FINAL REPORT

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Embassy Suites Hotel, Springfield Virginia

Embassy Suites Hotel

Springfield, Virginia | Dominick Lovallo | Structural Option

Building Statistics

Size: 185,000 Square Feet
Building Height: 93 Feet

Number of Stories: 7, 6 Stories Above Grade + Basement

Project Delivery Method: Design-Bid- Build

Project Cost: $31.5 Million

Construction Dates:
Start- November 2011,
Completion- July 13th 2013

Owner: Miller Global Properties, LLC

Architect: Cooper Carry

Structural Engineer: SK & A Structural Engineers, PLLC

Construction Manager: Balfour Beatty Construction

Architectural

- **219 , Two Room Guest Suites**
- **Glass Enclosed Open Air Atrium**
- **Large Pool Area and Fitness Room**
- **Retail Store Space**
- **Adhered Concrete Stone Veneer and EIFS Facades**

Structural

- **8" Two - Way Flat Plate Floor System**
- **Reinforced Concrete Columns**
- **Lateral Force Resisting System - Ordinary Reinforced Concrete Moment Frames**
- **Mud Matt Foundation System**

MEP

- **Split System Air Conditioning**
- **100% Rooftop Outside Air Unit**
- **Custom LED Pendant Lighting Fixtures**

Sustainability

- **Use of Local Construction Materials**
- **LEED Certification**
- **Waste Management Control**
- **Partial Green Roof System**
- **Recycling of Materials**

http://www.engr.psu.edu/ae/thesis/portfolios/2012/DZL5049/index.html

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Acknowledgements

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Cooper Carry

The Pennsylvania State University

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Lastly, I would like to thank my friends and family for their continued support and encouragement through the senior thesis process.

Executive Summery

The Embassy Suites Hotels is a 7 story all-suite hotel located in Springfield Virginia. The Embassy Suites contains 219 guestrooms and the building will also contain many retail stores located on the lower level. The building stands at 91feet 10 inches and is approximately 185,000 square feet. The typical story height is 9 feet except for the ground storefront level and the roof level, having heights of 18 feet and 10 feet respectively.

The purpose of the final thesis report is to delve into an alternate proposed area of study for the Embassy Suites Hotel Project. The existing building contains reinforced concrete moment frames and flat slab construction. A redesign of the structural system proposed a conversion of the current concrete framing system to a steel framing system. As a result of this new design, steel reframing members and a composite floor system were selected and designed in looking to achieve an alternate design as efficient and functional as the existing system in place. One standard W14 x 74 column size was selected to resist lateral and gravity loads that were applied to the building, trying to limit the architectural impacts to the overall layout of the building. The steel gravity and moment frame systems resulted in a decrease in overall building weight, which reduced the base shear in the determination of seismic loads. Additionally W 10 x 26 beams were designed for flooring members trying to limit the overall increase to the building height, choosing beams with the shortest depth that would adequately resist the loads . All framing members were designed and met ASCE 7-05 serviceability conditions including allowable story drift. Overall the use of the steel framed system proved to be an adequate design and was able to resist the loads applied to the structure.

In addition to the structural depth, two alternative areas of study were investigated. The first study examined looked at the acoustics of a typical guest room. The Sound Transmission Classes of walls between guest rooms were calculated using Transmission loss data plotted over select frequencies. The results showed that both the current curtain wall, existing and resigned floor system were adequate for the recommended sound levels.

The second topic looked into the impact of changing the structure of the building on the construction site layout. A site layout plan was developed for the erection process of the steel framing members. Additionally a crane was specked that would be able to handle the erection of the steel framing of the redesign.

The ultimate goal of redesign was to try to design an effective and efficient structural system that would be comparable to the original concrete framed structure. Overall due to zoning limitations and height restrictions the original design would be the best option for the Embassy Suites project however this redesign could be a viable option if circumstances were different.

Introduction: Embassy Suites Hotel

Figure: Site Map. (Photo taken from Google Earth)

The Embassy Suites Hotels is the newest, 7 story, luxury, hotel to become part of the Miller Global, LLC family. Along with Miller Global, the owner the collaborative construction team on this venture include, Cooper Carry, architect; SK & A Structural Engineers, PLLC , structural designers; Balfour Beatty Construction, construction manager; Jordan and Skala, MEP firm; Christopher Consultants, LTD, civil engineering firm. The site is located at the junction of I-95 and Fairfax County Parkway. The location lies in the Springfield region of Fairfax County, Virginia. The site is approximately 16 miles away from the heart of downtown Washington, D.C... Patrons will also be in close proximity to both the Fort Belvoir Main Army Post and the National Geospatial-Intelligence Agency (NGA) facility. The construction delivery method was design –bid - build. Construction began in November 2011 and will be completed July 13th 2013.

Figure: **Facade. (Photo taken from Miller Global, LLC website)**

Upon its completion, this 31.5 million, 185,000 square foot, hotel will feature many amenities. These include a large open air atrium and spacious two room suites. The hotel will serve as a model for comfort and convenience. The building's architecture boasts long flowing curved lines that give it immense visual appeal and a unique flow. The hotel's ground floor will contain a 1300 square foot pool area, a fitness center along with multiple meeting areas, a bar, a lounge and over 1400 square feet of retail space.

The ground level and upper floors store front materials will be made up of manufactured masonry (adhered concrete stone veneer). It is comprised of boral cultured stone country ledge stone along with architectural adhered precast concrete panels. It also contains 1" insulated glass windows with aluminum frames and automatic entrances. The upper levels the exterior façade will feature an exterior insulation finish system (EIFS).

This report will be describing the structural redesign of the Embassy Suites Hotel project feature the design and analysis of the gravity and lateral load resist systems along with the methods and materials used for calculation.

Existing Structural Systems

Foundation

Prior to construction, subsurface exploration and geotechnical engineering analysis were conducted on the future Embassy Suites Hotel site and was completed in January11, 2011 by ECS Mid- Atlantic, LLC. The report indicates a number of test borings were performed on 3 separate occasions. The test borings were drilled at depths ranging from 2.5' to 79' to determine the soil composition in different areas of the site. ECS Mid- Atlantics results showed fill soil material was found in ten boring locations around the site. The fill material was composed of

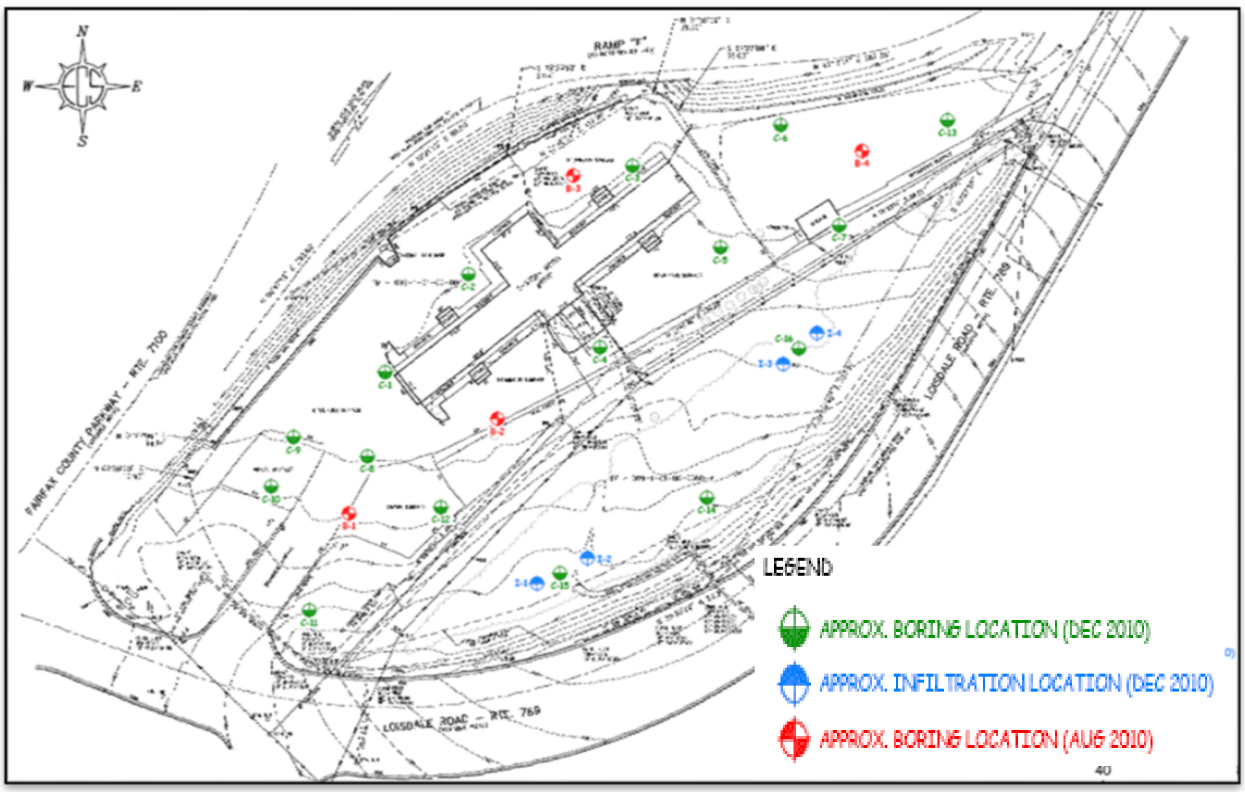

Figure: Core Boring Locations

silty sand and clay from depths of 6.5' to 8.5' below the ground surface. Further down the borings indicated the existence of natural soils that were mainly composed of clayey sand, silt and fat clay. A weather rock material was found at 77' to 78.6'and ground water was encountered at of 18.5' to 65'.

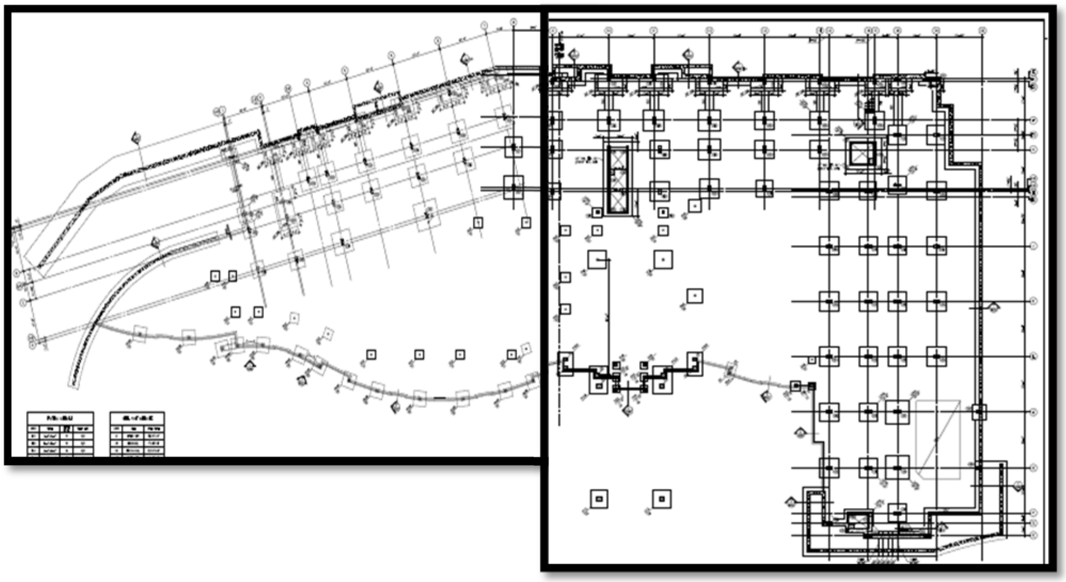

Figure: Foundation Plan

Due to the variability in soil composition, the project team had to employ a partial mud matt system to equalize the soil capacity around the site in some areas. A mud matt system is a thin layer of lean concrete mix (in this case 2000 psi) placed over the existing soil below and allows a stable base for construction.

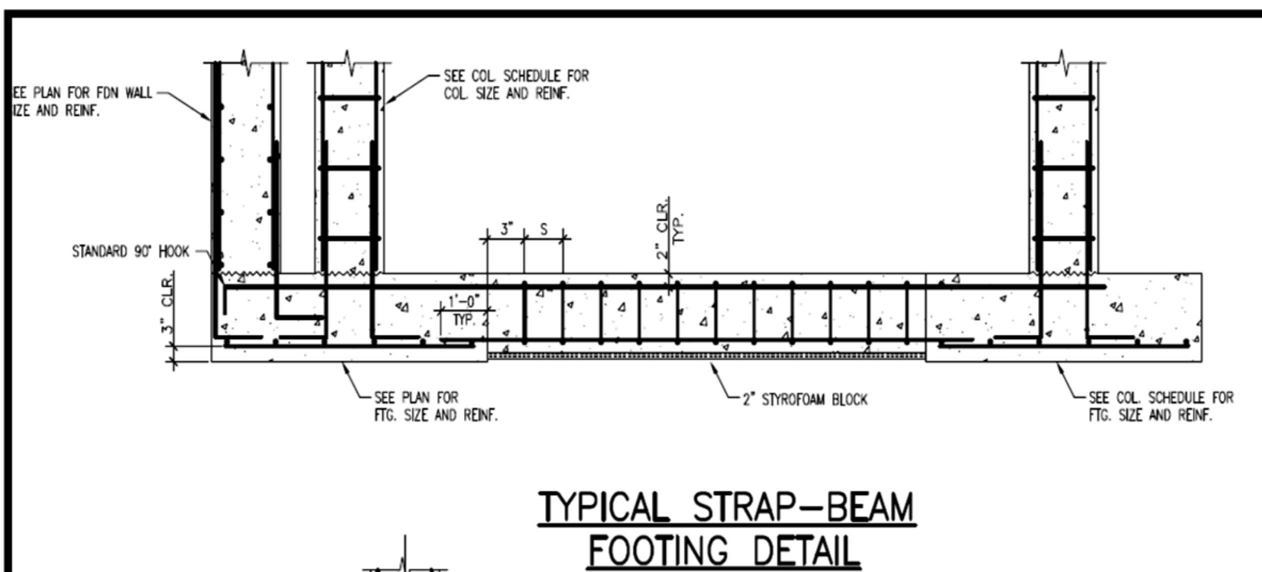

Figure: Strap Beam Detail

The spread footings were designed to have an allowable bearing capacity 6000 psi. The size of footings range from 3' by 3' to 12' by 8' and extend 2' below the slab on grade. To tie the footings together, longitudinally placed strap beams ranging from 36 width x 24 depths to 42 width x 24 depth beams were used. A strap beam is a structural element used to connect to isolated footings together. These beams help distribute the building load to the footings and eventually the ground. The beams range in size and have varied vertical and horizontal reinforcing.

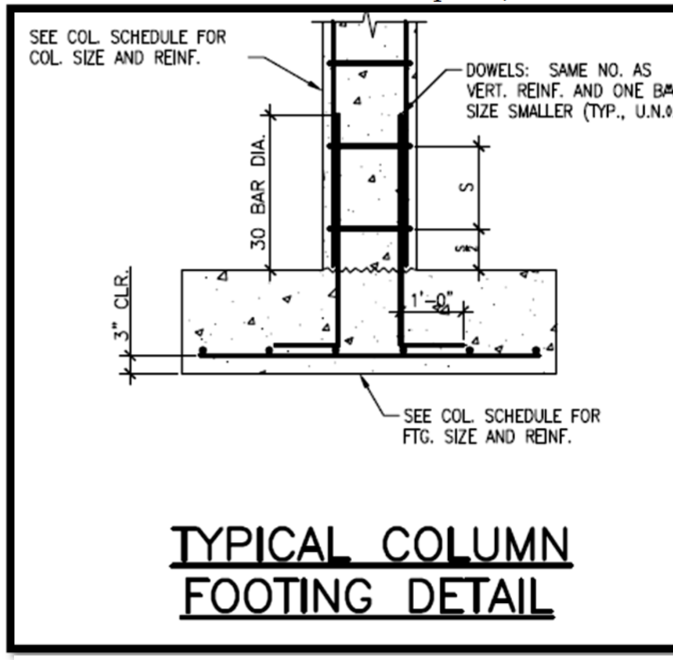

Figure: Footing Detail

The typical slab on grade is a minimum of 5 inches in depth and sits on 4 inches of washed crushed stone. The capacity of the slab is 3500 psi for the interior portions and 5000 psi for exterior slab conditions. The slab contains 6x6 – W 2.0 x W2.0 welded wire fabric and has number 4 reinforcing steel bars spaced 12 inches on center each way.

Floor System

The Embassy Suites Hotel is made up of a typical flat slab construction. The two way slab thickness is 8 inch and the compressive strength of the normal weight concrete is 5000 psi. The slab reinforcing includes number 4 reinforcing bars spaced at 10 inches on center, either way and run the full length from column to column. The floor system also uses drop panel system around one of the interior columns to provide increased negative moment capacity and to protect against punching shear. Punching shear is a failure mechanism were the slab separates from the column due to concentrated shear force. Drop panels are 3.5 inches thick (total slab thickness around column on typical floor is 11.5 inches) and extend 5 feet from either side of the columns.

Framing System

In the image below, a typical framing plan section is shown for floors of the Embassy Suites Hotel (Floors 3 to 7). A typical bay size is 23’ by 18’ for floors containing the guest suites. The columns chosen in for the framing plan were almost all 14” x 30” rectangular reinforced concrete columns. The majority of the columns have a minimum compressive strength of 6,000psi. There are no beams running in between the interior and exterior columns. The only reinforced beams found are located in stairwell openings and elevator shafts.

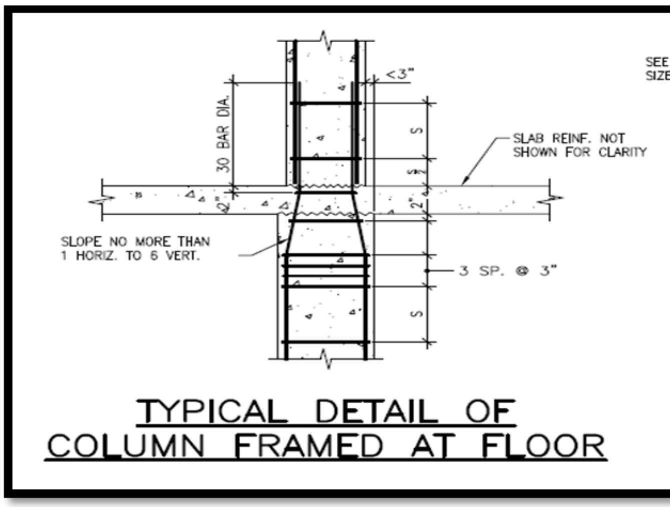

Figure: Column Framing Detail

Due to the increased load on the second floor, large concrete transfer girders had to be used to accommodate for the fitness and pool area. Level 2 also contains HSS columns along with a variety of wide flange shape beams. These are located in the section of the hotel where future retail stores will be housed.

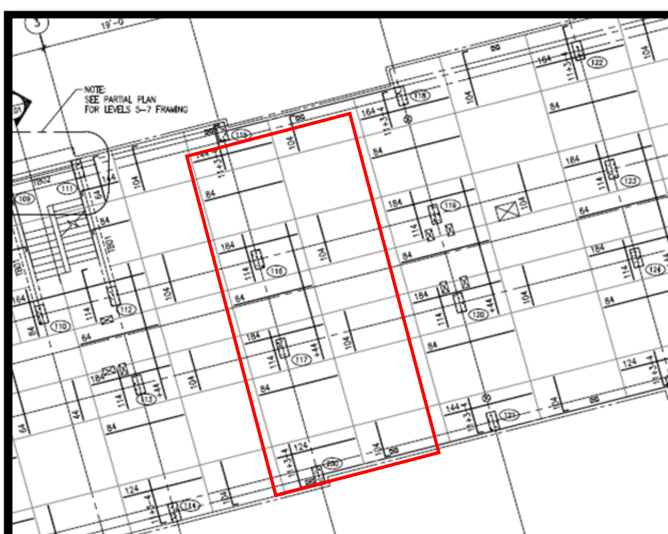

Figure: Typical Bay

Lateral System

To resist lateral forces due to wind and seismic loads the structural engineers employed reinforced concrete moment frames moment frames. The concrete moment frames are the main lateral force resisting system in the building. The lower storefront levels have welded steel moment connections as shown in welded moment detail. The moment connections were designed to develop the full capacity of the member. The connections use high strength ¾ or 7/8 inch ASTM A325 or A490 threaded bolts. The bolts connect the ¼ x 1 inch plates to the beams were the plates are butt and penetrate welded.

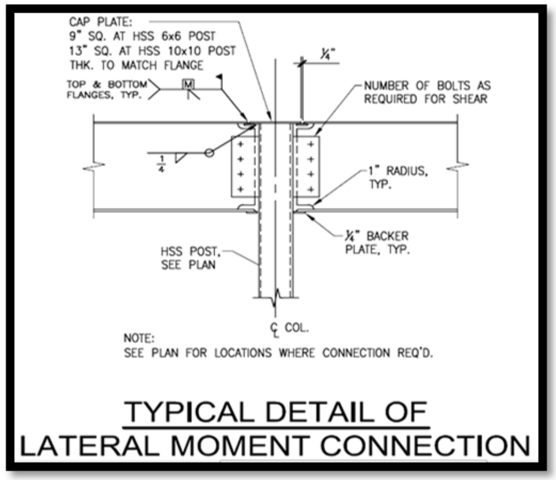

Figure: Welded Moment Connection

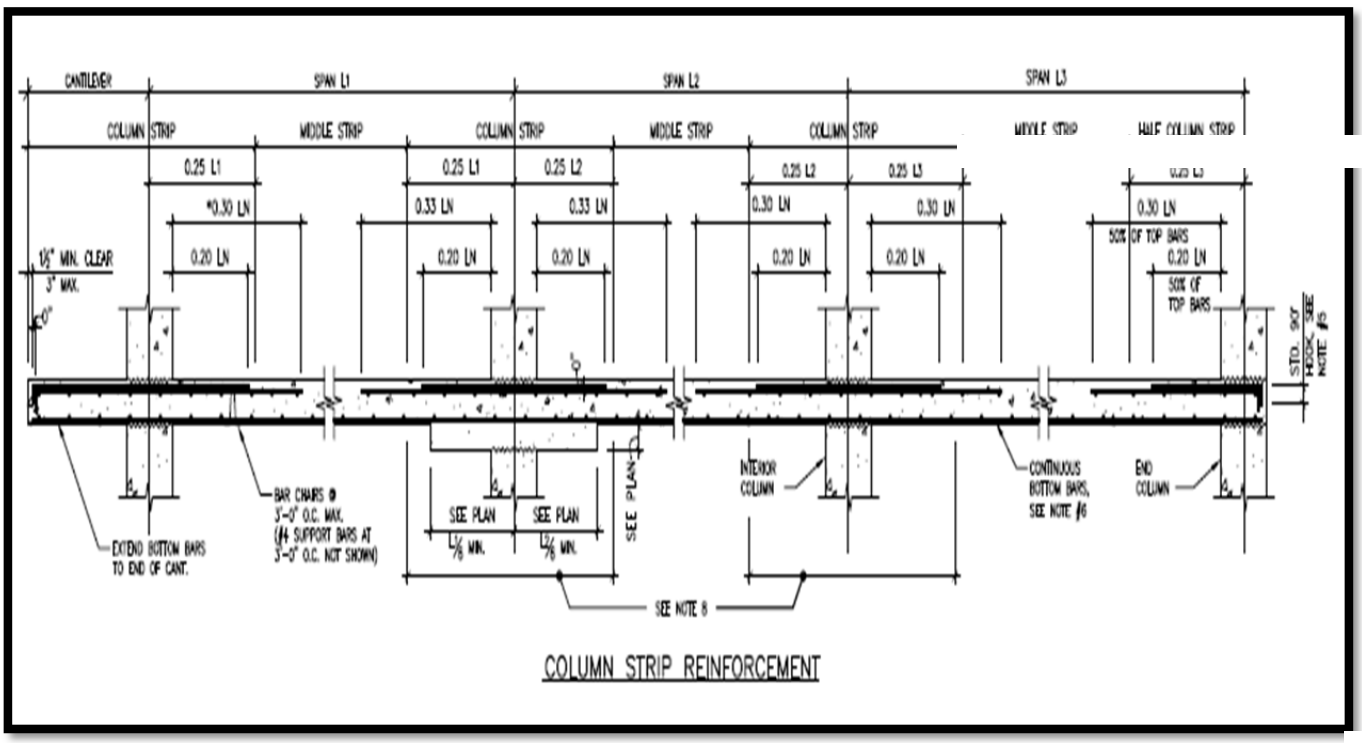

Figure: Main Lateral Force Resisting System

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Roofing System

The high level roofing system consists of 3.25 inch light weight concrete slab. This slab has a compressive strength of 3,500 psi. The lower level roof (top of retail space) is made of 1.5 inch deep 20 gauge Type B cold formed metal deck. The roof deck systems are supported by wide flange beams, concrete reinforced beams varying in size and open web steel joists. The lower level roof system is comprised of a thermoplastic membrane fully adhered with heat welded seams and vapor retarder over a metal deck. Part of the lower level roof (top of part of the second floor) contains a green roof system that includes a pre-vegetated 50 percent extensive and a 50 percent intensive system that is placed upon a protective mat.

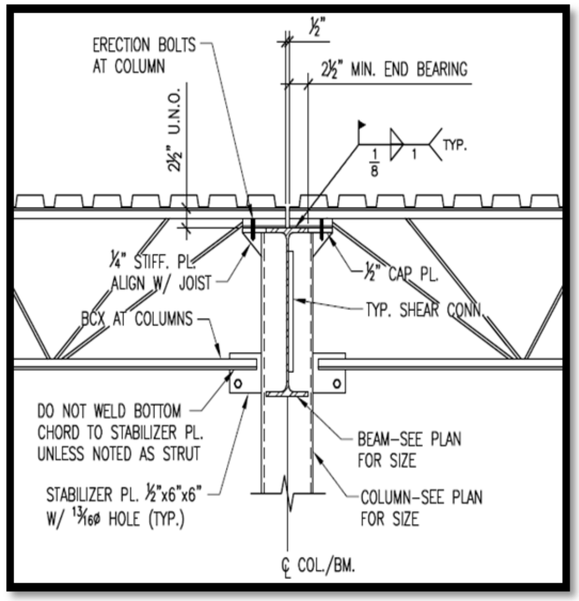


Figure 1: Lower Roof System Connection

Problem Statement

In examining the Embassy Suites Hotel, a predominantly concrete structure, it was determined that the system in place is the most practical and efficient design possible, having building system components that adequately carry the loads applied to the gravity and lateral force resisting systems.

Having delved in many aspects of the design and analysis of this reinforced concrete system and gaining in depth knowledge of this topic, it draws the question if there is an alternative material that could be as efficient as structural concrete for the existing design. It is important to note that due to the exceptional performance of the current structure a comparable alternative design may not be found. To attempt to answer this question a redesign of the Embassy Suites Hotel framing system using steel construction will be studied. The effectiveness and impacts of this new material on other components of the building design will be compared to existing system.

Problem Solution

For the topic of the redesign, the selection of an alternative material for the redesign of the Embassy Suites framing system will be examined. It is known that by selecting a steel system that this would affect building height and the possibility of losing a floor. For this report it will be chosen to keep the same amount of stories even if the zoning limitations are determined to be exceeded. Converting the Embassy Suites Hotel into a steel framed structure in tern affects how the building responds to gravity and lateral loads. Design of steel moment frames to produce the lateral forces exerted on the building will have to be examined.

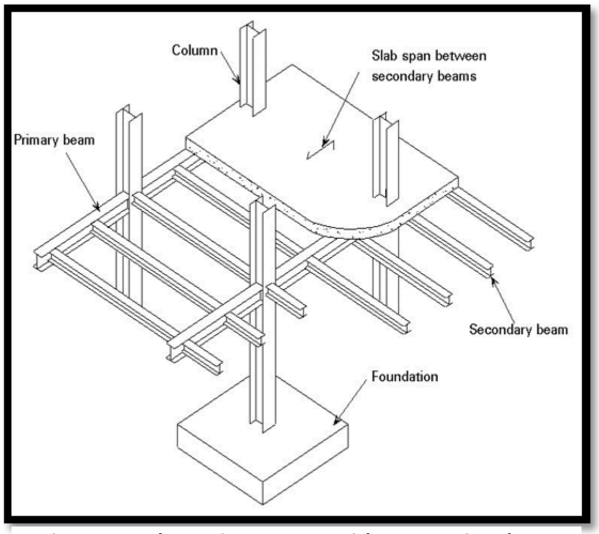


Figure: Steel Framing System with Composite Floor. (Photo taken from www.tatasteelconstruction.com)

In changing the framing system itself the components of this system will also be altered. Columns and beams at respective locations in the buildings will have to be designed and checked for adequacy and will be again compared to the existing structure for efficiency. With altering the columns of the Embassy Suites Hotel a look into the overall column placement will have to studied. In addition, if the building is converted to steel, alternative floor systems that work more efficiently with steel framed structures will be considered. A composite floor system along with a slim floor system will be studied in determined the best option for a floor system in a steel framed structure with height limitations.

The investigation into this material and its affect on the overall design will be compared to the existing design of the Embassy Suites Hotel and will determine whether this material can be as efficient as the existing structure material.

Structural Redesign

Design: Gravity Load Resisting System

This section will outline the redesign and analysis of the gravity load resisting system of the Embassy Suites Hotel project. As put forth in the problem statement, a steel framing system will replace the existing concrete structural system

Before any design calculations could begin load summary and design criteria had to be established in accordance with ASCE- 7-05 and the IBC 2009. Load values are used in design calculations are shown in the table. Loads are in psf unless otherwise noted.

Load Summary Table			
Live Load			
Element	Design Live Load	ASCE 7-05	Redesign Load
Guestroom Floors	40	40	40
Corridors	100	100	100
Mechanical Rooms	150	150	150
Partitions	15	15	15
Elevator Machine Room	125	125	125
Stairs and Exit Ways	125	125	125
Slab on Grade	125	125	125
Balconies	125	125	125
Roof Live	30	30	30
Dead Loads			
Reinforced Concrete	150 (pcf)	150 (pcf)	150 (pcf)
Steel	Varies	Varies	Varies
Composite Flooring System	-	-	63
Composite Roofing System	-	-	2.5
MEP	-	10	10
Snow Load			
Ground	20	20	20

Table: Redesign Load Summary

Column Spacing and Layout

To limit substantial architectural impacts to the Embassy Suites Hotel a uniform column layout and bay spacing was chosen to ensure consistency in the idea of attempting to produce an equally sound design. The column layout and bay spacing was altered slightly increasing the length of some bays for ease of calculation. Additionally it was determined to keep column locations in line with guest room partition walls as but forth in the original floor plan to again hinder architectural alterations. Shown in the in the image below is the column layout for the redesign for the Embassy Suites Hotel.

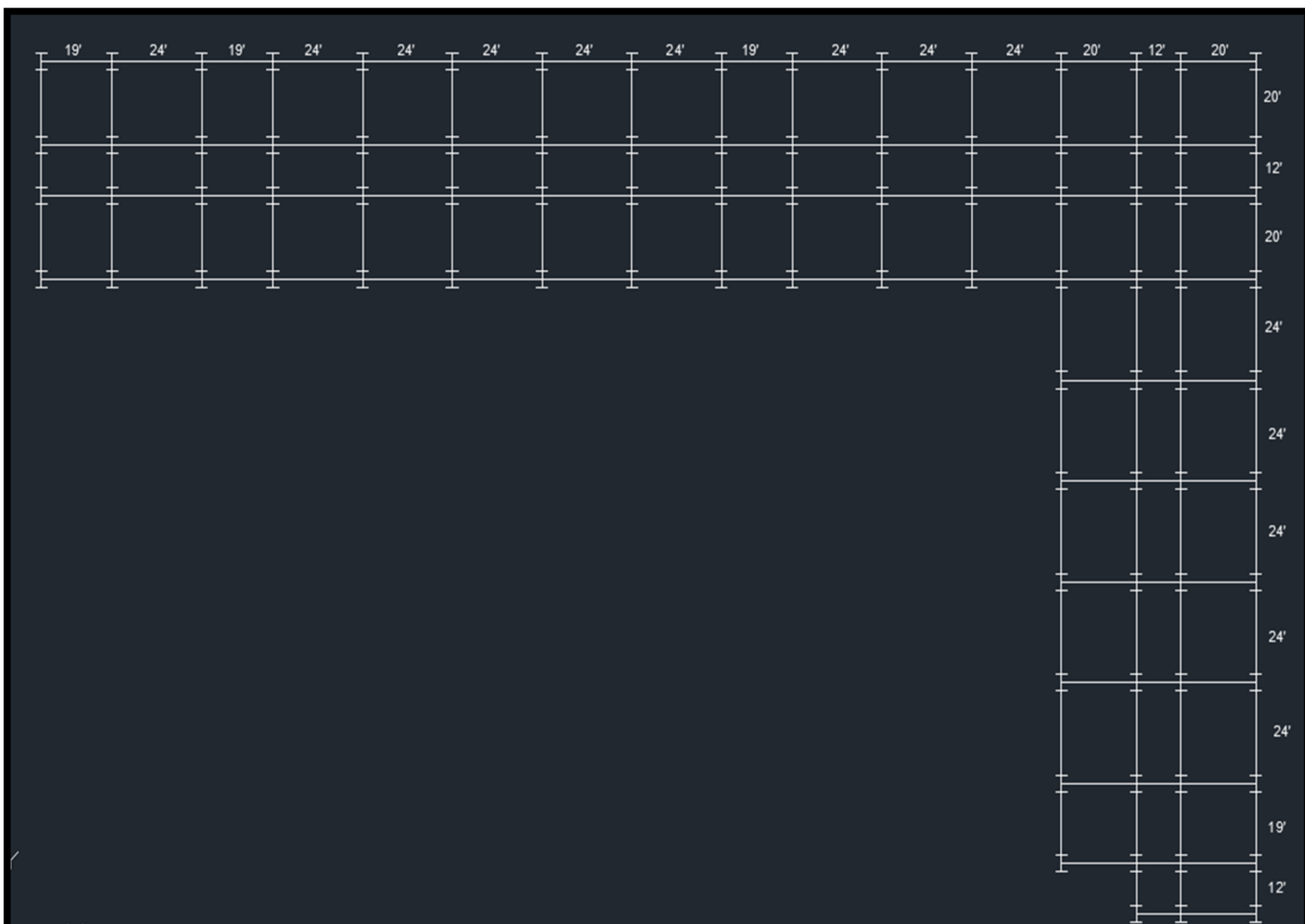


Figure: Bay Layout

Slim and Composite Floor Design

For the redesign, new alternative floor systems had to be considered with the building being changed from a reinforced concrete design to a steel framed structure. Two initial systems were chosen from investigation, a composite floor system on steel framing and a slim floor system or girder slab system. Slim floor system is a system that employs interior girder dissymmetric beams and prestressed hollow-core slabs. The planks are then connected the use of cementitious grout. Through going through the design and analysis it was determined that the slim floor system was an inadequate design choice due to the differential bay sizes in the building and loads applied to the building. To make the slim floor system feasible an economical special heaver beams would have to be manufactured for this project because the standard sizes available failed to pass serviceability conditions. The bay sizes would also have to be changes to one uniform size to make the slim floor design practical. With these factors present it was determined to move forward with the composite floor design.

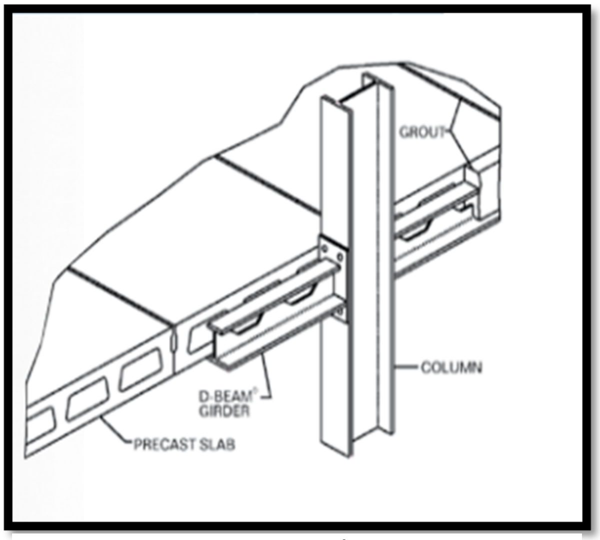


Figure: Girder Slab System (Photo taken from http://www.girder-slab.com/)

The Vulcraft Design manual was used in determined the composite floors system. The system was designed in accordance with ASCE 7-05 loads and survivability limitations. A Vulcract 3VLI 20 floor with a 3.5 in topping thickness was chosen with a 3 span condition and a maximum construction span of. 11'- 9". A typical 20' x 24' guestroom and 12' x 24 bays were examined in the design of the floor system. The layout of the composite floor system can be seen in the image on the following page.

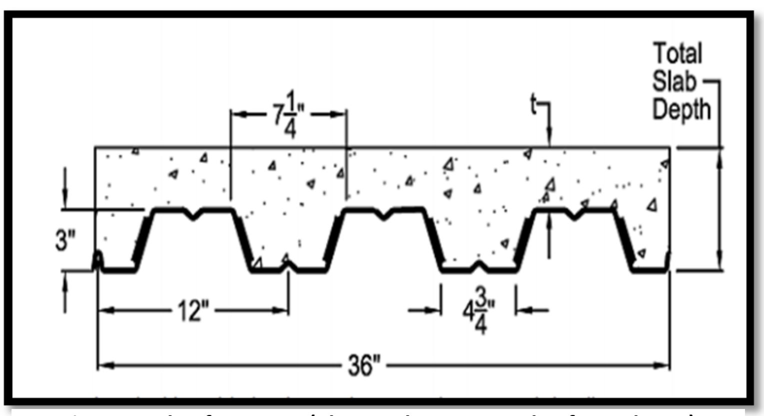


Figure: Vulcraft 3VLI 20 (Photo Taken From Vulcraft Catalogue)

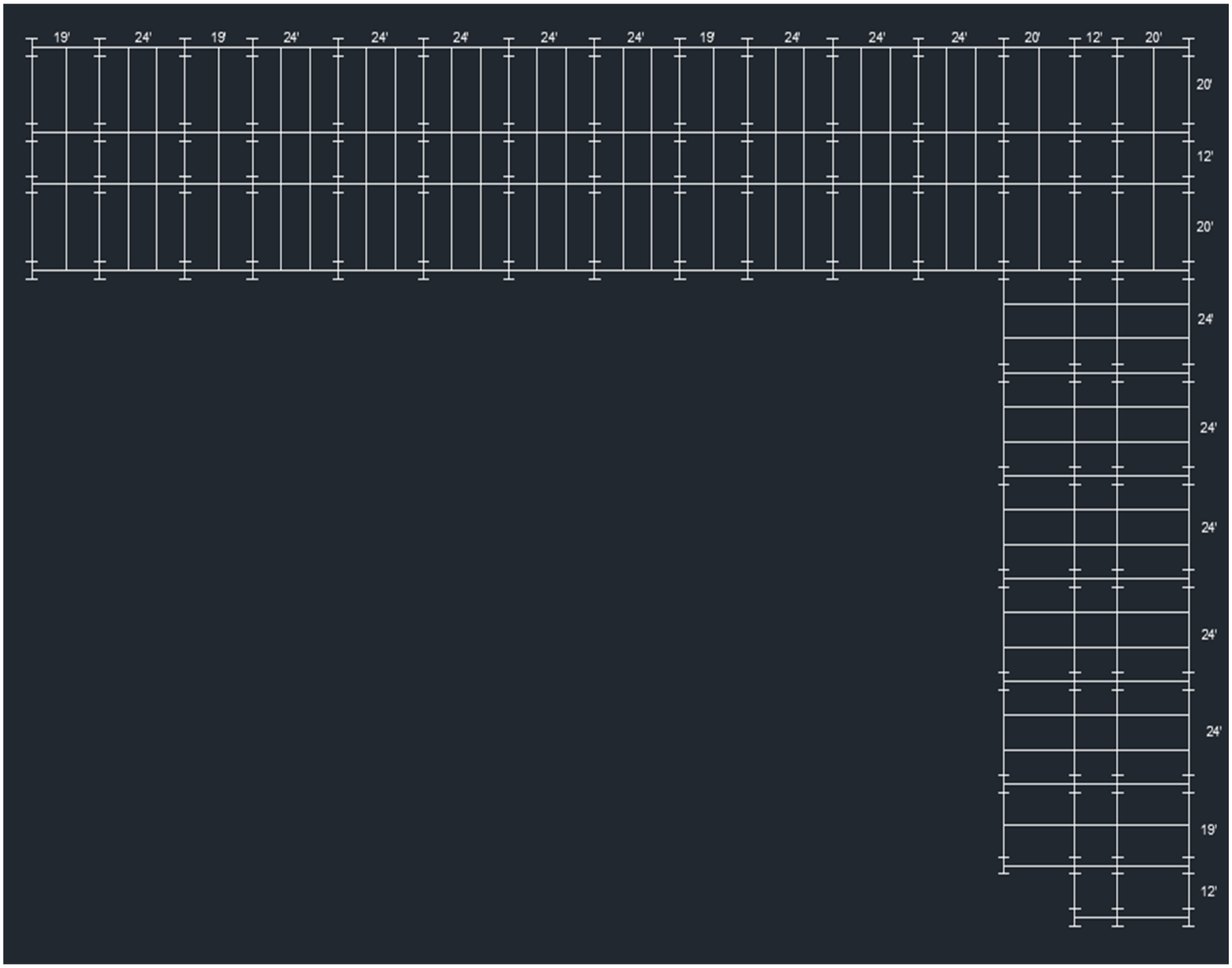


Figure: Typical Floor Bay Layout

Beam and Girder Design

With the composite floor system chosen and validated, the steel framing members of the gravity load system could be designed. Hand calculations were performed in accordance with Load and Resistance Factor Design (LRFD) method put forth in the AISC Steel Construction Manual and applicable ASCE 7-05 load combinations focusing on typical bays in the structure. It was determined that W10 X 26 beams and girders would support the gravity loads acting on the flooring system. Computers models were developed using STAAD Pro structural analysis software to confirm the adequacy of the members. Detailed calculations can be found in Appendix

Frame Design: Gravity

Before framing members could be investigated, some initial parameters had to be established for the overall design of the building. It was determined frames would be designed for combined lateral gravity forces. With that in mind and with gravity loads established, values from previous lateral load analysis were used in obtaining an initial size for column members for the framing system. Hand calculations were performed in accordance with Load and Resistance Factor Design method put forth in the AISC Steel Construction Manual and applicable ASCE 7-05 load combinations focusing on typical frames in the structure. It was determined that W14 X 74 columns and girders would support the gravity loads acting on the flooring system. Computers models were developed using STAAD Pro structural analysis software to confirm the adequacy of the members. Further analysis with new lateral loads would later be examined to confirm design. Through the use of computer gernerated models it was confirmed that the framing system can resist the loads due gravity applied to the building.

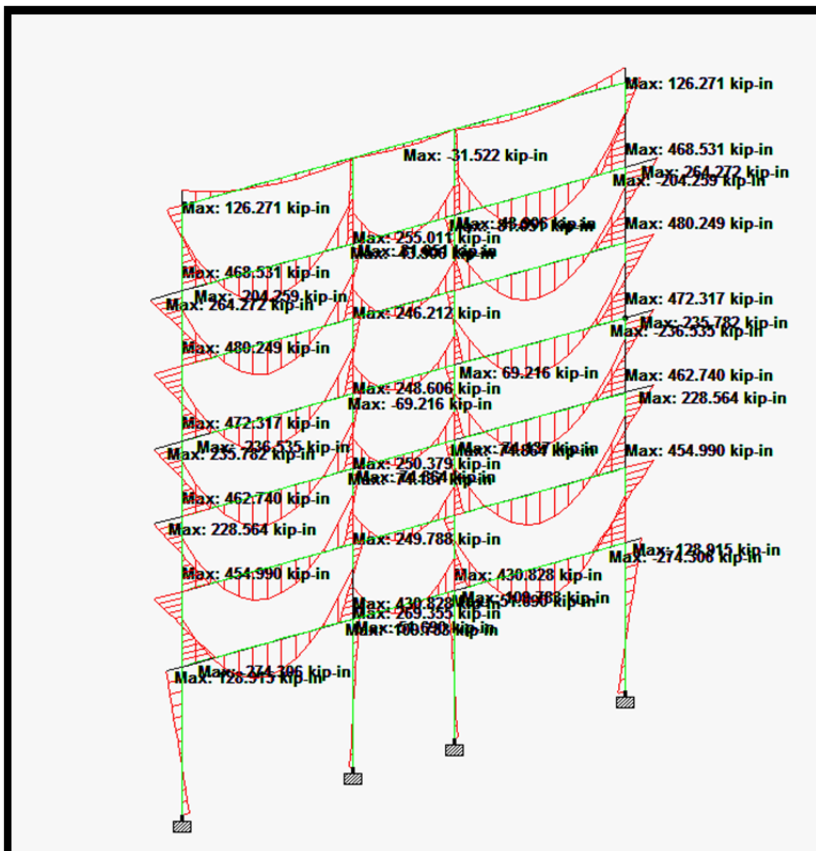


Figure: 3 Bay Frame Gravity Load Analyses

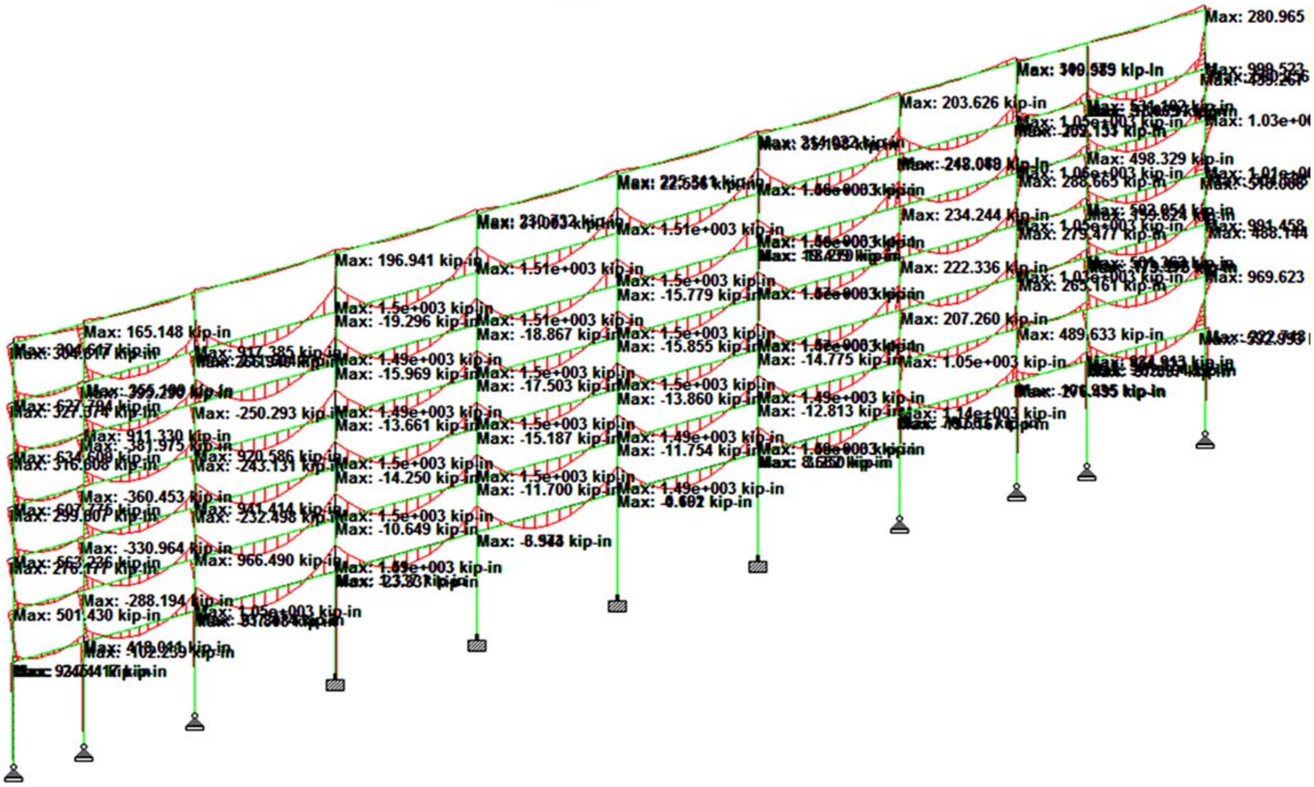


Figure: 10 Bay Frame Gravity Load Analyses

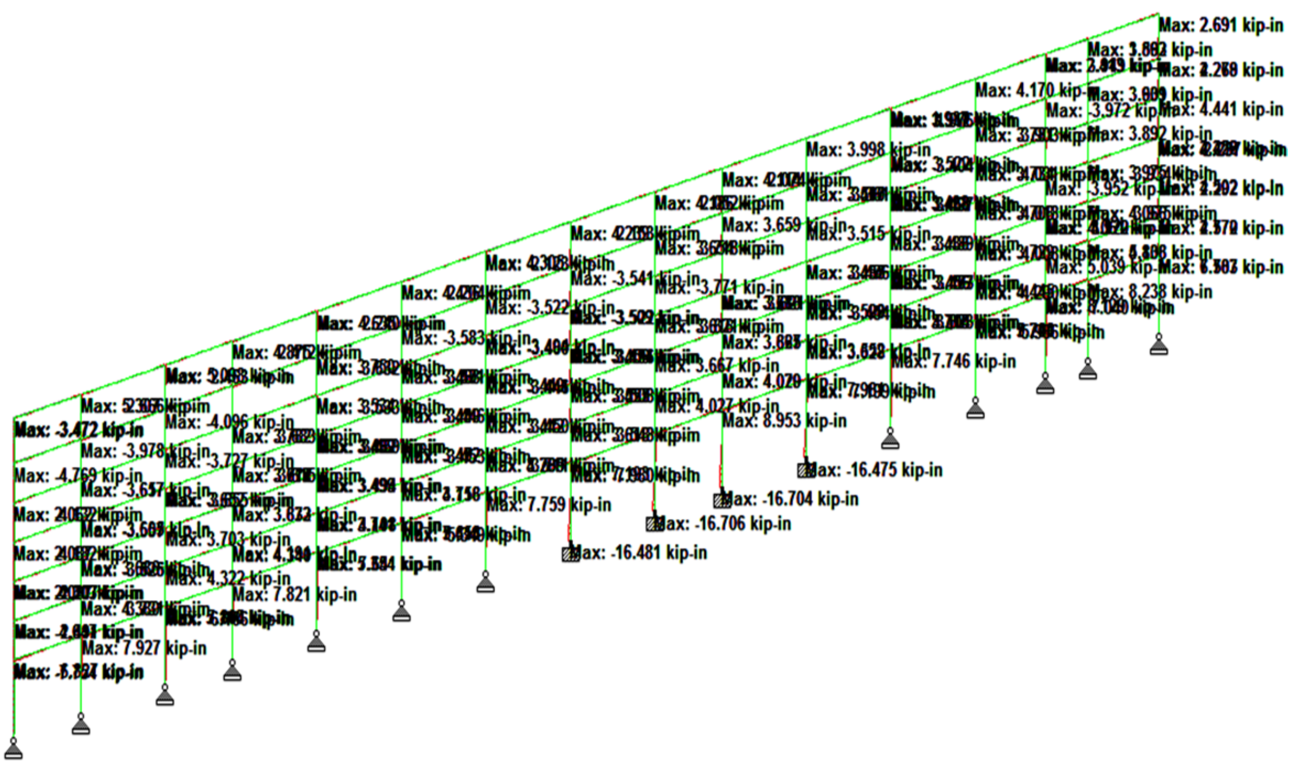


Figure: 15 Bay Frame Gravity Load Analyses

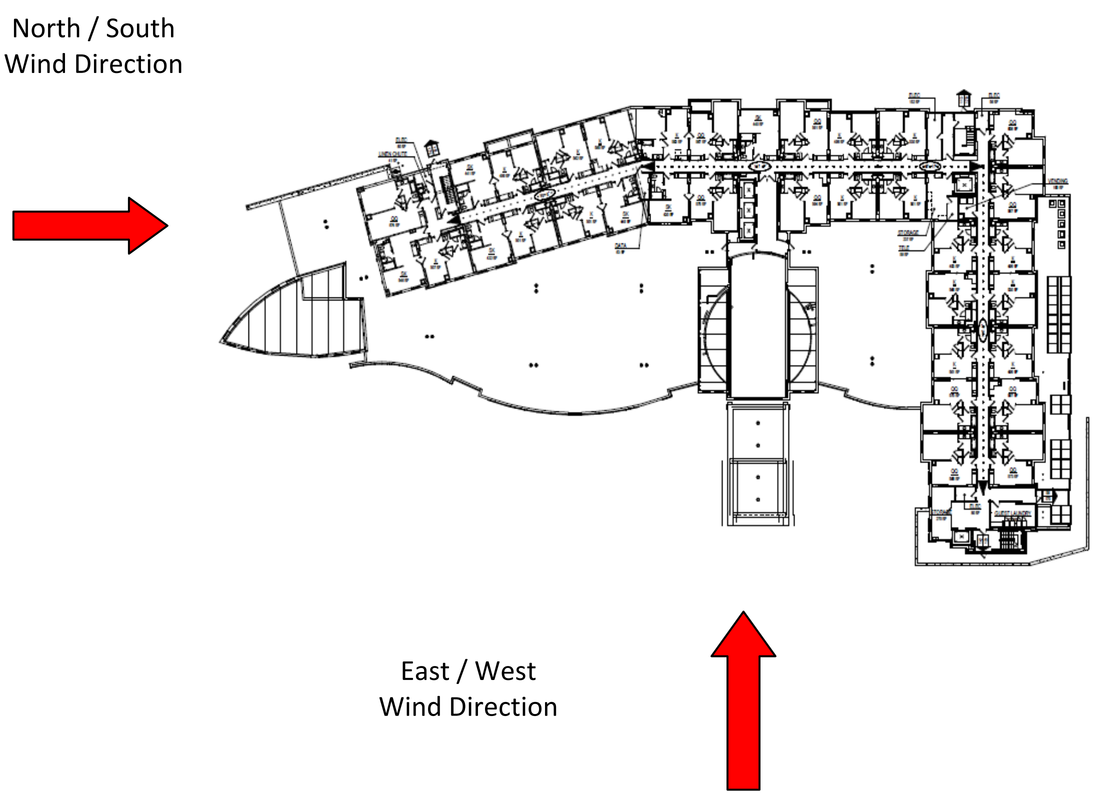
Design: Lateral Load Resisting System

In this section, the redesign of the lateral load resting system will be discussed. The current ordinary concrete moment frames will be replaced with ordinary steel moment frames to resistt the lateral loads applied to it due to wind and seismic forces.

Wind Loads

A reexamination of the wind loads was performed on the Embassy Suites Hotel. It carried out in accordance with Chapter 6 of ASCE 7-05, *Wind Loads.* Due to the fact, that overall building height of the hotel exceeds 60 feet, it is necessary to use the Analytical Method of analysis, examining g the four main cases highlighted in ASCE7-05. Appendix holds detailed wind analysis procedure.

The greatest wind pressure exerted on the building are the wind pressures in the East/ West direction due to the building having an L- shape design and having a long slender facade. This direction is the most critical due to the greater length of building in contact with the wind forces in the East/ West direction. Figures highlight the wind story force distributions in each respective direction the wind analysis data for the analytical procedure can be found in the table.



Wind Analysis Data			
Element	**Symbol**	**Value**	**ASCE7-05 Reference**
Basic Speed	V	90 mph	Figure 1
Directional Factor	Kd	0.85	Table 6-4
Importance Factor 1.0	I	1.0	Table 6-1
Occupancy Category		II	Table 1-1
Exposure Category B		B	Section 6.5.6.3
Enclosure Classification		Enclosed, Partially Enclosed	Section 6.5.9
Topographic Factor	Kzt	1.0	Section 6.5.7.2
Velocity Pressure Exposure Coefficient Evaluated @ Height Z	Kz	Varies	Table 6-3
Velocity Pressure @ Height Z	qz	Varies	Equation 6-15
Velocity Pressure @ Mean Roof Height	qh	.938	Equation 6-15
Gust Effect Factor	G		Section 6.5.8.1
Product of Internal Pressure Coefficient & Gust Effect Factor	GCpi	+/- 0.18, +/-.55	Figure 6-5
External Pressure Coefficient (Windward) (East /West Direction)	Cp	.8	Figure 6-6
External Pressure Coefficient (Leeward) (East /West Direction)	Cp	-.5	Figure 6-6
External Pressure Coefficient (Windward) (North /South Direction)	Cp	.8	Figure 6-6
External Pressure Coefficient (Leeward) (North /South Direction)	Cp	-.362	Figure 6-6

Table: Wind Analysis Data

Seismic Loads

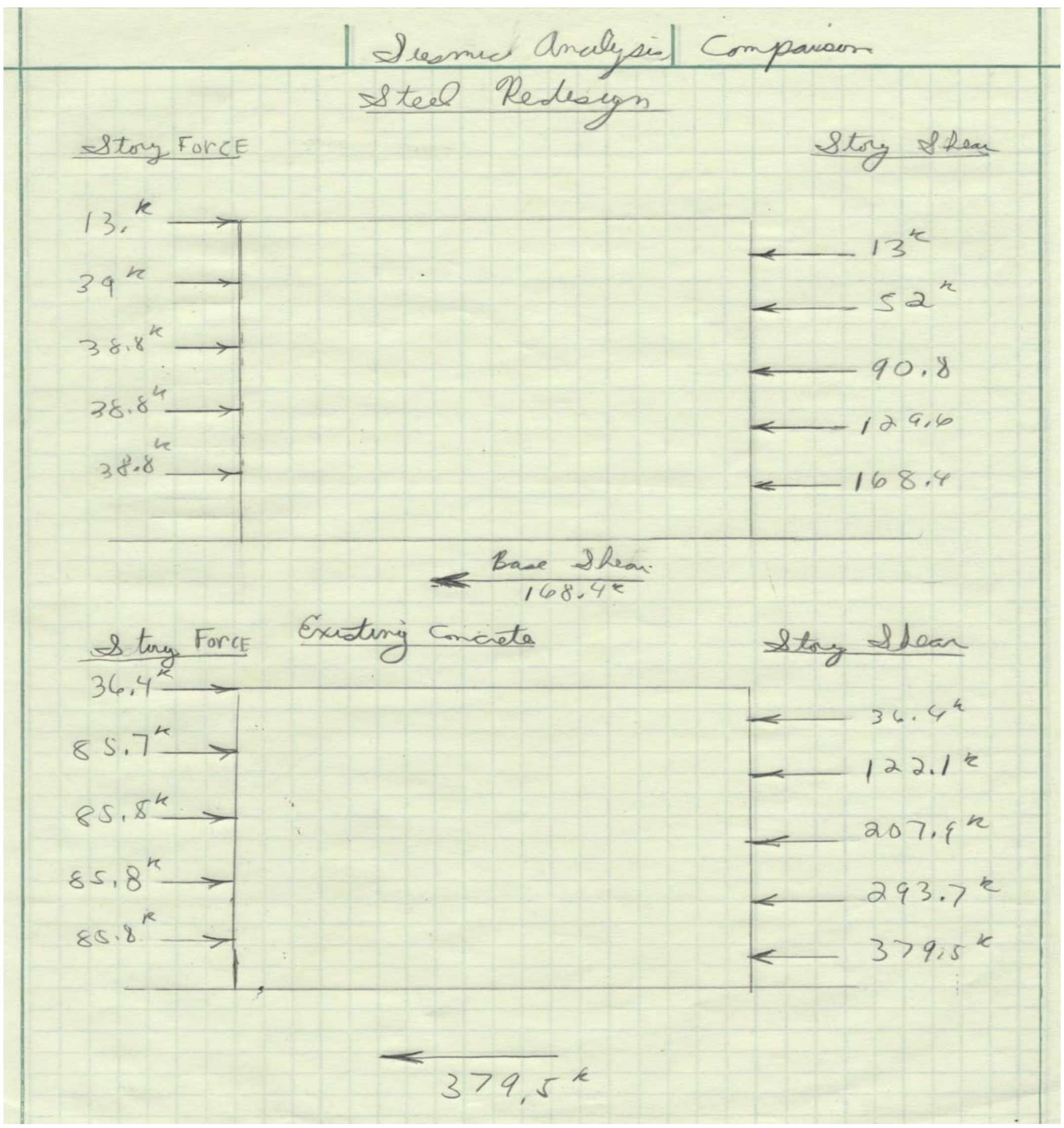
A reexamination of the seismic loads was performed on the Embassy Suites Hotel. Chapters 11 and 12 of ASCE 7-05 were used in the analysis of the seismic loads on The Embassy Suites Hotel. The hotel was designed to withstand the effects of seismic loads having the seismic design class designation B from section 1613.5.6 of the IBC 2009 and a site class designation of D from section 1613.5.2 of the IBC 2009. It is important to mention the assumed base level for calculating the building load was taken at level 2 to giving the total height above grade to be 56 feet. Below is the Seismic Analysis Data used to determine the effects seismic forces acting on the building.

Seismic Analysis Data			
Element	Symbol		ASCE 70-5 References
Site Class		D	Table 20.3-1
Occupancy Category		II	Table 1-1
Importance Factor		1	Table 11.5-1
Structural System		Ordinary Reinforced Steel Moment Frames	Table 12.2-1
Spectral Response Acceleration, short	Ss	0.155	USGS
Spectral Response Acceleration	S1	0.051	USGS
Site Coefficient	Fa	1.6	Table 11.4-1
Site Coefficient	Fv	2.4	Table 11.4-2
MCE Spectral Response Acceleration	Sms	0.248	Eq. 11.4-1
MCE Spectral Response Acceleration	Sm1	0.122	Eq. 11.4-2
Design Spectral Acceleration	Sds	0.165	Eq. 11.4-3
Design Spectral Acceleration	Sd1	0.081	Eq. 11.4-4
Seismic Design Category	Sdc	B	Table 11.6-2
Response Modification Coefficient	R	3.5	Table 12.212
Approximate Period Parameter	Ct	.016	Table 12.8-2
Building Height (above grade)	hn	56 feet	
Approximate Period Parameter	x	.9	Table 12.8-2
Approximate Fundamental Period	Ta	.599	Table 12.8-7
Long Period Transition Period	TL	8 s	Figure 22-15
Seismic Response Coefficient	Cs	0.055	Eq. 12.8-2
Structural Period Exponent	k	1.0	Eq. 12.8-3

Table: Seismic Analysis Data

When reexamining the seismic force distribution to each story of the structure it was necessary to recalculate calculate the buildings total weight, which was done by summing building load elements for each of the floors and adding those together. The redesign of the structure utilizing a steel frame system opposed to the existing concrete flat slab system. It was show that the overall building weight and base shear, decreased by roughly 50. The table shows a caparison between the existing buildings resign. Figures highlight the seismic story force distributions in each respective direction

Seismic Analysis Comparison		
	Existing Building	Redesign
Weight	14202.5	8600(kip)
Base Shear	379.5	168 (kip)



Lateral Load Distribution

To determine the story stiffness due to a unit load a 1 kip force was applied at the top of each of the moment frames to obtain a displacement. The formula K = P/ delta , was used where P is the 1 kip unit load applied to the top of the frame and delta is the displacement of the frame at its respective story level in inches due to the unit load. K – Value calculations can be found in Appendix

Center of Rigidity and Center of Mass

The center of rigidity is defined the stiffness centroid in a structure. A reverence point was chosen in the south west corner of the building to find distances in the x and y directions. A more simplified L - shape layout of the floors was chosen for ease of calculation to determine the distances to each moment frame. The center of mass is defined as the mass centroid in a structure. A reverence point was chosen in the south west corner of the building to find distances in the x and y directions. A more simplified L - shape layout of the floors was chosen for ease of calculation to determine the distances to each moment frame. Detailed calculations of the center of mass and center of rigidity can be found in Appendix

The formulas used to calculate the center of rigidity and centers of mass are as follows:

CR (x, y direction)

$$\frac{\sum kix}{\sum ki} \qquad \frac{\sum kiy}{\sum ki}$$

CM (x, y direction)

$$\frac{A_1x_1+A_2x_2}{A_1+A_2} \qquad \frac{A_1y_1+A_2y_2}{A_1+A_2}$$

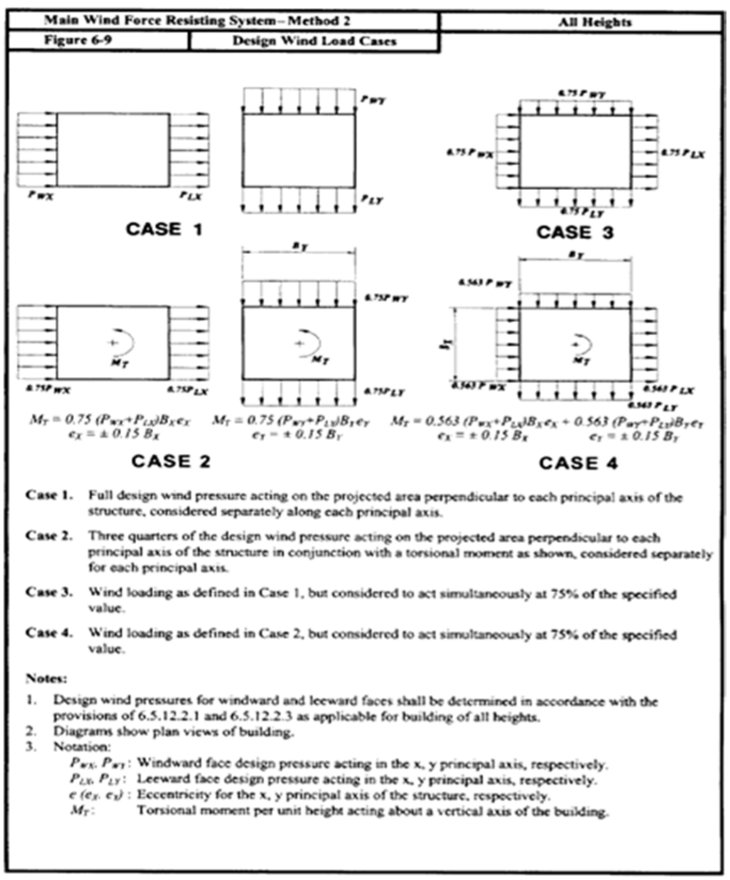
Dominick Lovallo
Structural Option
Dr. Hanagan Advisor
Penn State University

Torsion

Torsional effects in a building structure are caused when the center of rigidity and the center of mass are offset causing a twisting moment that is subjected to the lateral force resisting systems. This is especially prevalent for L- shaped structures. These effects have to be accounted for in the design of the lateral systems. To gain a better understand of torsion and how it is distributed to a building one must look at individual frames in respective direction due to wind and seismic loads

Torsional Shear: Wind Loading

In ASCE 7-05, figure 6-9 highlights 4 different wind load cases on a building. For this report all four wind load cases were considered ion the redesign of the structure. The controlling load case was determined to be Case 1 having the greatest wind forces acting on the structure



Direct Shear: Wind Loading

The direct shear distributed to each frame was calculated for the East/West and North/South directions. The story stiffness for each lateral force resting frame was used to compute the distributed forces. The building is essentially comprised of 3 types of frames, 3 bay and 10 bay frames that resist loads in the East/West direction and 3 bay and 15 bay frames in the North / South direction. In the table below are the direct shear forces distributed to each frame due to east / west and north /south wind loadings respectively. There are a total of 16 lateral force resting frames in the East/West direction and 10 frames that resist load in the North / South direction. The direct shear of each frame was calculated by taking the stiffness factor for that frame over the sum of the stiffness factors in the direction of the force multiplied by the story force. The formula is as follows:

$$\frac{\sum ki}{\sum K} * \mathrm{P}$$

Floor	Story Pressure	Wind Direction	K Value- 3 Bay	K Value- 10 Bay	Total K Value	% Load to 3 Bay Frame	% Load to 8 Bay Frame	Load to 3 Bay	Load to 8 Bay
7th	60.30	E/W	23.8	58.8	521.0	0.0457	0.1129	2.8	6.8
6th	51.50	E/W	27.0	66.7	591.0	0.0457	0.1128	2.4	5.8
5th	50.30	E/W	31.3	71.4	660.7	0.0473	0.1081	2.4	5.4
4th	48.80	E/W	35.7	83.3	761.9	0.0469	0.1094	2.3	5.3
3rd	47.10	E/W	43.5	90.9	885.4	0.0491	0.1027	2.3	4.8
2nd	44.90	E/W	52.6	100.0	1031.6	0.0510	0.0969	2.3	4.4

Table: Wind Force Frame Distribution E/W Wind

Floor	Story Pressure	Wind Direction	K Value- 3 Bay	K Value - 15 Bay	Total K Value	% Load to 3 Bay Frame	% Load to 15 Bay Frame	Load to 3 Bay	Load to 15 Bay
7th	33.60	N/S	23.8	76.9	450.5	0.0528	0.1707	1.8	5.7
6th	28.60	N/S	27.0	90.9	525.8	0.0514	0.1729	1.5	4.9
5th	27.80	N/S	31.3	100.0	587.5	0.0532	0.1702	1.5	4.7
4th	26.90	N/S	35.7	100.0	614.3	0.0581	0.1628	1.6	4.4
3rd	25.80	N/S	43.5	111.1	705.3	0.0616	0.1575	1.6	4.1
2nd	24.50	N/S	52.6	125.0	815.8	0.0645	0.1532	1.6	3.8

Table: Wind Force Frame Distribution N/S Wind

After the loads were calculated and distributed to the frames, the total forces to each frame, combining direct and torsional effects, could be determined. As mentioned previously, the Embassy Suites Hotel will be comprised of a total of 16 lateral force resting frames in the East/West direction and 10 frames that resist load in the North / South direction. Typical frames were chosen in analysis for each of the wind direction. (Frames 5 and 15, highlighted in **red** for the East/ West Direction and Frames 2 and 6 highlighted in **blue** for the North South Direction) The total shear forces that act on the frames for the East / West and North/ South Directions were calculated can be found in the table on the following page.

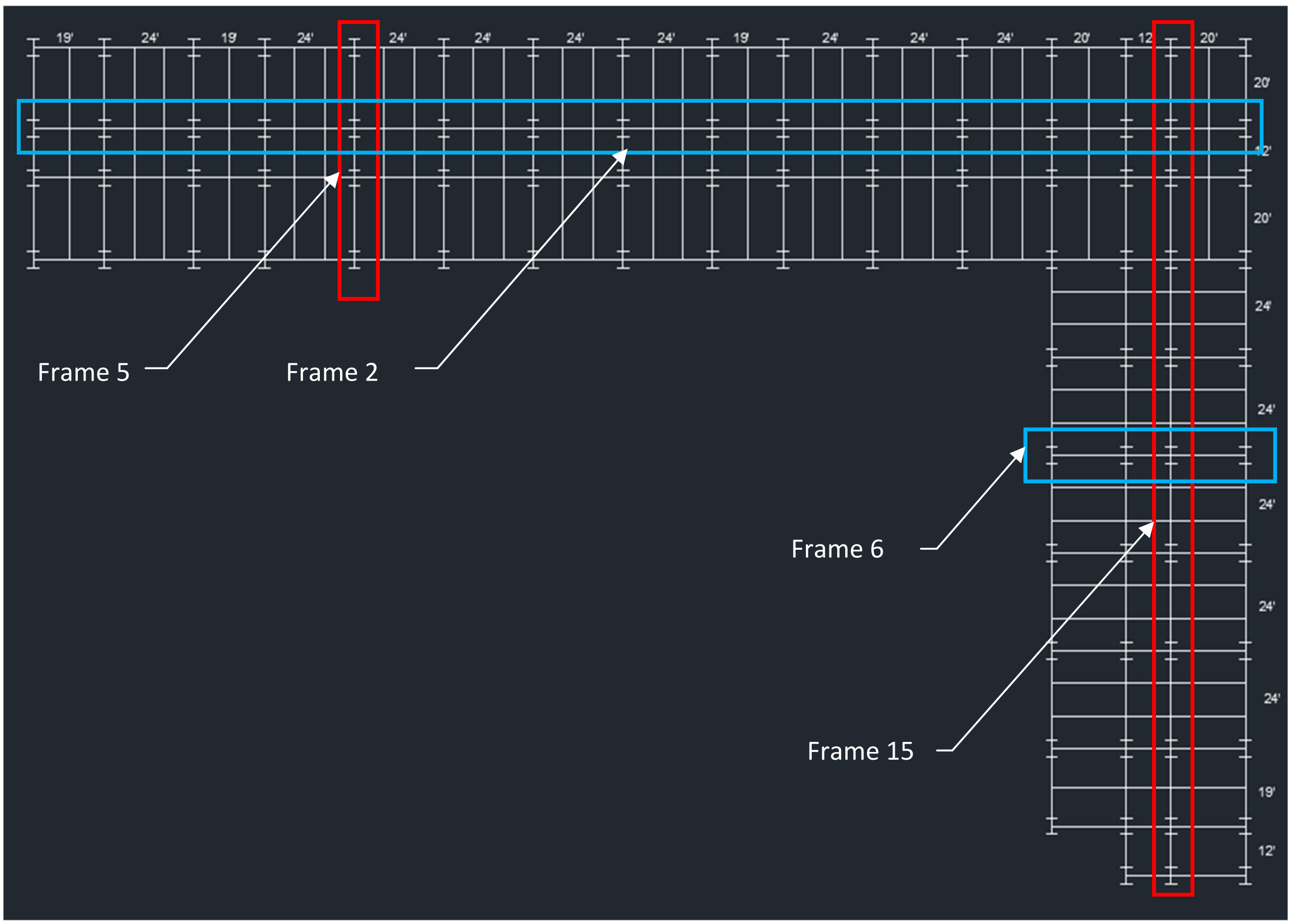


Figure: Typical Frames for Analysis

Lateral Wind Force E/W Direction Case 1						
Level	Load to 3 Bay	Load to 10 Bay	Frame 5	Frame 15	Total Lateral (Kip)	
7th	2.76	6.81	-1.00	2.87	1.76	9.68
6th	2.36	5.81	-0.96	2.09	1.40	7.90
5th	2.38	5.44	-0.97	1.92	1.41	7.36
4th	2.29	5.34	-0.93	1.92	1.36	7.26
3rd	2.31	4.84	-0.93	1.73	1.38	6.57
2nd	2.29	4.35	-0.92	1.55	1.37	5.90

Table: Total Wind Force per Story for Typical Frames E/W Direction

Lateral Wind Force N/S Direction Case 1						
Level	Load to 15 Bay	Load to 3 Bay	Frame 2	Frame 6	Total Lateral (Kip)	
7th	5.74	1.78	-2.91	0.85	2.83	2.63
6th	4.94	1.47	-2.51	0.72	2.43	2.19
5th	4.73	1.48	-2.35	0.71	2.38	2.19
4th	4.38	1.56	-2.08	0.72	2.30	2.28
3rd	4.06	1.59	-1.86	0.71	2.20	2.30
2nd	3.75	1.58	-1.68	0.69	2.07	2.27

Table: Total Wind Force per Story for Typical Frames N/S Direction

Torsional Shear: Seismic Loading

It is important to note that the seismic story force acts at the center of mass and the eccentricity of the moment is from the center of mass to the center of rigidity. Even though seismic loads are not directional in nature and are applied to the whole building at once, it is important it to examine it in this manner to determine controlling load cases.

Direct Shear Seismic Loading

The direct shear distributed to each frame was calculated for the East/West and North/South directions. The story stiffness for each later force resting frame was used to compute the distributed forces. In the table are the direct shear forces distributed to each frame due to east / west and north /south wind loadings respectively. The same typical frames used in wind analysis were used in the analysis of seismic forces acting on the frames. The direct shear of each frame was calculated by taking the stiffness factor for that frame over the sum of the stiffness factors in the direction of the force multiplied by the story force. The formula is as follows:

$$\frac{\sum ki}{\sum K} * \mathrm{P}$$

Floor	Story Pressure	Wind Direction	K Value - 3 Bay	K Value- 8 Bay	Total K - Value	% Load to 3 Bay	% Load to 8 Bay	Load to 3 Bay	Load to 8 Bay
7th	13.00	E/W	23.8	58.8	521.0	0.0457	0.1129	0.6	1.5
6th	39.00	E/W	27.0	66.7	591.0	0.0457	0.1128	1.8	4.4
5th	38.80	E/W	31.3	71.4	660.7	0.0473	0.1081	1.8	4.2
4th	38.80	E/W	35.7	83.3	761.9	0.0469	0.1094	1.8	4.2
3rd	38.80	E/W	43.5	90.9	885.4	0.0491	0.1027	1.9	4.0

Table: Seismic Force Frame Distribution E/W Direction

Floor	Story Pressure	Wind Direction	K Value - 3 Bay	K-Value 15 Bay	Total K - Value	% Load to 3 Bay	% Load to 15 Bay	Load to 3 Bay	Load to 15 Bay
7th	13.00	N/S	23.8	76.9	450.5	0.0528	0.1707	0.7	2.2
6th	39.00	N/S	27.0	90.9	525.8	0.0514	0.1729	2.0	6.7
5th	38.80	N/S	31.3	100.0	587.5	0.0532	0.1702	2.1	6.6
4th	38.80	N/S	35.7	100.0	614.3	0.0581	0.1628	2.3	6.3
3rd	38.80	N/S	43.5	111.1	705.3	0.0616	0.1575	2.4	6.1

Table: Seismic Force Frame Distribution N/S Direction

Total seismic loads were calculated and distributed to the frames, the total forces to each frames, combining direct and torsional effects could now determined. The typical frames used in the Wind analysis were also used in checking adequacy for seismic loading for constancy.

Lateral Seismic Force E/W Direction						
Level	Load to 3 Bay	Load to 10 Bay	Frame 5	Frame 15	Total Lateral (Kip)	
7th	0.59	1.47	-0.02	0.05	0.57	1.52
6th	1.78	4.40	-0.07	0.00	1.71	4.40
5th	1.84	4.19	-0.07	0.13	1.77	4.32
4th	1.82	4.24	-0.07	0.13	1.75	4.37
3rd	1.91	3.98	-0.07	0.13	1.84	4.11

Table: Total Seismic Force per Story for Typical Frames E/W Direction

Lateral Seismic Force N/S Direction						
Level	Load to 15 Bay	Load to 3 Bay	Frame 2	Frame 6	Total Lateral (Kip)	
7th	2.22	0.69	-0.09	0.03	2.13	0.72
6th	6.74	2.00	-0.20	0.06	6.54	2.06
5th	6.60	2.06	-0.18	0.06	6.42	2.12
4th	6.32	2.26	-0.18	0.06	6.14	2.32
3rd	6.11	2.39	-0.18	0.06	5.93	2.45

Table: Total Seismic Force per Story for Typical Frames N/S Direction

Load Combinations

After the lateral loads were found for both wind an seismic effects and these load were distributed to the frames, a series of basic load combinations where taken into consideration when analyzing values that were to determine drift The controlling load combination for this design is highlighted in red. The load combinations can be found in ASCE 7-05 chapter 2. The ASCE 7-05 load combinations are as follows:

2.3 COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 Applicability. The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Figure: ASCE Load Combinations

Initial parameters had to be established for the overall design of the building. It was determined frames would be designed for combined lateral gravity forces. In initial size for column members for the framing system. Hand calculations were performed in accordance with Load and Resistance Factor Design method put forth in the AISC Steel Construction Manual and applicable ASCE 7-05 load combinations focusing on typical frames in the structure. It was determined that W14 X 74 columns would be used to resist the lateral forces exerted on the frames. A typical 3 bay frame design was developed, have all columns assist in the resistance of load. In changing the Embassy Suites to steel frame, it is important to note that to keep the floor to ceiling heights the same the floor to floor height and the overall building height had to increase. A height comparison summary can be found in the table. Computers models were developed using STAAD Pro structural analysis software to confirm the adequacy of the members using loads from the controlling load combination. In the figures below it is shown the parts of the frame (highlighted in red) that will contain the moment resisting elements, the lateral forces in the East/ West and /North South directions. Detailed hand calculations can be found in Appendix

Building Height and Floor Thickness Comparison						
Level	Existing Story Height (ft.)	Redesign Story Height (ft)	Percent Increase (%)	Floor Thickness Existing(in)	Floor Thickness Redesign(in)	Percent Increase (%)
7	10.375	11.09	6.4	3.25	11.8	72.4
6	9.125	9.61	5	11.5	16.8	35
5	9.125	9.61	5	11.5	16.8	35
4	9.125	9.61	5	11.5	16.8	35
3	9.125	9.61	5	11.5	16.8	35
2	9.125	9.61	5	11.5	16.8	35
1	18	18.48	2.6	11.5	16.8	35

Table: Building Height per Story and Floor Increase Summary

	Total Story Height (ft)	**Overall Building Height(ft)**
Existing	74	91.82
Redesign	77.62	95.45

Table: Overall Height Increase Summary

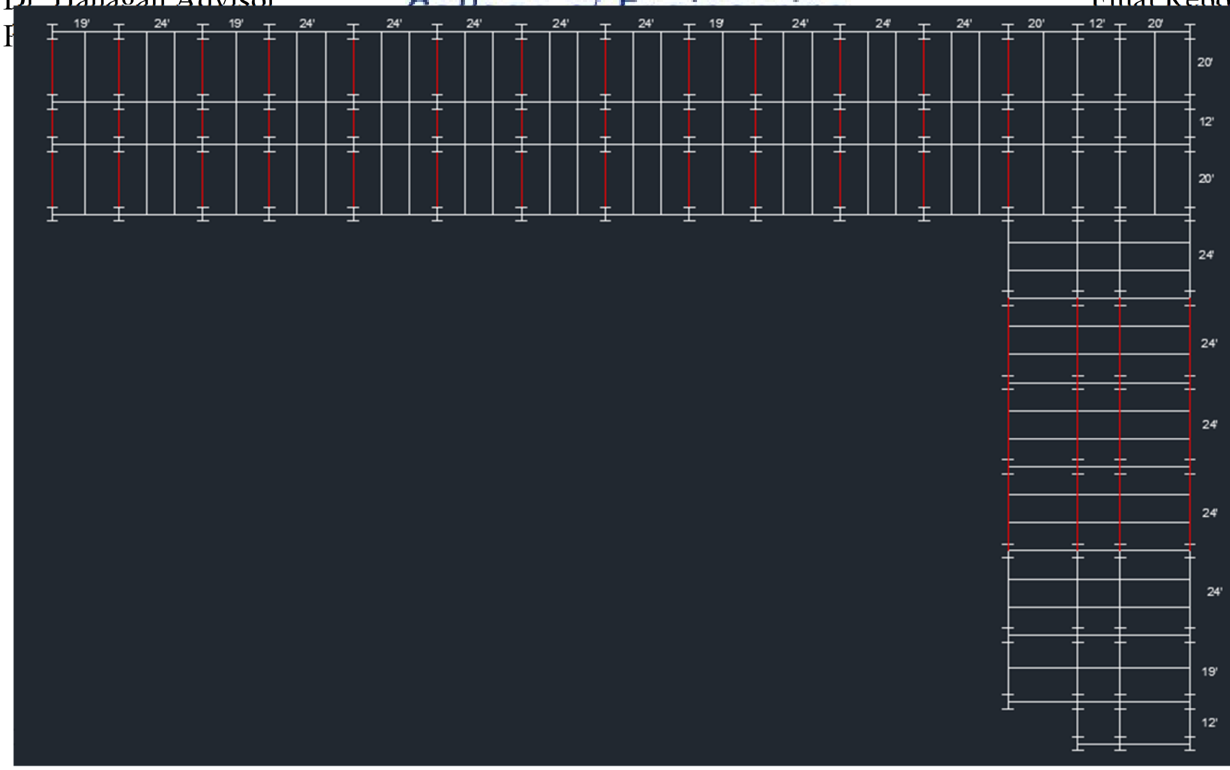


Figure: Lateral Force Resisting Frames E/W Direction

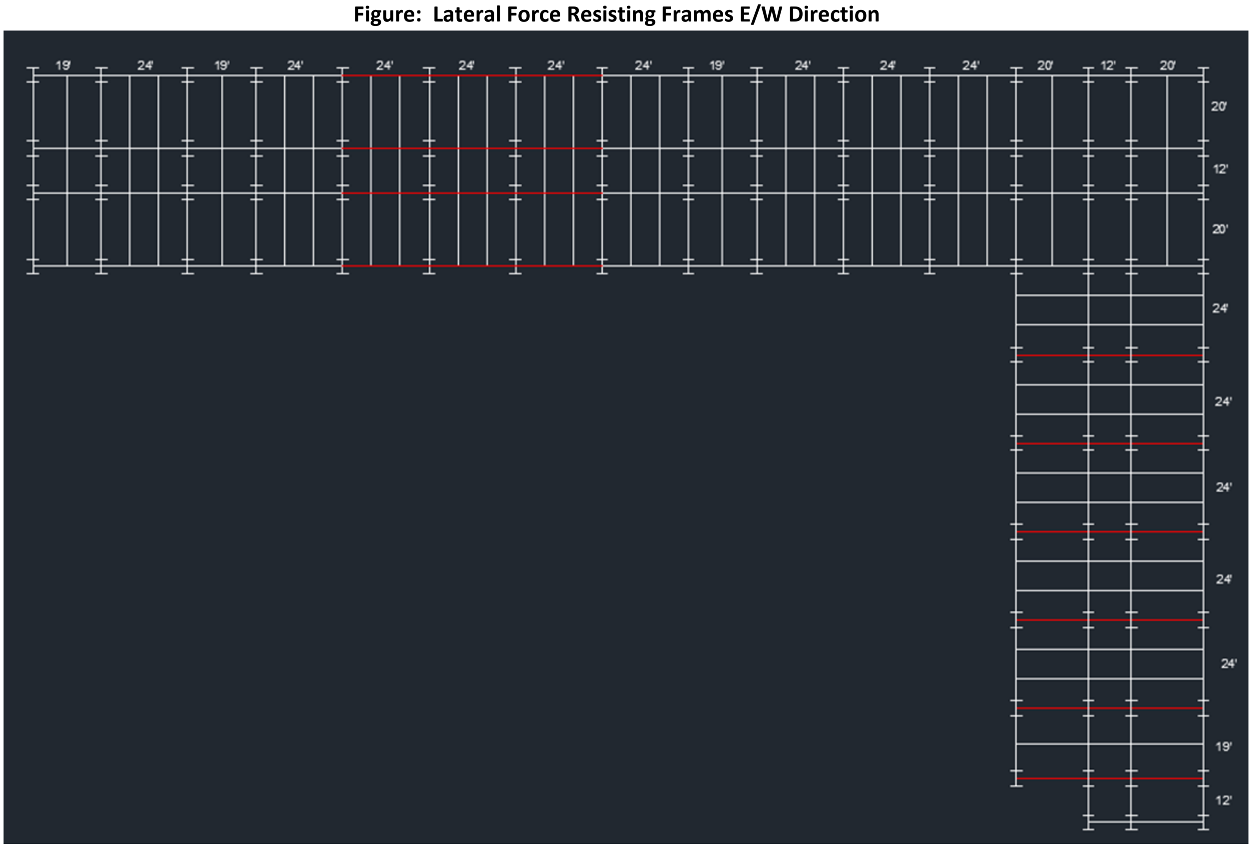


Figure: Lateral Force Resisting Frames E/W Direction

Story Drift and Lateral Displacement

A series of 2D Frames were modeled with STAAD Pro using values calculated in direct and torsional shear analysis to analysis the maximum drift for both wind and seismic loads. A three and ten bay frames were modeled with load orientated in the east and west direction and a three and fifteen bay frame in the north and south direction. These frames considered the maximum percentage of load when they were modeled using the controlling load combination to get and address the possibility of the largest drift. The images show the drift values when loads were applied to the frames.

The deflections for wind were compared to a limit of L/ 400 as a conservative assumption outlined in ASCE 7-05 Appendix C. For Seismic loads the maximum drift was compared to .02 times the height of the frame are which specified in ASCE 7-05 table 12.12-1.

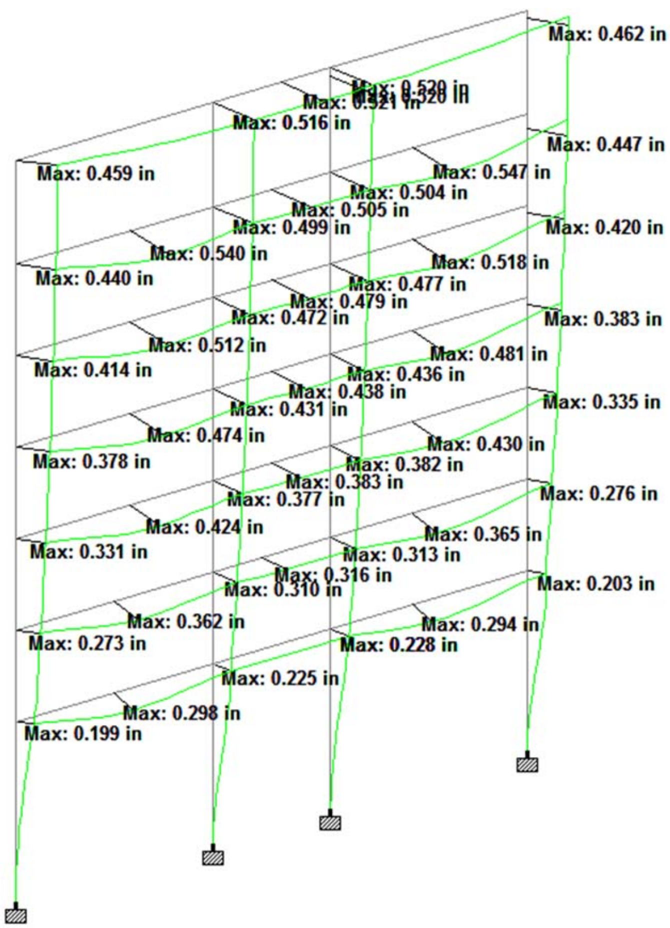


Figure: Drift Values for a Typical 3 Bay Frames E/W Wind Direction

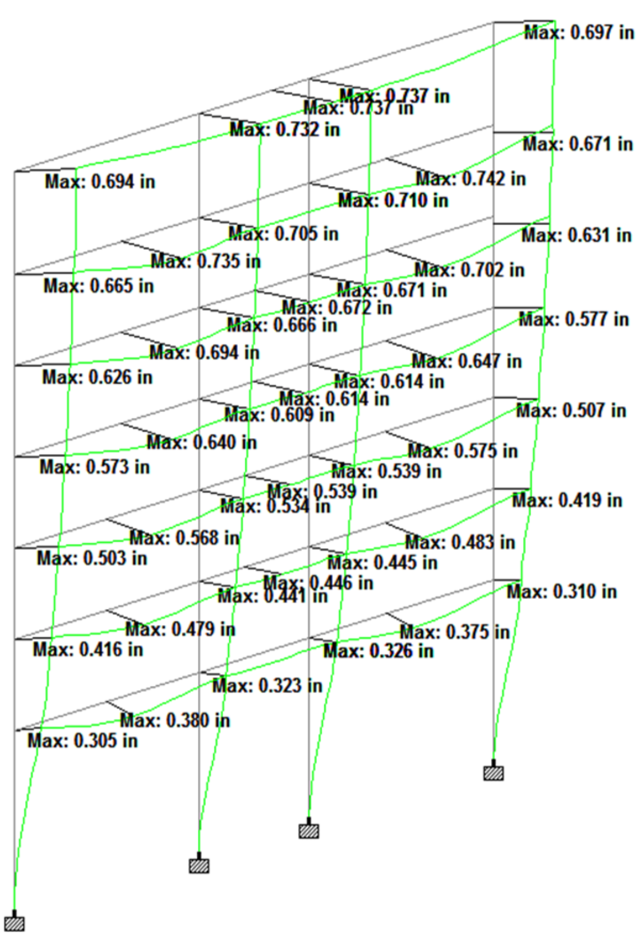


Figure: Drift Values for Typical 3 Bay Frames N/S Wind Direction

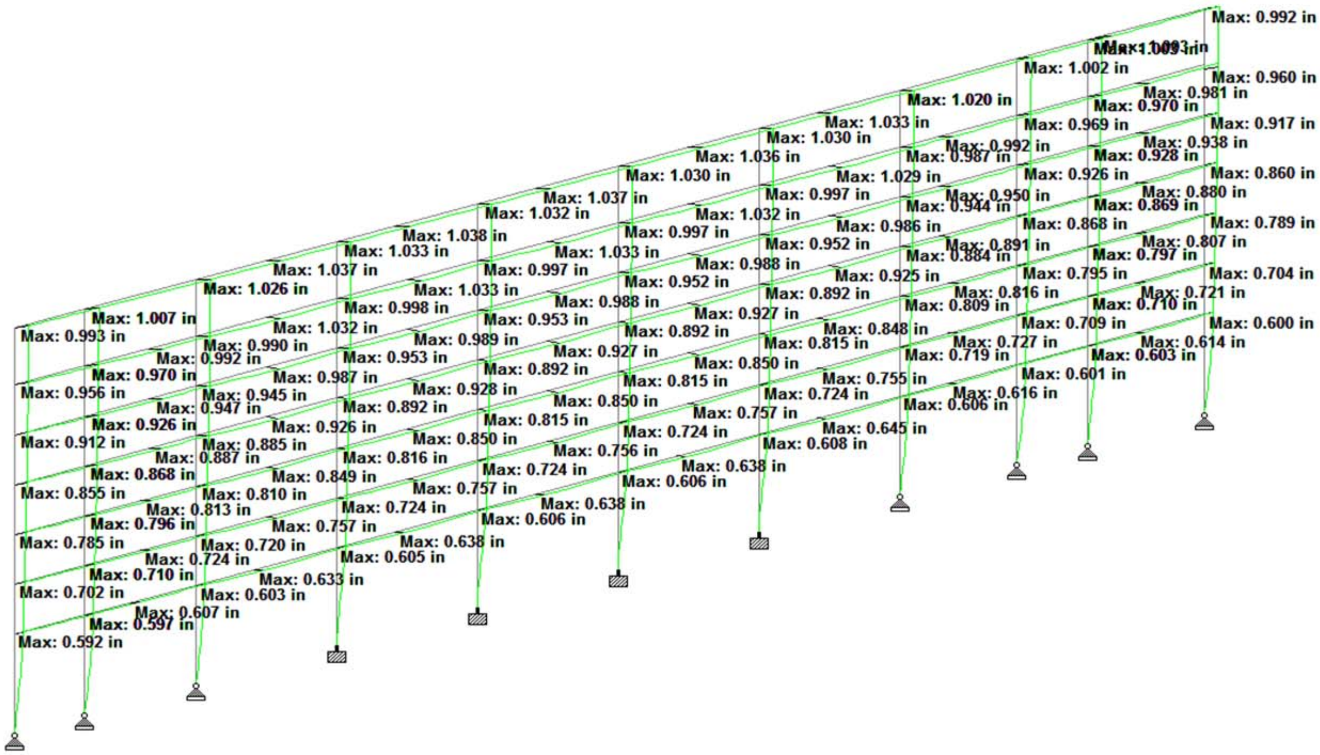


Figure: Drift Values for Typical 10 Bay Frames E/W Wind Direction

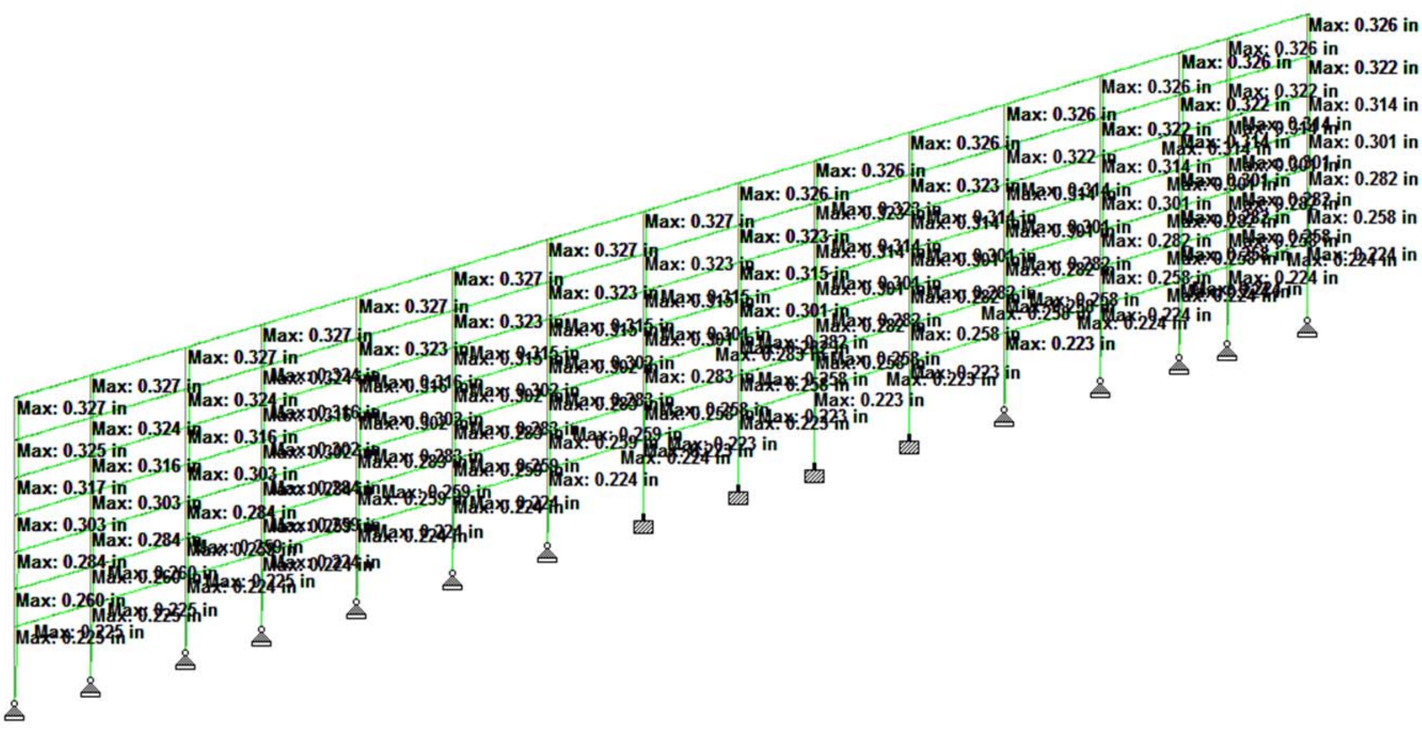


Figure: Drift Values for Typical 15 Bay Frames N/S Wind Direction

Through the shear and torsional analysis it was determined that the controlling lateral forces would be due to Wind forces. Although wind forces are the controlling lateral load entity, it is important to examine the effects of lateral forces exerted on the building to ensure that the frames meet drift limitations outlined in ASCE 7-05. Again, a series of frames typical frames were analysis to attain drift values per story of the building. Below are drift values for typical frames from a STAAD Pro computer analysis.

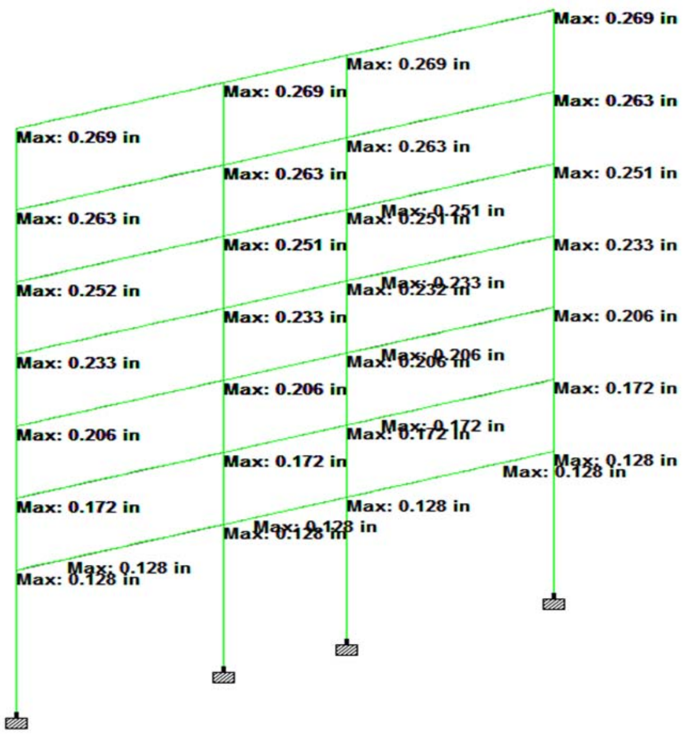


Figure: Drift Values for Typical 3 Bay Frame E/W Direction - Seismic

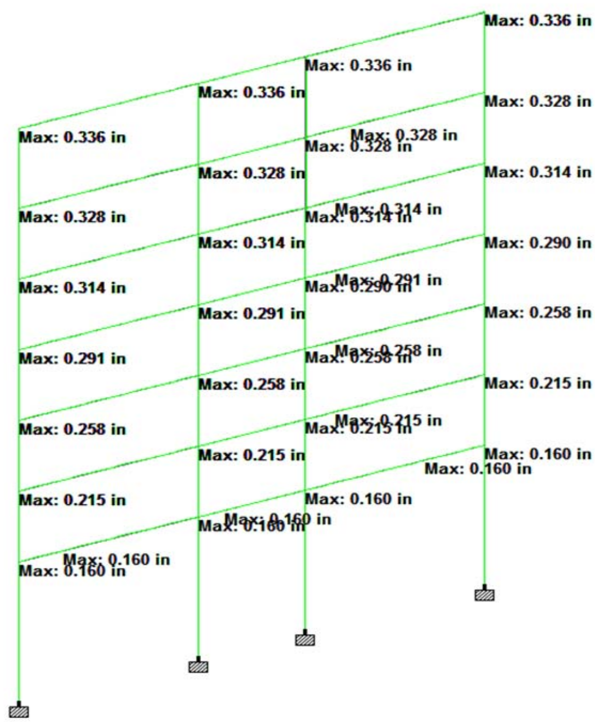
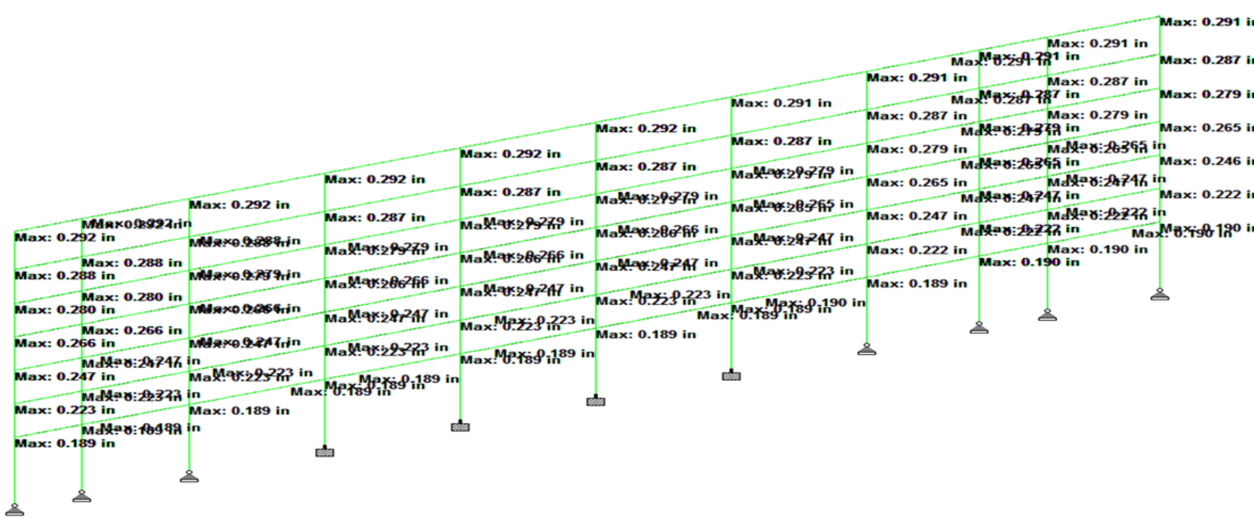


Figure: Drift Values for Typical 3 Bay Frame N/S Direction - Seismic



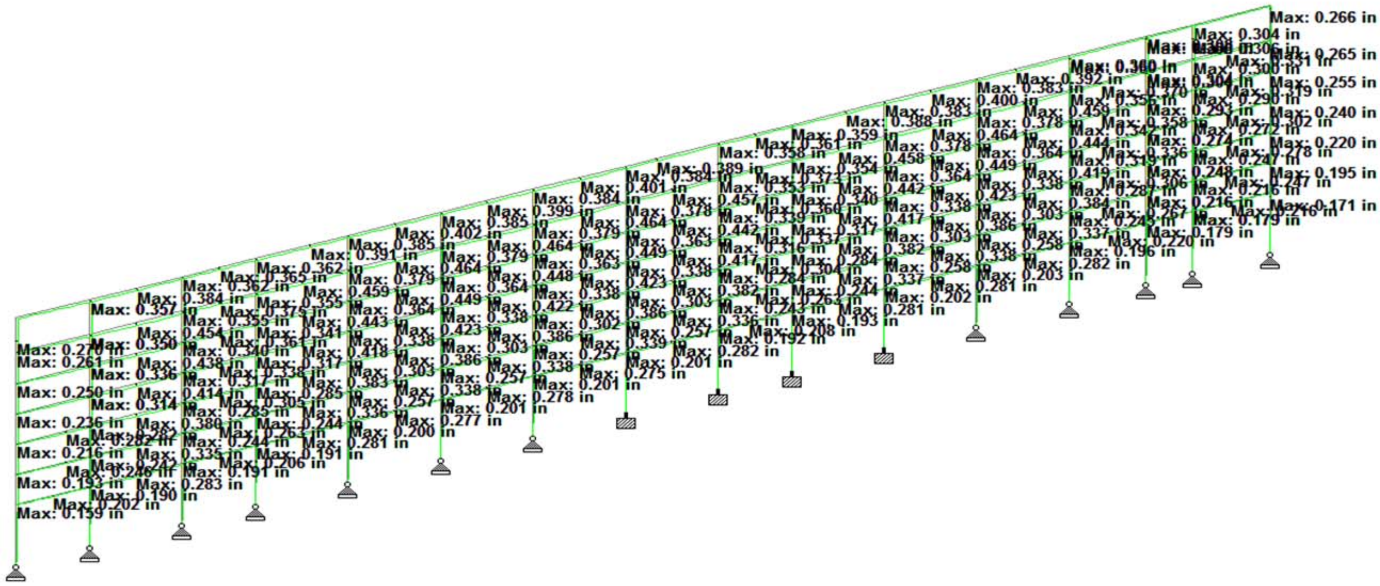


Figure: Drift Values for Typical 3 Bay Frame N/S Direction - Seismic

Direction	**Lateral Force**	**Frame**	**Maximum Drift (in) Steel Frame**	**Drift Limit (in) Steel Frame**
E/W	Wind	3 Bay	.462	2.33
E/W	Wind	10 Bay	1.03	2.33
N/S	Wind	3 Bay	.737	2.33
N/S	Wind	15 Bay	.326	2.33
E/W	Seismic	3 Bay	.269	1.12
E/W	Seismic	10 Bay	.291	1.12
N/S	Seismic	3 Bay	.326	1.12
N/S	Seismic	15 Bay	.4	1.12

Breath Studies

Breath Study I: Acoustics

For hotels, acoustics play a significant part in the planning of a structure to ensure the guest privacy and comfort. In the early phases of construction, an acoustical study was done to determine the sound vibrations due to the location of the hotel near major highways and air force base. This breadth study will delve into the effects of having a steel framed structure on the acoustics of the buildings and what practices and solutions could be put forth to ensure that sound and noise levels will be controlled and maintained. A room acoustics evaluation was performed to determine the noise criteria (NC) levels in a typical guest room.

To better understand how noise criteria will be determined it is important to look at a parameter called transmission loss (TL). Transmission loss is how much sound energy is not transmitted through a partition, in this case a typical guest room partition wall. The equation is given as:

$$TL = 10 \log (1/\tau)$$

Where τ is transmission coefficient in decibels (dB) .

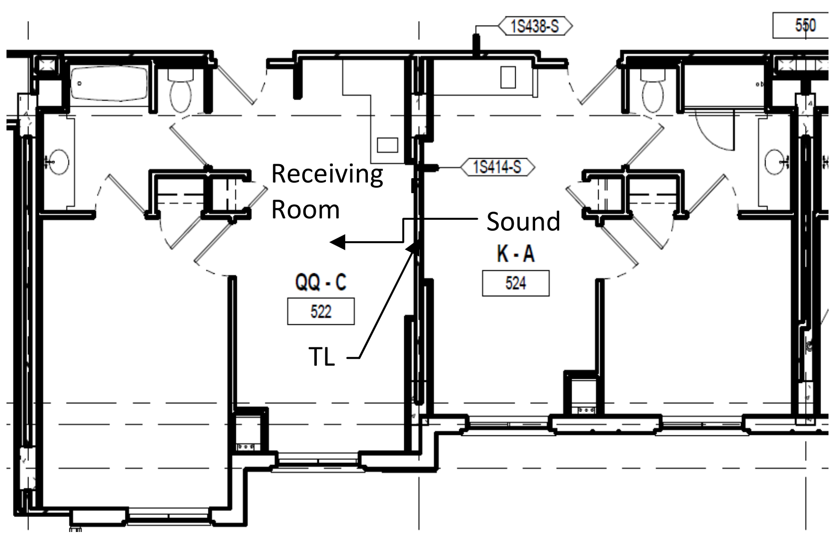


Figure: Typical Guest Room Layout

The transmission coefficient is how much sound actually gets passed through the wall partition.

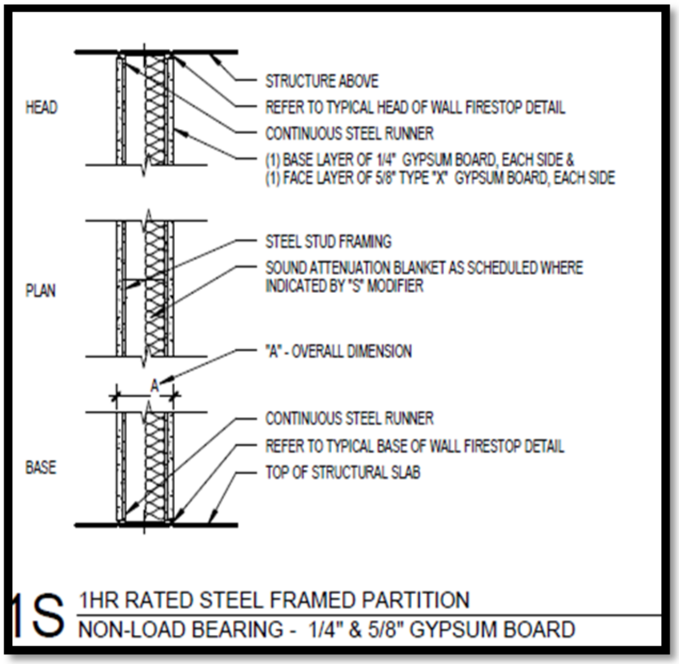


Figure: Typical Partition Guest Room Wall Assembly

The wall construction is composed of 2.5”, 25 gauge metal studs spaced at 24” on center. The wall also consist of two layers of gypsum wall board on either side of the stud with the innermost layer having thicknesses of ¼” and an outer layer of 5/8” sounds attenuation blanket. This aids in sound absorption. The fire rating of the partition wall is 1 hour. The overall wall thickness is 4.25”.

The next step was then to determine the Sound Transmission Class of the typical partition wall system. The STC is a single number transmission loss rating for a particular assembly. For the mentioned wall assembly the STC recommended is 54 dB. To measure this value a scatter plot of the transmission loss values in 1/3 octave bands were determined ranging from 125-4000HZ. Values for transmission loss data were taken for Appendix J of Architectural Acoustics: *Principals and Design*. Values highlighted in table were used to create the plot.

Freq (Hz)	TL (db)	Contour (dB)	Deficiency (dB)	Exceeds Max Deficiency
125	37	38	1	No
160	37	41	4	No
200	41	44	3	No
250	46	47	0	No
300	50	50	0	No
400	53	53	0	No
500	55	54	0	No
630	55	55	0	No
800	59	56	0	No
1000	60	57	0	No
1250	58	58	0	No
1600	56	58	2	No
2000	51	58	7	No
2500	51	58	7	No
3150	54	58	4	No
4000	58	58	0	No
		Total =	28	

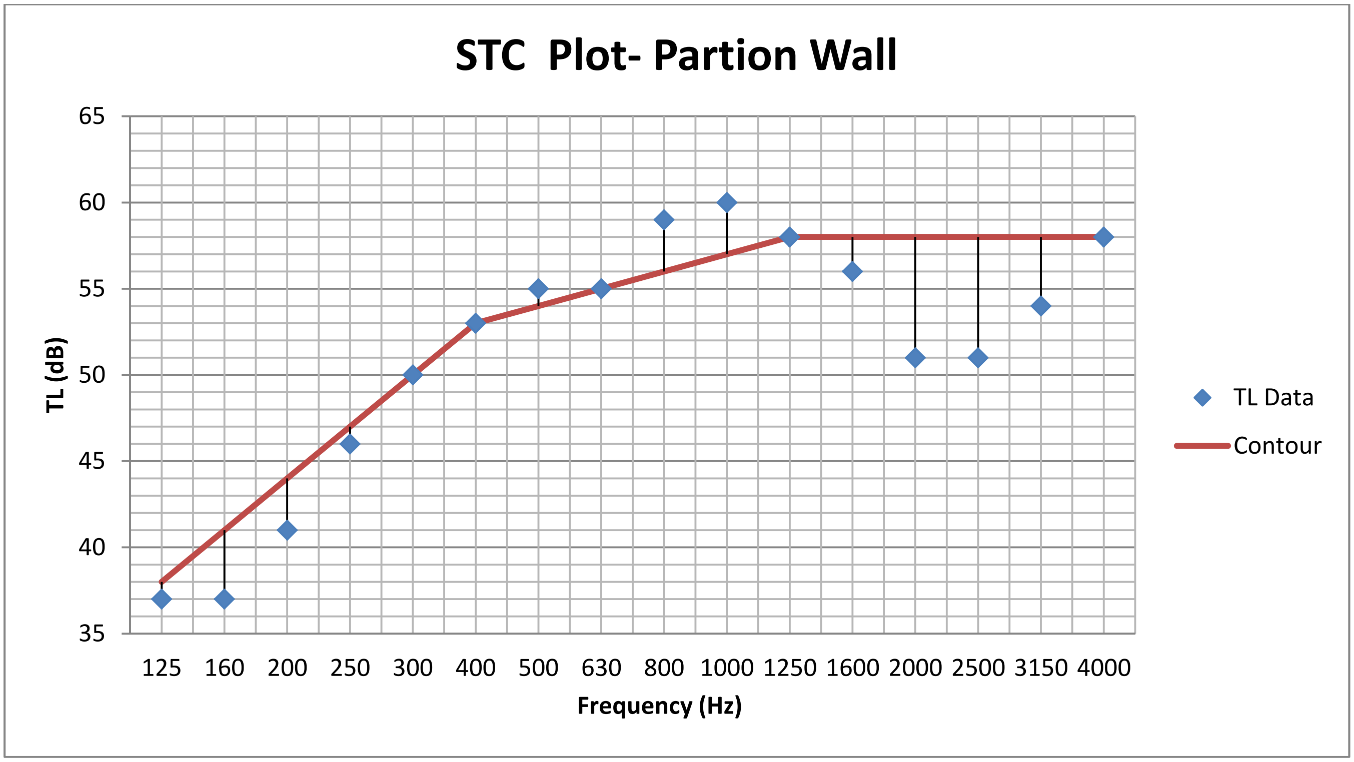


Figure: Guest Room Wall Assembly Plot

To determine the sound transfer coefficient of the wall, two parameters have to be met. No two plotted points can fall more than 8 dB below the contour line and the sum of all the deficiencies below the contour line can be no more than 32 dB. In this case, the sum of the total t deficiencies was found to be 21dbs which is acceptable for the criteria. The wall has a rating of STC-54. To give a better understanding of what the STC – 54 class falls in Loud audible speech is essentially blocked out by the wall assembly (in red). Since the wall assembly not was altered in anyway, even with the columns being converted to steel significant change to the STC was not to be expected.

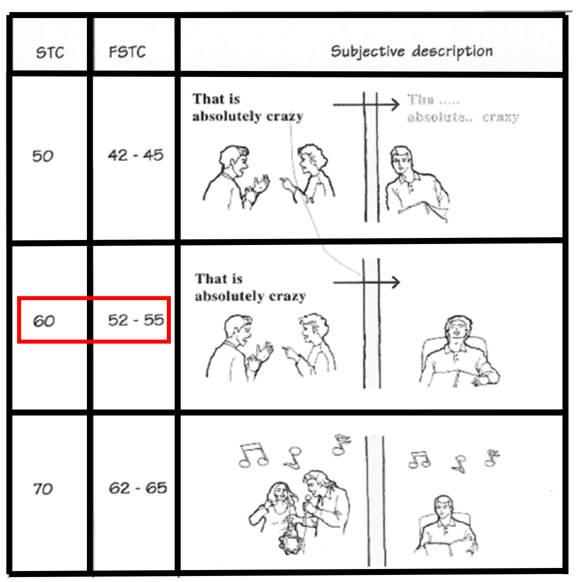
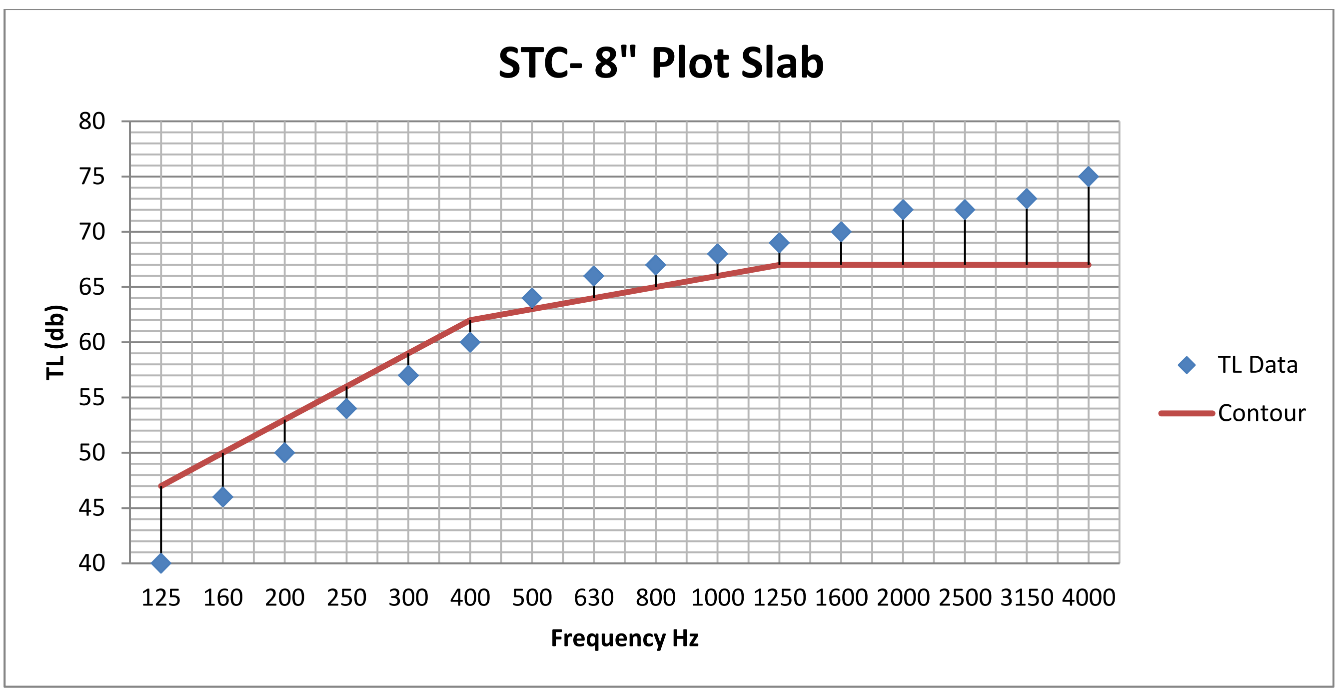


Figure: STC Class Illustration

For the redesign, the floor system was changed from an 8" concrete slab to a 6.5" concrete slab on metal deck. Sound Transmission Class was then determined for both the existing slab and the resigned composite floor. Assumptions were made in Appendix J of Architectural Acoustics: *Principals and Design* for the floor system data used, choosing TL data from similar floor assemblies that closely matched the existing and redesigned floor systems. The related floor systems used were (1) 8" solid concrete slab with 2x 2 wood furring, fiber glass insulation and 5/8 inch gypsum wall board for the existing floor and a (2) 6" solid concrete slab with 2x 2 wood furring, fiber glass insulation and 5/8 inch gypsum wall board for the composite floor. The sound transmission classes have values of 63 and 62 respectively. For a typical guest room floor system a value of 60 db or above for the sound transmission class is recommended. Again determined the STC a scatter plots of the transmission loss values in 1/3 octave bands were determined ranging from 125-4000Hz.

Freq (Hz)	TL (dB)	Contour (dB)	Deficiency (dB)	Exceeds Max Deficiency
125	40	47	7	No
160	46	50	5	No
200	50	53	3	No
250	54	56	2	No
300	57	59	2	No
400	60	62	2	No
500	64	63	0	No
630	66	64	0	No
800	67	65	0	No
1000	68	66	0	No
1250	69	67	0	No
1600	70	67	0	No
2000	72	67	0	No
2500	72	67	0	No
3150	73	67	0	No
4000	75	67	0	No
		Total =	21	



Freq (Hz)	TL (db)	Contour (dB)	Deficiency (dB)	Exceeds Max Deficiency
125	42	47	5	No
160	44	50	6	No
200	47	53	6	No
250	51	56	5	No
300	56	59	3	No
400	59	62	3	No
500	60	63	3	No
630	62	64	2	No
800	63	65	1	No
1000	65	66	0	No
1250	68	67	0	No
1600	69	67	0	No
2000	69	67	0	No
2500	72	67	0	No
3150	75	67	0	No
4000	76	67	0	No
		Total =	34	

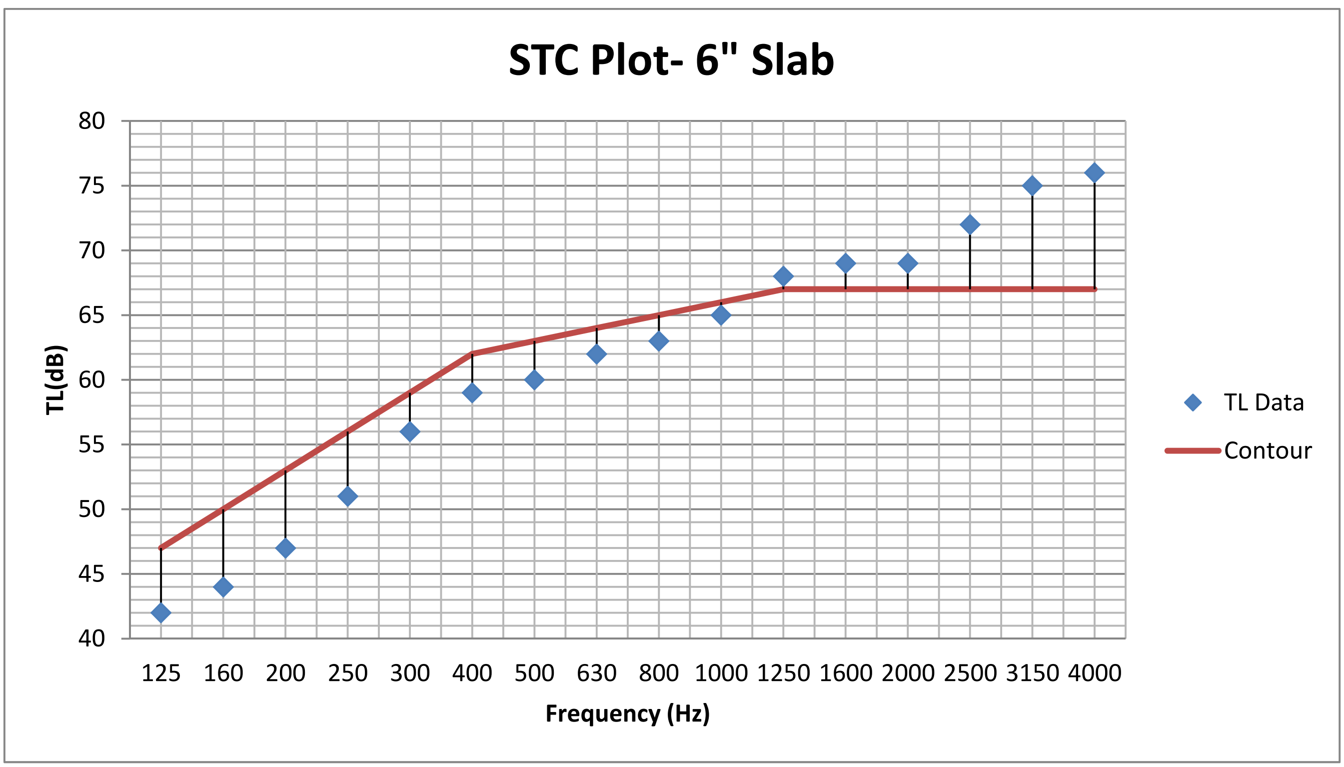


Figure: Kinetics Soundmatt

To ensure that the redesign floor meets the standards of the existing system a sound proofing material for the floor system would be suggested.

The Kinetics Soundmatt is a floor mat system that is underplayed under the floor assembly used to control sound transmission of both impact noise and noise in floor systems. The Soundmatt has a thickness of 5/16" (8 mm) which is made of pre-compressed molded glass fibers. This material puts forth a system that contains enough stiffness to prevent grout cracking in tile floors. It is also is dense enough to enough to reduce noise traveling through the floor systems. Having this material will greatly improve the overall STC values, making it exceed the recommended value for hotels. With the addition of another layer this will help limit impact sounds from the floors above. An example of impact noise in hotel would be guests walking on upper floors

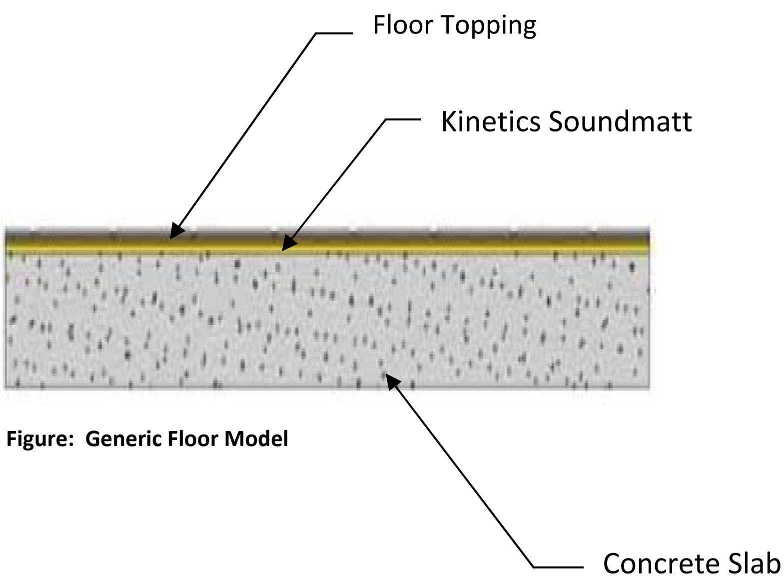


Figure: Generic Floor Model

Breathe Study II: Construction Breadth

With the proposed redesign, it raises many questions about how the Embassy Suites Hotel project construction management process. By changing the concrete framing system to steel construction it was examined how altering the predominant material in the building will affect the way the project is constructed and the overall construction layout and steel erection process. Additionally look into how the sizing of equipment and other essential materials needed on site.

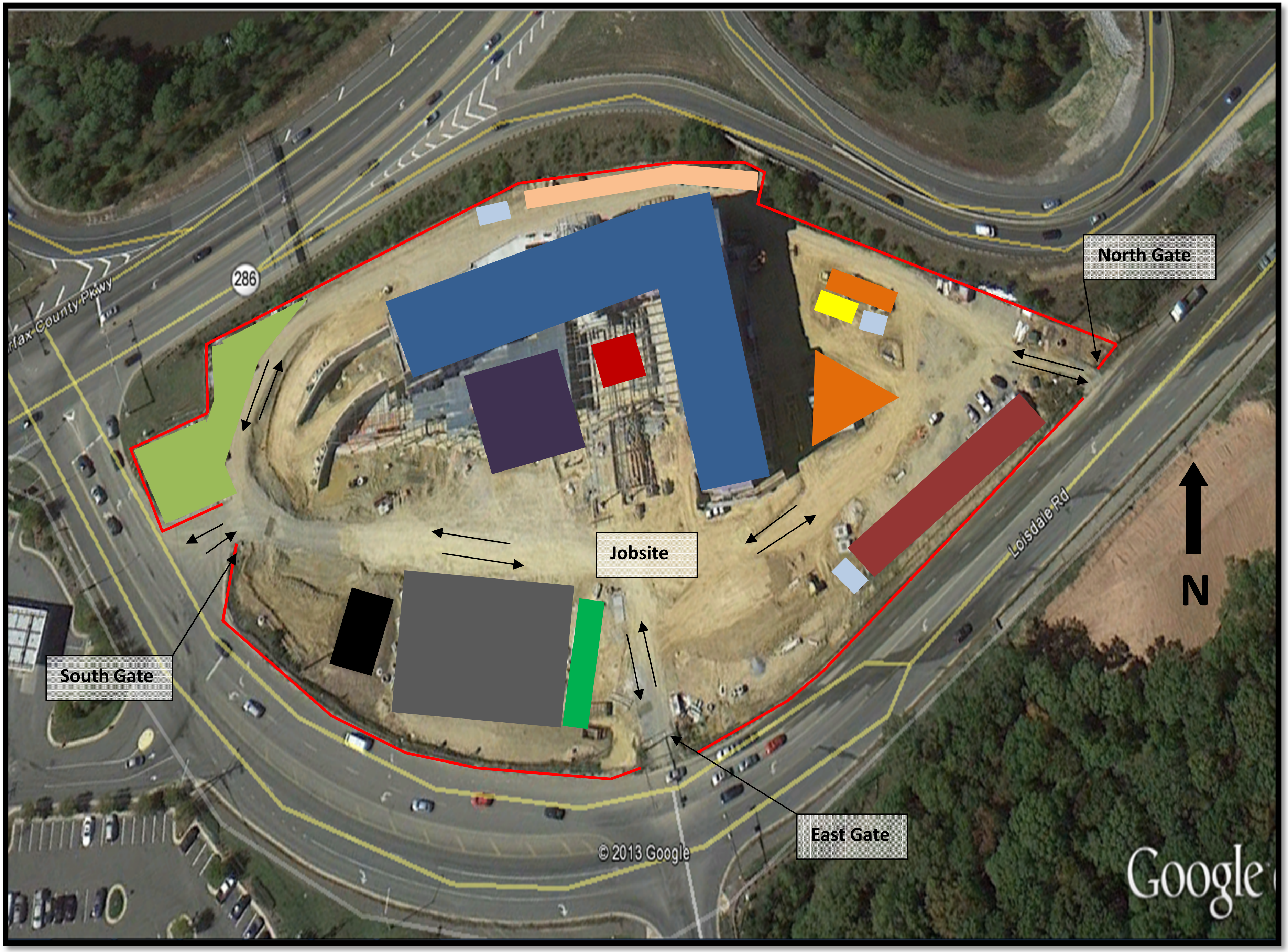
Existing Sight Conditions

Before initial planning of the site layout, the existing landscape of the project location was examined. The site of the Embassy Suites Hotel jobsite contains a two story existing motel structure. The site lies between two major highways, I-95 Loisdale Road and Route 7100 Road. The site is also located in close proximity to Fort Belvoir and Davidson Army Airfield located roughly 5 miles to the east.

Site Layout

On any construction site security is an issue. A metal chain link chain linked fence will be placed around the perimeter of the site to ensure safety no unwanted traffic of vehicles and persons. The site will feature tree entrance points, gates at the north, south and east directions. These entrance points that will allow for maximizing construction flow of vehicle delivery of materials and easy maneuverability around the construction project. The construction flow pattern is highlighted by the white arrows indicated on the sight layout drawing. A color coordinated site layout can be found before for the jobsite. The map legend can be found below.

Temporary Power		Temporary Construction Fence	
Building Foot Print		Steel Lay down Area	
Material Storage		Equipment Storage	
Delivery / Staging Area		Portable Toilets	
Sump Area		Job Trailers	
Worker Parking		Dumpster / Recycling Area	
Delivery/ Site Vehicle Flow	→	Crane Area	


North Gate
South Gate
East Gate
Jobsite
N
286
Fairfax County Pkwy
Loisdale Rd
© 2013 Google
Google

Throughout the course of the project many import placement of basic construction site feature s had to be considered. Portable toilets were located in the staging area of the project because of its central location on the site to allow for easy access. Temporary power can be located in out out near the equipment storage near the east site of the proposed to have close proximity to required tools and machinery. he south entrance of the site, Equipment storage is place near temporary power outlets to allow for ease of access power supply and to reduce the overall usage and length of temporary wiring. The power itself will be tapped in tapped into an existing power line running along the east edge of the site of the site. Recycling and waste dumpster areas are to allow for easy disposal and removal. The jobsite trailers were placed at the north end of the sigh to limit vehicle congestion but at the same time allow for maneuverability on and off site for construction staff.

Peak traffic hours for the interstate I-95 7:30 am to 8:30 am, 1130am to 12:30 pm and 4:30 pm to 5:30 pm. The steel framing members should be delivered sometime between these to time values to avoid delay in project schedule. The steel should be delivered and at either the east or south gates and placed at the staging / delivery are for relocation to the temporary steel stockpile located next to the crane. For the most effective erecting sequence of the steel the crane should be placed in the center of the L- shaped design building the middle sections then expanding outward to either ends of the building.

Figure: Manitowoc TMS800E

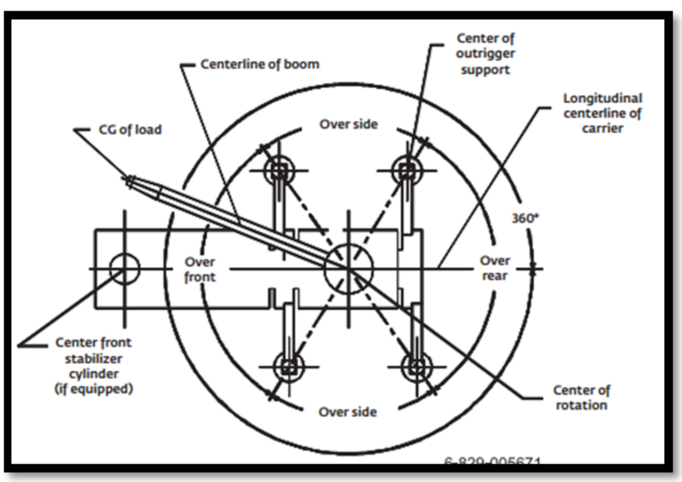


Figure: Crane Work Area Diagram

With the Embassy Suites Hotel being converting from a concrete structure to a steel framed structure one of the biggest challenges would the determination strategic placing of the crane. The erection of the structural steel is one of the most critical components of the redesign of the hotel. The project roughly the amount of structural steel that is need to be erected is about 194,000 pounds per floor of the building. For this job a Manitowoc TMS800E crane was chosen. This crane features a four section boom with a maximum extension length of 128 feet. The crane also features a 56 foot bifold swingway. The overall crane base has dimensions of 50 feet in length and 24 feet in width taking up an area of 1200 square feet on the site.

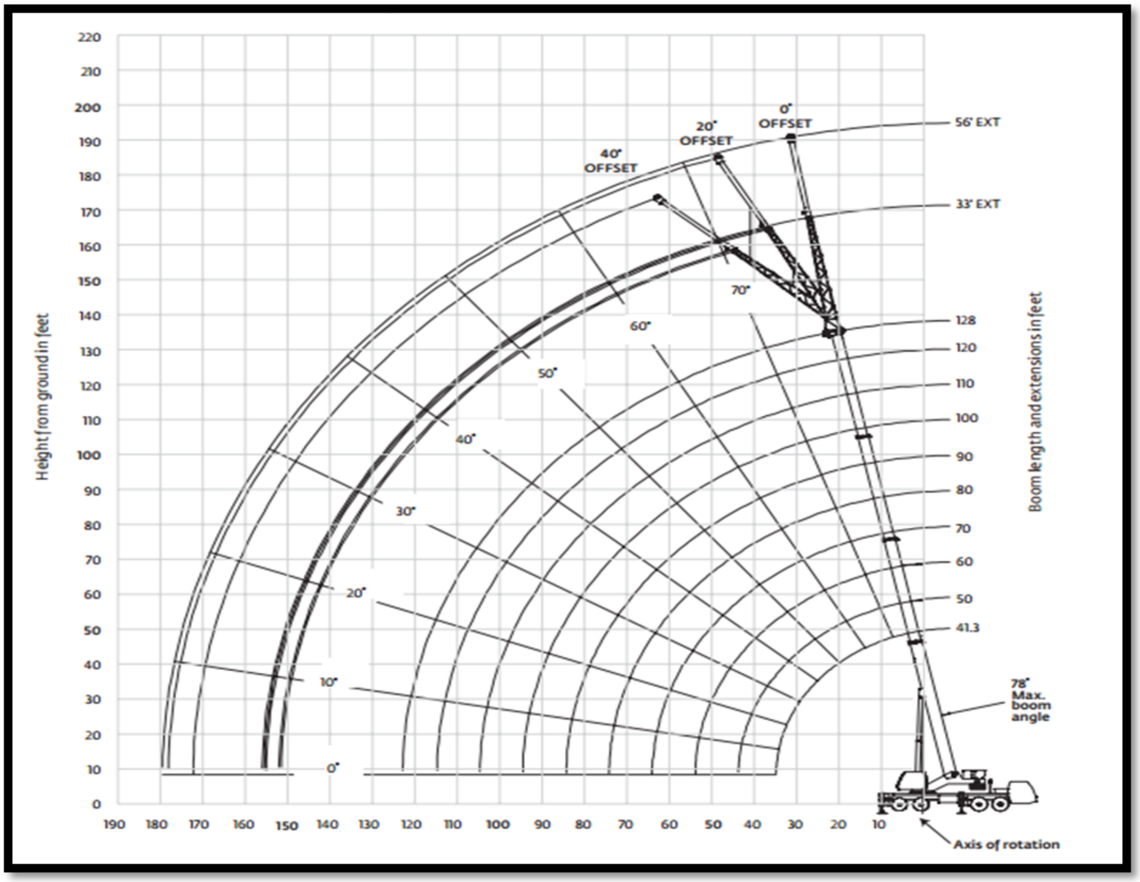


Figure: Crane Extension Diagram

The amount of load that the crane can hold at any one time varies with the given extension length of the boom. The maxim average amount of structural steel that could be hoisted at one time is 17,160 pounds This crane design, because of the crane being mounted on a mobile platform, will allow for quick deliver assembly and movement of the crane around the jobsite.

Conclusion

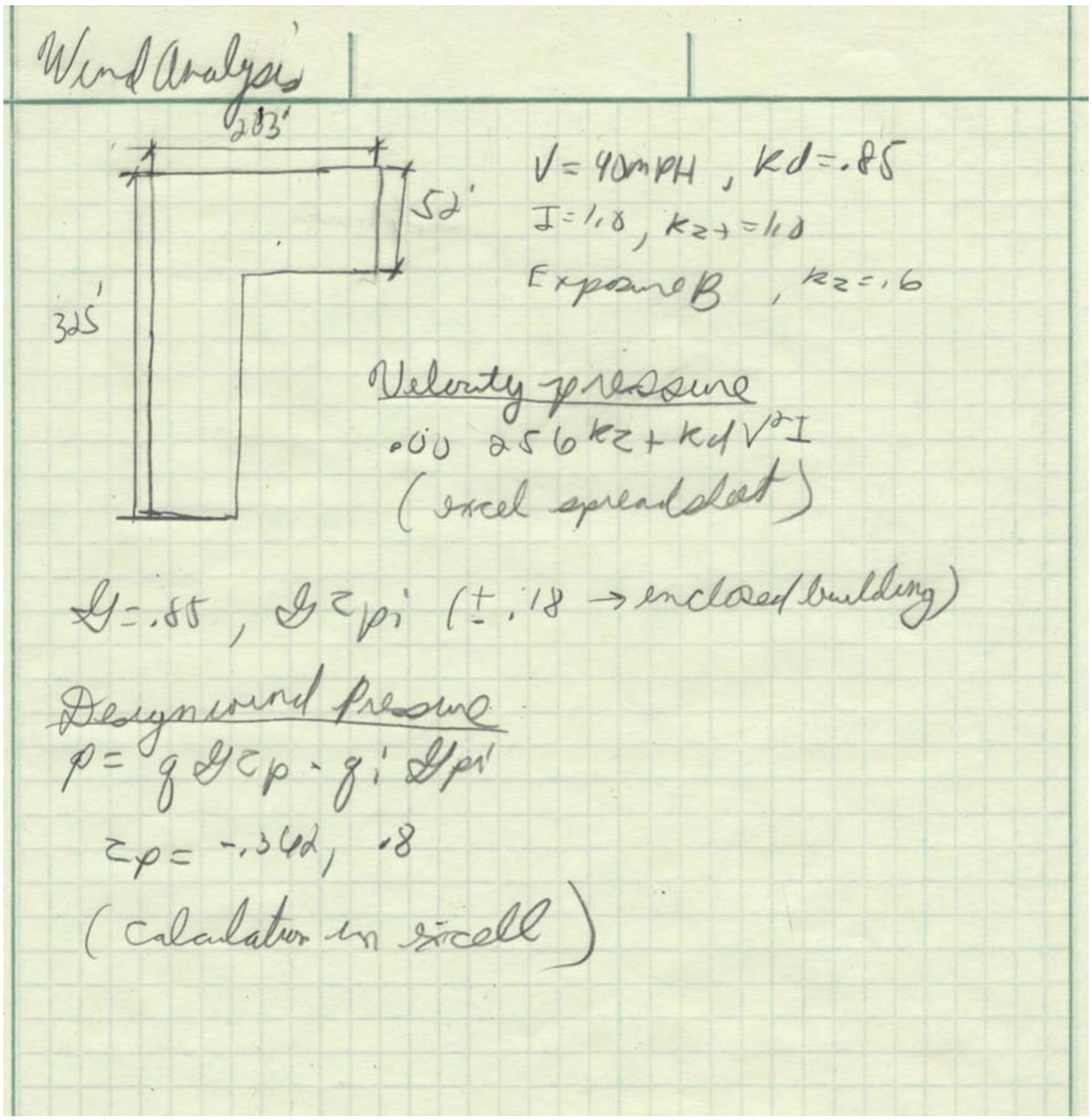
The main goal for the depth of the proposal was met through the design steel framing structure two floor systems were compared slab beam system and a composite floor system was determined that a 6.5” composite floor system would best option for their design due to the . For the lateral framing system W14 x 74 steel columns and W10 x 26 beams would make up the moment resting frames that would be put in place compared to the original concrete system. With the conversion of the building to steel, the result is an overall building weight reduction of roughly 50 percent and an overall a reduction in the overall base shear of for lateral load distribution was also reduced. The bay layout corresponds with the existing hotel floor plan with minor adjustments to the sizes of bays. The lateral frames and members are easily laid using the original locations making this configuration a sufficient system to keep the building drift within code limitations and try to limit architectural impacts. One noticeable drawback to the conversion of steel material is the increase in floor-to-floor of the building. The increase per floor is and total height increase is the. The building height increase with the given height limitations due to zoning would cause a potential problem if the steel system were chosen to be implemented.

In changing the material of a building there are many factors to consider for hotels acoustics plays an important role in ensure the comfort of a guest. For a typical guest room, the sound transmission class was determined for a wall that adjoins guest rooms. Both the existing the redesign floor system and the sum of the total deficiencies were found acceptable for the criteria for hotel rooms. Since the wall assembly not was altered in any way it was shown even with the column material change the STC of the wall was not altered. To ensure the redesign floor had sufficient sound transmission proprieties a Kinetics Soundmatt system would be installed to ensure that sound levels are attained.

Additionally the impact of changing the structure of the building on the construction site layout examines Changing the steel framed systems brings out sight coordination problems mainly in the placement for the crane. A site layout plan was developed for the erection process of the steel framing members. A crane was specked that would be able to handle the erection of the steel framing of the redesign. Due to the relative size of the site the relocation of the crane would no t be an issue.

The overall goal of redesign was to try to design an effective and efficient structural system that would be comparable to the original concrete framed structure. It was shown through calculation and research that an adequate alternative could be developed for the Embassy Suites Hotel. Overall, due to zoning limitations and height restrictions the original design would be the best option for the Embassy suites project b however this redesign could be a viable option if project parameters were different.

Appendix A: Wind Loads


Wind Analysis

283'
52'
325'

V = 90mPH , Kd = .85
I = 1.0 , Kzt = 1.0
Exposure B , Kz = .6

Velocity pressure
.00256 Kz + Kd V² I
(excel spreadsheet)

G = .85 , GCpi (± .18 → enclosed building)

Design wind Pressure
p = qGCp - qi GCpi
Cp = -.342, .8
(Calculation in excell)

Appendix B: Seismic Loading

Seismic Analysis

$S_1 = .051$ $S_S = .155$, $F_V = 2.4$, $F_a = 1.6$

$S_{MS} = .248$ $S_{DS} = .165$ $S_{M1} = .1224$ $S_{D1} = .0816$

*use soil class "B" (# from previous analysis)

– Base Shear

$T_a = .028(56')^{.8} = .701$

$T_a = 1.7(.701) = 1.192$

$C_s = \frac{.166}{(3.5/1)} = .047$ $C_s = \frac{.0816}{1.192(3.5)} = .01955$

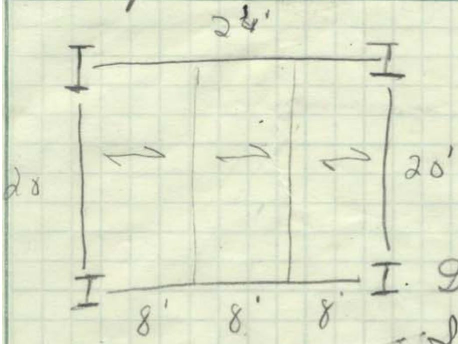
$V = (.01955)(8600^k) = 167.9^k$

$FL = .075(168) = 13^k$
$.232(168) = 39^k$ → story shear
$.231(168) = 38.8^k$

(remainder of calculation done in excel)

Appendix C: Floor Design

Composite Plan



$W_{LL} = 40 psf + 15 psf = 55 psf$
$W_{DL} = 10 psf + 63 psf_{SW}$
3VLI 20 → 3 SPAN condition
Deck span (8') < Max Construction SPAN / 9-
- Superimposed LL (251 psf) > 128 psf

- Beam Design

$W_u = [1.2(73 psf) + 1.6(55)](8') = 1.4 klf$

$V_u = \frac{(1.4 klf)(20)}{2} = 14^k$, $M_u = \frac{(1.4)(20)^2}{8} = 70^{k\text{-}ft}$

beff (MIN) | $\frac{20(12)}{8} = 30''$ → controls → beff = 60''
$\frac{(8')(12)}{2} = 48''$

- member size
- want to put PNA in Conc.
$A_s F_y = .85 f'_c\, b_{eff}\, a$
- assume $a = 1''$, $F_y = 50$ ksi, $f'_c = 5000$
$Y_2 = t - a/2 = 6.5 - 1/2 = 6''$
Try W10x26 → $\phi M_p = 200$

$a = \frac{127}{.85(5)(60)} = .498 < .5$, $Y_2 = 6$ ok ✓

$V'_c = .85(5ksi)(60)(3.5") = 892.5$

$V'_s = (7.61)(50ksi) = 380.5^k$

$V'_c > V'_s$, $a = \frac{380.5}{.85(5)(60)} = 1.5"$

$\phi M_N = \phi A_s F_y \left[\frac{d}{2} + hr + t - (a/2)\right]$

$(.9)(380.5)\left(\frac{10.3}{2} + 3" + 3.5" - (1.5/2)\right] = 346.7^{k\text{-}ft}$

$\phi M_N > M_u$ ok ✓

- wet concrete Deflection

S.W BEAM

$W_{wc} = 63psf(8') + 26plf = .53klf$

$\Delta_{wc} = \frac{5(.53)(20')^4(1728)}{384(29{,}000)(144)} = .457"$

$l/240 = \frac{20(12)}{240} = 1" > .457"$ ok ✓

- unshored Strength

$W_{LL} = C_{LL} = 20psf(8') = .160$

$W_{DL} = (.53klf$, $W_u = 1.2(.53) + 1.6(.160) = .892$

$M_u = \frac{.892(20)^2}{8} = 44.6^{k\text{-}ft} < \phi M_p = 117^{k\text{-}ft}$ ok ✓

- Shear:

$\phi V_N = \phi(.6)(50)(.26)(10.3") = 72.3^k$

$\phi V_N > V_u$ ok ✓

- Shear studs
  - assume 3/4" dia. STUDS, Deck ⊥ beams
  1 stud / rib, weak position

$Q_n = 17.2$, # of studs $= \frac{\Sigma Q_n}{Q_n} = \frac{386.5}{17.2} = 24$

use 24 studs / beam

- LL Deflection

$W_{LL} = (55)(8') = .44$

$\Delta_{LL} = \frac{5(.44)(20)^4(1728)}{384(29{,}000)(960)}$

$Y_2 = 6.5 - 1.5/2 = 5.75$, $Y_2 \approx 6$

$\Delta_{LL} = .12''$

$l/360 = \frac{20(12)}{360} = .667 > .12$ ok ✓

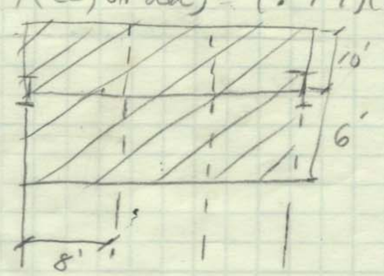
Girder Design

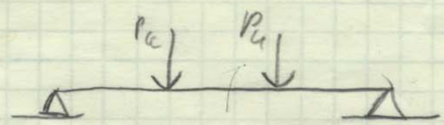
$W_{T(DL)} = (63_{DSD} + 10\ psf)(8') = .584\ klf$

$W_{T(LL)} = (.44\ klf$

$W_{T(DL,\ Girder)} = (.584)(24)(1.2) = 16.8$

$W_{T(LL,\ Girder)} = (.44)(24)(1.6) = 16.8$





$P_u = 33.6$

$M_u = (8')(33.6) = 268.8$

b_{eff} | $\frac{24'(12)}{8} = 36$ → Controls $b_{eff} = 72'$

| $\frac{16(12)}{2} = 96 =$

assumed $6'' = y_2$

Try W10x26

$\phi M_N = 346.7^{k\text{-}ft}$

- check unshored strength

$P_u = [1.2(.026 + .026) + 1.6(20)(8')](16')$

$P_u = 4.1^k (8)$, $M_u = (4.1)(8) = 32.8^{k\text{-}ft}$

$\phi M_N = 177 > 32.8$ ok

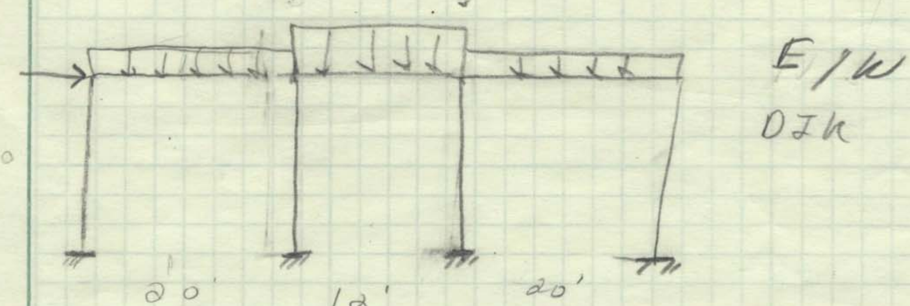
∴ use W10x26

Appendix D: Framed Design

3 BAY Frame Design

- estimate column size

3 BAY FRAME @ Level 1



Load

LL = 55 Guestrooms, 100 corridor

DL = 10psf + 73 psf + 26plf (BEAM LOAD)

Load Combos → 1.2D + 1.0E + .5L + .2S
1.2D + 1.0W + L + .5S

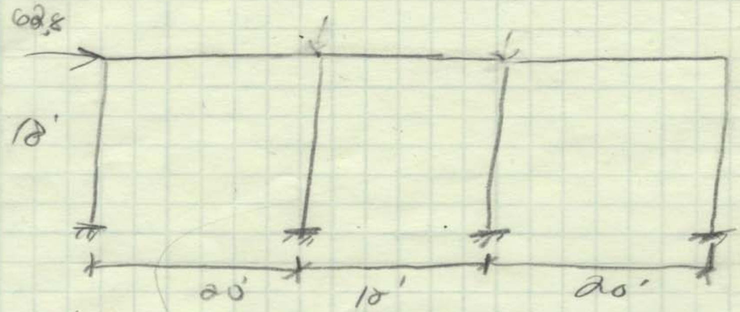
int column

LL → 55(10)(24) + 100(6')(24') = 27.6k

24'
16' TRIB

DL → 73(24)(16) + (16')(26) = 28.4k

LAT LOAD → 62.8k



$\frac{L}{600} = \frac{18'(12)}{600} = .36$

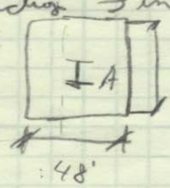
Load on int col → 27.4^k (LL) + 28.45 (DL)

Load on ext col → (55)(20')(24') + [(73)(10')(24') + 40)(20')

13.2^k + 17.78^k

6 FLOORS + ROOF

LL reduction → int col → $LL = L_0 \left(.25 + \frac{15}{\sqrt{I \cdot A}}\right)$



IA = 1536 ft² → can reduce

$LL = 27.6^k \left(.25 + \frac{15}{\sqrt{1536}}\right) = (27.6)(.632)$

$LL = 17.5^k$

ASCE 7-05

1.2D + 1.6W + L + .5(Lr)

1.2D + 1.0E + L + .2S

DL (FL) = 1.2(28.9)(6 FL) = 204.5^k
(ROOF) 1.2(12.49 psf)(24)(16') = 5.75^k

LL (FL) = 17.5^k(6 FL) = 105^k
ROOF = (30)(24)(16)(.5) = 5.76^k
= 321.1^k

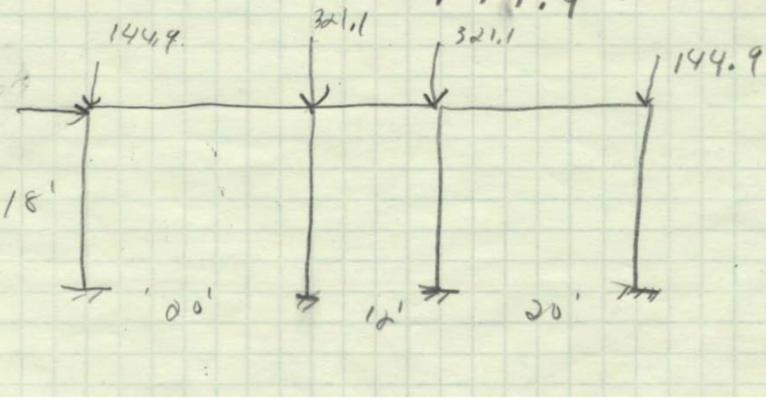
Exterior Column

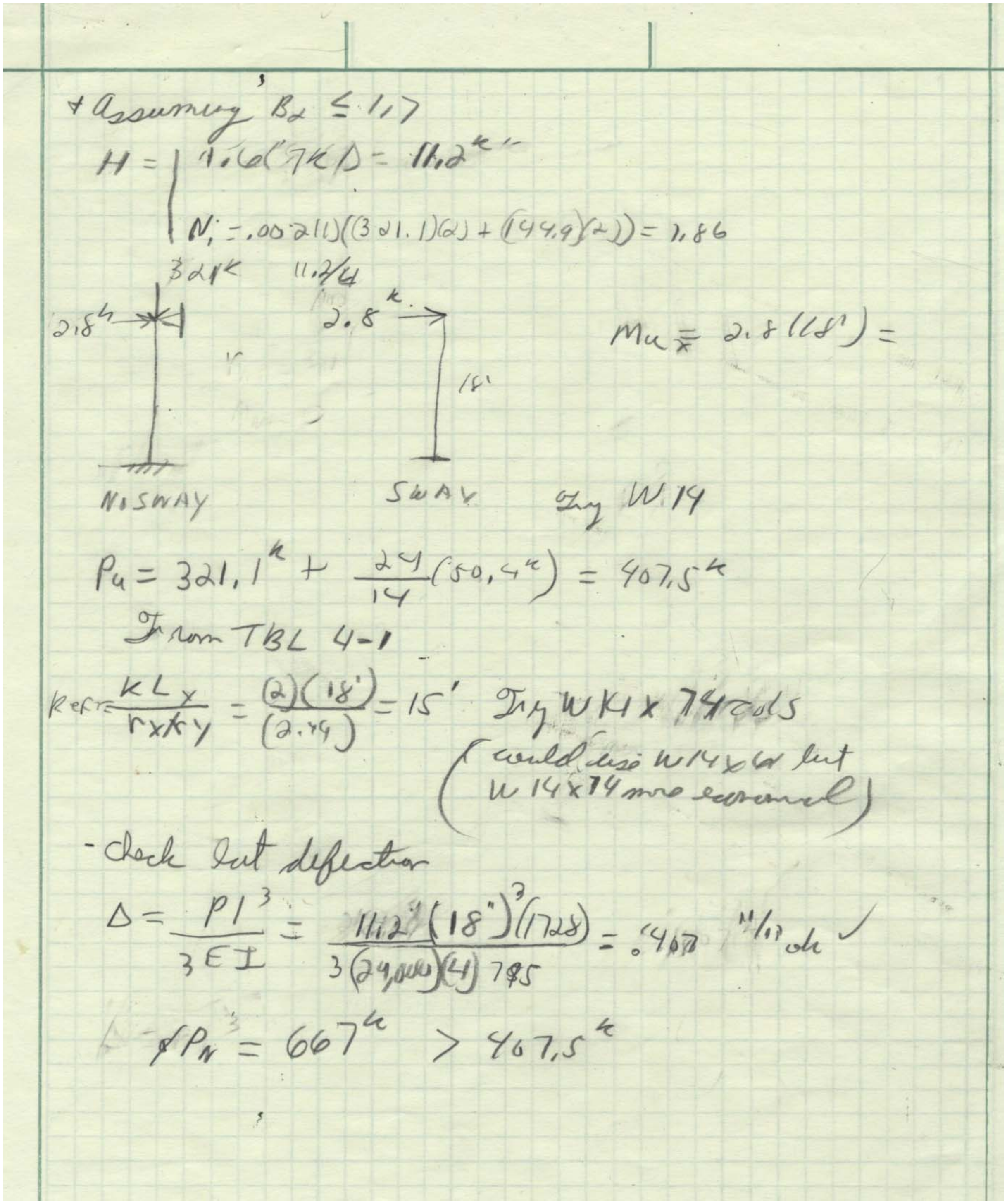
24 × 10 TRIB

DL = 1.2(17.78) = 21.3^k(6 FL) = 128^k
1.2(12.49)(24')(10') = 3.6^k
LL (Roof) = (30)(24)(10)(.5) = 3.6^k

20' × 48' I.A

LL (FL) = LL = 13.2(.25 + 15/√960) = .73(13.2)
= 9.7^k
144.9^k




+ Assuming $B_2 \leq 1.7$

$H = 1.6(7^k) = 11.2^k$

$N_i = .00211((321.1)(2) + (44.9)(2)) = 1.86$

$M_u = 2.8(18') =$

Try W14

$P_u = 321.1^k + \frac{24}{14}(50.4^k) = 407.5^k$

From TBL 4-1

$K_{eff} = \frac{KL_x}{r_x/r_y} = \frac{(2)(18')}{(2.44)} = 15'$ Try W14 x 74 cols

(would use W14x61 but W14x74 more economical)

- check lat deflection

$\Delta = \frac{Pl^3}{3EI} = \frac{11.2(18')^3(1728)}{3(29{,}000)(4)795} = .407$ in/in ok ✓

$\phi P_n = 667^k > 407.5^k$

$P_{nt} = 321.1^k$ $\quad P_{lt} = 2.8^k$

$M_{nt} = 76^{k\text{-}ft}$ $\quad M_{lt} = 50.4^k$

(FROM FC DESIGN)

$P_{story} = \Sigma P = (2)(321.1) + (144.9)(2) = 932^k$

$R_m = .85$ for moment frame

$$P_{e\,story} = \frac{.85\,(11.2)(18')(12''/ft)}{.407/.8} = 4041.4$$

$$B_2 = \frac{1}{1 - \frac{932}{4041.4}} = 1.3 < 1.7 \text{ ok for assumption}$$

$P_r = 321.1 + 1.3(11.2) = 335.66^k$

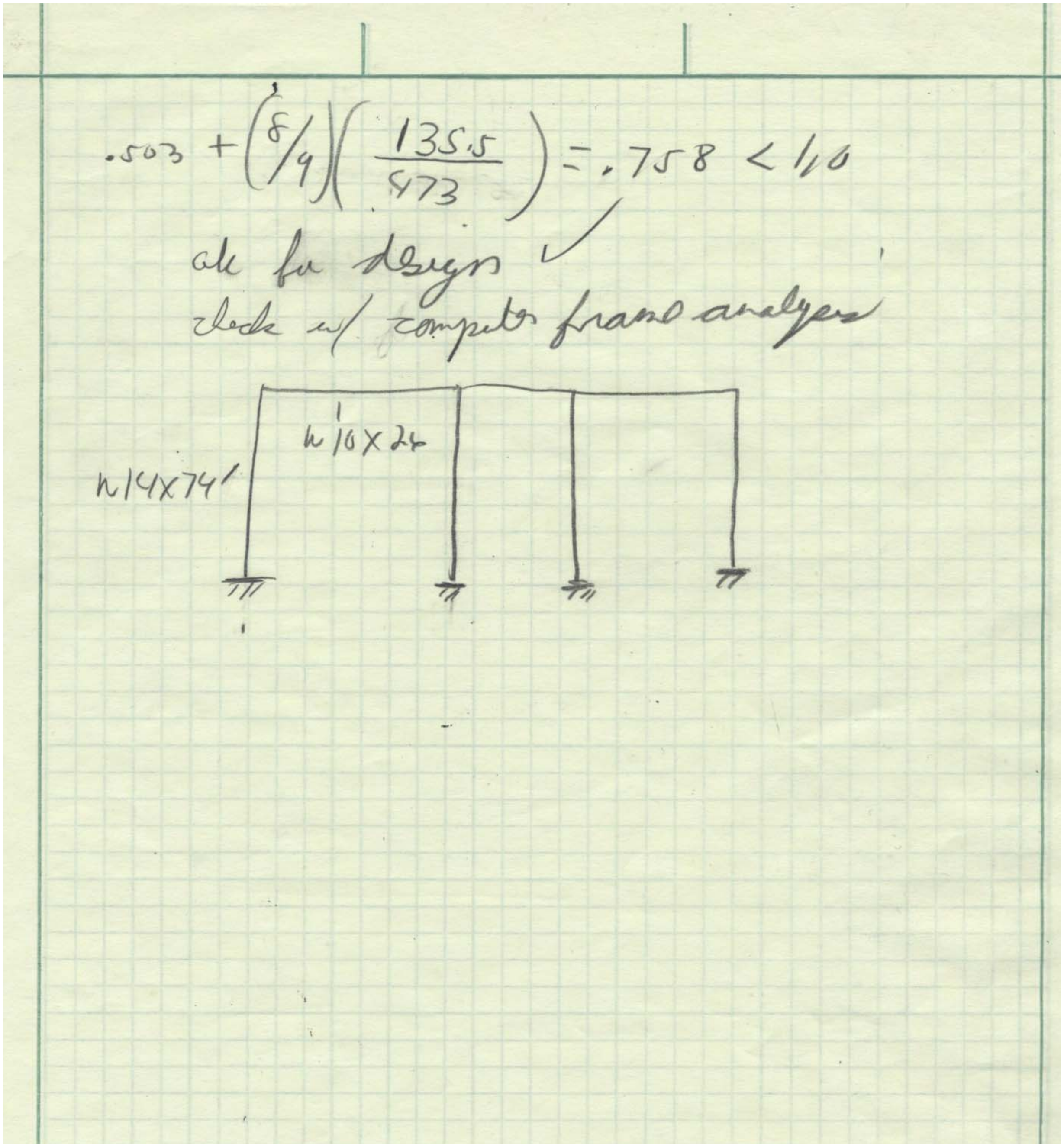
$C_m = (.6) - (.4) = .2$

$$P_{e1} = \frac{\pi^2 (29{,}000)(795)}{[(2)(15')(12)]^2} = 1755$$

$$B_1 = \frac{.2}{\left(1 - \frac{335.66}{1755}\right)} = .247 \rightarrow \text{use } 1.0$$

$M_r = (1.0)(76) + (1.3)(50.4) = 135.52$

$$\frac{P_r}{\phi P_n} = \frac{335.46}{667^k} = .503 \rightarrow \text{use eq (H1-1a)}$$


.503 + (8/9)(135.5/473) = .758 < 1.0
ok for design
check w/ computer frame analysis
W14X74