4/7/2010

SENIOR THESIS
FINAL REPORT

FAIRFIELD INN & SUITES, MARRIOT
PITTSBURGH, PA

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Structural Option

FAIRFIELD INN & SUITES

Project Information

Location: 228 Federal St Pittsburgh, PA

Occupancy: Hotel

Total Size: 80,000 SF

Stories: 10 stories + 1 below grade

Project Delivery: Design-Bid

Project Cost: Unavailable

Project Team

Owner: Federal St. Hospitality Assoc.

Developer: Krasta Properties

Architect: Burt Hill

General Contractor: Massaro Corp.

Structural: Atlantic Engineering

MEP : Allen & Shariff

Landscape /Civil: Burt Hill

Architecture

- 135 guest rooms hotel with indoor pool and fitness center
- 18'-0" ceiling heights in lobby
- Lobby windows shaded by 19'x10' steel supported, tempered glass canopies
- Exterior façade cast stone up to 4th level then brick veneer continues to roof level
- Glass curtain wall system extends above lobby
- Roof composed of 8" concrete plank structural slab, TPO roofing membrane, tapered insulation

Structural

- Concrete floor and wall system allowing for the lobby ceiling height and providing an acoustic barrier in guest rooms
- 12" thick mat concrete slab
- 8" untopped precast concrete plank floors
- Concrete masonry bearing walls
- Lateral resistance provided by concrete masonry shear walls
- Auger cast concrete pilecaps and grade bream foundation
- Transfer beams ranging in sizes from W33x118 to W40x149

Mechanical/Electrical

- 1600 cfm air handling unit in basement
- 4200lb natural gas rooftop unit with 3400 cfm airflow
- Pool dehumidifier removers 15 lbs/hr of moisture
- Automatic sprinkler system throughout entire building
- 208/120V 3 phase 4 wire and emergency electrical system
- Florescent and HID lighting with dimmer system

Amanda Smith
Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2010/

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ACKNOWLEDGEMENTS

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Lastly, a very special thank you to my parents and fiancé for all your continued love and support.

EXECUTIVE SUMMARY

The Fairfield Inn and Suites is a 10 story hotel located in downtown Pittsburgh, Pennsylvania. The building is approximately 80,000 square feet and reaches a height of 102' above grade with a typical floor to floor height of approximately 9'4".

The current site of the Fairfield Inn and Suites was chosen because it's adjacent location to PNC Park and close proximity to Heinz Field in Pittsburgh. For these reasons, the hotel was kept on the existing site. Upon investigation of the soil classification for the site, it was determined that the soil is classified Site Class D. This will significantly impact the base shear value of the building, due to the poor soil the foundation will rest on.

This final thesis study examined the implications related to redesigning the gravity and lateral systems of the Fairfield Inn and Suites. The current design of the building includes load bearing concrete masonry walls, transfer beams, and an auger cast pile foundation. The redesign completed in the structural depth study explored steel moment frames rather than the load bearing concrete masonry walls. This would eliminate the use of the transfer beams in the current design. The design also examined a modified layout in the shear walls that result in the lateral force resisting system of the building.

The steel gravity system resulted in a decrease to the overall building weight. Along with the decrease to the overall building weight, the construction time to erect the steel building structure was sufficiently lower than the concrete masonry bearing structure. The shorter construction time does sacrifice an increase in cost. Structurally, the redesign of the gravity system does prove to be an efficient option for the building. The decrease in building weight resulted in a reduced base shear value on the building. A lateral optimization study was included as part of the structural depth study to see if a modified shear wall layout would provide greater resistance to the loads. The modified layout proved to be the optimal design as it reduced the overall torsion present on the building and reduced the required number of piles in the foundation.

The façade breadth study focuses on improvements in guest comfort with respect to natural daylight penetration verse heat transfer through the wall system. By implementing the brick veneer system, the heat transfer through the wall would not be affected, as opposed to using the larger curtain wall system façade option which would increase the heat transfer but allow for more natural daylight. A lower heat transfer rate façade proves to me a more efficient system for the building.

The goals of this thesis were to create an efficient optional gravity and lateral system for the building. Based on the results discussed, these goals are clearly met. If cost was not an issue, it is the recommendation of the author to implement the changes proposed, as each study does impact the building in a positive way.

BUILDING OVERVIEW

Function

Fairfield Inn and Suites is a ten story hotel that provides a nice, convenient place to stay for visitors to Pittsburgh. The hotel is located in the heart of Pittsburgh within walking distance to Heinz Field (football stadium), the new Rivers casino, downtown Pittsburgh, plus many other Pittsburgh attractions. The hotel's closest attraction, directly across the street, is the Pittsburgh Pirates baseball stadium, PNC Park. Being in such a prime location, this hotel with accommodate thousands of guests visiting the area throughout the year making it an essential addition to the community.

Architecture

The hotel occupies 135 guest rooms in addition to an indoor pool and fitness center for its guests. There will be a variety of typical king/queen size rooms to king/queen suites to satisfy the needs of all guests. Guests to the hotel will enter into an 18' lobby off of Federal St. where the main entrance exists. The lobby consists of a large reception desk for check-in/out, a breakfast area, and a large seating area featuring a cherry finished wood fireplace. The hotel holds a basement below grade that consists of the electrical, mechanical, and maintenance rooms, along with the laundry room and break room for employees.

The façade of the building is similar for all views (north, south, east and west). The exterior walls are all composed of concrete masonry blocks. Cast-stone veneer against the CMU block decorates the exterior façade of the building from the first level to the top of level three. Brick veneer than extends from level four to the roof of the building against the CMU to decorate the rest of the building's facade. On the north façade, there are two 56' x 18' bond faced brick detailed rectangles accenting this view of the building from the highway.

As one approaches the 18' lobby entrance, large glass windows and doors greet them, opening up the lobby area. The windows and doors lining the front of the building along the lobby (west façade) are part of a glass curtain wall system. In addition, a spandrel glass curtain wall then extends two stories above the lobby entrance adding verticality to the building. The lobby entrance is emphasized by a 19'x10' steel supported, tempered glass awning shading the curtain wall. The remaining curtain wall along the street level is shaded by additional glass awnings. Windows throughout the rest of the building, for the hotel rooms, are all aluminum window systems. A metal louver and cast stone sill line the bottom of each hotel room window.

At the top of the west and south building façades there are attached illuminated “Fairfield Inn and Suites” signs identifying the hotel. The north façade, which faces the highway, is a larger illuminated sign with lighting fixtures in the bond face brick detail on that façade.

Construction Management

The construction of the Fairfield Inn and Suites started in late October 2008 and is set to be completed in the spring of 2010. The general contractor is Massaro Corporation and it is a design-bid project.

Being a design-bid project, the design phase of the project overlaps with the construction phase of the project making it difficult to get an exact cost of the building. An estimated building cost, as of December 2009, was $19 million. Please refer to Figure 1.1 below to view a site plan of the Fairfield Inn and Suites.

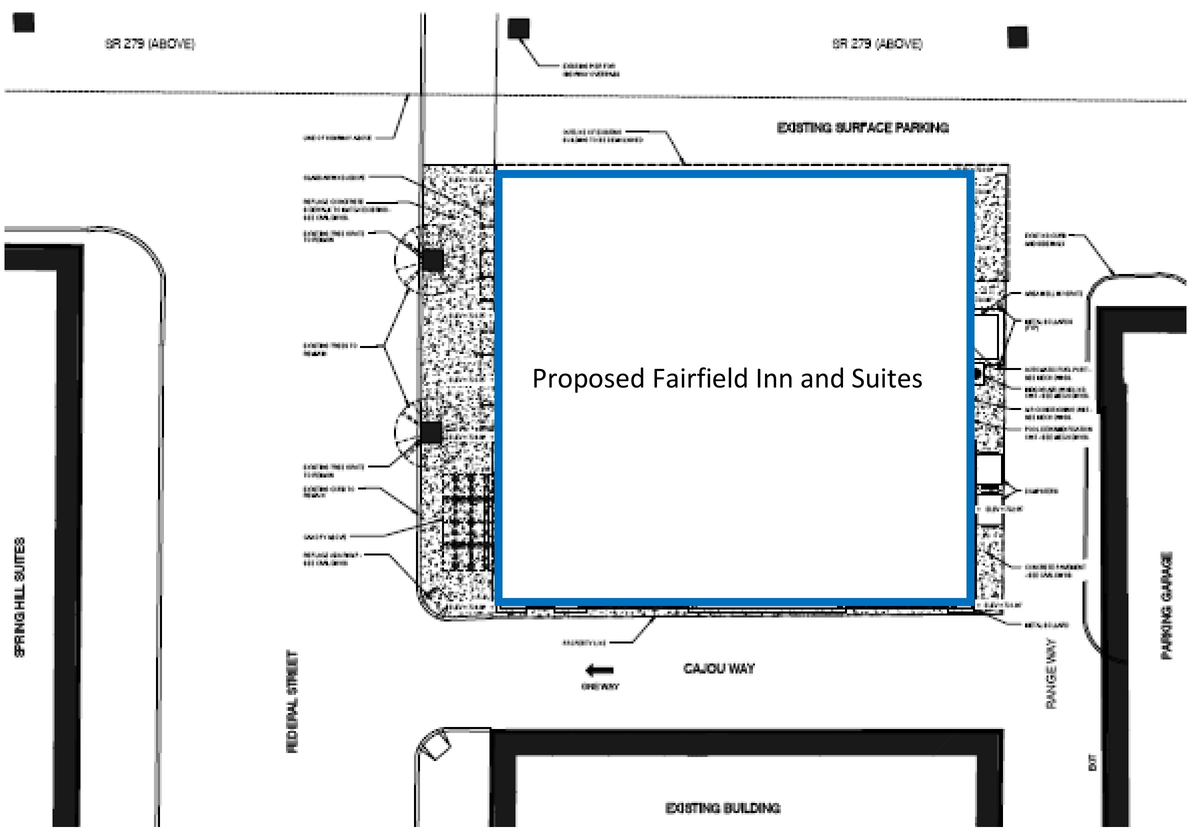


Figure 1.1: Fairfield Inn and Suites Site Plan

Mechanical System

The Fairfield Inn and Suites' mechanical system is designed for multiple areas of the hotel. An indoor air handling unit placed in the basement of the hotel services the heating and cooling of the 1st floor and corridors of the hotel. The air handling unit has airflow of 1600 cfm. Each guest room is equipped with their own mini A/C units with airflows of 530 cfm. Boilers are also placed in the basement to serve the purpose of heating the remainder of the building. The mechanical room also houses an indoor air cooling chiller and outdoor condenser to service the building.

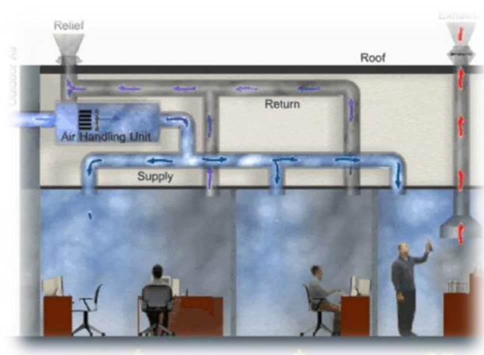


Figure 1.2: Typical Air Handling Unit process

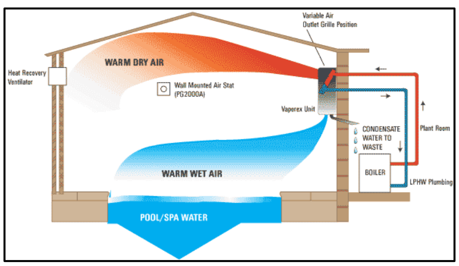


Figure 1.3: Typical Pool Dehumidifier Process

The roof of the hotel holds a 4200 lb natural gas rooftop unit with airflow of 3400 cfm. The mechanical system does not place any significant weight on any of the other floors in the building. With an indoor luxurious pool for its guests located on the first floor, the building mechanical system also incorporates a pool dehumidifier on the first floor that removes 15lb/hr of moisture from the air and airflow of 1620 cfm.

Lighting & Electrical System

The electrical service to the building is a 208/120V 3 phase 4 wire system and an emergency backup electrical system. The common distribution switch board is a 208/120V 3 phase 4 wire system with a 250 Amp bus. Each story of the hotel is supplied with a 3 panel switch board located in the electrical room on each floor. There are also six additional panel boards throughout the building that supply the mechanical rooms, pool, fitness center, and elevators.

The lighting system used throughout the building is mainly comprised of florescent and HID lighting; recessed, pendant, and industrial strip fixtures. There is also a dimming system incorporated in the lighting systems of the building to allow guests to change the brightness of their hotel room lights by wall switches. The dimming system will ultimately allow the hotel to save money if used properly by guests.

EXISTING STRUCTURAL SYSTEM

Foundation

A geotechnical soils report was conducted for the Fairfield Inn and Suites site on November 27, 2007 by Construction Engineering Consultants. In the study, it was found that the typical soil found on site is brown silt, clay, and sand. The reported water level was approximately 25'-0" on site. The depth of the basement is 12'-8" below grade, therefore there shouldn't be a concern regarding the uplift pressures on the foundation due to the water level. Due to the moderate depth to bedrock and precaution taken in regards to water level, the deep foundation system consists of auger cast friction piles and grade beams. With the foundation not extending below 33 ft., the net allowable bearing pressure on site is 200 psf.

The ground floor rests on a 6" concrete slab which is 5 ksi normal weight concrete (NWC). The slab increases in thickness from 6" to 12" within the core shear walls where the elevator pit and stair wells are located. The slab reinforcement consists of W/ 6x6-W1.2xW1.2 welded wire fabric and #5 bars located 12" o.c. top and bottom and each way. The slab depth is approximately 12'-0" below grade, while the elevator pit extends to 17'-5" below grade.

The auger cast piles extend beneath a pilecap topping below the slab and are spaced approximately between 26' to 31' apart. The typical size of the foundation pilecaps are a 7'-6" square approximately 4' deep with four 16" diameter auger cast piles per cap. The core shear walls incasing the stairs and elevator have additional rectangular pilecaps and piles for more support. Pilecaps are reinforced with #8 bars at 6" o.c. The typical column piers extending from the pilecaps are concrete 24"x24" piers with horizontal ties and vertical bar reinforcement that support each column. (See Figure 2.1)

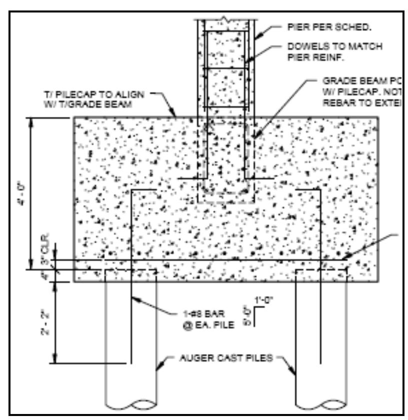


Figure 2.1: Typical Detail thru Pilecap

Grade beams run between pilecaps transferring the loads from the façade of the building and interior shear walls to the piles. (Refer to Figure 2.2). The depth of the beams range between 36" and 48" depending on location in foundation. Reinforcement and sizes vary per grade beam.

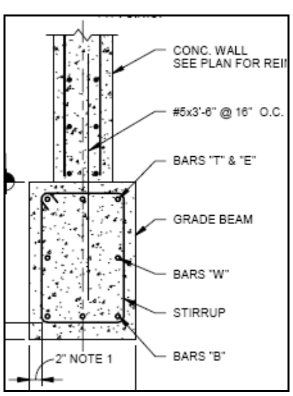


Figure 2.2: Typical Grade Beam Detail

Floor System

Fairfield Inn and Suites typical floor system is a precast concrete plank floor with a thickness of 8” untopped. The hollow core concrete plank floor allows for the building to be supported without the use of columns or beams on floors two thru ten and longer spans. Concrete compressive strength for the floors is f’c=5000 psi. The typical spans of the precast plank floor are 31’-0” and 26’-0”. The planks are supported by load bearing concrete masonry walls.

The floor system for the first floor is a combination between 4” slab on grade and the 8” precast concrete plank floor. There is no basement below the first floor running along the south wall and the lobby entrance on the west wall of the building (see Figure 2.3). Due to a pool being located in this area, the hollow core plank floor would not be sufficient in supporting the weight of the pool and lobby live loads. Therefore, the floor system is a 4” slab on grade with W/6x6-W1.4xW1.4 weld wire fabric reinforcement.

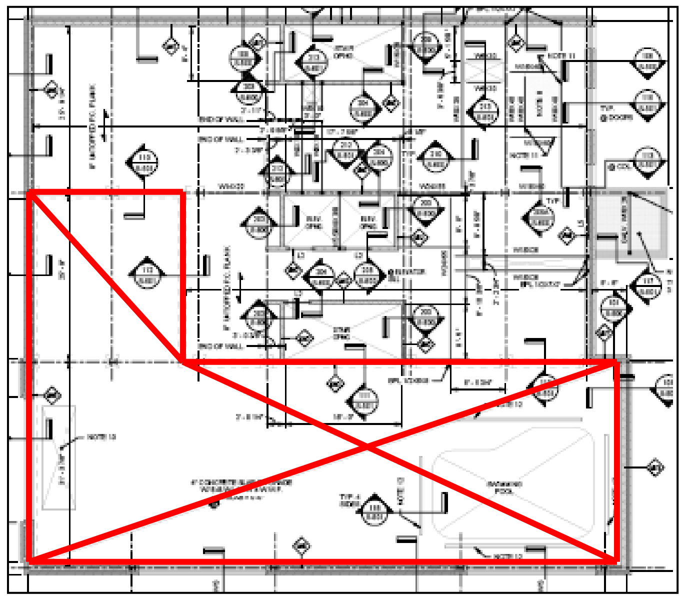


Figure 2.3: Partial First Floor Slab

Since the floor system is a precast plank floor, there are a limited number of steel beams and girders throughout the structure. With no columns to support floors two thru ten, the majority of the beams present are transfer beams on the second floor that transfer loads from the load bearing walls that support the planks on the floors above. The transfer beams transfer those loads to the columns extending from the pilecaps and thus transferring all loads to the foundation system. The transfer beams run along the north wall of the elevator shafts from the west wall to the east wall, and along the south wall of stair B extending from the west wall to the east wall (see Figure 2.4). Transfer beams range in size from W 33x118 to W 40x149. Girders run along the first floor supporting mechanical equipment loads and tying into the beams and shear walls supporting the first floor. Girders and beams throughout the building are non-composite systems.

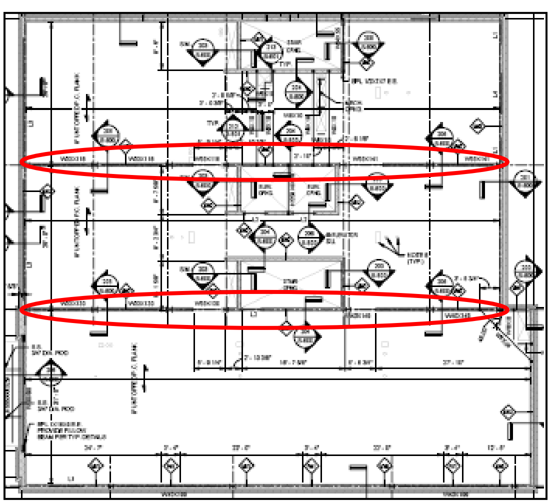


Figure 2.4: Second Floor Transfer Beams

Roof System

The roof system and smaller high roof system use the same 8" untopped precast concrete plank systems. At two locations on the roof, the plank floor is modified to support 150 psf of weight for a mechanical unit. W8x28 and W8x18 beams run along the top of the shear walls enclosing the elevator and stairwell shafts on the roof. Hoist beams support the top of the elevator shaft in the high roof system. There are a total of six drains located on the roof for the drainage system.

Columns

The only columns used in the Fairfield Inn and Suites are the ones extending from the pilecaps and concrete piers to the second floor supporting the 18' first floor. The columns connect into the transfer beams and distribute the loads. The columns range in size from W10x100's to W 12x120's depending on location. All columns connect into a pilecap or concrete pier, where the weight on each column transfers the load down to the foundation (refer to Figure 2.5). The base plates range from ½" – 1" thick and typically 14"x14". Each plate utilizes a standard 4 bolt connection using 1" A325 bolts.

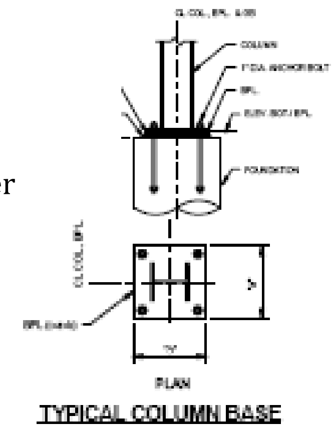


Figure 2.5: Typical Column Base Detail

Lateral System

The lateral system for the Fairfield Inn and Suites is a combination of ordinary reinforced concrete masonry shear walls. The exterior shear walls are 10" concrete masonry block and the core shear walls are 8" concrete masonry block. The core shear walls surround the staircases and elevator shaft. On floors two thru ten, two additional load bearing masonry walls extend from the west wall to the east wall running along the south wall of staircase B and the north wall of the elevator shafts (see Figure 2.6).

Shear walls supporting the ground floor to the fourth floor support a compressive strength of f'c=8000 psi. All other shear walls support a compressive strength of f'c=5000 psi. The typical vertical reinforcement in both the 10" and 8" shear walls is #5 bars at 16" o.c., 24" o.c., or 32" o.c. with bars centered in cells and solid grout. The plank floor system rests on the load bearing shear walls.

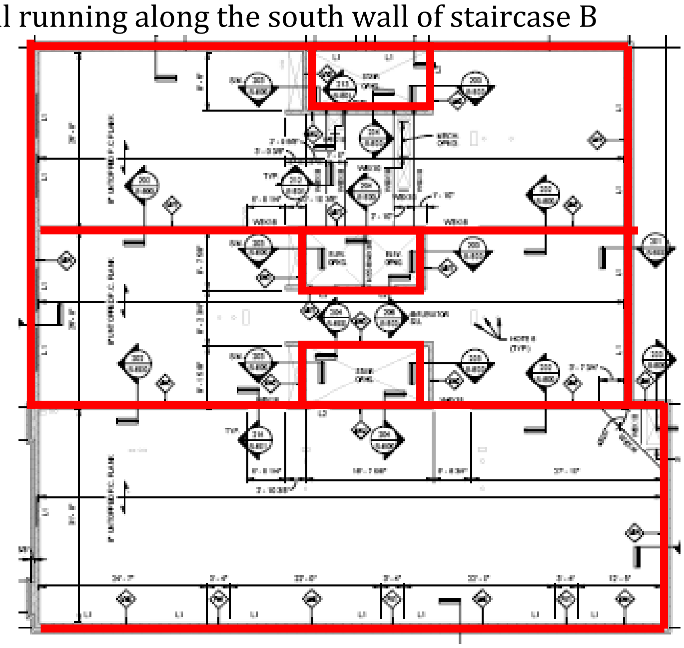


Figure 2.6: Shear Wall layout

Detailed connections of the plank to shear walls can be found in Figure 2.7 and 2.8.

The wind and seismic loads, as well as gravity loads, reach the foundation by first traveling through the rigid building diaphragm (floor system) to the shear walls and transfer beams.

From there all loads travel through the columns into the grade beams and auger cast pile foundation. This load path is governed by the concept of relative stiffness.

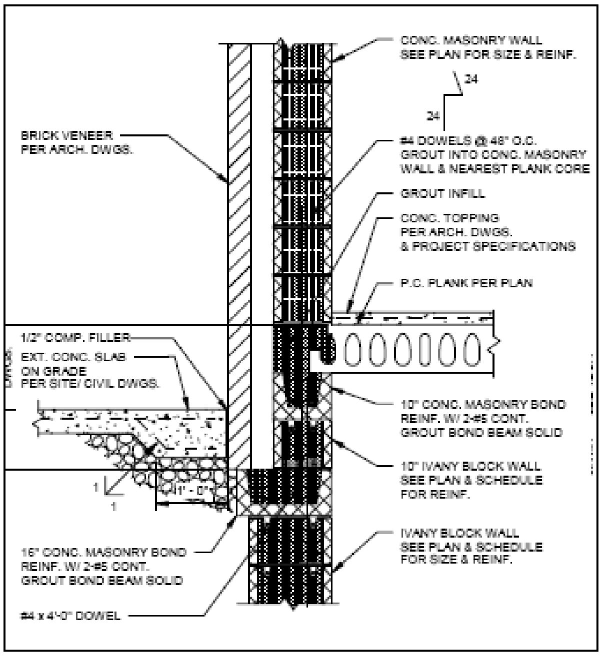


Figure 2.7: Typical Exterior CMU wall connection to precast plank floor

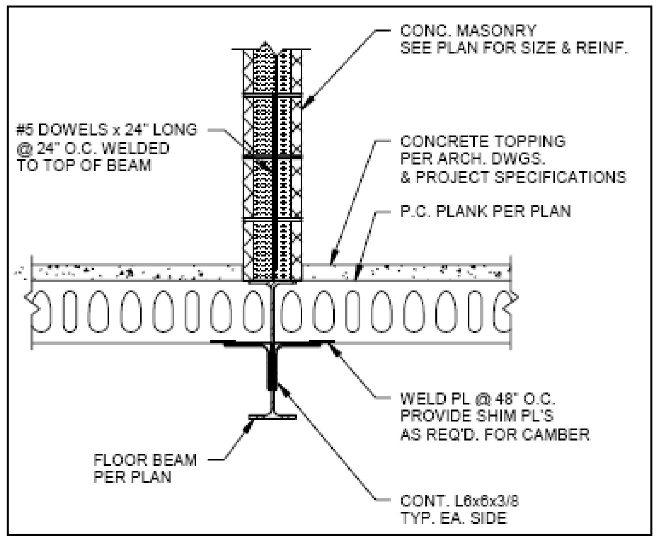


Figure 2.8: Typical precast plank floor connection to steel beam and interior CMU wall

ARCHITECTURAL & STRUCTURAL PLANS

The following figures are provided for side by side reference of architectural function and structural framing for each floor of the Fairfield Inn and Suites.

Columns   Beams

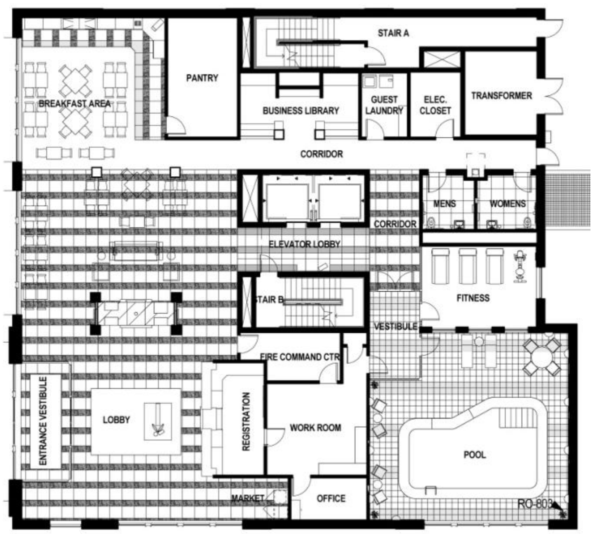


Figure 3.1a: First Floor Architectural Plan

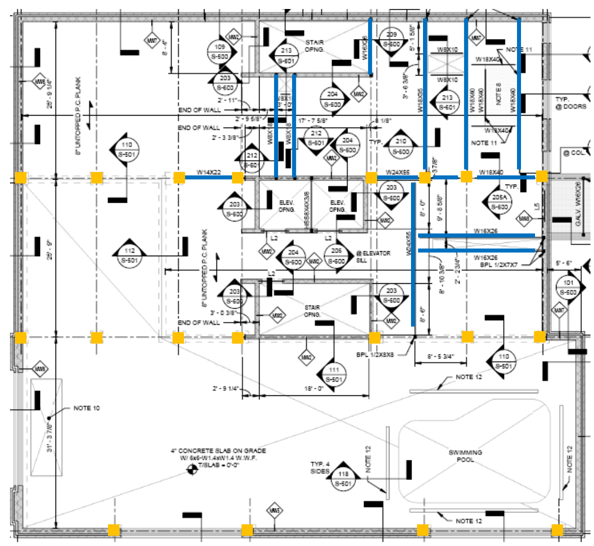


Figure 3.1b: First Floor Framing Plan

Figure 3.2a: Second Floor Architectural Plan

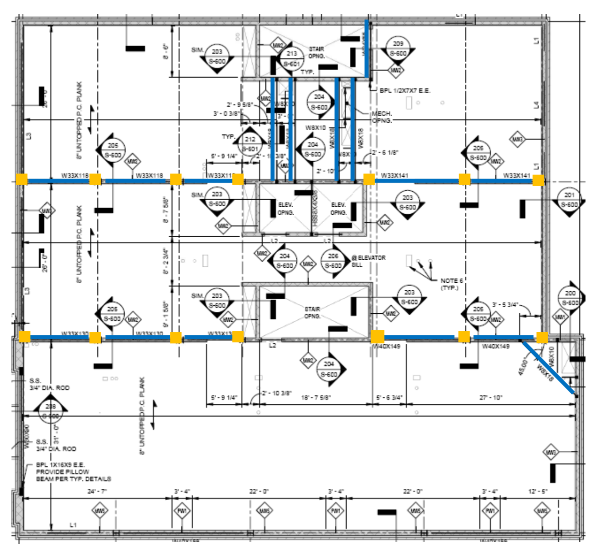


Figure 3.2b: Second Floor Framing Plan

Columns Beams

Figure 3.3a: Third thru Tenth Floor Architectural Plans

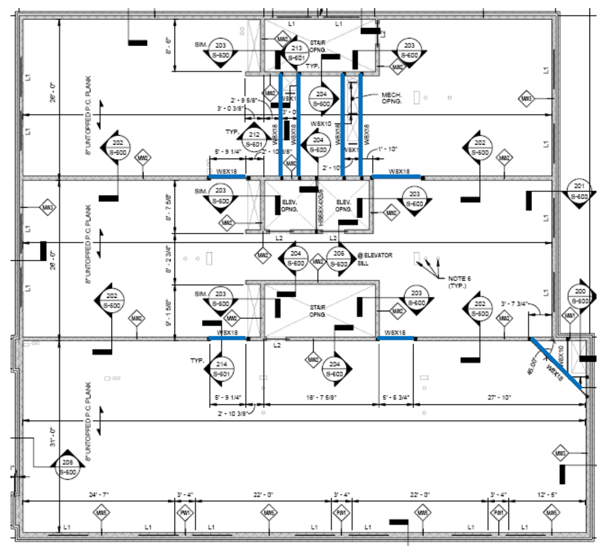


Figure 3.3b: Third thru Tenth Floor Structural Plans

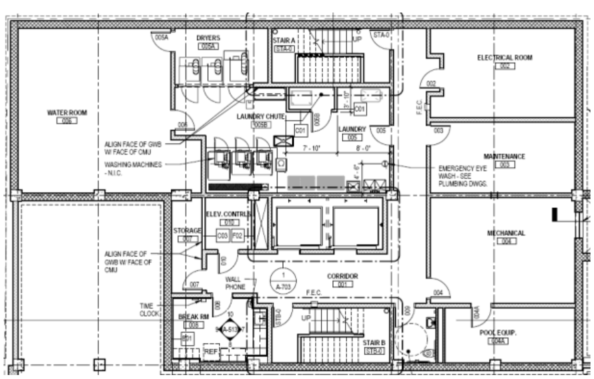


Figure 3.4a: Basement Architectural Plans

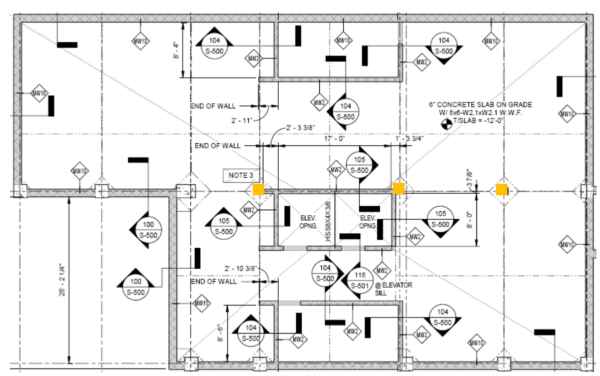


Figure 3.4b: Basement Structural Plans

PROPOSAL BACKGROUND AND PROJECT GOALS

Problem Statement

The nature of this site for the Fairfield Inn and Suites had an impact on the structural design of the building. Based on the field and laboratory test data within the geotechnical report for the site, it was determined that the soil on site is poor and classified as soil class D. This significantly impacted the base shear value, leading the building to be seismically controlled even when torsion effects were considered. The auger cast piles design of the foundation system for the building was designed to best fit this criterion and support the building in this soil class. The possibility of increased loads to the building and other implications resulting from the implementation of any new system, will require checks to be done at the foundation to verify it is sufficient to withstand changes or whether or not alterations must be made if at all feasible.

The existing structural design of the building meets all required design requirements as per ASCE 07 and any code requirements concerning restrictions due to location or zoning. Therefore, when considering an alternative design to this building, the final decision may not prove to be more effective compared to the existing design. However, a further investigation of other options to the building design should be considered.

With the number of load bearing concrete masonry walls making up the building, it results in a very high overall building weight that must be supported in such a poor soil site. For such a poor soil class, a lighter building weight would be suggested for the design to enhance its supporting foundation. A redesign of the structural system of the existing Fairfield Inn and Suites will be designed in an attempt to find an equally effective and efficient building system. To determine whether a different system is equally efficient and effective, it will be compared to the existing system in a number of categories.

Proposed Solution

Due to the nature of the soil, steel may be the best viable solution for the design of the structural system. Concrete is a heavier material by nature, therefore the steel could only decrease the weight of the building, creating a lighter base shear value. As a result, a viable alternative structural system for the Fairfield Inn and Suites is altering the framing system to a steel frame. This consequently will affect the foundation and construction management issues like schedule and cost. The architecture of the building could also be impacted without the exterior shear walls present. In addition, since the controlling lateral load case is seismic, changing the building frame to steel may reduce those loads due to stiffness.

With a steel framing system, an alteration to the lateral force resisting system and gravity resisting system will be considered. The current hollow core plank floor system will remain as it proved to be the most efficient floor system solution. The plank floor system will sit on steel non-composite girders rather than the existing load bearing masonry walls. The majority of the load bearing masonry walls will be eliminated from the structural design. The shear walls will only remain in the core of the building surrounding the staircases and elevator shaft. The core shear walls will now be what make up the lateral force resisting system. The shear walls will be redesigned in order to withstand the lateral loads. An optimization study will be performed to verify the new design. Steel moment frames will be designed to resist the gravity loads placed on the building.

All relative structural elements of the building will have to be considered throughout this alternate design. Since the redesign incorporates a different primary material for the building, steel, the existing columns and transfer beams will be altered. The floor spans and location of the floor framing members will remain unchanged. The location of the interior load bearing masonry walls will be replaced by steel columns and non-composite moment frames. Instead of steel columns extending from the auger cast piles to the second floor transfer beams, they will now extend to the roof of the building. This will ultimately eliminate the use of transfer beams at the second floor. Finally, an analysis will be done to the foundation to ensure the new lateral loads and building weight can be resisted by the auger cast piles or if a redesign is required. The purpose of making these alterations to the structure is simply to investigate the overall affects they have on the project, whether the results are positive or negative.

Project Goals

1. Reduce the weight of the overall building by optimizing the gravity system
2. Optimize the lateral force resisting system, in coordination with the gravity system
3. Verify the impact on the foundation system
4. Research façade options available to the building design
5. Determine the impact an altered design has on the construction schedule and cost

GRAVITY SYSTEM REDESIGN

This following section discusses the redesign and analysis process of the gravity system. As discussed in the proposal it was decided to explore the use of steel as a framing material to resist gravity loads as opposed to concrete.

Design Loads & Criteria

In order to redesign the gravity system of the Fairfield Inn and Suites, the gravity loads applied on the building were determined. The gravity loads were determined in accordance with American Society of Civil Engineers (ASCE) 7-05. The design criteria for the gravity design of the building can be found below. A summary of the gravity loads used in the redesign of the structural system can be found in Table 1. The table clearly defines the loads that were used in the redesign analysis, as well as used by the design engineer. These loads are included to emphasize that the loading specific to the analysis and design of thesis course work varies in comparison to the loads used for the original design.

Table 1 - Gravity Loads

Description	ASCE 7-05	Design Engineer	Design Value
Dead Loads (DL)			
Concrete	150 pcf	150 pcf	150 pcf
Plank	84 pcf	84 pcf	84 pcf
Steel	490 pcf	490 pcf	490 pcf
Roof	20 psf	20 psf	20 psf
Live Loads (LL)			
Public Areas	100 psf	100 psf	100 psf
Lobbies	100 psf	100 psf	100 psf
First Floor Corridors	100 psf	100 psf	100 psf
Corridors above First Floor	80 psf	80 psf	80 psf
Hotel Rooms	40 psf	40 psf	40 psf
Stairs	100 psf	100 psf	100 psf
Roof	20 psf	75 psf	75 psf
Mechanical	150 psf	150 psf	150 psf
Superimposed Dead Loads (SDL)			
Partitions	15 psf	15 psf	15 psf
MEP, Finishes, Misc	10 psf	15 psf	10 psf
Snow Loads (S)			
Snow	25 psf	30 psf	30 psf

Strength Design Criteria: ASCE 7-05 LRFD Load Combinations

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S)
3. 1.2D + 1.6L_r + 0.5L

The controlling load combination is 1.2D + 1.6L + 0.5L_r. The loads produced by this case were used in designing the steel moment frame member sizes.

Serviceability Criteria: Deflection

Non-Composite:

Dead Load ………………………….l/360

Live Load …………………………..l/360

Total Load ………………………….l/240

Economy Criteria: Camber

Beams that do NOT camber: Beams that are less than 25ft
Beams that requires less that ¾" of camber
Beams in braced frames

Design Process

Framing Plan

The structural redesign of the gravity system began with determining an initial framing plan. It was possible to use the existing column locations for all interior and exterior columns, as well as adding additional columns around the perimeter. The main effects of the redesign will be that the columns in the building no longer only extend to the second floor, but now extend the entire height of the building.

The plank floor system will still exist and now rest on steel girders rather than the load bearing walls. Girders will now connect the columns making up the moment frames and the perimeter of the building. The location of the interior moment frames was chosen because this is where the interior load bearing masonry walls existed. A steel framing system creates a thicker floor depth than having the 8” plank floor rest on load bearing walls. This would affect the floor-to-ceiling height and ultimately result in a higher overall building height. Even though the location of the building site would allow for the additional height, placing the moment frames along the line of where the interior load bearing walls existed than the floor-to-ceiling height would not be altered, and the overall building height would remain the same.

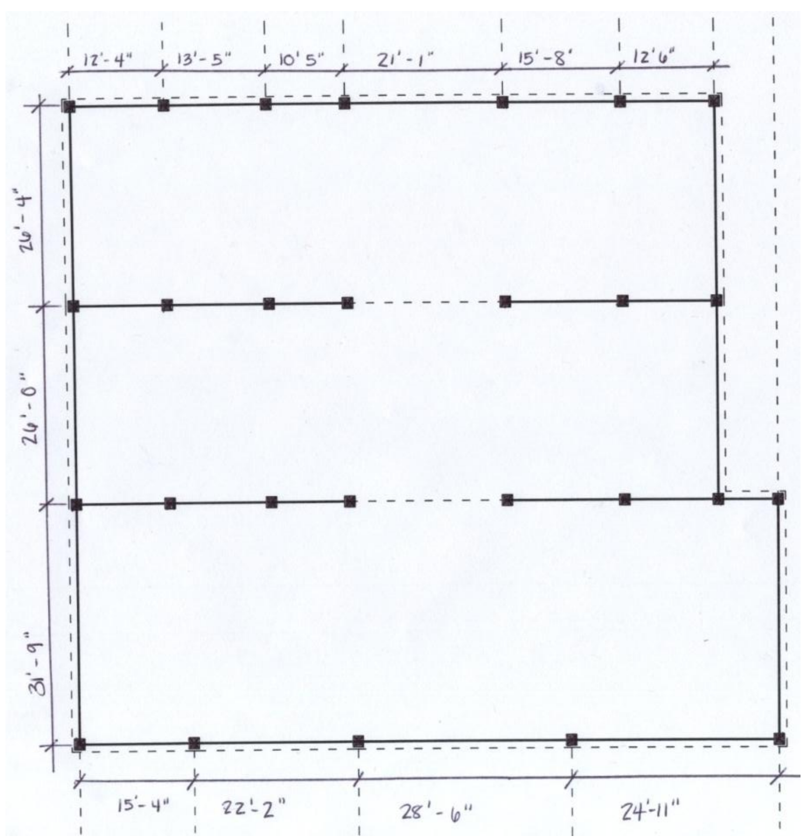


Figure 4.1: Schematic Framing Plan

Hollow-core Plank Design

The next step in the design process was determining if any alteration would need to be made to the existing plank floor. A topped plank floor system was looked at to determine if feasible for the building to create more fireproofing in the building since a steel frame would now be utilized. To keep an 8" slab depth, a 6" + (2") topped plank floor would not be able to support the loads for a plank span of 31' as required. In this case, if we wanted to add a topping to the plank floor, it would need to be an 8" + (2") topped floor and this would increase the slab depth resulting in a reduced floor to ceiling height. In the redesign of the gravity system, altering the building height and floor-to-ceiling heights was avoided, therefore keeping the existing 8" untopped plank floor design. Hollow-core plank design specifications are summarized in Figure 4.2 below. Hand calculations can be found in Appendix A.

Reference from the PCI Design Handbook/Sixth Edition

Strand Pattern Designation
76-S

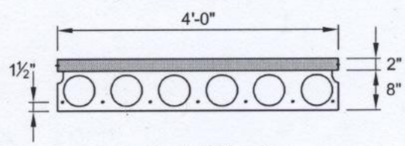
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped	Topped
A =	215 in.^{2}	311 in.^{2}
I =	1,666 in.^{4}	3,071 in.^{4}
y_b =	4.00 in.	5.29 in.
y_t =	4.00 in.	4.71 in.
S_b =	417 in.^{3}	581 in.^{3}
S_t =	417 in.^{3}	652 in.^{3}
wt =	224 plf	324 plf
DL =	56 psf	81 psf
V/S =	1.92 in.	

4HC8

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

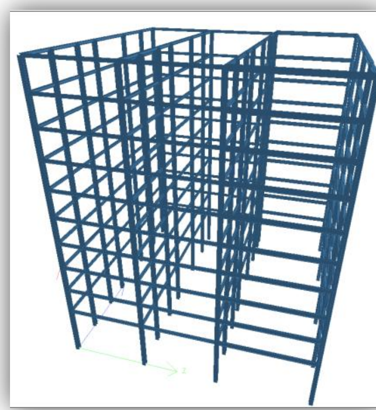
Strand Designation Code	Span, ft 11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
66-S	458	415	378	346	311	269	234	204	179	158	140	124	110	98	87
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1
76-S	470	424	387	355	326	303	276	242	213	188	167	149	133	119	106
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3
58-S	464	421	384	352	323	300	280	260	244	229	211	194	177	160	144
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3

Strand Designation Code	Span, ft 26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	77	69	61	54	48	43	38	33	29						
	0.2	0.1	0.0	0.0	–0.1	–0.2	–0.3	–0.5	–0.6						
	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.4						
76-S	95	86	77	69	62	55	50	44	39	35	31	26			
	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9			
	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.6	–0.8	–1.1	–1.4	–1.7	–2.0			
58-S	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.4	–0.3	–0.5	–0.7	–0.9
	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4
68-S	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3
	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	–0.2	–0.5	–0.8	–1.1	–1.5
78-S	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	–0.3	–0.7

Figure 4.2: Hollow-core Plank Design Specifications

Beam and Girder Design

With the plank floor system confirmed, the supporting steel framing members could be designed for the given loads. Beams and girders were designed in accordance with Load and Resistance Factor Design (LRFD) methods and the AISC Steel Construction Manual, 13th edition. In accordance with ASCE 7-05, loads were multiplied by a load factor combination that incorporated both the situations in which the loads would occur simultaneously at their maximum level and the margins against which failure if the structure is measured. Since the planks sit on the steel girders, there was no use for composite steel girders and non-composite steel members were designed to support the floor system. The system of hollow-core plank floors on structural steel frames is economical, easy to design, and fast to erect.



Staad.Pro was used as the primary computer analysis software for the framing design. A 3D structural model of the gravity system was constructed as a design aid to efficiently determine optimal member sizes. Staad was chosen for the steel moment frame design because it is known to be a reliable and user friendly design aid for steel structures. Designed girder sizes ranged from W12's to W14's. Member sizes obtained through Staad were spot checked with hand calculations for strength and serviceability criteria. In all cases optimal member sizes determined by hand calculations matched those determined through Staad. All calculations can be found in Appendix A.

Column Design

The columns were designed in accordance with LRFD methods and AISC Steel Construction Manual. Column design followed the same procedures as the girder design procedures. The columns designed resist gravity loads only. The columns are spliced every two to three stories. The main column splice occurs at the fifth level where there is a change in the optimal member size. For simplification of calculations, the optimal member sizes were determined for stories one thru five and stories six thru ten.

Optimal column sizes were designed through the use of Staad. All columns were designed to be W14's. Select column sizes obtained through Staad were spot checked with hand calculations. For hand calculations, column load take downs were performed to determine the loads each column must support. In all cases optimal column sizes determined by hand calculations matched those determined through Staad. All calculations can be found in Appendix A.

Gravity System Final Design

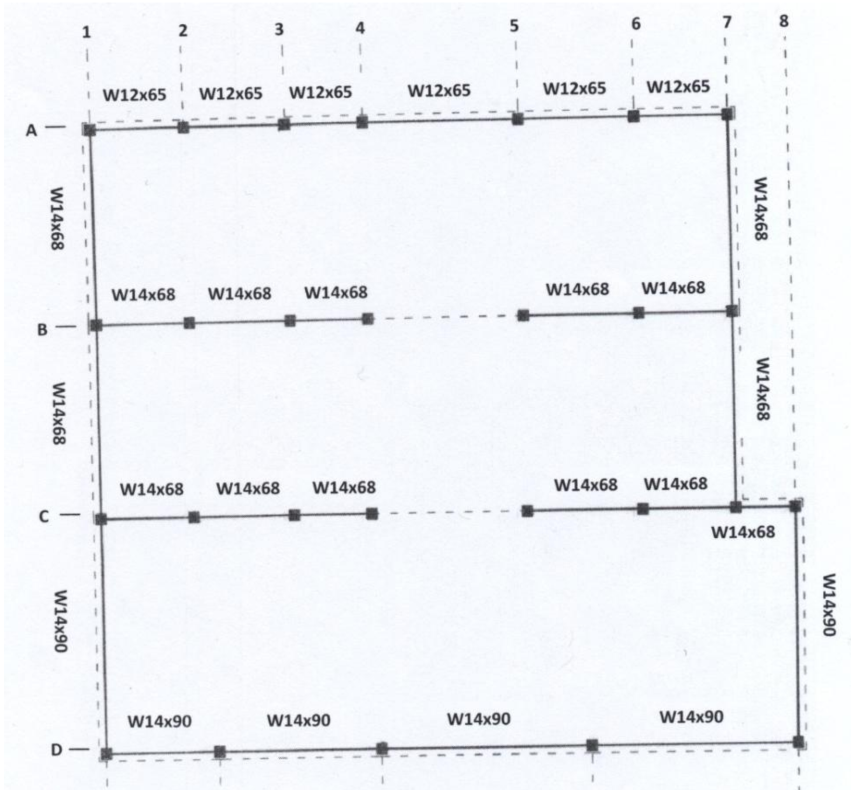


Figure 4.3: Typical Floor Beam/Girder Plan

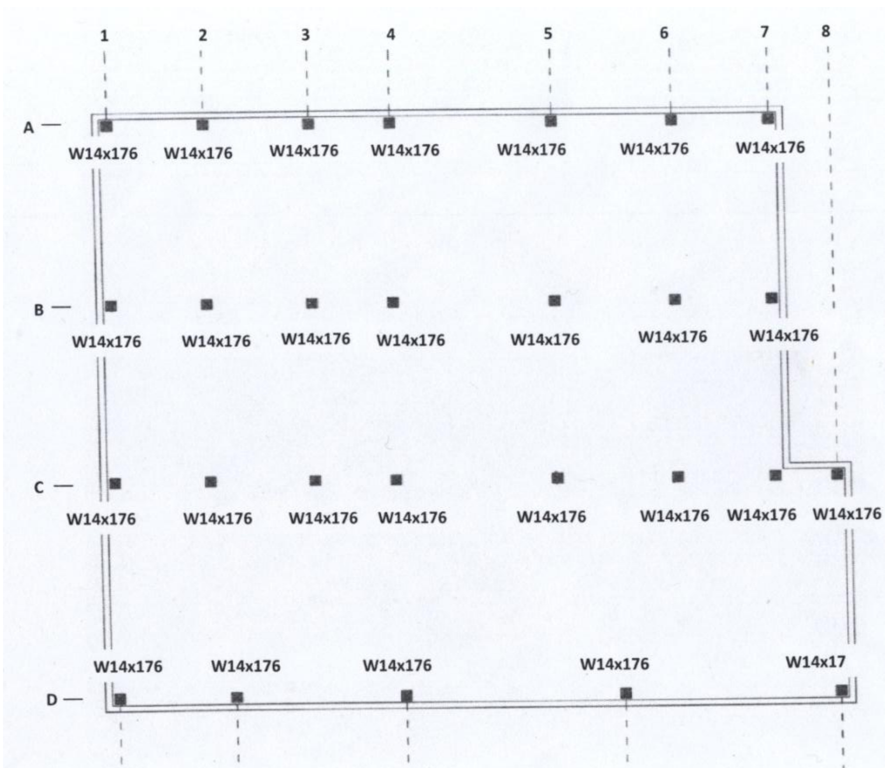


Figure 4.4: Column Layout for Floors 1-5

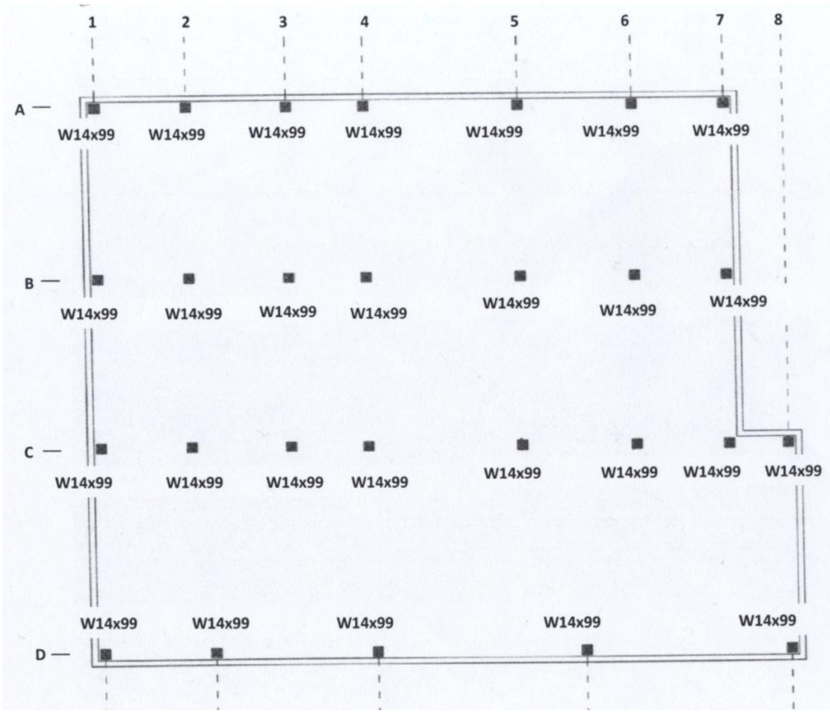


Figure 4.5: Column Layout for Floors 6-10

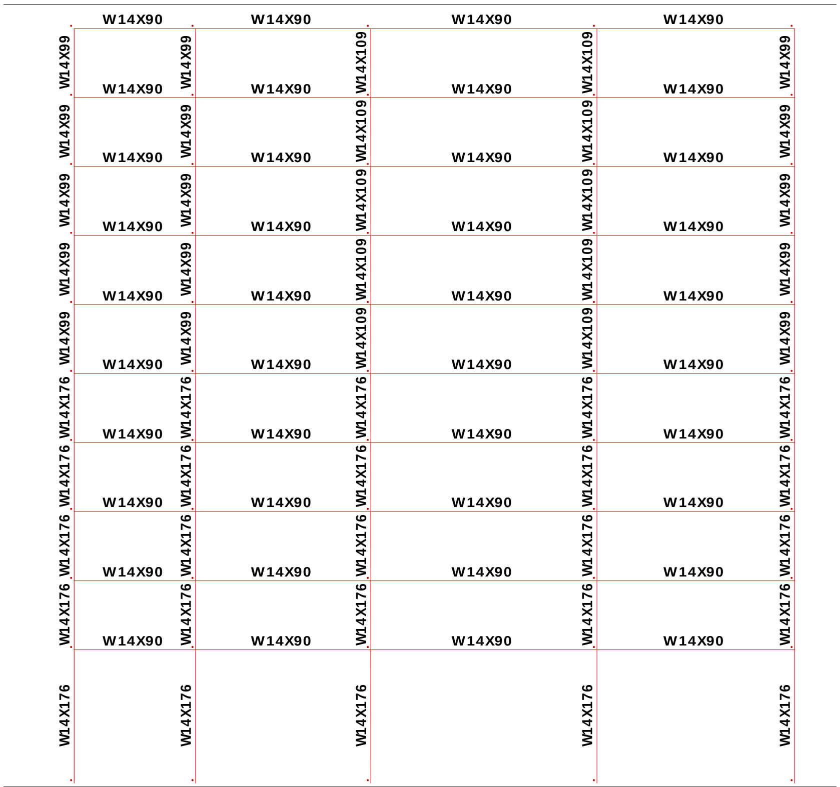


Figure 4.6: Column Line D Framing Elevation

Connection Design

The steel moment frames have an intermediate moment frame connection. A typical interior moment frame connection was designed for the framing system. Moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design. The moment connection is a beam to column flange connection and was designed as a 4 bolt unstiffened extended end plate connection. The extended end plate connection consists of a plate of length greater than the beam depth, perpendicular to the longitudinal axis of the supported beam. The connection details can be seen in Figure 4.7. The design calculations can be found in Appendix A.

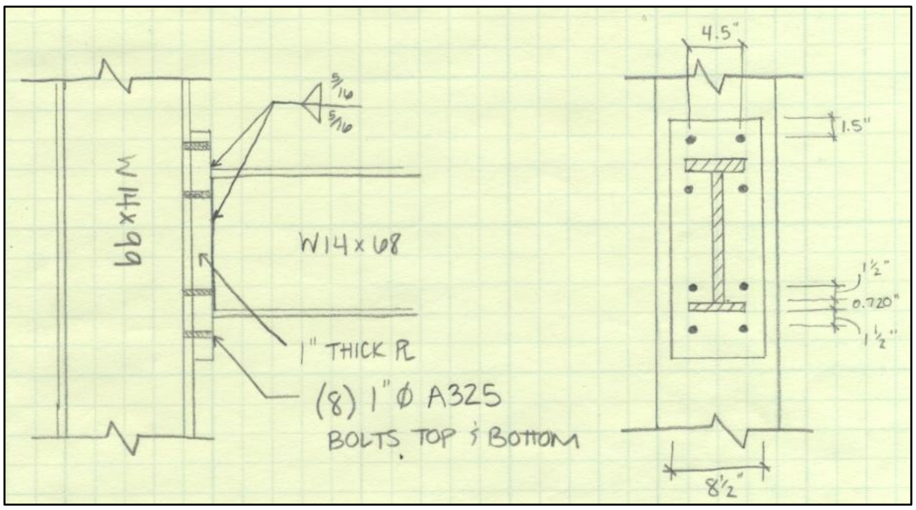


Figure 4.7: Moment Frame Connection

The exterior steel framing for the building will have sections of beam to column web connections. A typical exterior connection is designed to be an all bolted unstiffened seated connection for the system. For a beam to column web connection, the seated connection was chosen because it can simplify the erection process with ample erection clearance provided. A typical seat connection to column web can be seen in Figure 4.8. The design calculations can be found in Appendix A.

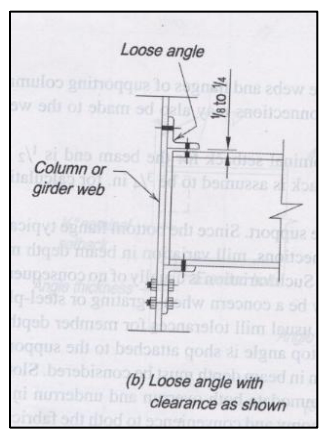


Figure 4.8: Typical Seat Connection Detail

At the bottom of each column in the building, a column base plate is attached to each column and attaches into the concrete column piers that extend up from the foundation pilecaps. The base plate is often attached to the bottom of the column

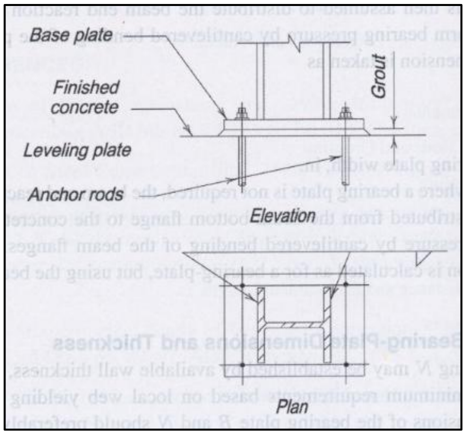


Figure 4.9: Typical Column Base Plate Detail

in the shop. A column base plate is made up of a plate with a minimum of four anchor rods. A typical base plate was designed for a W14x176 column in the building. The axial compression loads were determined through column load takedowns. The design resulted in a 1-1/2" thick plate by 24" by 24". A typical column base connection detail can be found in Figure 4.9. The design calculations can be found in Appendix A.

In the design of the steel columns of the building, it was economically advantageous to change the column size half way up the height of the building. The columns above the fifth floor do not need to hold as much weight as the lower columns, therefore a column change was necessary and two columns needed to be spliced together. The column splice occurred where the W14x176 column changed to a W14x99. A combination bolted and welded flange plated column splice was designed for this connection. The connection details can be found in Figure 4.10. The design calculations can be found in Appendix A.

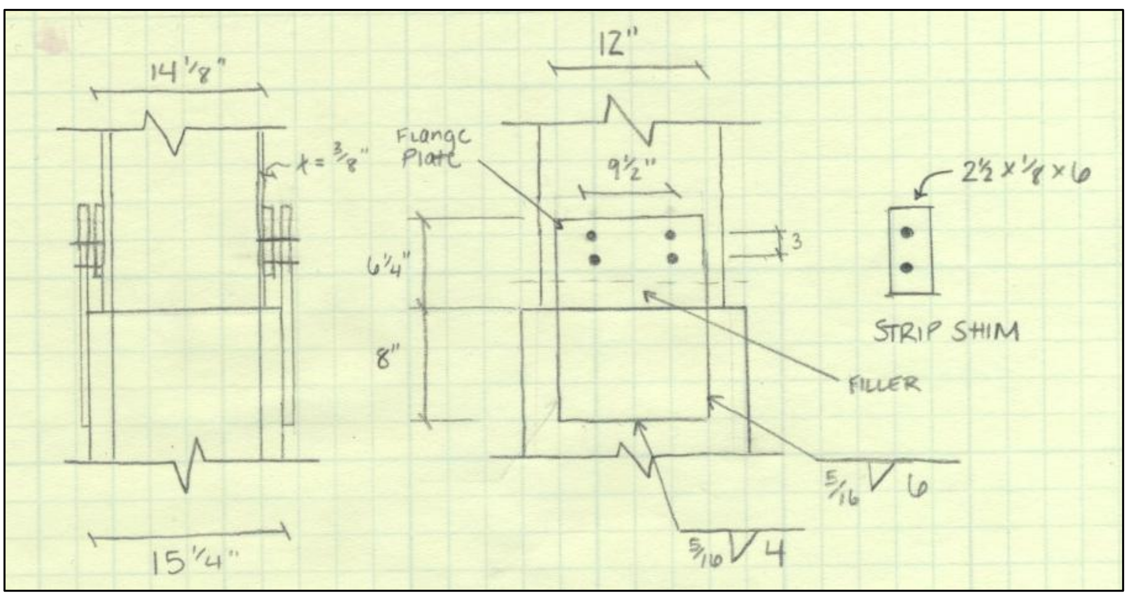


Figure 4.10: Column Splice Connection

In the gravity system design for the building, the planks sit on the top flange of the steel beams. The planks would be notched to fit around each of the columns. The plank to steel connection can vary from region to region, but for this design a typical plank to steel beam connection is used from the PCI Design Manual. The grout connection allows the system to transfer internal diaphragm forces and can provide lateral bracing for the steel beam. The connection details can be found in Figure 4.11.

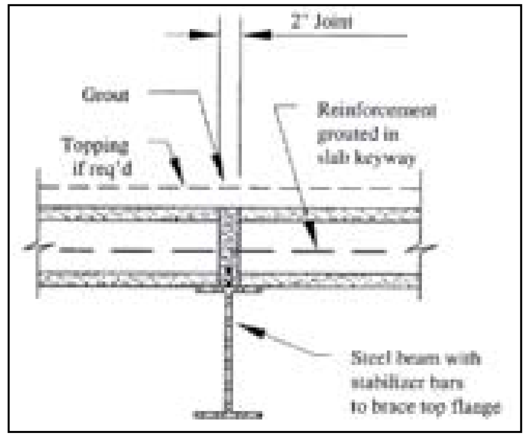


Figure 4.11: Plank to steel beam connection

Summary and Conclusion

The steel moment frames designed for the depth study met all structural requirements for resisting all gravity loads. Deflection was also determined and fell within the limits set forth by the code. Each member designed through Staad coordinates with hand calculations proving that optimal member sizes were designed. Steel proves to be a much lighter building material, which will ultimately improve the foundation system as it will not need to support so much weight. As you can see in Figure 4.12, the new load path for the gravity system will be through the floor diaphragm, into the beams the planks sit on, and distributed to the connecting columns that make up the steel moment frames then down to the foundation. Structurally, the steel gravity system proves to be an efficient optional design for the structural system of the Fairfield Inn and Suites.

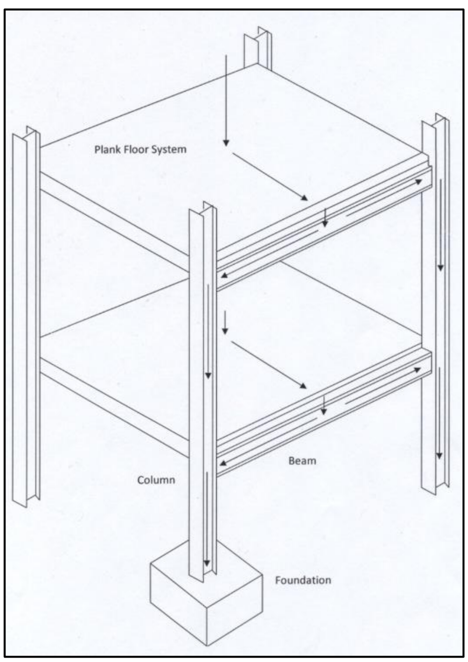


Figure 4.12: Gravity Load Path

LATERAL FORCE RESISTING SYSTEM REDESIGN

This following section discusses the redesign and analysis process of the lateral force resisting system. As discussed in the proposal it was decided to keep shear walls as the lateral system, but to reduce the number of walls used in the system compared to the original design.

Design Loads & Criteria

Wind Loads

Wind loads were calculated in accordance with ASCE 7-05, Chapter 6. To examine the wind loads in the North/South direction and the West/East direction, the Analytical Procedure – Method two described in Section 6.5, was used to find design pressures. The variables used in this analysis are located in Table 2A. Please refer to Appendix B for equations and base calculations used for the execution of this procedure.

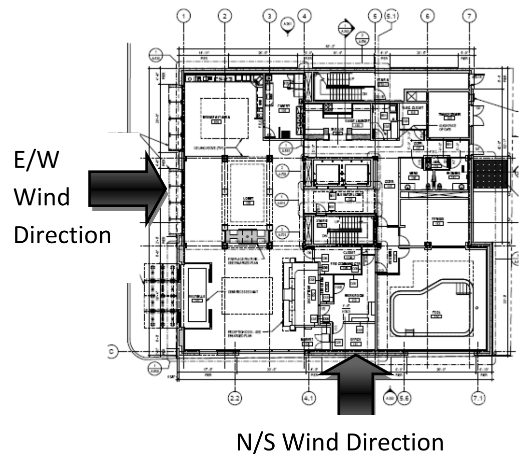


Figure 5.1: Wind Direction

Table 2A - Wind Variables			ASCE References
Basic Wind Speed	V	90	Fig. 6-1
Directionality Factor	K_d	0.85	Table 6-4
Importance Factor	I	1.15	Table 6-1
Exposure Category		C	§ 6.5.6.3
Topographic Factor	K_{zt}	1.00	§ 6.5.7.1
Velocity Pressure Exposure Coefficient evaluated at Height Z	K_z	Varies	Table 6-3
Velocity Pressure at Height z	q_z	Varies	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	20.47	Eq. 6-15
Equivalent Height of Structure	>	64.6'	Table 6-2
Intensity of Turbulence	$I_{\dot{z}}$	0.268	Eq. 6-5
Integral Length Scale of Turbulence	$L\dot{z}$	208.81	Eq. 6-7
Background Response Factor (East/West)	Q	0.792	Eq. 6-6
Background Response Factor (North/South)	Q	0.788	Eq. 6-6
Gust Effect Factor (East/West)	G	0.808	Eq. 6-4
Gust Effect Factor (North/South)	G	0.806	Eq. 6-4
External Pressure Coefficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coefficient (E/W Leeward)	C_p	-0.03	Fig. 6-6
External Pressure Coefficient (N/S Leeward)	C_p	-0.05	Fig. 6-6

Tables and calculations of wind pressures in each direction can be found in Appendix B as well. The most prevalent wind loads on site are the wind pressures in the North/South direction. This direction is adjacent to an existing building and a major highway, which neither structure is significant enough to block the building from receiving full wind loads. In the East/West direction, there are currently adjacent buildings blocking the wind on the lower levels on the hotel, but wind in this direction must be examined in the case that these buildings will not be present in the future and the full wind load will be applied to the building. Basic loading diagrams for wind forces in each direction are provided for reference in Appendix B.

Seismic Loads

An assumption was made in this seismic analysis that the Fairfield Inn and Suites employs a rigid diaphragm and therefore allows the use of the Equivalent Lateral Force procedure found in Chapters 11 and 12 of ASCE 7-05. Upon investigation of the geotechnical report, the Fairfield Inn and Suites falls under the Site D classification. The variables needed to calculate base shear according to ASCE 7-05 are located in Table 2B.

Table 2B - Seismic Design Variables			ASCE References
Site Class		D	Table 20.3-1
Occupancy Category		II	Table 1-1
Importance Factor		1.00	Table 11.5-1
Structural System		Ordinary reinforced masonry shear walls	Table 12.2-1
Spectral Response Acceleration, short	S_s	0.125	USGS
Spectral Response Acceleration, 1 s	S_1	0.049	USGS
Site Coefficient	F_a	1.6	Table 11.4-1
Site Coefficient	F_v	2.4	Table 11.4-2
MCE Spectral Response Acceleration, short	S_{ms}	0.2	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 s	S_{m1}	0.1176	Eq. 11.4-2
Design Spectral Acceleration, short	S_{ds}	0.133	Eq. 11.4-3
Design Spectral Acceleration,1 s	S_{d1}	0.0784	Eq. 11.4-4
Seismic Design Category	S_{dc}	B	Table 11.6-2
Response Modification Coefficient	R	2.0	Table 12.2-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height (above grade)	h_n	112.66	
Approximate Period Parameter	x	0.75	Table 12.8-2

Calculated Period Upper Limit Coefficient	C_u	1.70	Table 12.8-1
Approximate Fundamental Period	T_a	0.692	Eq. 12.8-7
Fundamental Period	T	1.17	Sec. 12.8.2
Long Period Transition Period	T_L	12	Fig. 22-15
Seismic Response Coefficient	C_s	0.034	Eq. 12.8-2
Structural Period Exponent	k	1.335	Sec. 12.8.3

In order to calculate the base shear, the effective seismic building weight needed to be determined. An excel sheet was set up to determine the total weight that accumulated at each floor above grade. A summation of each floor resulted in the effective building weight which was used to determine the base shear and overturning moments due to seismic loads. The detailed calculations used to obtain the overall building weight can be found in Appendix B.

It is important to note that the use of a steel frame system, rather than the load bearing masonry walls, is a much lighter system. Therefore, this dramatically reduced the overall building weight. In turn, the base shear was reduced. Please refer to Table 2C for a comparison of the original seismic values and the new design values.

Table 2C - Seismic Comparison

	Original Building Design	New Building Design
Building Weight	16679 lbs	11359 lbs
Base Shear	583.5 kips	397.6 kips
Total Moment	40116 ft-kips	27962 ft-kips

The base shear and overturning moment calculations for each floor can be referenced in Appendix B. The story shear calculations determined for each level can be found in Appendix B along with the existing story shear seismic calculations.

Even with the lighter overall building weight and reduced seismic loads, the seismic forces exceed the forces present due to the wind pressures. Seismic loads control the building in both directions.

Load Combinations

The list below shows the various load cases according to ASCE-07 section 2.3 for factored loads using strength design and from the International Building Code, 2006 edition. These were the load cases used in the analysis of the lateral system for this report.

1.4D
1.2D +1.6L +0.5Lr
1.2D +1.6W +1.0L +0.5Lr
1.2D + 1.0E + 0.5L
0.9D + 1.6W
0.9D + 1.0E

The controlling load combination in each direction is 0.9D + 1.0E.

Drift Criteria

The following allowable drift criteria that will be used to check deflection for the redesign of the buildings lateral system will be in accordance with the International Building Code, 2006 edition.

(Allowable Building Drift) $\Delta_{wind} = H/400$

(Allowable Story Drift) $\Delta_{seismic} = 0.015H_{sx}$

Since the seismic loads control in both directions, the story drift will be governed by the allowable seismic drift equation.

Material Properties

The material strengths for the lateral system redesign are as follows:

Normal Weight Concrete
- f'_c = 8000 psi (for walls supporting ground to 4th floor)
- f'_c = 5000 psi (for walls supporting 5th floor and above)
- E_c = 5700 kis

CMU Block
- f'_m = 1500 psi

Reinforcing Steel
- f_y = 60 ksi
- E_s = 29000 ksi

Design Process

Modified Shear Wall Layout

The existing lateral system of concrete masonry shear walls was chosen to remain as the lateral force resisting system. In changing the building gravity system to steel, the exterior shear walls for the building were eliminated. Therefore, the first step in the design process was to determine a layout for the redesigned lateral system. After eliminating the exterior shear walls, the remaining shear walls in the building are the core shear walls that surround the staircases and elevator shaft. The new lateral system will now consist of those core shear walls. No shear walls will be present along the perimeter of the building. There will be five shear walls approximately 21 ft long and six walls approximately 8 ft long. The new shear wall layout can be seen in Figure 5.2.

The exterior shear walls that were eliminated made the most impact on resisting lateral loads to the building. Now that only the core shear walls remain as the lateral force resisting system, the walls will need to be redesigned in order to resist the seismic loads present on the building.

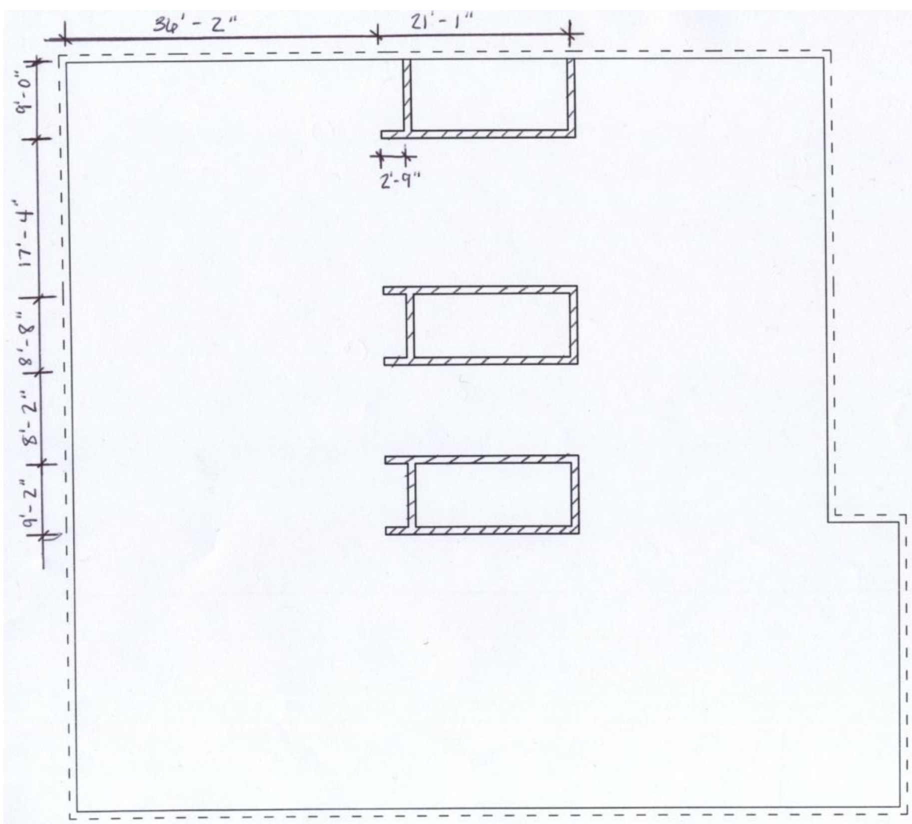


Figure 5.2: Shear Wall Layout

Preliminary Shear Wall Thickness

The next step in the design process was determining a preliminary wall thickness for the shear walls. The minimum thickness of the shear walls was limited by the shear strength of concrete. The wind and seismic loads calculated using ASCE 7-05 were used in the determination of the preliminary wall thicknesses, which were calculated by the following equation:

$$t = \frac{\rho(V_x)}{\emptyset\left(3\sqrt{f'c}\right)l_w}$$

- t = wall thickness (in.)
- ρ = fraction of story shear force resisted by shear wall
- V_x = factored total shear force at level x (lbs)
- Φ = 0.75 for wind loads
- Φ= 0.6 for seismic loads
- $3\sqrt{f'c}$ = approximate shear stress of wall (psi)
- l_w = length of wall (in.)

The required thicknesses based on the seismic loads were much larger than those based on wind loads. The required preliminary thicknesses determined in association to the seismic loads can be found in Table 2D for reference. The table is broken up in different sections; the required area in shear for each story, the required area in shear for each wall, and then the preliminary thickness for each wall. The minimum thickness required for the shear walls was 7” as highlighted below in the table. To be conservative in the design, the use of a 10” concrete masonry block was chosen for the design. Since the original design made use of 8” concrete masonry walls, the redesigned 10” thick walls will not affect the layout of the building.

Table 2D: Preliminary Shear Wall Thickness

Determination of Preliminary Shear wall thickness to resist seismic forces				
Top of Level	Story Force (k)	Total Shear (lbs)	Total Shear/0.60 (lbs)	Required area in shear (in^2)
Roof	5.41	5410	9017	42.53
10	69.66	75070	125117	590.17
9	68.33	143400	239000	1127.36
8	59.30	202700	337833	1593.55
7	50.61	253310	422183	1991.43
6	42.27	295580	492633	2323.74
5	34.33	329910	549850	2593.63
4	26.83	356740	594567	2804.56
3	19.81	376550	627583	2341.73
2	13.39	389940	649900	2425.00
1	7.67	397610	662683	2472.70

Determination of Preliminary Shear wall Thickness to Resist Seismic Forces										
Required area in Shear per Wall (in^2)										
E/W Direction - 20% to each wall					N/S Direction - 16.67% to each wall					
Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall A	Wall B	Wall C	Wall D	Wall E	Wall F
0.00	10.63	10.63	10.63	10.63	0.00	10.63	10.63	0.00	10.63	10.63
118.03	118.03	118.03	118.03	118.03	98.38	147.54	147.54	98.38	147.54	147.54
225.47	225.47	225.47	225.47	225.47	187.93	281.84	281.84	187.93	281.84	281.84
318.71	318.71	318.71	318.71	318.71	265.65	398.39	398.39	265.65	398.39	398.39
398.29	398.29	398.29	398.29	398.29	331.97	497.86	497.86	331.97	497.86	497.86
464.75	464.75	464.75	464.75	464.75	387.37	580.94	580.94	387.37	580.94	580.94
518.73	518.73	518.73	518.73	518.73	432.36	648.41	648.41	432.36	648.41	648.41
560.91	560.91	560.91	560.91	560.91	467.52	701.14	701.14	467.52	701.14	701.14
468.35	468.35	468.35	468.35	468.35	390.37	585.43	585.43	390.37	585.43	585.43
485.00	485.00	485.00	485.00	485.00	404.25	606.25	606.25	404.25	606.25	606.25
494.54	494.54	494.54	494.54	494.54	412.20	618.17	618.17	412.20	618.17	618.17

Determination of Preliminary Shear wall Thickness to Resist Seismic Forces										
Preliminary Thickness (in)										
E/W Direction					N/S Direction					
Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall A	Wall B	Wall C	Wall D	Wall E	Wall F
0.000	0.043	0.043	0.041	0.041	0.000	0.111	0.097	0.000	0.111	0.097
0.457	0.480	0.480	0.457	0.457	0.965	1.537	1.341	0.965	1.537	1.341
0.874	0.917	0.917	0.874	0.874	1.842	2.936	2.562	1.842	2.936	2.562
1.235	1.296	1.296	1.235	1.235	2.604	4.150	3.622	2.604	4.150	3.622
1.544	1.619	1.619	1.544	1.544	3.255	5.186	4.526	3.255	5.186	4.526
1.801	1.889	1.889	1.801	1.801	3.798	6.051	5.281	3.798	6.051	5.281
2.011	2.109	2.109	2.011	2.011	4.239	6.754	5.895	4.239	6.754	5.895
2.174	2.280	2.280	2.174	2.174	4.584	7.304	6.374	4.584	7.304	6.374
1.815	1.904	1.904	1.815	1.815	3.827	6.098	5.322	3.827	6.098	5.322
1.880	1.972	1.972	1.880	1.880	3.963	6.315	5.511	3.963	6.315	5.511
1.917	2.010	2.010	1.917	1.917	4.041	**6.439**	5.620	4.041	**6.439**	5.620

Shear Wall Design

Shear reinforcing for the shear walls was determined by hand methods and it was determined that only the minimum amount of reinforcing according ACI-530 was required for the shear walls. See Table 2E below for a sample calculation of Wall 2. The table is broken into two parts defining the loads and defining the design. The full hand calculations and design for every shear wall can be found in Appendix C. The concrete masonry shear walls designed are normal weight concrete blocks with solid grout fill. The required number of reinforcing bars and their spacing were determined. The load combination 1.2D+0.5L+1.0E was the controlling load case in determining axial and flexural strength in finding the required reinforcement.

Table 2E: Shear Wall 2 Design

t_w (in) =	10
h_w (in) =	216
l_w (in) =	246
l_w (ft) =	20.5
f'm (psi) =	1500
fy (psi) =	60000
d (in) =	196.8

NWC-Solid Grout (psf) =	104
w_d (plf)=	1239
w_L (plf) =	991
w_{Lr} (plf) =	929

Level	Height (ft)	F (k)	V_E (k)	Load Combination: 1.2D+0.5L+1.0E			
				P_d (k)	P_L (k)	P_u (k)	M_u (k-ft)
PH Roof	111.97	1.26	1.26	25.4	20.3155	45.21	12.5512
Roof	101.97	16.16	17.42	72.1	40.631	120	175
10	92.64	15.85	33.27	116.1	60.9465	191	485
9	83.31	13.76	47.03	161.3	81.262	263	924
8	73.98	11.74	58.77	206.7	101.5775	336	1473
7	64.65	9.81	68.57	252.6	121.893	410	2112
6	55.32	7.96	76.54	297.2	142.2085	481	2827
5	45.99	6.22	82.76	342.5	162.524	554	3599
4	36.66	4.60	87.36	388.5	182.8395	628	4415
3	27.33	3.11	90.47	433.1	203.155	699	5281
2	**18**	**1.78**	**92.25**	**496.9**	**223.4705**	**797**	**6918**

Shear Wall Design: Wall 2

Bar Size	Area
3	0.11
4	0.2
5	0.31
6	0.44
7	0.6
8	0.79

Required Vertical Shear Reinforcing

$\rho_{t\,min} = A_v / (s*h) = 0.0025$

$A_{v\,req'd} =$ 0.40 in^2

$A_v =$ 0.62 > 0.40 OKAY

Flexural	Axial+Flexural Strength		Vertical Reinforcement							
ΦM_n (k-ft)	ΦP_n (k)	ΦM_n (k-ft)	M_n (k-ft)	V_u (k)	< V_n (k)	A_s/s	Spacing	$A_{v,req'd}$	Design Reinf.	A_v
6	42	22	26	3	296	0.000	16	0.00	(2) #5 bars	0.62
88	124	306	333	41	296	0.004	16	0.06	(2) #5 bars	0.62
243	202	850	908	78	296	0.007	16	0.11	(2) #5 bars	0.62
462	279	1618	1724	110	296	0.009	16	0.15	(2) #5 bars	0.62
736	355	2577	2753	137	296	0.012	16	0.19	(2) #5 bars	0.62
1056	430	3696	3965	161	296	0.014	16	0.22	(2) #5 bars	0.62
1413	502	4947	5334	181	296	0.015	16	0.24	(2) #5 bars	0.62
1799	573	6298	6831	196	296	0.017	16	0.27	(2) #5 bars	0.62
2207	643	7726	8433	209	296	0.018	16	0.28	(2) #5 bars	0.62
2641	711	9242	10151	217	296	0.018	16	0.29	(2) #5 bars	0.62
3459	**801**	**12107**	**13413**	**224**	**296**	**0.019**	**16**	**0.30**	**(2) #5 bars**	0.62

The shear reinforcing for all the walls consisted of two #5 bars at a minimum spacing of 8”. After the shear reinforcing was designed by hand, it was verified with an interaction diagram. The ultimate factored moments and axial loads were plotted on interaction diagrams to check that they were within the shear wall’s capacity. The reinforcing in all walls is sufficient to carry the applied loads. An interaction diagram for Wall 2 can be referenced in Figure 5.3.

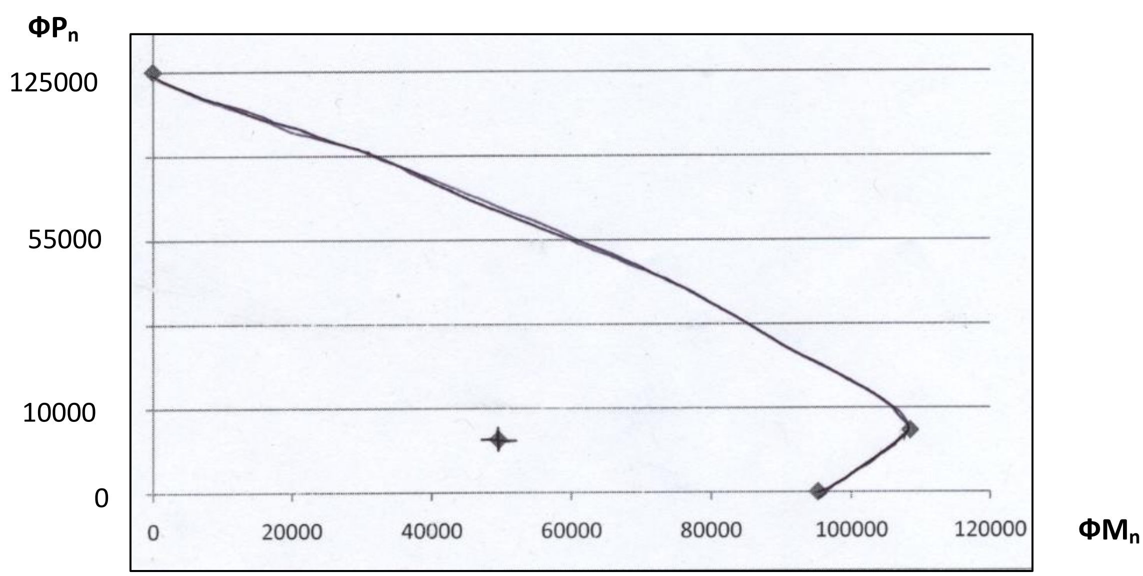


Figure 5.3: Interaction Diagram for a wall with (2) #5 bars @ 16" OC

Optimization Study of Redesign Lateral System

ETABS Model

ETABS is a computer modeling and analysis program developed by Computer and Structures, Inc. One of the advantages of this program is the ability to look at each floor of the building strictly as a rigid diaphragm against lateral loading. Therefore, for the analysis, the building's lateral system and diaphragms were the only components modeled. As seen in Figure 5.4, the shear walls and floor slabs were the only elements modeled. Material properties and geometric properties were inputted for the floor slabs and each shear wall. The simplification of only modeling lateral components allowed for the gravity loads to be applied as additional area masses to the diaphragms. Both wind and seismic loads were applied about the centers of rigidity of the structure for analysis. The results from this model were compared to values produced by hand calculations of the center of mass, centers of rigidity, and story displacements. The overall building drift and controlling loads in each direction were also pulled from the model analysis.

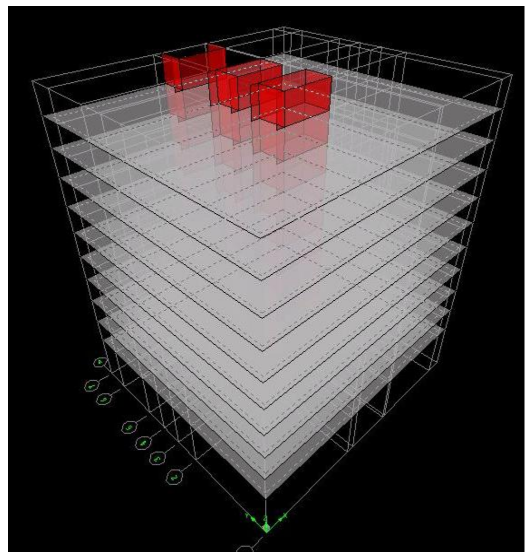


Figure 5.4: ETABS Model of Shear Walls

Load Path

The wind and seismic loads that act against the building need a way of traveling through the structure into the foundation, ultimately reaching the ground. This load path is assumed to be governed by the concept of relative stiffness. The members that are most rigid in a building draw the forces to them. As the lateral forces come in contact with the building, the loads are transmitted through the rigid floor diaphragms, to the core shear walls. Diaphragm forces are transferred to shear walls parallel to the force direction as referenced in Figure 5.5. The shear walls react to the lateral loads and ultimately distributed the loads down through the foundation.

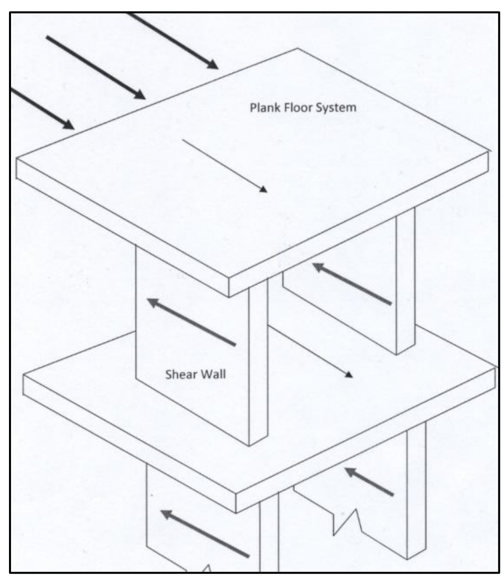


Figure 5.5: Lateral Load Path

Center of Rigidity and Mass

The Fairfield Inn and Suites has shear wall core. The shear walls surround the two staircases and the elevator shaft. The assigned designation to each shear wall can be found in Figure 5.6 for reference as the shear walls are discussed throughout the analysis. The shear walls are all a thickness of 10" throughout their heights. The walls do vary in length and are located different distances from the center of rigidity of the building. The thickness, height, and distance from the center of rigidity for each shear wall affect the rigidity of the wall and alter the relative stiffness of each wall.

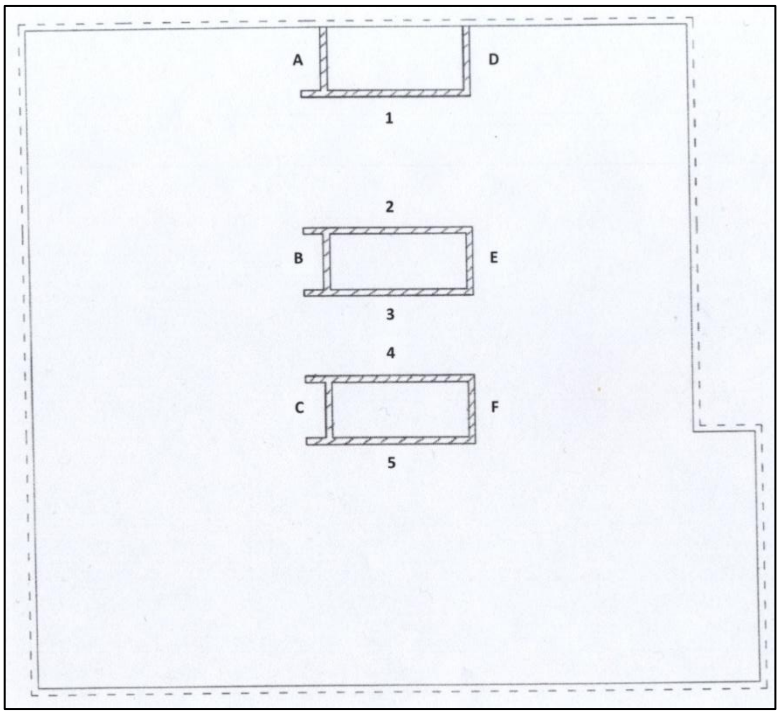


Figure 5.6: Numbered Shear Wall Layout

Tables in Appendix D define the rigidities of Walls 1-5 which are parallel to the East/West lateral forces and of Walls A-F which are parallel to the North/South lateral forces. The rigidities of each wall were calculated using the following equation:

$$R = \frac{\mathrm{E\,t}}{4\left(\frac{\mathrm{H}}{\mathrm{L}}\right)^{3} + 3\left(\frac{\mathrm{H}}{\mathrm{L}}\right)}$$

The equation has to take into account that walls supporting up to floor 4 have an f'c = 8000 psi and the walls above floor 4 have an f'c = 5000 psi. The rigidities of each wall can then be used to determine the center of rigidity of each floor through the following equation:

$$\text{Center of Rigidiy} = \frac{\Sigma[(\mathrm{R})(\text{distance between origin and element})]}{\Sigma R}$$

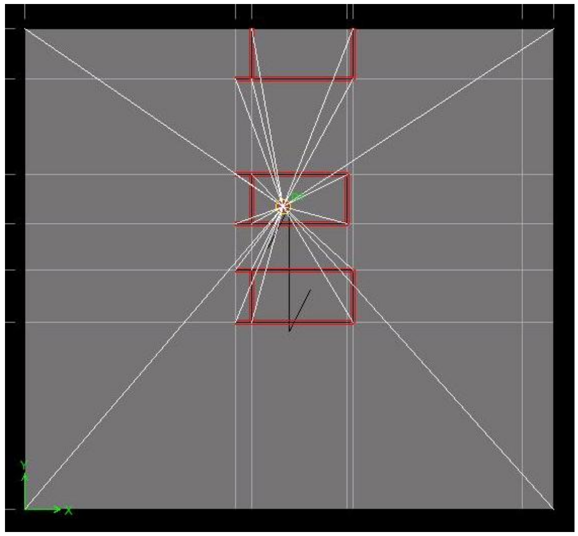


Figure 5.7: ETABS Rigidity Layout

The values for the center of rigidity and center of mass for the original and modified shear walls can be found in Table 2F. The values in the table were found using hand calculation methods. The rigidity calculated by hand assumes only the shear walls are to be considered, but the ETABS model takes into account the building diaphragms when determining the rigidity as seen in Figure 5.7. This study aims to only look at the shear walls as lateral resisting elements, therefore only the hand calculated values will be used throughout the analysis. The ETABS values and detailed hand calculations can be found in Appendix D.

Table 2F - Original vs. Modified Comparison								
	Center of Rigidity				Center of Mass			
	Original		Modified		Original		Modified	
Level	x	y	x	y	x	y	x	y
Roof	614.11	485	544.60	471.68	555	504	554.97	504
10	608.04	535.68	545.38	394.92	555	504	554.97	504
9	599.93	533.96	545.38	394.91	555	504	554.97	504
8	580.41	531.92	545.38	394.90	555	504	554.97	504
7	563.13	529.50	545.38	394.89	555	504	554.97	504
6	549.52	526.63	545.38	394.87	555	504	554.97	504
5	539.18	523.17	545.38	394.84	555	504	554.97	504
4	531.36	518.96	545.37	394.80	555	504	554.97	504
3	525.42	513.65	545.37	394.73	555	504	554.97	504
2	520.85	506.63	545.36	394.60	555	504	554.97	504
1	514.66	497.07	545.35	394.37	555	504	554.97	504

Relative Stiffness

With the rigidity of the walls determined, we can use them to find the relative stiffness of each wall at each floor. The relative stiffness dictates what percentage of the lateral force is distributed to each wall. The relative stiffness will not be consistent throughout the entire height of the building. This can be calculated using the following equation:

$$\text{Relative Stiffness} = \frac{\text{R}}{\Sigma R}$$

The values for all the walls at every floor in the modified design can be found in Table 2G. Knowing the relative stiffness of each wall, the values can be directly applied to the loads at each floor to determine how much of the load each wall will have to resist. Table 2H provides the relative stiffness values for the original design. The relative stiffness of all the walls is much greater which was to be expected by eliminating the exterior shear walls.

Table 2G - Modified Relative Stiffness (%)											
	North - South						East- West				
	Wall A	Wall B	Wall C	Wall D	Wall E	Wall F	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Roof	0.00	19.98	30.02	0.00	19.98	30.02	0.00	23.25	23.25	26.75	26.75
10	16.20	13.51	20.30	16.20	13.51	20.30	21.10	18.35	18.35	21.10	21.10
9	16.20	13.51	20.29	16.20	13.51	20.29	21.10	18.35	18.35	21.10	21.10
8	16.20	13.52	20.29	16.20	13.52	20.29	21.09	18.36	18.36	21.09	21.09
7	16.20	13.52	20.28	16.20	13.52	20.28	21.08	18.38	18.38	21.08	21.08
6	16.20	13.53	20.28	16.20	13.53	20.28	21.07	18.39	18.39	21.07	21.07
5	16.20	13.53	20.26	16.20	13.53	20.26	21.05	18.42	18.42	21.05	21.05
4	16.21	13.55	20.24	16.21	13.55	20.24	21.03	18.46	18.46	21.03	21.03
3	16.21	13.58	20.21	16.21	13.58	20.21	20.98	18.53	18.53	20.98	20.98
2	16.23	13.64	20.13	16.23	13.64	20.13	20.90	18.65	18.65	20.90	20.90
1	16.27	13.79	19.94	16.27	13.79	19.94	20.75	18.87	18.87	20.75	20.75

Table 2H - Relative Stiffness (%)											
	North - South						East - West				
	Wall A	Wall B	Wall C	Wall D	Wall E	Wall F	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Roof	0	23.30	26.7	0	23.3	26.7	0	21.8	21.8	28.2	28.2
10	1.73	1.51	2.04	1.73	1.51	2.04	0.81	0.71	0.71	0.81	0.81
9	0.74	0.63	0.91	0.74	0.63	0.91	0.87	0.76	0.76	0.87	0.87
8	0.44	0.37	0.54	0.44	0.37	0.54	0.94	0.82	0.82	0.94	0.94
7	0.31	0.26	0.39	0.31	0.26	0.39	1.04	0.91	0.91	1.04	1.04
6	0.25	0.21	0.31	0.25	0.21	0.31	1.18	1.03	1.03	1.18	1.18
5	0.22	0.18	0.27	0.22	0.18	0.27	1.38	1.21	1.21	1.38	1.38
4	0.19	0.16	0.24	0.19	0.16	0.24	1.68	1.48	1.48	1.68	1.68
3	0.18	0.15	0.23	0.18	0.15	0.23	2.16	1.91	1.91	2.16	2.16
2	0.17	0.14	0.21	0.17	0.14	0.21	2.94	2.63	2.63	2.94	2.94
1	0.16	0.13	0.20	0.16	0.13	0.20	4.18	3.80	3.80	4.18	4.18

Torsion

When the center of rigidity and the center of mass do not occur at the same location, torsion is present. The difference between the center of rigidity and center of mass is the eccentricity. Moments are produced by this eccentricity and torsional shear becomes an additional force on the building.

For rigid diaphragms, like Fairfield Inn and Suites, two separate moments need to be taken into account when determining torsion in a building. According to ASCE 7-05, torsion in rigid diaphragms is the sum of the inherent moment and the accidental moment. The inherent moment, M_t, is caused by the eccentricity between the center of rigidity and the center of mass. The lateral force exerted on the building at that level; times the eccentricity of the floor gives the inherent moment. The accidental moment, M_{ta}, is due to the rigidity of the slab. The accidental moment takes into account an assumed displacement of the center of mass. The displacement is a distance equal to 5% of the center of mass dimension each way from the actual location perpendicular to the direction of the applied force. Torsional moments for the original design produced by forces in both directions can be seen in Table 2I.

Table 2J shows the overall building torsion for the modified design. With the altered shear wall layout, the distances between the center of rigidities and the center of mass have decreased. In turn, the Mt moment decreased in the North/South direction and translated into a 55% decrease in the overall torsion on the building. This is a significant decrease in torsion and is a great advantage of the modified shear wall design. A decrease in overall building torsion occurred in the East/West direction as well, but at a smaller percentage. Detailed calculations of this method can be found in Appendix D.

Table 2I: Original Building Torsion

Original Overall Building Torsion										
	North/South Direction					East/West Direction				
	Factored Lateral Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)	Factored Lateral Force (k)	COR-COM (ft)	Mt (ft-k)	Mta (ft-k)	Mt,tot (ft-k)
Roof	78.56	4.93	387.05	362.95	750.00	78.56	0.94	73.88	343.31	417.18
10	105.83	3.75	396.34	488.93	885.27	105.83	0.80	84.29	462.48	546.76
9	91.85	2.12	194.58	424.35	618.93	91.85	0.63	57.54	401.38	458.93
8	78.39	0.68	53.20	362.16	415.36	78.39	0.43	33.33	342.56	375.90
7	65.47	-0.46	-29.81	302.47	272.66	65.47	0.19	12.15	286.10	298.25
6	53.17	-1.32	-70.04	245.65	175.60	53.17	-0.10	-5.43	232.35	226.92
5	41.55	-1.97	-81.80	191.96	110.16	41.55	-0.45	-18.84	181.57	162.73
4	30.69	-2.46	-75.62	141.79	66.17	30.69	-0.90	-27.49	134.12	106.62
3	20.74	-2.84	-59.00	95.82	36.82	20.74	-1.48	-30.71	90.63	59.93
2	11.87	-3.36	-39.9	54.84	14.95	11.87	-2.28	-27.03	51.87	24.84
				Total =	3345.92				Total =	2678.06

Table 2J: Modified Building Torsion

Modified Overall Building Torsion										
	North/South Direction (x)					East/West Direction (y)				
	Factored Lateral Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)	Factored Lateral Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
PH Roof	5.41	-0.86	-4.67	24.99	20.32	5.41	-2.73	-14.79	22.72	7.93
Roof	69.66	-0.80	-55.68	321.83	266.15	69.66	-9.13	-636.11	292.57	-343.54
10	68.33	-0.80	-54.62	315.68	261.07	68.33	-9.13	-624.01	286.99	-337.02
9	59.30	-0.80	-47.40	273.97	226.57	59.30	-9.13	-541.59	249.06	-292.53
8	50.61	-0.80	-40.46	233.82	193.36	50.61	-9.13	-462.28	212.56	-249.72
7	42.27	-0.80	-33.79	195.29	161.50	42.27	-9.14	-386.17	177.53	-208.63
6	34.33	-0.80	-27.45	158.60	131.16	34.33	-9.14	-313.71	144.19	-169.52
5	26.83	-0.80	-21.46	123.95	102.50	26.83	-9.14	-245.27	112.69	-132.59
4	19.81	-0.80	-15.85	91.52	75.68	19.81	-9.15	-181.22	83.20	-98.01
3	13.39	-0.80	-10.72	61.86	51.14	13.39	-9.16	-122.63	56.24	-66.39
2	7.67	-0.80	-6.15	35.44	29.28	7.67	-9.18	-70.39	32.21	-38.18
				Total =	1518.73				Total =	-1928.22

Direct Shear

In order to determine the shear forces on each level of the building, the direct and torsion forces need to be calculated. The combination of the two forces is the overall shear force occurring at each level. The direct shear forces relate to relative stiffness of the shear walls. The torsion forces relate to the torsion moments produced on each floor due to the wind or seismic loads.

The lateral forces acting on a building must be distributed among the shear walls in the structure to be directed down through the load path. The distribution of these forces is the direct shear force that occurs at each level of a building. The story shear forces are distributed dependent on the relative stiffness of each shear wall. The greater the stiffness of the wall, the greater the load the wall can receive. The direct shears applied to each wall can be seen for the modified design in Tables 2K & 2L. Since we eliminated a couple of walls for the new shear wall layout, as opposed to the original design, the direct shear forces applied to each wall are much greater than the original forces applied to those walls. Detailed calculations of obtaining the direct shears in both directions can be found in Appendix D, as well as the original direct shear forces applied to each wall.

Table 2K – Modified North/South Direct Shear

Load Combination 0.9D + 1.0E	Force (k)	Factored Force (k)	Wall A	Wall B	Wall C	Wall D	Wall E	Wall F
Roof	5.41	5.41	0.00	1.08	1.62	0.00	1.08	1.62
10	69.66	69.66	11.28	9.41	14.14	11.28	9.41	14.14
9	68.33	68.33	11.07	9.23	13.87	11.07	9.23	13.87
8	59.3	59.3	9.60	8.01	12.03	9.60	8.01	12.03
7	50.61	50.61	8.20	6.84	10.27	8.20	6.84	10.27
6	42.27	42.27	6.85	5.72	8.57	6.85	5.72	8.57
5	34.33	34.33	5.56	4.65	6.96	5.56	4.65	6.96
4	26.83	26.83	4.35	3.64	5.43	4.35	3.64	5.43
3	19.81	19.81	3.21	2.69	4.00	3.21	2.69	4.00
2	13.39	13.39	2.17	1.83	2.70	2.17	1.83	2.70
1	7.67	7.67	1.25	1.06	1.53	1.25	1.06	1.53

Table 2L – Modified East/West Direct Shear							
Load Combination 0.9D +1.0E	Force (k)	Factored Force (k)	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Roof	5.41	5.41	0.00	1.26	1.26	1.45	1.45
10	69.66	69.66	14.70	12.78	12.78	14.70	14.70
9	68.33	68.33	14.42	12.54	12.54	14.42	14.42
8	59.3	59.3	12.51	10.89	10.89	12.51	12.51
7	50.61	50.61	10.67	9.30	9.30	10.67	10.67
6	42.27	42.27	8.91	7.77	7.77	8.91	8.91
5	34.33	34.33	7.23	6.32	6.32	7.23	7.23
4	26.83	26.83	5.64	4.95	4.95	5.64	5.64
3	19.81	19.81	4.16	3.67	3.67	4.16	4.16
2	13.39	13.39	2.80	2.50	2.50	2.80	2.80
1	7.67	7.67	1.59	1.45	1.45	1.59	1.59

Torsional Shear

Due to the torsion present in the structure, an additional force is present on the building. Each shear wall within in the building will have to resist a torsional shear force. The torsional shear is due to the torsion moments produced on each floor caused by the eccentricity. The total torsional shear present at each wall also relates to the relative stiffness of each shear wall. Once again, the greater the relative stiffness, the greater the shear force will be against that wall. To determine the torsional shear values the following equation is used:

$$T = \frac{\text{Vtot e di Ri}}{\text{J}}$$

- V_{tot} = total story shear
- e = eccentricity (distance from center of rigidity to center of mass)
- d_i = distance from center of rigidity to shear wall
- R_i = relative stiffness of shear wall
- J = torsional moment of inertia

The torsional shear forces were determined for the shear walls supporting floor 3 for the modified design and can be found in Table 2M. The torsional forces in the E/W direct were much higher than the forces in the N/S direction. This is due to the further distance between the center of mass and the center of rigidity in that direction on Floor 3. The torsional forces present in the original design can be found in Appendix D, which for those shear walls the torsion is negligible. Further detailed calculations of how to determine the torsional shear can be found in Appendix D.

Table 2M - Torsional Shear in Shear Walls Supporting Floor 3							
		Factored Story Shear V_{tot} (k)	Relative Stiffness R_i	Distance from COM to COR e (in)	Distance from Wall i to COR d_i (in)	$(R_i)(d_i^2)$	Torsional Shear (k)
Wall 1	E/W	390	0.210	111.8	287.5	17340.7	50.589
Wall 2	E/W	390	0.185	111.8	86.4	1383.3	13.429
Wall 3	E/W	390	0.185	111.8	17.6	57.4	2.736
Wall 4	E/W	390	0.210	111.8	115.6	2803.5	20.341
Wall 5	E/W	390	0.210	111.8	225.6	10677.5	39.697
Wall A	N/S	390	0.162	9.57	142.4	3287.8	1.658
Wall B	N/S	390	0.136	9.57	142.4	2753.3	1.388
Wall C	N/S	390	0.202	9.57	142.4	4097.8	2.066
Wall D	N/S	390	0.162	9.57	141.6	3251.0	1.648
Wall E	N/S	390	0.136	9.57	129.6	2280.5	1.263
Wall F	N/S	390	0.202	9.57	141.6	4051.9	2.054
Torsional Moment of Inertia $J = \Sigma (R_i)(d_i^2)$ =						51984.7	

Drift

The overall drift of a building should be limited as much as possible. The drift is a serviceability consideration that relates to the rigidity of each of the shear walls. The higher a building, the more important the overall drift of a building becomes a factor. The wind drift is limited to an allowable drift of $\Delta = \ell/400$. The seismic forces control the drift in the both directions. The seismic drift is limited to an allowable drift of $\Delta = 0.015h_{sx}$. For the Fairfield Inn and Suites the allowable building drift limit (at the top of the building) will be:

$$\Delta_{limit} = 0.015 \text{ x } (1224") = 18.36"$$

Each floor will be examined independently to determine an approximate story displacement and story drift, adding up to overall building drift. A hand calculation was done to determine the displacements on each floor, keeping in mind that the modulus of elasticity and rigidity change as the f'c of shear walls supporting up to level 4 changes from f'c = 8000 to f'c = 5000. The hand calculations done were determined using the following equation:

$$\Delta_{cantilever} = \Delta_{flexural} + \Delta_{shear}$$

The hand calculations done according to drift are an approximation. In order to computer the story drift and displacements of all the shear walls working together by hand would be very intricate. ETABS does analyze the drift and displacements with all the shear walls working together as a lateral resisting system, therefore, the values computed by hand can't be directly compared with the ETAB results.

For the ETABS model, the building drifts were taken in the x-direction which related to the east/west forces, and in the y-direction which related to the forces in the north/south direction. The overall building drift in the x-direction was 1.04", and 1.66" in the y-direction. Drifts in both directions are less than 18.36"; therefore well within the seismic drift limits enforced. The drift limit for a typical story is 1.68", which each story drift is well within as you can see in Table 2N.

Table 2N - ETABS Overall Story Drifts			
Story	X-direction	Y-direction	Allowable Story Drifts (0.015h)
11	0.0471	0.026	1.8
10	0.561	0.131	1.8
9	0.0574	0.143	1.68
8	0.0584	0.154	1.68
7	0.0586	0.165	1.68
6	0.0578	0.173	1.68
5	0.0555	0.177	1.68
4	0.0516	0.174	1.68
3	0.0458	0.164	1.68
2	0.0375	0.143	1.68
1	0.0142	0.21	3.24
Building Drift	1.0449	1.66	20.28

The original design had a maximum drift in the x-direction (due to east/west forces) = 0.61” and a maximum drift in the y-direction (due to north/south forces) = 1.84”, both of which are well below the limit. Due to all the walls in core acting as a unit, a slight decrease in the drift in the north/south direction was also noted.

The actual hand calculations used to determine the drift and displacement can be found in Appendix D and tables for walls 5 and C.

Summary and Conclusions

In modifying the layout of the shear walls, it added pros and cons to the overall building structure. The use of a reduced number of shear walls caused the thickness of the walls to increase, which even though the program layout of spaces would allow for this, it could be viewed as a disadvantage. An advantage associated with this change was a slightly reduced building drift, but since the drift produced by the original design was well within the allowable limits, it is not necessary to decrease this value. A clear advantage of the modification was the decrease in overall building torsion. This in turn makes the use of only the core shear walls an efficient system.

IMPACT ON FOUNDATION

Overturning and Building Weight

Moments caused against the building could result in overturning affects. The lateral forces against the building result in overturning moments. The foundation for the Fairfield Inn and Suites would experience the most impact from overturning moments. The dead load of the building would serve as the system to resist the overturning. The moments due to the seismic loads in both directions can be found in Table 3A. In the both directions, the seismic loads controlled. These moments are transformed into axial loads and transmitted through the lateral elements to the auger cast pile foundation. A rough estimate was done to check if the overturning would be an issue to the Fairfield Inn and Suites. Stresses due to the lateral loads were compared with the stresses due to the self weight of the building resisting. The stresses from the lateral loads are a small fraction of the stresses from the dead loads; therefore the foundation will have minimal overturning affects. Since moments are present, there will however be a force along the perimeter of the building with a small uplift force on the windward sides and a slight downward force on the leeward sides. Detailed calculations of the overturning check can be found in Appendix E.

Table 3A – Overturning

Level	Height Above Ground - Z (ft)	Story Height (ft)	N/S Forces		E/W Forces	
			Lateral Force F_x (k)	Moments M_x (ft-k)	Lateral Force F_x (k)	Moments M_x (ft-k)
Roof	112.00	10.00	5.41	582.83	5.41	582.83
10	102.00	9.33	69.66	6802.95	69.66	6802.95
9	92.66	9.33	68.33	6012.50	68.33	6012.50
8	83.33	9.33	59.30	4664.93	59.30	4664.93
7	74.00	9.33	50.61	3508.70	50.61	3508.70
6	64.66	9.33	42.27	2535.80	42.27	2535.80
5	55.33	9.33	34.33	1739.22	34.33	1739.22
4	46.00	9.33	26.83	1108.80	26.83	1108.80
3	36.66	9.33	19.81	634.00	19.81	634.00
2	27.33	9.33	13.39	303.43	13.39	303.43
1	18.00	18.00	7.67	69.00	7.67	69.00
		Totals:	397.61	27962.16	397.61	27962.16

Foundation Piles

To evaluate the impact of the redesign of the building on the foundations, the required number of piles to support the new steel structural system was compared to the number of piles used in the original design to support the load bearing masonry walls.

Floor loads to each column were determined by hand calculations and totaled to give the load on each column at the foundation level. See Figure 6.1 for column numbers and locations.

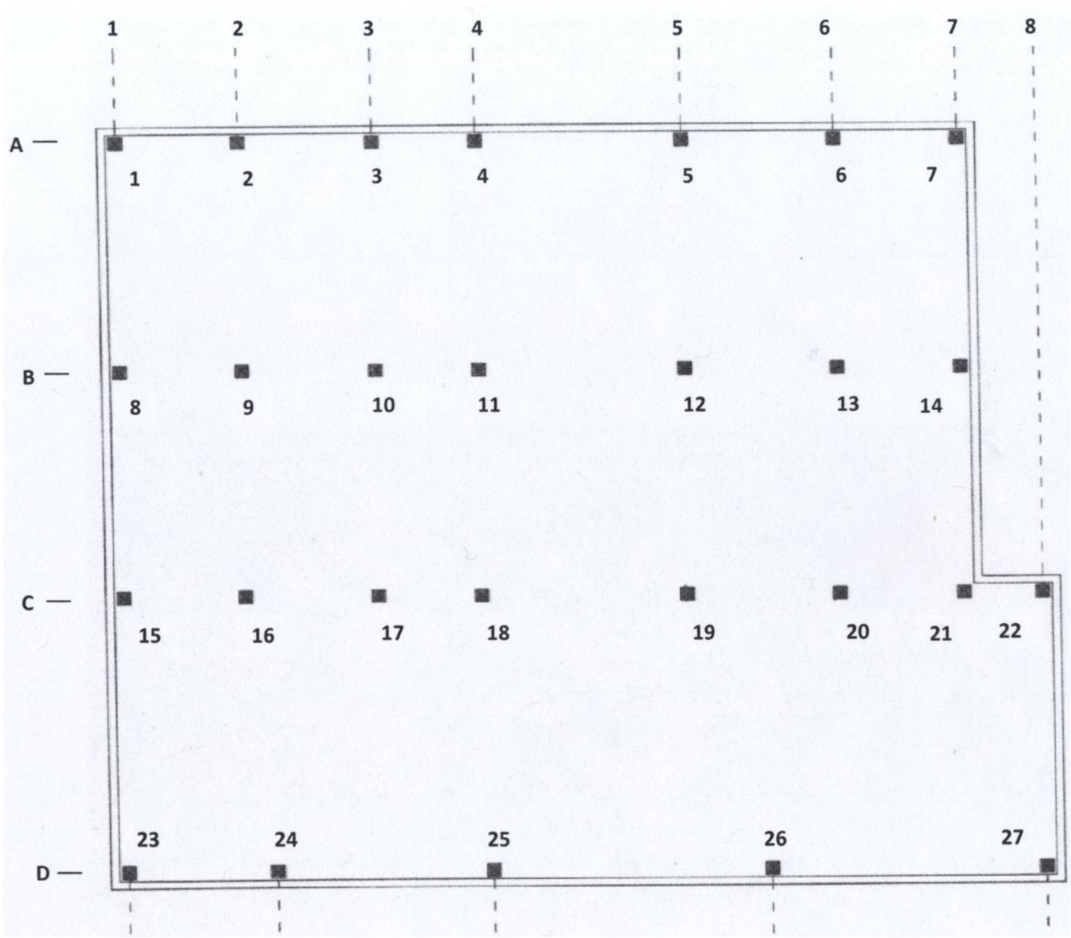


Figure 6.1: Column Numbers Impacting Foundation

The original design utilize 110 ton, 16" diameter, pre-stressed precast concrete piles. The load on each column was divided by the 110 ton capacity of the piles to determine the required number of piles to support each column load. The resulting number of piles was compared with the number of piles required to support the original load bearing walls. See

Table 3B for a comparison of the number of piles required for each system. A breakdown of the number of piles required per column can be found in Appendix E.

Table 3B - Comparison of the required number of piles	
Structural System	Number of Required Piles
Redesigned Steel System	68
Original Concrete/Masonry System	104
Overall % Decrease =	35

Steel structural systems are generally lighter than a load bearing concrete masonry systems, and it is expected that the foundation will be over designed because the loads that if must handle with be lower. The number of piles required to support the core shear walls will remain the same as the original system because they were not altered in the design change. On average there was a 28% decrease in the number of piles that were required per each column load. Overall there is a 35% decrease in the number of piles required for the entire redesign of the structural system.

BREADTH STUDY I: Facade Study

Figure 7.1: Original Facade Brick Veneer

In the original design concept, the load bearing concrete masonry walls did not allow for many design options. The exterior façade consisted of cast stone veneer and brick veneer against the CMU walls. With a new steel structural system designed, this breadth study will focus on adding to and extending the existing curtain wall system on the building. For the purpose of the breadth study, the façade changes will only occur along the west façade of the building. A comparison of the curtain wall system or a brick veneer system on the redesigned structure will be compared with the existing façade design. The wall systems will be compared in respect to the thermal gradient, cost, and construction time.

Figure 7.2: Redesigned Brick Veneer Facade

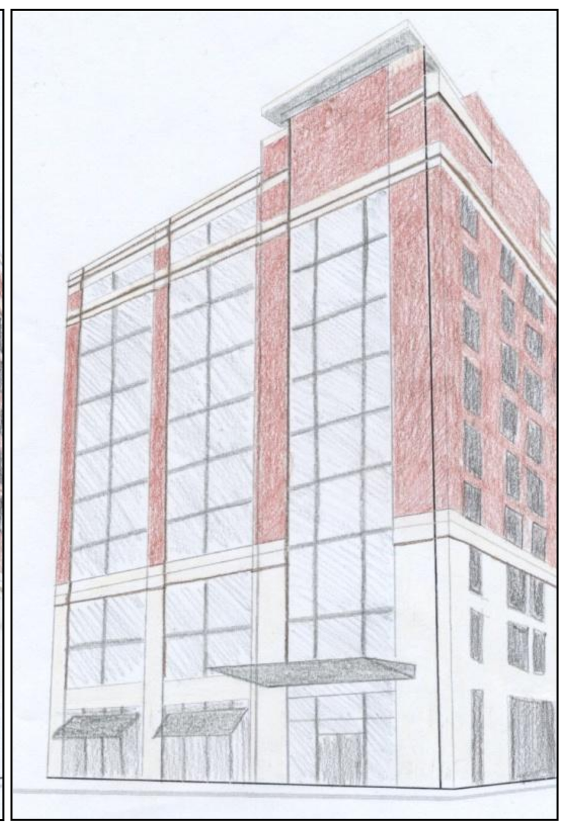


Figure 7.3: Redesigned Curtain Wall Facade

Thermal Gradient Comparison

The thermal gradient for each wall system was determined by establishing the thermal resistance (R-value) for each material within the wall. The R-values for the brick veneer system and the original design were determine in accordance with the 2001 ASHRAE Handbook – Fundamentals. The curtain wall system is an EFCO Series 5900 Wall System and the R-values were determined from the product specifications. Once the R-values were know, the temperature difference between the materials was determined by the following equation:

$$T_x = T_{outdoor} + (T_{indoor} - T_{outdoor})(\Sigma R_{o\text{-}x}/\Sigma R_{o\text{-}i})$$

The following assumptions were made for these calculations:

1. The outdoor air temperature ($T_{outdoor}$) was taken as 0°F
2. The indoor air temerpature (T_{indoor}) was taken as 70°F
3. The relative humidity was taken as 50%

The thermal gradients for the original façade, curtain wall, and brick veneer wall systems are shown in Figures 7.4, 7.5, & 7.6. Please refer to Appendix F for detailed calculations showing how exactly these values were determined.

Existing CMU/Masonry System		
Between Material	**$\Sigma R_{o\text{-}x}$ (°F ft^2 h/BTU)**	**Temperature (°F)**
o - 1	0.17	0
1 - 2	0.28	0.931
2 - 3	1.54	5.12
3 - 4	2.86	9.51
4 - 5	16.61	55.24
5 - 6	19.81	65.88
6 - i	20.37	67.74
	21.05	**70**
U = 0.0475 (BTU/°F ft^2 h)		

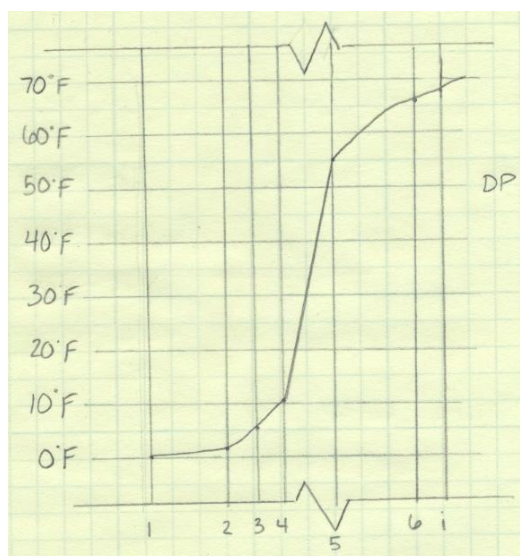


Figure 7.4: Existing Facade Thermal Gradient

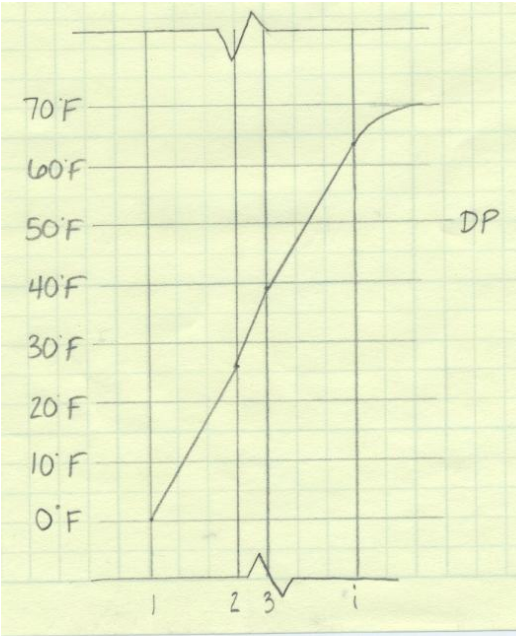


Figure 7.5: Curtain Wall Thermal Gradient

Curtain Wall System		
Between Material	**ΣR_{o-x} (°F ft^2 h/BTU)**	**Temperature (°F)**
o - 1	0.17	0
1 - 2	2.34	25.84
2 - 3	3.49	38.53
3 - i	5.66	62.49
	6.34	**70**
U = 0.158 (BTU/°F ft^2 h)		

Brick Venner System		
Between Material	**ΣR_{o-x} (°F ft^2 h/BTU)**	**Temperature (°F)**
o - 1	0.17	0
1 - 2	0.28	0.848
2 - 3	1.54	4.67
3 - 4	2.86	8.67
4 - 5	21.86	66.2
5 - i	22.54	68.3
	23.10	**70**
U = 0.0433 (BTU/°F ft^2 h)		

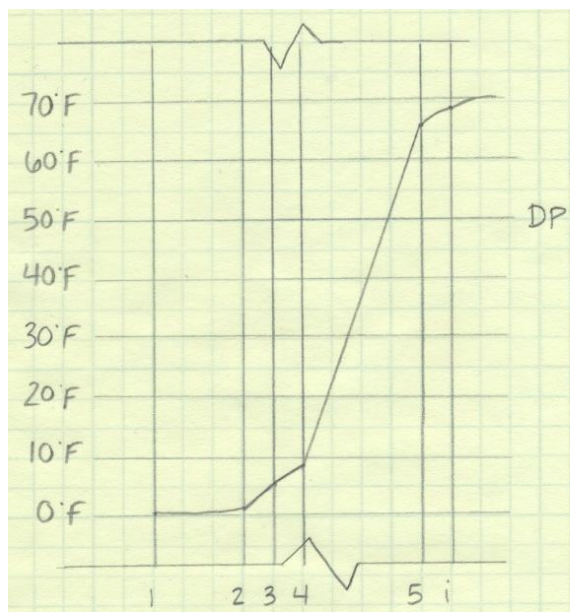


Figure 7.6: Brick Veneer Thermal Gradient

Cost and Construction Time Comparison

A rough estimate of cost and construction time of the EFCO 5900 Curtain Wall System and brick veneer system were prepared using RS Means to compare to the existing brick façade system. The estimate for the brick veneer is based solely on square footage. For a more in depth estimate, scaffolding would need to be considered when setting brick veneer. However, for this breadth, a simple square footage analysis is sufficient to get the overall idea of the differences between each system. The estimate for each wall system is summarized in Table 4 below.

Table 4 – Façade Comparisons

Façade of Existing Structural Design							
Wall System	**S.F.**	**Crew Size**	**Material Cost**	**Labor Cost**	**Total Cost**	**Daily Output**	**Construction Time**
CMU/Brick Veneer System	5734	3 Layers	$10.92	$14.13	$143,637	540	11 days
Façade Systems with Redesigned Structural System							
Wall System	**S.F.**	**Crew Size**	**Material Cost/SF**	**Labor Cost/SF**	**Total Cost**	**Daily Output**	**Construction Time**
Curtain Wall System	5734	2 Glaziers	$30.49	$6.94	$214,624	410	14 days
Brick Veneer System	5734	3 Brick Layers	$6.95	$8.94	$91,113	660	7 days

Conclusions

The thermal gradient of the brick veneer wall system is very gradual due to the batting insulation used within the system. This is the same with the original CMU/Brick veneer system. In determining which system would be most efficient with the redesign of the structural system, the comparison of the heat transfer values (U-values) determines that the curtain wall system transfers approximately 27.4% more BTU/hr than the brick veneer system. Therefore, it can be concluded that utilizing the brick veneer system along the west façade would minimize the heat loss within the guest rooms. This would take away from the esthetics the curtain wall would bring to the room by allowing more light to enter, but ultimately occupant comfort would be improved by utilizing the brick veneer system.

In addition, the brick veneer wall is more cost efficient with respect to construction time. As previously stated, scaffolding for the brick veneer would add to overall cost, however it can still be concluded that the installation of the brick veneer system is more efficient than that of the curtain wall system. It is recommended that the façade on the redesigned structural system implements the brick veneer.

BREADTH STUDY II: Construction Management

To further determine which structural system would be most viable for the Fairfield Inn and Suites, a cost and schedule comparison was conducted between the load bearing concrete masonry walls and the steel framing. In utilizing a steel structural system, the erection time of steel is much faster than CMU load bearing walls. The reduction of shear walls that would need to be constructed will impact the schedule as well. Ideally, this would save significant time on the construction schedule which will ultimately allow the Fairfield Inn and Suites to open sooner.

The redesign of the structural system did not greatly impact the foundation or the interior layout of the building. Therefore, the assumption will be made that only the structural system would impact the construction schedule and alter the cost of the building.

For these reasons, a schedule and cost analysis was prepared for both the existing structural system and the redesigned structural system.

Construction Schedule Comparison

Construction Schedule of Existing Structural System

The existing structural system of the Fairfield Inn and Suites was scheduled to start on February 26, 2009. The construction of the masonry bearing structure was estimated to take approximately six months by being completed by August 13, 2009.

A schedule for the structural system construction coordinates the placement of the block walls and the setting of the precast plank floors, in addition to the minimal transfer beam erection. A summary of the construction time is provided in Table 5A. Please refer to Appendix G to view a more detailed construction schedule for the existing structure.

Construction Schedule of Redesigned Structural System

The redesigned structural system will schedule to have the same start date of February 26, 2009. The steel erection and construction of the shear walls was estimated to take approximately 2 months by being completed on April 24, 2009.

A lot of construction time was saved by utilizing the steel structural system. Steel erection impacted the construction schedule of the structural system by cutting it in half. A mock construction schedule for the structural system was created that coordinates the erection of the steel, placement of the block walls, and setting the precast plank floor. For a detailed construction schedule of the redesigned structural system, please refer to Appendix G. It must be noted, the steel erection does greatly reduce the construction schedule, but it may also increase the lead time of the project by coordinating all the steel members that are now required.

Please refer to Table 5A to view a side by side comparison of the construction time for the existing and redesign system.

Table 5A - Construction Time Comparison			
Component	**Existing System (days)**	**Redesigned System (days)**	**Savings (+)**
Shear Walls	150	21	+ 129
Steel Frame	15	45	-30
Total			**+ 99**

Cost Comparison

A simplified cost comparison of the existing structural system and the redesign structural system for the Fairfield Inn and Suites was conducted using values obtained in the RS Means Cost Data 2009. As the estimate was conducted for comparison purposes, items that were consistent in both systems were omitted.

- Precast plank floors remained the same in each system, therefore unaccounted for in cost
- The foundation was not altered, therefore not included in the cost comparison
- Additional cost due incurred by altering the connections of structural members was ignored

Cost of Estimate of Existing System

Shearwalls	Amt.	Unit	Mat'l Cost/Unit	Labor Cost/Unit	Equip Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	TOTAL COST
8" CMU Block	50019	SF	2.27	3.62	0	5.89	8	$400,152
10" CMU Block	9716	SF	3.06	4.47	0	7.53	10.15	$98,617.40
Reinforcement	17	Ton	810	420	0	1230	---	$20,910.00
Steel	**Amt.**	**Unit**	**Mat'l Cost/Unit**	**Labor Cost/Unit**	**Equip Cost/Unit**	**Total Cost/Unit**	**Total Cost w/ O&P**	**TOTAL COST**
Columns	300	LF	217	4.075	2.18	223.3	241	$72,300.00
Baseplates	72	SF	45	0	0	45.0	---	$ 3,448.12
Beams	417	LF	199.5	3.475	1.41	204.4	219	$91,323.00
Fireproofing	1434	SF	1	1	1.2	3.2	---	$ 4,588.80
Crane			36			300		$10,800.00

Cost Estimate of Redesigned Structural System

Shearwalls	Amt.	Unit	Mat'l Cost/Unit	Labor Cost/Unit	Equip Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	TOTAL COST
10" CMU Block	18473	SF	3.06	4.47	0	7.53	10.15	$187,500.95
Reinforce.	7	Ton	810	420	0	1230	---	$ 8,610.00
Steel	**Amt.**	**Unit**	**Mat'l Cost/Unit**	**Labor Cost/Unit**	**Equip Cost/Unit**	**Total Cost/Unit**	**Total Cost w/ O&P**	**TOTAL COST**
Columns	2753	LF	145	3.26	2.18	150.44	168	$462,504.00
Baseplates	108	SF	45		0	45		$ 5,172.18
Beams	4807	LF	89.5	3.13	2.09	94.72	106	$509,542.00
Fireproofing	7559	SF	1	1	1.2	3.2		$ 24,188.80
Crane	76					300		$ 22,800.00

Detailed material takeoffs can be found in Appendix G to support all cost estimate calculations.

After completing the cost estimate of the separate systems, Table 5B was put together to summarize the overall comparisons between the costs of both structural systems.

Table 5B - Overall Cost Comparison

Component	Existing System	Redesigned System	Additional Cost
Shear Walls	$519,680	$196,111	-$323,569
Steel Framing	$171,660	$1,001,407	$829,747
Crane	$10,977	$22,822	$11,845
TOTAL	**$702,317**	**$1,220,340**	**$518,023**

Conclusion

The use of the steel structural system significantly impacts the cost and construction time of the Fairfield Inn and Suites. Since steel erection can be installed at a higher quantity per day than CMU, the construction time was reduced by approximately 66% in regards to the structural system construction only. In addition, with the high reduction in the construction schedule, the steel structural system is not the most cost efficient for the building. The redesigned system costs approximately 42% most than the existing structural system.

In the event that the hotel would want to be completed faster to make a necessary deadline, the use of the steel structural system could be an efficient option, but the shortened construction time will result in an increase in cost the owner.

CONCLUSION AND RECOMMENDATIONS

The main focus of this final thesis report is to optimize the gravity and lateral systems of the Fairfield Inn and Suites. Classified as Seismic Site Class D soil, it was necessary at the time of the design to utilize a steel structural system in order to reduce the load on the supporting foundation at such a poor soil site. While this design exhibits no problems structurally, both systems are possible areas of optimization for the building.

The gravity system proves that the use of steel moment frame system considerably reduces the overall building weight. The framing plan conducted in the redesign conformed easily to the existing architectural layout, while not affecting the structural depth of the floor system. The system does cut done on construction time, but would not be cost effective. Without having any negative effects on the layout of the building, the reduced building weight does improve the efficiency of the gravity system on the foundation and soil class.

While seismic loads still controlled despite the reduced loads, the base shear value was decreased by approximately 32%. This allowed for a reduction of shear walls that made up the lateral force resisting system. The use of shear walls in the redesign of the lateral system was chosen to keep consistency around the core areas of the building, similar to the original design. As a result, the core shear walls were designed slightly thicker to compensate for the loss of exterior shear wall, without adding weight to the building. The lateral system optimization study favors the modified shear wall layout in comparison to the original design because less building torsion is present on the building while drift is still well within the limits set forth by the code. This in fact would reduce the number of piles required by the foundation system to support the overall redesigned structural system.

The façade breadth study focuses on improvements in guest comfort with respect to natural daylight penetration verse heat transfer through the wall system. By implementing the brick veneer system, the heat transfer through the wall would not be affected, as opposed to using the larger curtain wall system façade option which would increase the heat transfer by approximately 70%. Therefore, choosing the system with a lower heat transfer rate verse the amount of natural daylight it allows in the room is more efficient for the building.

The goals of this thesis were to create an efficient optional gravity and lateral system for the Fairfield Inn and Suites. Based on the results discussed, these goals are clearly met. If cost was not an issue, it is the recommendation of the author to implement the changes proposed, as each study does impact the building in a positive way from a feasibility standpoint.

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