams based on the study, it is recommended that in the end the height of the floor cavity should be increased for there is still limited room left after the steel. The double loaded corridor allowed this to be more acceptable for there was more space in the corridor ceiling but any MEP systems going to the adjacent bays would need to be carefully designed to fit into the smaller remaining space.

For the lateral system redesign the overall process and goal was met for all the required checks and procedures for designed in an SDC of D were performed. The seismic loading was determined to be very high and thus controlled the overall lateral design for the wind didn't change significantly in the move to San Diego. A computer model was created and the lateral loads and load combinations were inputted and the lateral system as designed. In the end the lateral frames were designed to meet the requirements for strength and drift. The resulting members of the SCBF's were on the large size but considering the magnitude of the force.

The final lateral system design is acceptable for Building 7 and is more efficient and recommended that the alternatives be considered in the future and further developments for they can help the lateral system for the better. While this layout is acceptable it required a redundancy factor of 1.3 because there was no perimeter framing in each direction, also the inherent torsion was also large for the limited space to place the SCBF's within the plans which would not affect the architecture. For this reason it is recommended that further developments of perimeter lateral frames be used to reduce the redundancy factor and also try and limit the torsion more.

Both gravity and seismic connections were looked at for this study. The two gravity connections were both in the end determined to be shear tab connections for the ease and speed of construction of these connections made them ideal. For the lateral seismic connections typical connections for a SCBF were considered and designed. In the end the seismic connections became very complex and large due to the required forced for the members to behave correctly under seismic loading.

Lastly fireproofing issues and cost savings from not fireproofing the decking were considered along with an alternative brace configuration at the top of the lateral frames. The alternative configurations was in the end a better choice for the members were smaller and work better with the architecture along with the cost of this configuration was less. Another alternative or consideration after completing this thesis report and study is that due to drift being an issue with the SCBF that an alternative study of Buckling Restrained Braced Frames would be a valid choice for the reduced connections and better control of under loading as proved by the Hysteresis Loops.



Structural Goal Evaluation

Goal 1: Design an overall structure made of steel and has limited use of propriety systems.

This goal was achieved for the design is using rolled shaped members and plates. It should be noted though that the other systems not considered in this report may have to be propriety systems but the major contributors to the structural system are not now.

Goal 2: Design a gravity system that does not require a change in the building height while still being acceptable.

This goal was achieved with the new design by using a double loaded corridor which gave room along the corridor to run the MEP systems. This goal has its disadvantages for the space is still limited and would most likely result in an inefficient design of the MEP systems. In the end this works but the ceiling/floor cavity should still be increased.

Goal 3: Move the location of Building 7 to a high seismic to better understand and work with seismic requirements in detail.

This goal was achieved for the move in the location gave a very good understanding of the seismic requirements involved in a SDC of D. All relevant codes and design practices were used when acceptable and research was needed when it came to detailing of the connections.

Goal 4: Pick a single lateral system that will work for the new location and design it while trying to optimize it.

This goal was achieved for only SCBF's were used as the new lateral system. The overall design of the SCBF's was approaching large sizes members due to drift requirements and torsion issues. It is recommended that a further study involving a perimeter system as well changing the SCBF to BRBF for they have small connections and behave better.

Breadth Studies

Breadth 1: Green Roof Study

The first breath topic is looking at a green roof added to the top of Building 7. This was considered because it can count towards a LEED/green design which the owner and architect want the building to be certified at a minimum. Originally the green roof was not considered due to cost reasons but this study as be performed as if the budget wasn't the issue. Green roofs have benefits with being able to recycle the water from the roof and can collect excess runoff to be used throughout the building for the sanitary system. For this breadth a green roof will be looked and the best choice will be picked. Waterproofing issues, weight issues, and the collection tanks with the piping will be designed for this breath. Since the building changed location the green roof will be designed such that the design is valid for both locations.

Materials and Considerations

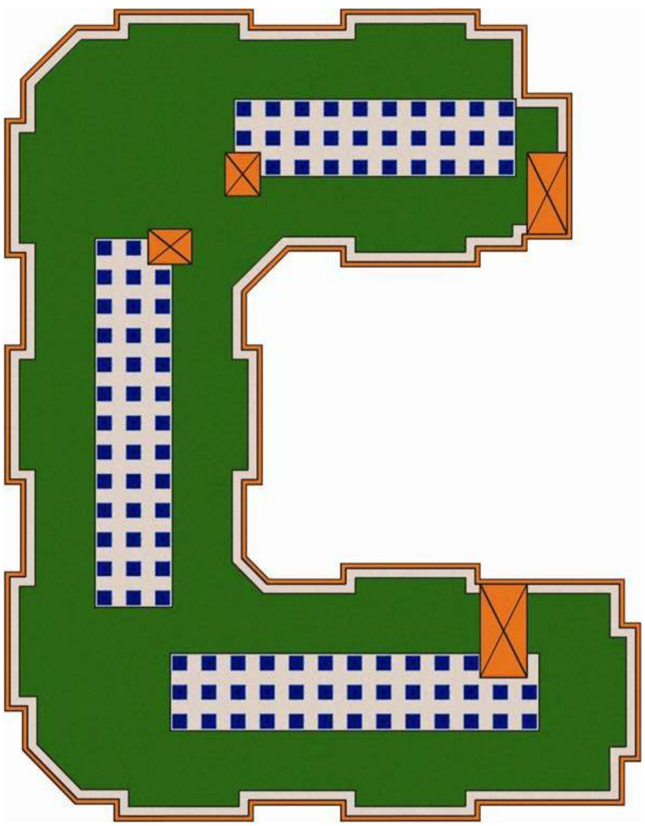
After researching the different types of green roofs it has been determined that an extensive green roof is the best option. The reason for this choice is due to the massive amount of mechanical units take up valuable space on the roof plus since the overall roof wasn't suppose to have people on it this roof makes it the best option. Extensive green roofs are designed to be virtually self-sustaining and they require only a minimum of maintenance. Typically it's a yearly weeding and an application if a slow release fertilizer. Green roofs have also been found to dramatically improve a roof's insulation value. These improvements can be as much as 25% reduction in summer cooling while also gaining approximately the same reduction in winter heat losses. This type of green roof can be expected to lengthen a lifespan of the roof by up to two times.

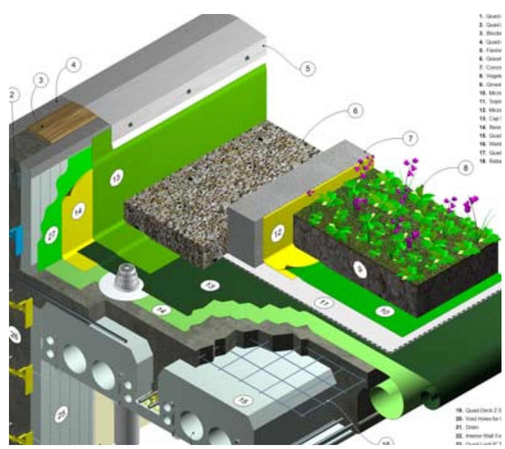
After looking at the different types of plant life available and most well suited for both locations it was found that the best species for a green roof is the sedum species and mosses. This specie is very low maintenance and also very hardy. There are multiple types of plant within this range that are better for cold weather and some that are better for warm weather. The price difference between them isn't a relevant difference. Shown in the pictures below are two varieties that would be used for this thesis.

Roof Layout and Details

When looking at the roof plan so to figure the best layout and style of green roof, it was found that there were many mechanical units occupying the area. These mechanical units, 111 units in total were placed in three primary areas while some of them were scattered in different small locations. For the design of the green roof to be most efficient, all of the scattered units were moved and aligned in with the rest of collected areas forming three key patches, one on each side of the U. The units were shifted so that the area was centered in the wide of that section so to provide more room around the room.

Above the concrete slab are two rubber waterproofing membranes that are sealed chemically so that water cannot penetrate into the slab. Above this is a mesh that allows for excess water that the soil cannot hold and safely moves it along to a drain. This mesh is the barrier between the membrane and the soil. From here 3" of soil is placed on top followed by the plant life listed. Around the perimeter is a 1' wide strip of gravel followed by a curb so to ensure that the edge drains properly so there are no water issues along the exterior wall. Along with the gravel edge this gravel was placed around the mechanical units so that they did not interfere with the grass or any possible maintenance that may occur on the units. In the figures below are the final layouts of the new green roof for Building 7.



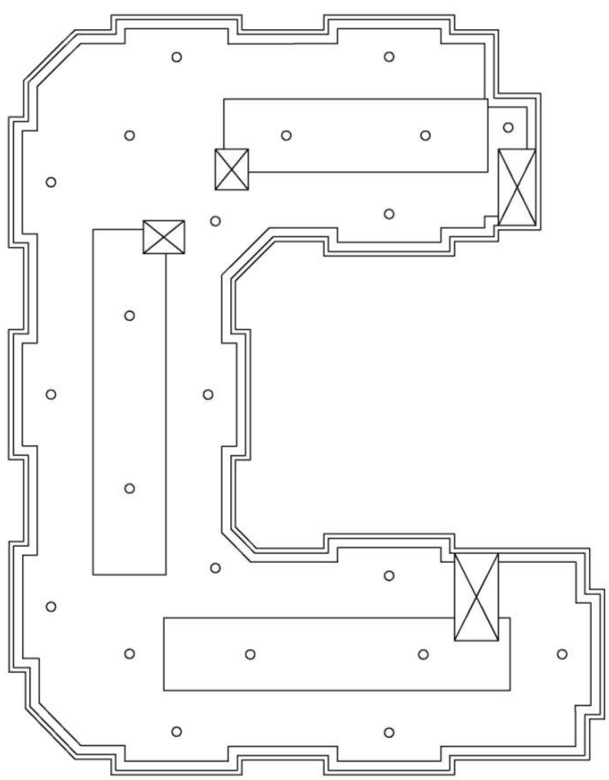


The figure on the right, though it does not have the same structural system as the new Building 7, this figure accurately shows the multiple layers of water proofing membranes and also all of the materials and their order for constructing the green roof.

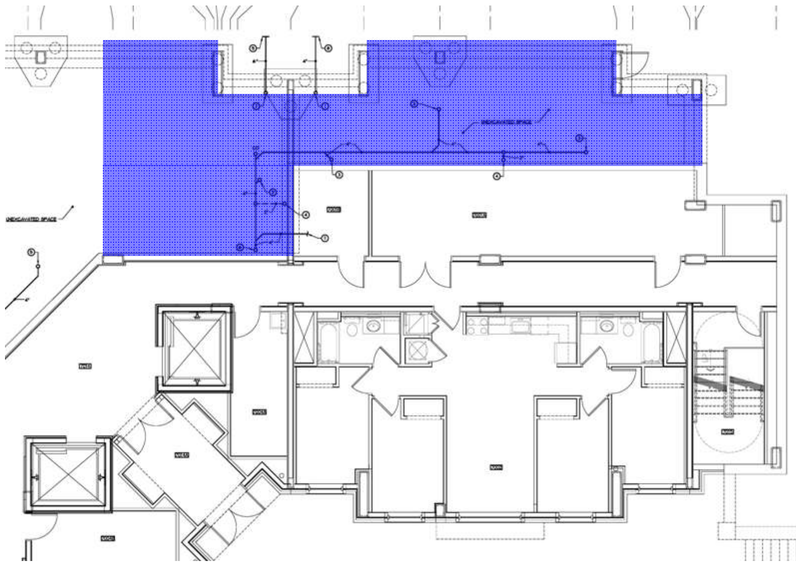
Water Collection Design

When considering using the excess water runoff of water from the roof as a means of using it for other gray water systems in the building the average annual rainwater was considered instead of the rainfall in a single storm. This number was found on National Oceanic and Atmospheric Administration (NOAA) and it was determined to be 44” for College Park, MD and 5” for San Diego, Ca. To determine how much water that would need to be stored the roof area was calculated then the required water that the roof would need was subtracted from the overall, the rest is the extra to be stored in the lower ground level.

Once the rain level was determined it was then possible to design the drains and the piping that would be going to the tanks. The numbers of drains and their locations were determined and the plan below shows their locations. After the number of drains were determined it was then possible to calculate the size of the pipes. The average sizes of the drains are 6” within the main regions of the roof and 3” along the perimeter within the 1’ gravel strip (these 3” drains were not shown on the plan for difficulty of reading).



The location of the holding tanks was determined to be best placed in the mechanical and electrical room on the terrace/ground floor. The room as seen in the figure below is not fully excavated for that space was originally not needed. This unexcavated space (shown in blue) is the perfect place for the tanks and is large enough for both locations.



From here the tanks were chosen based on the amount of water they had to hold at each location. The tanks for San Diego, CA were determined to be 1-500 Gallon tank while the size for College Park needed to be 2-1000 Gallon tanks. It should be noted that these tanks sizes are recommended but not required for the tanks are designed that once they are filled the excess water that may want to come into the will be redirected into the storm water removal system. Refer to Appendix E for calculations of the tank sizes and also for the LEED score card for the original design.

LEED and Benefits

This new design that was created is beneficial to the environment as well as to the building owners and occupants. Because of the green roof and water collection design it was possible to gain 2 LEED points. The First LEED point is the reduction in the heat island effect on the roof, this point was obtained by the function of the green roof itself to help isolate the thermal properties. The second LEED point was the reduction in the water consumption, this point is hard to justify since details of water consumption rates for the building were not know but the storage tanks would certainly reduce the overall consumption. The reduction of water consumption varies on the two locations, College Park being more efficient mainly due to that location having more rain. So overall this green roof has many benefits for Building & in both locations and with the 2 LEED points places it at a LEED Gold.

Breadth 2: Acoustic Study

The second breath topic will be to look at a single apartment/dorm room and will study the impact of the change in the structural material to steel and see if sound isolation is an issue. If sound isolation is a problem then the walls and floors will be designed to prevent as much of the noise as possible from being transmitted from one area to another. If sound isolation is not an issue a higher standard from the minimal will be used/designed since the building is a dorm and the noise environment tends to be quite loud at times and specialty equipments/treatments can be looked at mechanical duct isolators, resilient channels on the walls, etc.

Sound Isolation Verification

Two different locations were looked at when verifying the sound isolation of the new system to see if it meets the requirements and is a good design. The locations for these two locations are the walls between each room within the apartment and also the walls from one apartment to another, both side by side and above and below. The original structure was not looked at for it was felt that the original met the requirements as part of the actual design process for Building 7. Sound Transmission Classes (STC's) and Impact Isolation Classes (IIC's) were determined using the Building Constructions tables in *Architectural Acoustics.* Also the standard/recommended values for between two areas for both STC and IIC were used from this same source. The figure here shows the typical apartment and the locations of the rooms within each.

The new structural system has an increased deck slab thickness thus resulting in it having a higher STC and IIC; this is shown in the summary table below and shows that the new system is ok for sound issues. Since the new structural system has no bearing walls the section of these walls being looked at till change, primarily the Apartment to Apartment are now different since the old was the shear wall but now a wall with a brace frame going through it. The other wall barriers between the rooms within the apartment are sufficient and passed is excellent for they did not change from the original design.

Since this wall changed a new type needed to be considered. From the determined the requirements for this barrier wall it was concluded that 2 rows of 3-5/8" steel studs at 24" O.C. with 2 layers of 5/8" gypsum board with glass insulation along with the air space between the braced frame was the best choice. The reason for not choosing a masonry wall was because for the likely hood of the braces damaging them from seismic drift.

Space/ Location	Building Construction	STC		IIC		Pass
		Actual	Recommended	Actual	Recommended	
Bedroom-Bedroom Same Apt.	3 5/8"steel stud 24"OC with 2 layers of 5/8"Gyp board with glass insulation	57	50	-	-	Pass
Bedroom-Living Room	2 1/2" steel stud 24"OC with 2 layers of 5/8"Gyp board with glass insulation	51	55	-	-	Pass
Bedroom-Bathroom	2 1/2" steel stud 24"OC with 2 layers of 5/8"Gyp board with glass insulation	51	55	-	-	Pass
Bedroom-Bedroom Diff. Apt.	2 rows of 3 5/8"steel stud 24"OC with 2 layers of 5/8"Gyp board with glass insulation and the air space the braced frame takes	65	50	-	-	Pass
Level to Level with Ceiling	6" Conc. Slab w/ a resilient suspended ceiling	47	55	35	52	Pass

Advanced Measures in Isolating Noise

The above Verification of the wall and ceiling systems show that new bedroom to bedroom wall design and the existing wall designs along with the ceiling meets the STC and IIC. Even though these meet the criteria it is often the fact that sound leaks can occur and also a problem with low frequency sounds, such as base players. Due to Building 7 being a dorm often noise leaks and low frequency sound transmissions are over looked, these can be a problem for the resident's tend to enjoy low frequency music.

For this reason the following techniques listed to keep the sound from creeping through into other locations and also to try and cut on the low frequency transmission the following modifications to the design of Building 7 should be done. Note most of these were not employed with the original design for lack of reason based on price concerns but for this project should be used for cost isn't an issue for these along with making the dorm more luxurious and inviting to the quite crowd.

Sound Leak Solutions

* For the doors change them to solid panel wood doors and use a threshold gasket around the entire perimeter of the opening.

* All outlets in the same wall but on different side at least 2 ft away and ensure a stud barrier is between the two with insulation around the entire back of the outlet.

* Caulk the perimeter of the base plate of the wall around the gypsum board to stop all leaks.

* Ensure all partitions go clean to the floor slab above the ceiling to act as a barrier. If not place neoprene barrier film between the partition and the slab

Isolation Components

* Place the rooftop mechanical units on ribbed neoprene pads along with unrestrained springs so to isolate the vibrations of the units.

* Use isolation hangers for all primary pipes and ducts so to reduce the sound of turbulence.

* Use wire isolation hangers for the ceiling along with 2 5/8" gypsum board ceiling that has been caulked all around the perimeter with acoustical sealant in all rooms and common areas.

* Place turning vanes in all air ducts to ensure smooth transitions around bends. Also ensure the proper distance and bend length for all ductwork so that sound does not travel through the ducts from apartment to apartment.

* When attaching the drywall to the stud walls do not attach them directly, instead use resilient channels to absorb some of the noise and waves.

Conclusion

In Conclusion the depth study and two breadth studies completed in this technical report. It was determined after looking at everything that the changes in to the design and the goals for this thesis study were for the better. A greater understanding of how a building should be designed and how it behaves in a high seismic zone such as San Diego along with the acoustical performance due to the change was learned. Also a greater appreciation of how everything in the building affects everything else was found and helped the in the overall study for this study.

Building 7 was redesigned from the original Hambro Composite Joists and bearing walls with light gage shear walls to a more standard and reliable structural steel system. Structural steel was chosen for back in Technical Report 2 it was determined to be the most efficient for the cost. A new gravity and lateral system was designed for a high seismic region. The gravity system was design and optimized with Ram Structural System and it was determined that a double loaded corridor was the best choice but in the end it is still recommended that the floor to floor height be increased.

With lateral redesign, the overall process and goal was met for all the required checks and procedures for designed in an SDC of D were performed. The seismic loading was determined to be very high and thus controlled the overall lateral design for the wind didn't change significantly in the move to San Diego. Special Concentric braced Frame had to be determined. A double loaded corridor was determined to be the best bay layout and the redesign was able to reduce the number of lateral frames as compared to the original (16 before to 10 at the end). Lateral connections were looked and were designed to meet the seismic requirements. The AISC Steel Construction Manual, 13th Edition and Steel Seismic Design Manual were used as a basis for all of the structural steel designs. A Ram Structural Model was created to help with the analysis and the design of both the gravity and the lateral systems. Advanced computer modeling along with connections were looked at for the MAE requirement.

Two breadth studies were conducted; the first was a green roof study. A green roof was designed to bring and add to the Green Standard and make the building more efficient. A water collection was also designed for both locations so that the roof runoff can be used to help reduce the water consumed by the sanitary system. It was determined that the green roof that was designed will work in both locations, College Park and San Diego. It is for this reason that this green roof system is an excellent source to make the building more earth friendly and also contributes to LEED. Two LEED points can be gain from this design. The first is the reduction in the Heat island effect and the second is the use of the gray water from the roof that will be used with the sanitary system.

The second breadth study was an acoustic study to see the impacts of changing the structural system to steel. It was determined that the new system is acceptable and recommendations were made to make the space more efficient at reducing sound leaks throughout. A new wall construction was created for the separation of the apartments that accounted for the lateral system within that wall, which meet and exceeded the minimum requirements for acceptable noise control.