Cambria Suites Hotel

Pittsburgh, PA

Senior Thesis Final Report

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Professor Linda Hanagan
April 7, 2011

Cambria Suites Hotel
@ Consol Energy Center

Pittsburgh, PA

Project Team

- Owner: Pittsburgh Arena Hotel Associates
- Developer: Horizon Properties Group
- GC: Snavely Building Company
- Architect: D.L. Astorino Horizon Architects
- Structural: Atlantic Engineering Services
- Geotech: Geomechanics, Inc.
- Civil: Civil & Environmental Consultants, Inc.
- Landscape Architect: Klavon Design Assoc.
- MEP/Fire Protection: Claitman Engineering Assoc.

Building Statistics

- Location: 1320 Centre Avenue Pittsburgh, PA 15213
- Size: 120,000 SF
- Occupancy Type: Hotel
- Number of Stories: 7 levels above grade
- Project Cost: projected at $25,000,000
- Project Delivery Method: Design-Bid-Build
- Dates of Construction: Nov. 2009—Sept. 2010

Architecture

- 142 -luxury suite hotel
- Indoor pool/spa and state-of-the-art fitness center
- Airy two story lobby
- Steel porte cochere which covers main entrance
- Composite metal panel cornice & fascia at roof level
- Cast stone band around 2nd & 7th floor level
- Roof composed of 10" precast concrete plank, TPO roofing membrane, & tapered insulation
- Light and dark brick vaneer

M.E.P.

- Multiple AHU's which service public & guest areas
- Mini A/C units equipped in guest rooms
- 6 ton natural gas rooftop units with 2000 CFM
- Pool ventilation unit removes 21 lb/hr of moisture
- 208/120V 3 phase, 4 wire & emergency back-up system
- Fluorescent & metal halide lighting
- Combined standpipe & automatic sprinkler system

Structure

- Precast concrete plank floor system
- 8" & 12" Concrete masonry bearing walls
- Steel transfer beams and columns
- Lateral resistance provided by reinforced concrete masonry shear walls
- Cast in place caissons & grade beam foundation
- Caissons vary in size; 13'8" - 35' below grade
- 4" thick slab on grade (8" in some areas)

Adam Kaczmarek

Structural

www.engr.psu.edu/ae/thesis/portfolios/2011/ack5057

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Acknowledgements

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Executive Summary

Cambria Suites Hotel is located in Downtown Pittsburgh. The building is approximately 120,000 square foot and is 7 levels above grade. Each story height ranges from 10' to 14', topping out at an overall building height of 102'-2". The current site of the Cambria Suites Hotel was chosen because of the recent construction of the CONSOL Energy Center. For this reason, the site location will remain the same as it serves as a popular attraction to visitors of the City of Pittsburgh and the CONSOL Energy Center.

The final thesis report examines the implications related to redesign the structural system of the Cambria Suites Hotel. The existing design of the building includes load bearing concrete masonry walls, an interior steel frame, hollow-core precast plank floor system, and concrete caisson foundation. The structural system redesign explores the Girder-Slab system which uses specially designed D-Beams and precast concrete floor plank, which eliminates the use of load bearing masonry walls along the exterior of the building. The redesign also examines the layout and design of the lateral force resisting system which comprises of concentrically braced frames.

The steel gravity system resulted in an overall decrease in building weight, which also reduced the base shear and total moment. Since the building weight was reduced, smaller loads will be transferred to the foundation, causing the caissons to be redesigned for the lighter loads. In addition, the total construction time to erect the steel structure was significantly lower than the existing concrete masonry structure. However, the modification to steel slightly increased the total construction cost of the structural system. The lateral force resisting system was sufficiently designed while maintaining an allowable building drift within code limitations. Structurally, the redesign of the gravity and lateral systems prove to be effective and efficient alternatives for the Cambria Suites Hotel.

The façade breadth focused on the architectural impact of changing the existing structural system to steel. This was done by comparing natural daylight penetration against heat transfer through a particular wall system for optimum guest comfort. By implementing the brick veneer system, it provided a lower heat transfer rate as opposed to the curtain wall system. Although the brick veneer system lacks natural daylight entering the building, it creates the most suitable indoor environment for hotel guests.

The overall goal of this thesis report was to design an effective and efficient structural system for the Cambria Suites Hotel. Through extensive research and design, the data and results throughout this report prove that the project goals were clearly met. If a minimal cost increase and minor floor layout changes were not an issue to the building owner, the alternative steel structural system could be implemented as the final design as each study impacts the building in a positive way.

Building Overview

Function

Cambria Suites Hotel is the newest, upscale, contemporary all-suite hotel located at 1320 Center Avenue in Pittsburgh, Pennsylvania. This luxury hotel is built adjacent to the new CONSOL Energy Center, home to the Pittsburgh Penguins hockey team, and numerous concerts and special events. Due to this prime location, the hotel will accommodate several Pittsburgh Penguins fans, as well as business and leisure travelers throughout the year.

Architecture

The hotel accommodates 142 guests and offers a state-of-the-art fitness center and relaxing indoor pool and spa at the Hotel Level. The hotel will have a variety of room suites, such as the double/queen suite, king suite, one bedroom suite, deluxe tower king suite, and hospitality suite. The Plaza Level will mainly consist of a few bistro-style restaurants which open to an outdoor terrace which will overlook the city of Pittsburgh and the CONSOL Energy Center. Guests will enter the Hotel Level from Center Avenue and be greeted by an airy two-story lobby, which consists of a reception desk, barista coffee bar, and a restaurant serving breakfast and dinner. In addition, there are two meeting rooms and a board room for guest use, as well as, a large kitchen/full-service bar off of the lobby entrance.

The exterior of the hotel is mainly brick and cast-stone veneer, architectural decisions made to resemble the bordering CONSOL Energy Center. A lighter color brick is used from the 2nd-Roof Floor levels, with the addition of a cast-stone band at the second and seventh floor level. The darker color brick is used from the second floor level and below, as well as vertical strips to separate window pairings and to accent building corners.

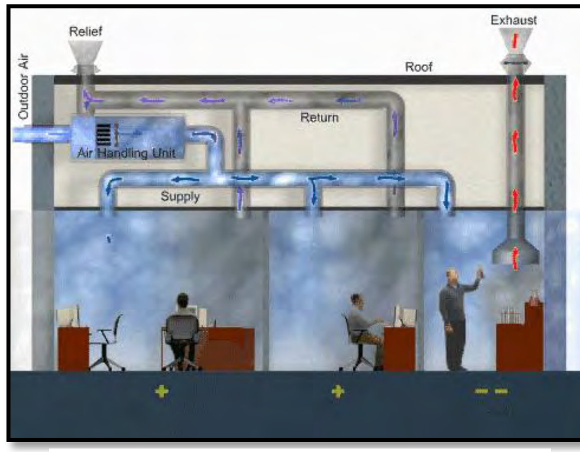
Construction Management

Cambria Suites was constructed as a design-bid-build delivery method. The project broke ground early November 2009, and was complete late September 2010. Hotel reservations began early December 2010, in time for the second half of the Pittsburgh Penguins season. The projected cost of Cambria Suites is $25,000,000 and Snavely Building Company was awarded the general contractor for the project. Cambria Suites is classified as 1B Modified Fire Resistive Construction due to its noncombustible or slow-burning exterior bearing walls and load-bearing portions of exterior walls. The site plan of Cambria Suites is shown is Figure 1.1.

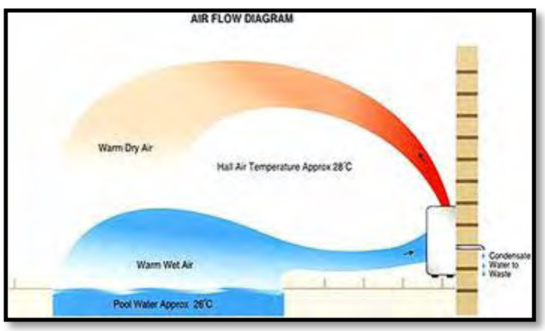
Cambria Suites Hotel Site Plan
Figure 1.1

Mechanical System

The mechanical system for the Cambria Suites Hotel was designed for multiple areas of the building; mainly guest rooms and public spaces. The public spaces located on both the Plaza level and Hotel level will be comprised of a variety of air handling units (AHU) ranging from 525-1400 CFM. In addition, air handling units will be on each remainder floor to service the corridors and tower suites of the hotel. All other guest rooms will be equipped with small room A/C units (PTAC) which have an airflow of 260 CFM. The pool area will be equipped with a pool ventilation unit (PPU) that removes 21 lb/hr of moisture from the air and produces an airflow of 2150 CFM.



<u>Air Handling Unit Process</u>
Figure 1.2



<u>Pool Dehumidifier Process</u>
Figure 1.3

The roof of the hotel will consist of two make-up air units (MUAU) with a rate of 4900 CMF, three rooftop units (RTU) ranging from 1000-2080 CFM, and several air cooled condensing units (ACCU) which are also located at the Hotel level and Second Floor level.

Lighting & Electrical System

The electrical service to Cambria Suites Hotel is a 150 kW, 208/120V, 3 phase, 4 wire system and an emergency back-up system. The typical distribution panel is a 208/120V, 3000A, 3 phase, 4 wire system which services other panels at different floor levels. Each level of the hotel is supplied with 6-7 panel boards located in their respective electrical room. Additional panels are added on the Plaza level to accommodate for the mechanical rooms, pool, fitness center, elevators, and the emergency generator.

The lighting system primarily consists of florescent luminaires with recessed, surface, pendant, and wall mounting. The roadway and parking is comprised of High Pressure Sodium (HPS) and Metal Halide (MH) ballasts.

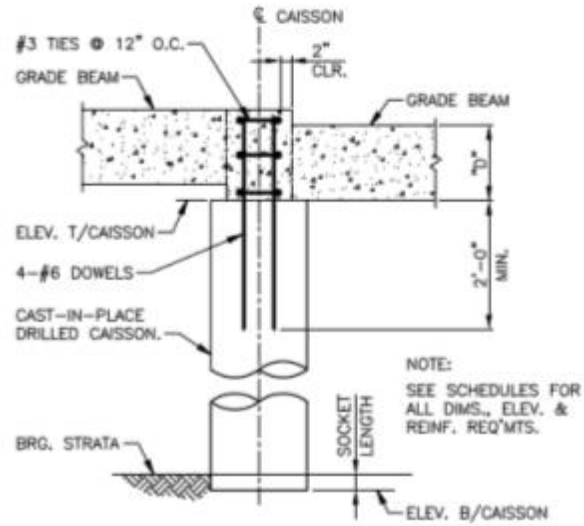
Existing Structural System

Foundation

The geotechnical engineering study for the Cambria Suites Hotel was completed by GeoMechanics Inc. on December 29, 2008. In the study, the site of Cambria Suites Hotel is underlain by sedimentary rocks of the Conemaugh group of rocks of Pennsylvania age. The Conemaugh group is predominantly comprised of clay stones and sandstones interbedded with thin limestone units and thin coal beds. The soil zone conditions consisted of materials of three distinct geologic origins: man-made fill, alluvial deposits, and residual soils. The fill in the hotel test borings was placed in conjunction with the recent demolition and regarding of St. Francis Hospital in order to build Cambria Suites and CONSOL Energy Center. Ground water exists locally as a series of perched water tables located throughout the sol zone and new the upper bedrock surface. Excavations in soils and bedrock can be expected to encounter perched water. The volume of inflow into excavations should be relatively minor, should diminish with time and should be able to be removed by standard pump collection/pumping techniques. The report also states that the most economical deep foundation solution for Cambria Suites included a system of drilled-in, cast-in-place concrete caissons with grade beams spanning between adjacent caissons to support the anticipated column and wall loads of the structure. With varying types of bedrock on site, the allowable end bearing pressure ranges from 8, 16, and 30 KSF. As for the floor slab, GeoMechanics Inc. recommended to place a ground floor slab on a minimum six-inch thick granular base and to provide expansion joints between the ground floor slab and any foundation walls and/or columns. This is done to permit independent movement of the two support systems.

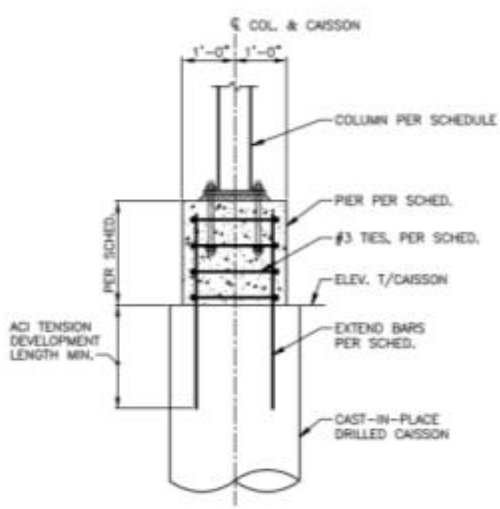
As a result of GeoMechanic's geotechnical study, the foundation of Cambria Suites Hotel incorporates a drilled cast-in-place concrete caissons and grade beams designed to support the load bearing walls and columns. The ground floor is comprised of a 4" concrete slab on grade, as well as, 10" precast concrete plank in the Southern portion of the building. The 4" concrete slab on grade is reinforced with 6x6-W1.4 welded wire fabric and has 4000 PSI normal weight concrete. The slab increases to 8" in thickness with #5 @ 16" O.C. in the South-West corner of the building, and increases to 24" with #5 @ 12" O.C. within the core shear walls where the elevator shaft is located. For the majority of the slab on grade, the slab depth is 14'-0" below finish grade.

The drilled cast-in-place caissons extend anywhere from 20-30 feet deep below grade and are socketed at least 3' into sound bedrock. Caisson end bearing capacity is 30 KSF (15 ton/SF) on Birmingham Sandstone bedrock. The caissons are designed with a compressive strength of 4000 PSI, range from 30-42 inches in diameter, and are spaced approximately between 15' and 30' apart (refer to Appendix A). Typical caissons terminate at the Plaza level and are tied into a grade beam with #3 ties @ 12" O.C. (horizontal reinforcement) and 4-#6 dowels (vertical reinforcement) embedded at least two feet into the drilled caisson. Where steel columns are located, a pier is poured integrally with the grade beam and reinforced with 8-#8 vertical bars and #3 @ 8" O.C. horizontal ties. (As shown in *Figures 2.1 & 2.2*)

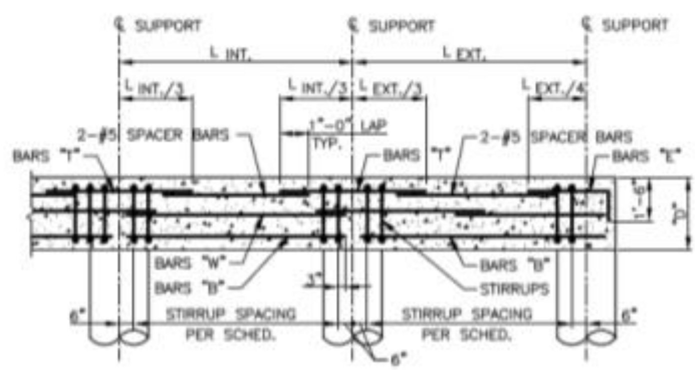


Typical Caisson & Grade Beam Detail
Figure 2.1

The grade beams have a compressive strength of 3000 PSI and range from 30-48 inches in width and 36-48 inches in depth. Each grade beam is reinforced with top and bottom bars which vary according to the size of the beam. Grade beams span between drilled cast-in-place caissons which transfer the loads from bearing walls, shear walls, and columns into the caissons. From the caissons, the loads are then transferred to bedrock. (As shown in *Figures 2.1 & 2.3*)



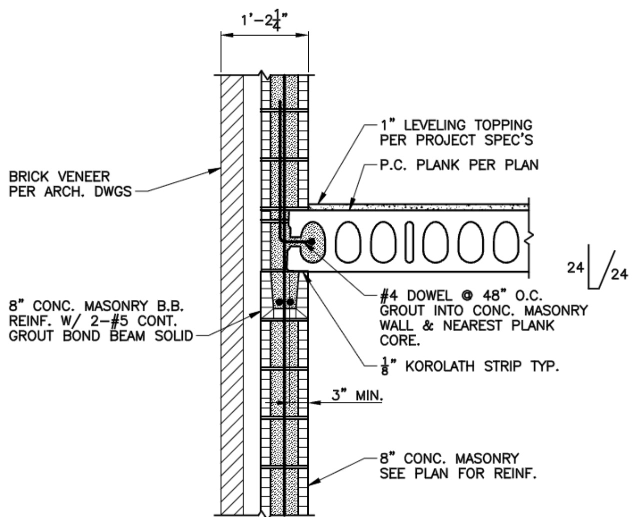
Typical Caisson Cap Detail
Figure 2.2



Typical Grade Beam Reinforcing Detail
Figure 2.3

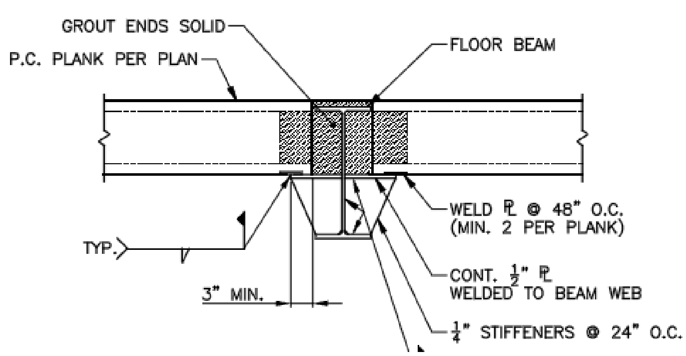
Superstructure System

The typical floor system of Cambria Suites Hotel consists of 10" precast hollow-core concrete plank with 1" leveling topping. The precast plank allowed for quicker erection, longer spans, open interior spaces, and serves as an immediate work deck for other trades. Concrete compressive strength for precast plank floors is 5000 PSI and uses normal weight concrete. The typical spans of the plank floors range from 30'-0" to 40'-0". The floor system is supported by exterior load bearing concrete masonry walls, as well as, interior steel beams and columns. Detailed connections of the plank to the exterior masonry walls and interior steel beams are shown in Figure 3.1 and 3.2.



<u>Typical Exterior CMU Wall Connection to Precast Plank</u>

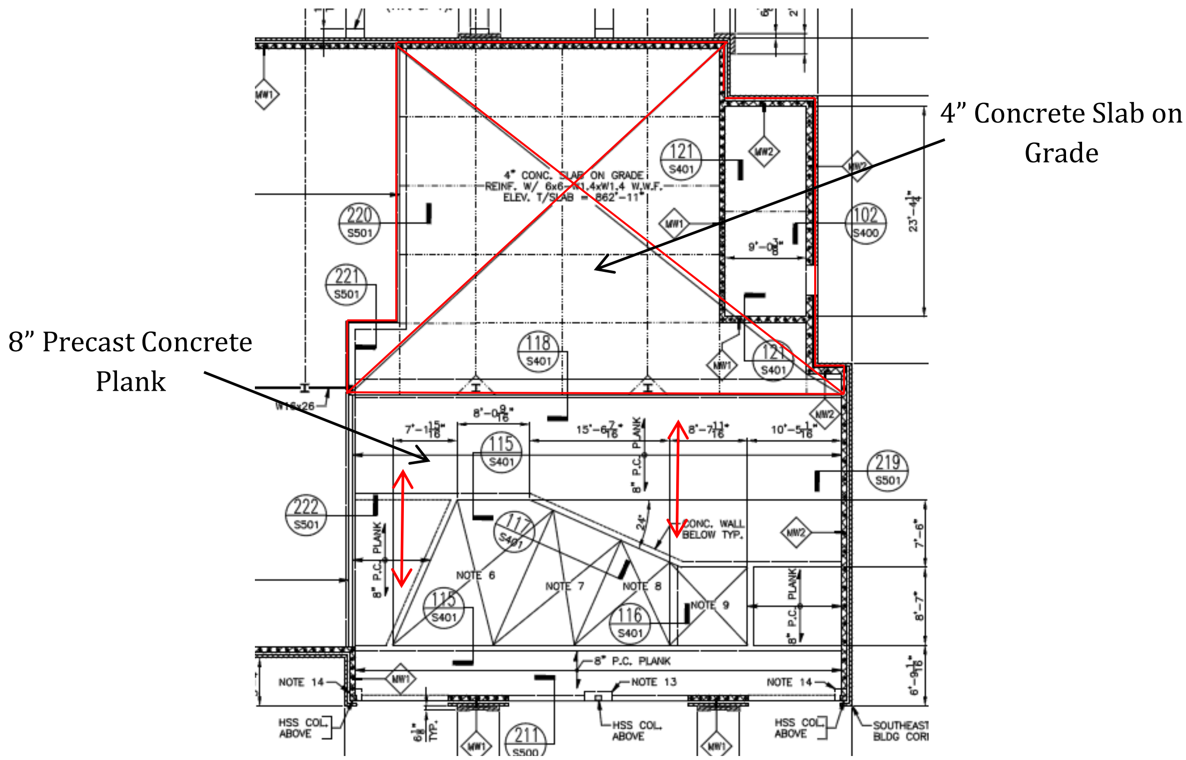
Figure 3.1



<u>Typical Exterior CMU Wall Connection to Steel Beam</u>

Figure 3.2

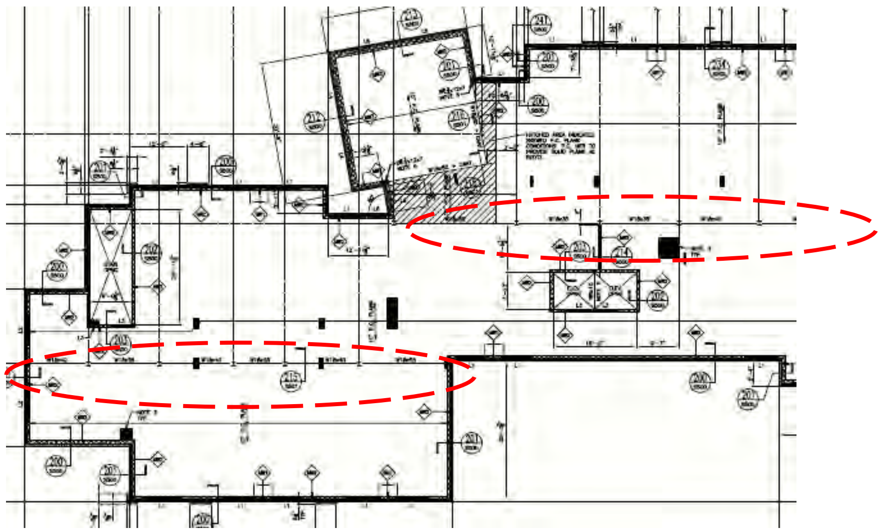
The Plaza level floor system is a combination of 10” precast concrete plank, 8” precast concrete plank and 4” slab on grade. Since there is no basement in the North-East section of the hotel due to the fitness center and pool, the site was excavated properly in order to place the 4” slab on grade and 8” precast concrete plank. The 4” slab on grade will be for the fitness center where as the 8” concrete plank will surround the pool area. (As shown in *Figure 3.3*)



Partial Hotel Level Floor Slab

Figure 3.3

Since the masonry bearing walls are typically located on the perimeter of the hotel, steel beams and columns were needed thru the center of the building to support the precast concrete plank floors. Steel beam sizes range from W16x26 to W24x94, and steel column sizes range from W8x58 to W18x175. Each column connects into concrete piers within the grade beams via base plates which vary in size. Base plates use either a 4-bolt or 8-bolt connection, typically using 1” A325 anchor bolts which extend 12” or 18” respectively into the concrete pier. The steel beams vary in length from 13’-0” to 19’-0” and typically span in the East-West direction. Exterior bearing masonry walls and the steel beams will take a reaction load from the precast concrete plank flooring, as well as other loads from levels above, which will then transfer thru steel columns and exterior bearing walls and thus transferring all loads to the foundation system. (As shown in *Figure 3.4*)



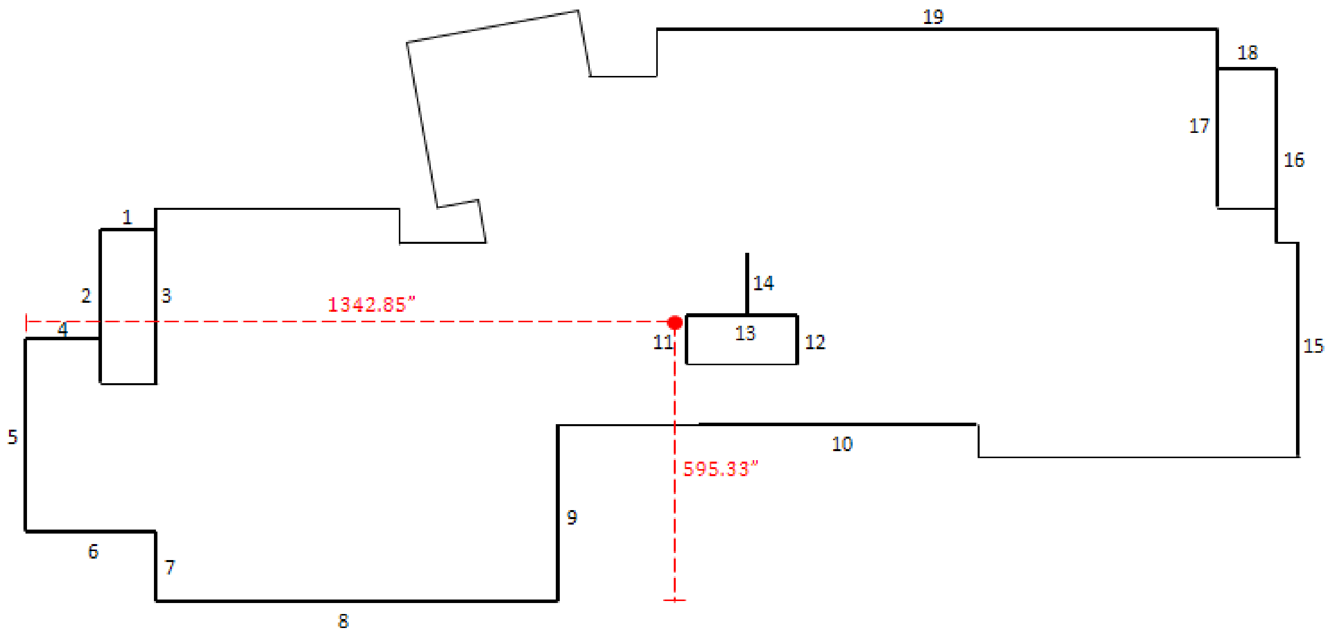
Typical Partial Floor Plan
Figure 3.4

The roof structural system at both the Second level and main Roof level uses untopped 10" precast concrete plank. Reinforced concrete masonry extends passed the Roof level to support a light gauge cornice which wraps the entire building. A high roof is constructed for hotel identification purposes and uses 10"-16 GA light gauge roof joists @ 16" O.C., supported by 8"-20 GA light gauge wall framing below. W8x21 hoist beams support the top of the elevator shaft which rest on ½"x7"x7" base plates. There are a total of eight drains located on the roof for the drainage system. (As shown in Appendix A)

Lateral System

The lateral system for the Cambria Suites hotel consists of reinforced concrete masonry shear walls. The exterior shear walls, as well as the core interior shear walls, are constructed of 8" concrete masonry, with the exception of a few 12" concrete masonry walls on the lower floor levels. All shear walls are solid concrete masonry walls which extend the entire height of the structure without openings for windows or doors. The core shear walls are located around the staircases and elevator shafts. The exterior shear walls are scattered around the perimeter of the building, as shown in Figure 4.1. Shear walls supporting the Plaza level to the Third Floor level have a compressive strength of 2000 PSI. All other shear walls support a compressive strength of 1500 PSI. In addition, all concrete masonry shall be grouted with a minimum compressive strength of 3000 PSI. Typical reinforcement for all shear walls is comprised with #5 bars at either 8" O.C. or 24" O.C.

Wind and seismic loads, as well as gravity loads, are transferred to the foundation by first traveling thru the rigid diaphragm; the precast concrete plank floor system. Loads are then transferred to the concrete masonry shear walls. From there, loads are transferred down to the preceding floor system and/or transferred the entire way to the grade beam foundation, finally travelling thru the concrete caissons which are embedded in bedrock.



Lateral Shear Wall System & Center of Mass

Figure 4.1

Codes and Design Requirements

- American Concrete Institute, *Building Code Requirements for Structural Concrete* (ACI 318-05)

- American Concrete Institute, *Specifications for Masonry Structures* (ACI 530.1)

- American Concrete Institute, *Specifications for Structural Concrete* (ACI 301-05)

- American Concrete Institute, *The Building Code Requirements for Masonry Structures* (ACI 530)

- American Institute of Steel Construction, *Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design* (AISC)

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

- ETABS Nonlinear v9.2.0, copyright 2007 (Research Engineers, Intl.)

- Geschwindner, L. (2008) *Unified Design of Steel Structures*, John Wiley and Sons, Inc., Hoboken, NJ

- Girder-Slab Technologies LLC, www.girder-slab.com

- International Building Code (IBC), 2006
  (As amended by the City of Pittsburgh

- Kawneer Building Systems, www.kawneer.com

- PCI Concrete (2004) *PCI Design Handbook: Precast/Prestressed Concrete Institute*, 6th Edition, PCI, Chicago, IL

- Pittsburgh Flexicore P.C. Plank Specifications

- RAM Structural System v14.03.01, copyright 2009 (Bentley Systems, Inc.)

- RS Means Construction Publishers and Consultants, Building Construction Cost Data 2008 66th Annual Edition, Reed Construction Data, Inc.: Kingston, MA, 2007.

- VULCRAFT Deck Catalog

Materials

Reinforced Concrete

Caissons & Piers	f'_c = 4000 PSI
Grade Beam Foundations	f'_c = 3000 PSI
Slabs on Grade	f'_c = 4000 PSI
Walls	f'_c = 4000 PSI
Exterior Bar or Wire Reinforcement Slabs	f'_c = 5000 PSI

Reinforcement Steel

Deformed Bars	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Structural Steel

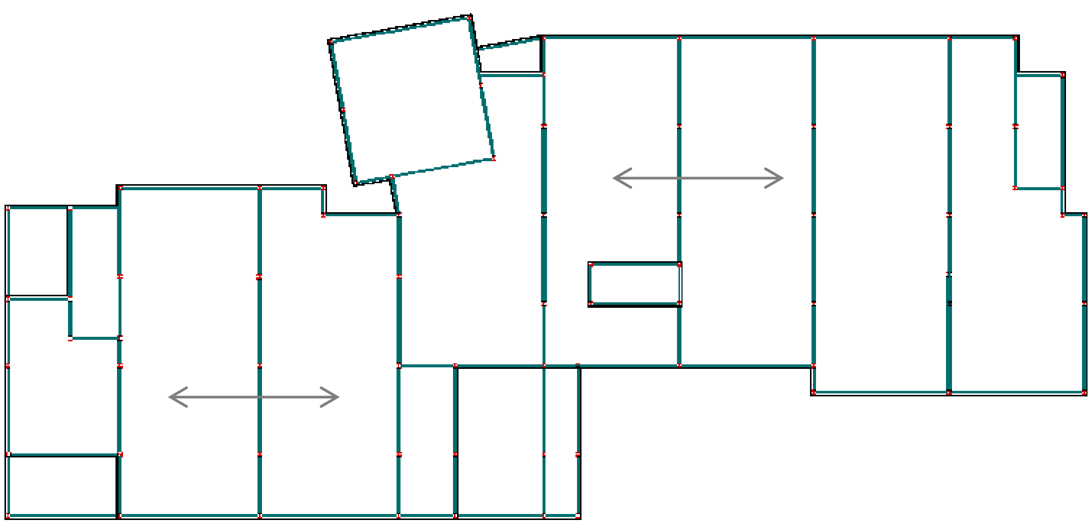
Structural W Shapes	ASTM A992
Channels	ASTM A572, Grade 50
Steel Tubes (HSS Shapes)	ASTM A500, Grade B
Steel Pipe (Round HSSS)	ASTM A500, Grade B
Angles & Plates	ASTM A36
Structural Shapes & Rods	ASTM A123
Bolts, Fasteners, & Hardware	ASTM A153

8" & 12" CMU	f'_m = 2000 PSI
Grout	f'_c = 3000 PSI

Architectural & Structural Floor Plans

Figures 5.1 and 5.2 provide a side-by-side reference of the typical architectural floor plan and the redesigned structural framing plan for the Cambria Suites Hotel. As seen, columns and beams are located within or along guest room partition walls.

Typical Architectural Floor Plan
Figure 5.1



Columns

Beams

Plank Span

Typical Structural Framing Plan
Figure 5.2

Proposal Background and Design Goals

Problem Statement

Upon the completion of analyzing the gravity and lateral force resisting systems present in the Cambria Suites Hotel, it is clear that the existing structural system chosen by the design team is currently the most efficient. It was also determined that the structural system meets all architectural, strength, and serviceability requirements governed by code. The gravity system, consisting of concrete masonry walls and an interior steel frame, were sufficiently designed to support the precast concrete planks. In addition, the current concrete shear walls were also efficient in keeping a minimal building deflection and resisting the torsional affects.

With the excellent performance of the current system, it will be difficult to find a comparable system which will replace the existing system. Therefore, when considering an alternative system design, the final design may not prove to be more efficient and/or effective compared to the existing system. That being said, a redesign of Cambria Suite's structural system will be designed in an attempt to find an equally effective and efficient building system.

In regards of the foundation system, it was verified that the existing design for the building was sufficient to transfer all loads for the specified soil class. However, the possibility of increased loads and other effects due to the proposed redesign will require foundation checks to verify it is sufficient for these changes.

Since Cambria Suite's structure is built primarily of concrete masonry walls, it results in a very high overall building weight. Since the hotel is located on a quit challenging site, it would be beneficial if a reduced building weight could be achieved. To determine whether a different building system is equally effective or efficient, it will be compared to the existing system in various categories. These categories will include code limitations, building performance, cost effectiveness, constructability, construction schedule, and material availability.

Proposed Solution

Since the existing concrete masonry wall structure is a heavier system by nature, steel could result in a decreased building weight, creating a lower base shear. As a result, a feasible alternative structural system for the Cambria Suites Hotel would be steel framing. This change will initially affect the foundation and construction management issues like cost and schedule, as well as architectural features such as the building façade due to the removal of exterior masonry walls.

With the modification to a steel framing system, the lateral force and gravity resisting systems will have to be considered as well. The current floor system comprised of hollow-core concrete planks will remain, but will be integrated as a Girder-Slab system using specially designed D-Beams. This innovative D-Beam girder was designed to allow the precast slab to set in on its bottom flange concealing its top flange and web. Once the slabs are set, grout is easily placed flowing around the D-Beam and through its trapezoidal shape web openings and into the slab cores. This process results in a system that develops composite action, enabling it to support residential live loads. This system also results in the removal of all load-bearing masonry walls in the building. The lateral force resisting system will now be comprised of braced frames surrounding the elevator shaft and staircases.

Since the redesign of the structural system uses a different material than the existing system, the existing beams and columns will be altered. The plank span will remain unchanged, whereas the column and beam locations for the girder-slab system may change slightly to line up between rooms. This is done so that the exterior columns do not alter the existing window locations. The redesign will then be thoroughly compared to the existing design to determine whether the alternative system is a more effective and efficient design solution.

Project Goals

The overall design goal of this project is to reduce the total building weight by optimizing the gravity system, as well as the lateral force resisting system. Additional goals to be met through the course of this study include:

- Limit alterations to architectural floor plans
- Reduce column and beam sizes where applicable
- Verify impact on the foundation system
- Research façade options for the proposed building design
- Determine the impacts of construction schedule and cost for the proposed redesign
- Use RAM and ETABS to perform in-depth gravity and lateral analyses
- Determine any architectural effects of structural changes
- Maintain/Reduce floor-to-ceiling height

Gravity System Redesign

This section focuses on the process of the redesign and analysis of the proposed gravity system. As discussed in the proposal, an all steel framing system to resist all gravity loads as opposed to an integrated concrete and steel system will be utilized.

Design Load Summary & Criteria

To fully understand the redesign of the gravity system, an analysis of the gravity loads was done according to ASCE 7-05. The ASCE 7-05 was also the code referenced by the structural engineers at Atlantic Engineering Services (AES), in the design of Cambria Suites Hotel. A summary of the gravity loads used in the structural redesign are shown in Table 1.

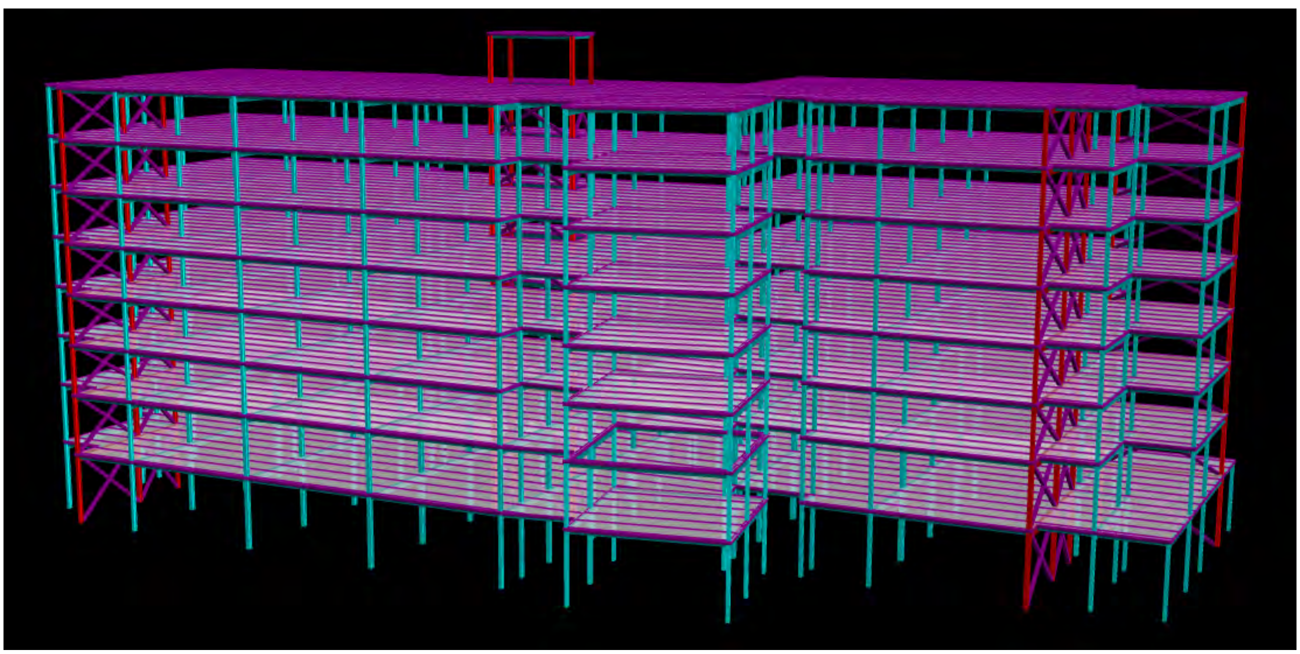
Table 1 - Design Load Summary			
Live Loads (LL)			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Public Areas	100	100	100
Lobbies	100	100	100
First Floor Corridors	100	100	100
Corridors above First Floor	40	40	40
Private Hotel Rooms	40	40	40
Partitions	15	≥15	15
Mechanical	150	150	150
Stairs	100	100	100
Roof	20	20	20
Dead Loads (DL)			
Material	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
8" Concrete Plank w/ topping	Unknown	Section 3.1	81
Steel	Unknown		varies
Partitions	Unknown		10
MEP	Unknown		10
Finishes & Miscellaneous	Unknown		5
Roof	Unknown		20
***Snow Load (SL)**			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Flat Roof	21	21	21

Design Process

Framing Plan

The redesign of the structural system began with determining the initial framing plan. To limit any major architectural changes to the floor plan and exterior façade, column locations were placed in-line with the guest room partition walls. This initial column layout also had no effect on the design of the D-Beams, which are limited in selection based on beam span and precast plank span. However, the addition of more columns will have an impact on a few areas of the Hotel Floor Level.

The hollow-core precast plank will still remain as the typical floor system, but will bear on specially designed D-Beams and Wide flange beams where necessary. The precast plank rests on the bottom flange of the D-Beam, while concealing its top flange and web. This creates a ready surface for either ceiling or floor finishes. The steel framing supporting the Hotel Floor Level will not incorporate the D-Beams due to the increased live load for lobbies. Since the location of the building allows for additional height, typical wide flange beams will be used in this area to carry the extra live load. See Figure 6.1 for the redesigned framing.



RAM Model of Steel Frame
Figure 6.1

Based on the structural redesign, it was necessary to determine whether the existing precast plank system would be reused or if a plank with a decreased thickness and weight would be adequate. Research was first done to determine if a topped floor system would be needed to achieve the 2-hour fire rating. It was found that topping was not necessary to obtain an unrestrained assembly rating, however a minimum topping of 1-1/8" is required for a 3-hour restrained assembly rating. Although a 3-hour rating is not required, topping was chosen for the redesign as it will help with floor vibration and rigidity, as well as create a smooth surface for floor finishes. Therefore, the floor system will be comprised of an 8" precast hollow-core plank with 2" topping. Hollow-core plank design specifications referenced from the PCI Design Handbook *6th Edition* are shown in Figure 6.2. Hand calculations performed to design the plank can be found in Appendix B.

Strand Pattern Designation
76-S

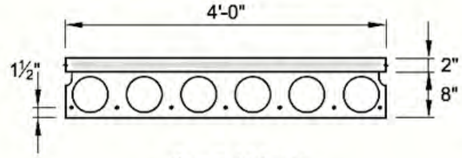
S = straight
Diameter of strand in 16ths
No. of Strand (7)

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 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	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	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	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	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	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	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	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	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	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	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

2–32

Hollow-Core Plank Design Specifications
Figure 6.2

D-Beam, Wide Flange Design

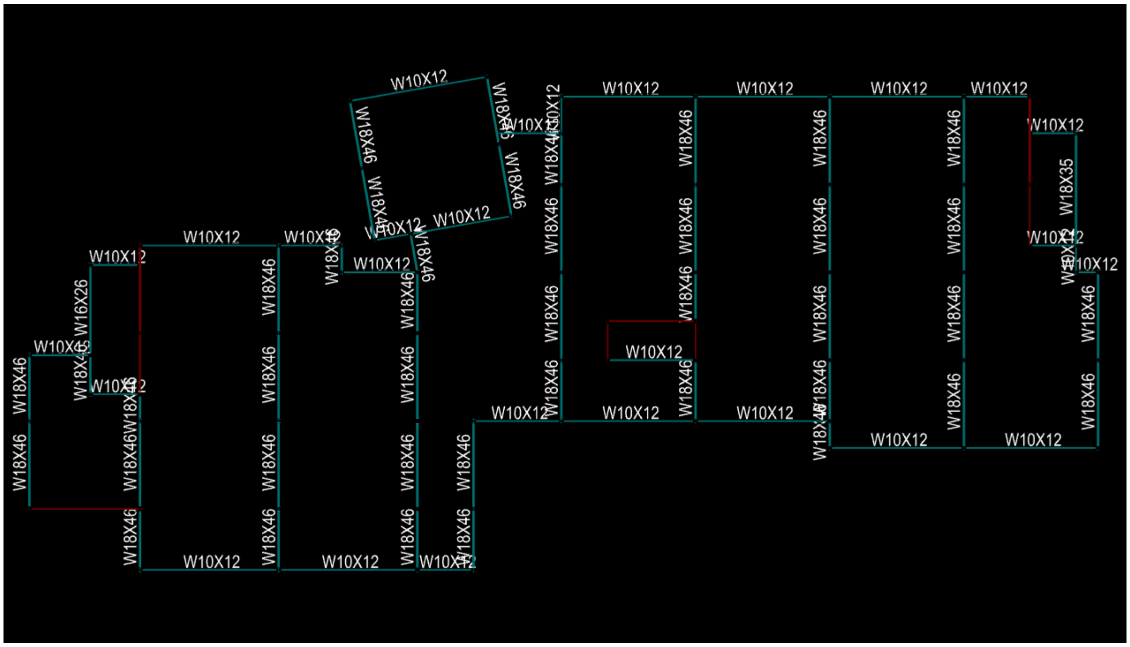
With the design loads and floor system confirmed, the steel members of the gravity system could be designed through the use of hand calculation and computer software. All beams and girders were designed in accordance with Load and Resistance Factor Design (LRFD) methods and the AISC Steel Construction Manual. In addition, to comply with ASCE 7-05, all loads were multiplied by a load factor so that their design strength equaled or exceeded the effects of the factored loads. The Girder-Slab system initially acts as a non-composite system, but turns into a composite steel precast system once the grout is placed around the D-Beam and into the slab cores.

RAM Structural System by Bentley Engineering was used as the primary computer analysis software for the gravity system. RAM was chosen for the redesign of the structural system because of its straightforward design aid for steel structures. However, RAM currently has not incorporated the use of D-Beams into their software. One solution for this problem would be create a steel section with the same properties as the D-Beam itself. A second solution (which was used for this analysis) would be to design all D-Beams as typical wide flange beams with the same weight so that the transferred loads through the structure would be accurate. D-Beams would then be hand calculated where applicable in the design. All D-Beams were designed for the worst case scenario, resulting in a DB 9x46. Spot checks for D-Beams and other wide flange beams were calculated for strength and serviceability criteria. In some cases, the most efficient beam hand calculated and designed in the computer software needed to be altered in order to reduce the floor to ceiling height. A typical D-Beam framing plan is shown in Figure 7.1. All relevant calculations can be found in Appendix B.

Column Design

All columns were designed to comply with LRFD methods and the AISC Steel Construction Manual. Note that all columns were designed to resist gravity loads only. The columns were spliced every two to three stories, occurring at the second and fifth floor levels. The most efficient column sizes were used between each column splice to simplify the design process. Through the use of RAM, all columns were designed to be either W10's or W12's. Column spot checks were performed for interior columns, exterior columns for an interior frame, and an exterior corner column. Column load take downs were performed to determine the loads to each of the columns which were spot checked. In all cases, optimal column sizes determined by hand corresponded to the designed columns by the computer software. Column layouts for all floors are shown in Figures 7.2 through 7.4. All relevant calculations can be found in Appendix B.

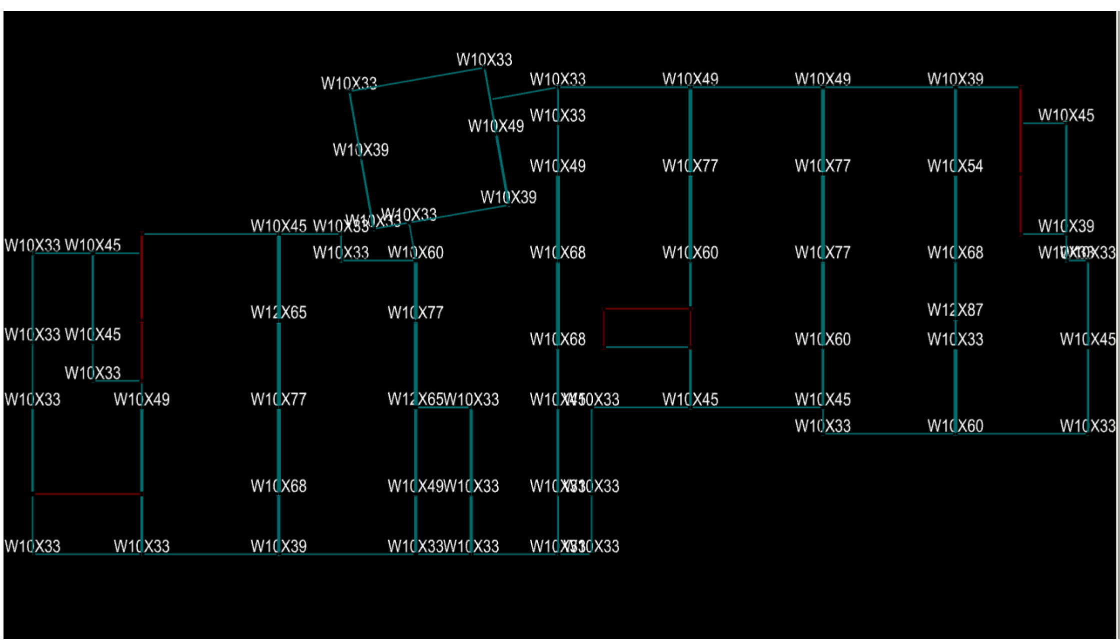
Gravity System Final Design



Typical D-Beam Floor Plan

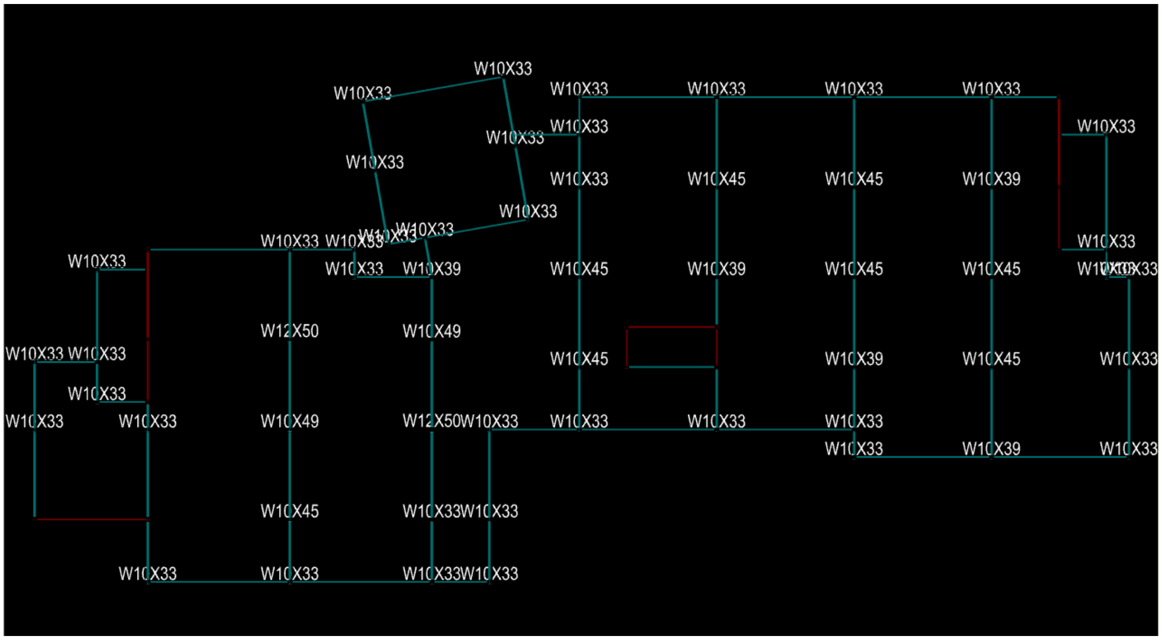
Figure 7.1

Note: DB 9x46 are labeled as W18x46



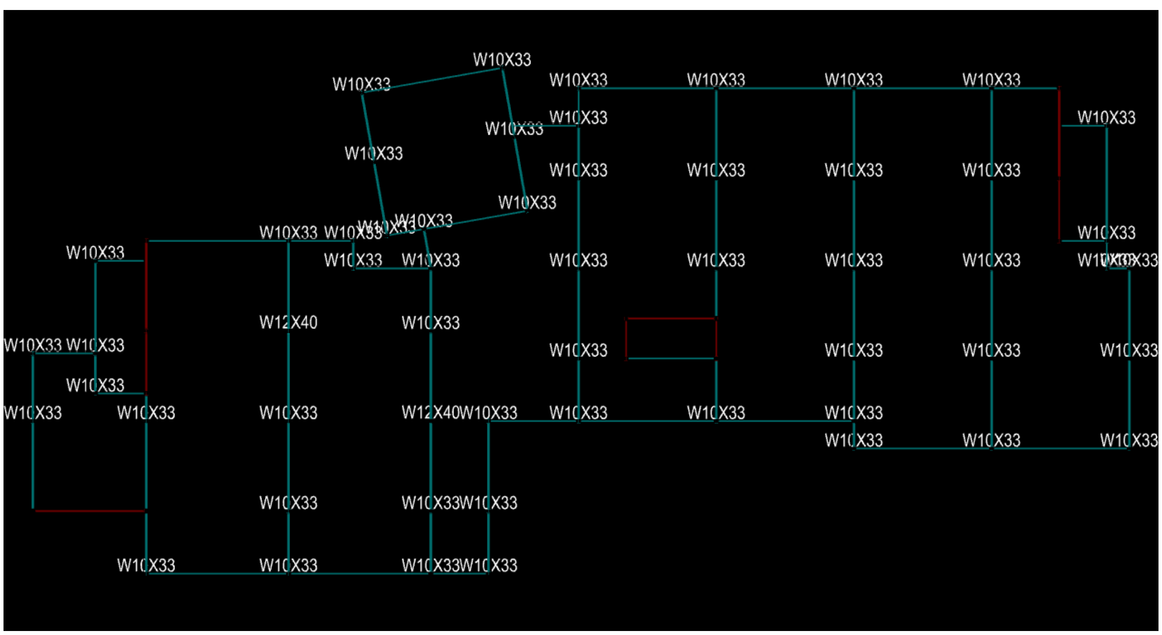
Column Layout for Plaza-Hotel Floor Level

Figure 7.2



Column Layout for 2nd-4th Floor Level

Figure 7.3



Column Layout for 5th-7th Floor Level

Figure 7.4

Lateral Force Resisting System Redesign

The following section discusses the redesign and analysis of the lateral force resisting system of the Cambria Suites Hotel and will determine if the redesign is more optimal than the existing system. As described in the proposal, it was decided to change the lateral force resisting system to concentrically braced frames since the building was being redesigned to a steel framing structure.

Design Loads & Criteria

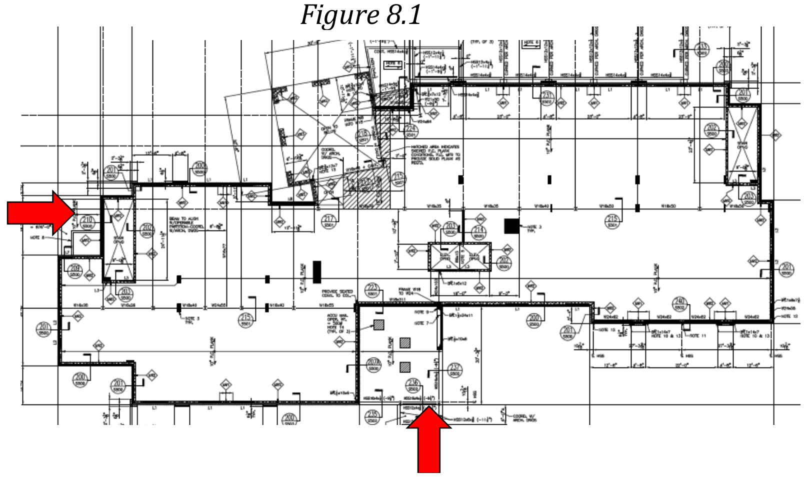
Wind Loads

In the following wind analysis, wind loads were determined according to ASCE 7-05, Chapter 6. Since the overall building height of Cambria Suites hotel reaches 86’-10” (High Roof extends to 102’-2”), it is required to determine the wind loads through the use of Section 6.5: Method 2 – Analytical Procedure because it exceeds the 60’-0” maximum building height stated in Section 6.4: Method 1 – Simplified Procedure. The wind variables used during this analysis to calculate the design wind pressures are located in Table 2. For detailed equations and base calculations used for this procedure, refer to Appendix C. The North/South and East/West wind directions are labeled on the typical floor plan in Figure 8.1.

Wind Directions

Figure 8.1

E/W
Wind



N/S
Wind

Table 2: Wind Variables

Wind Variables			ASCE Reference
Basic Wind Speed	V	90 mph	Fig. 6-1
Directional Factor	K_d	0.85	Table 6-4
Importance Factor	I	1.0	Table 6-1
Occupancy Category		II	Table 1-1
Exposure Category		B	Sec. 6.5.6.3
Enclosure Classification		Enclosed	Sec. 6.5.9
Building Natural Frequency	f_{n1}	1.47 (Rigid)	Eq. C6-19
Topographic Factor	K_{zt}	1.0	Sec. 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	17.1	Eq. 6-15
Gust Effect Factor	G	0.85	Sec. 6.5.8.1
Product of Internal Pressure Coefficient and Gust Effect Factor	GC_{pi}	0.18 -0.18	Fig. 6-5
External Pressure Coefficient (Windward)	C_p	0.80 (All Values)	Fig. 6-6
External Pressure Coefficient (Leeward)	C_p	-0.5 (N/S Direction, L/B = 0.45) -0.2 (E/W Direction, L/B = 2.22)	

***Equation C6 – 19:**

$f_{n1} = (150/H)$ where H = building height (ft.)

$f_{n1} = (150/102.167) = 1.47 \geq 1$ Hz $\quad\therefore$ The building is considered rigid

Tables and calculations of the wind pressures in each direction can be found in Appendix C. The North/South wind direction is of more concern since the wind contacts a building length of 219’-8”, compared to 98’-11” in the East/West direction. The direction of wind is adjacent to a road that services the front of hotel, and a parking garage that does not extend passed the Hotel level of Cambria Suites. Neither obstruction from the front or back of the hotel will cause a significant wind load blockage to the structure.

Seismic Loads

In the following seismic analysis, seismic loads were determined according to ASCE 7-05, Chapters 11 and 12. As identified in Section 1613.1 of the International Building Code (IBC), Cambria Suites Hotel is to be designed and constructed to resist the effects of earthquake motions. According to IBC 2006 criteria, site class for seismic design of "C" should be used for existing conditions. Other variables used in this analysis that are needed to calculate base shear and overturning moments, according to ASCE 7-05, are located in Table 3.

Table 3: Seismic Design Variables				ASCE References
Site Class		C		Table 20.3-1
Occupancy Category		II		Table 1-1
Importance Factor		1.0		Table 11.5-1
Structural System		Ordinary Reinforced Masonry Shear Walls		Table 12.2-1
Spectral Response Acceleration, short	S_s	0.125		Fig. 22-1 thru 22-14
Spectral Response Acceleration, 1 s	S_1	0.049		Fig. 22-1 thru 22-15
Site Coefficient	F_a	1.2		Table 11.4-1
Site Coefficient	F_v	1.7		Table 11.4-2
MCE Spectral Response Acceleration, short	S_{ms}	0.15		Eq. 11.4-1
MCE Spectral Response Acceeration, 1 s	S_{m1}	0.0833		Eq. 11.4-2
Design Spectral Acceleration, short	S_{ds}	0.100		Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{d1}	0.055		Eq. 11.4-4
Seismic Design Category	S_{dc}	A		Table 11.6-2
Response Modification Coefficient	R	2.0		Table 12.2-1
Building Height (above grade)(ft)	h_n	102.167		
		North/South	East/West	
Approximate Period Parameter	C_t	0.02	0.02	Table 12.8-2
Approximate Period Parameter	x	0.75	0.75	Table 12.8-2
Calculated Period Upper Limit Coefficient	C_u	1.7	1.7	Table 12.8-1
Approximate Fundamental Period	T_a	0.643	0.643	Eq. 12.8-7
Fundamental Period	T	1.09	1.09	Sec. 12.8.2
Long Period Transition Period	T_L	12	12	Fig. 22-15
Seismic Respose Coefficient	C_s	0.016	0.016	Eq. 12.8-2
Structural Period Exponent	k	1.295	1.295	Sec. 12.8.3

Note: Seismic Loads are the same in both North/South and East/West direction because the structural type is the same in both directions (Table 12.8-2)

To determine the base shear and total moment which acts on the building, the effective building weight of the redesigned structure needed to be calculated. An Excel spread sheet was created to determine the story weight of each individual floor (above grade), as well as the total building weight. Using the story weight values, the base shear and overturning moments due to seismic loads were then calculated. Please refer to Appendix C for detailed Excel spread sheet calculations.

Since the redesign of the building incorporates the use of steel framing as opposed to the existing concrete masonry, the overall building weight was decreased. Therefore, this reduction in building weight will affect the redesigned structure's base shear and total moment. Table 4, shown below, was created to compare the existing seismic values and the new design values.

All hand calculations for base shear and overturning moment for each floor can be viewed in Appendix C. In addition, Appendix C provides hand calculations for the existing and redesigned story shear for each level.

Table 4: Seismic Comparison

	Existing Building Design	**New Building Design**
Building Weight	20,223 kips	14,260 kips
Base Shear	508.03 kips	228.16 kips
Total Moment	29,463 ft-k	13,468 ft-k

Load Combinations

The following list shows the various load combinations according to ASCE 7-05 for factored loads using strength design and from the International Building Code *2006 edition*. These load combinations are used in the analysis of the lateral system for this report.

$1.4D$

$1.2D + 1.6L + 0.5L_r$

$1.2D + 1.6L_r + 1.0(L \text{ or } W)$

$1.2D + 1.6W + 1.0L + 0.5L_r$

$1.2D + 1.0E + 1.0L$

$0.9D + 1.6W$

$0.9D + 1.0E$

The wind load cases defined in Figure 6-9 of ASCE 7-05 were evaluated to account for torsion in the load combinations stated above. For this report, it is assumed that the ETABS analysis of the seismic load cases in the above load combinations accounted for inherent and accidental torsion. These additional torsional effects are examined in more detail in the report's Torsion section. Relevant calculations for the controlling ASCE 7-05 wind load cases can be referenced in Appendix C.

All load combinations were considered in the analysis of the ETABS model. After evaluating story displacements, shears, and drifts computed by ETABS for each of the above load combinations, it was concluded that the controlling load combination for the North/South direction was $1.2D+1.6W+1.0L+0.5L_r$ due to its large surface area. The controlling load combination for the East/West direction was $0.9D+1.0E$.

Drift Criteria

The following shows the allowable drift criteria according to the International Building Code *2006 Edition* which will be used to check deflection for the redesign of the lateral force resisting system.

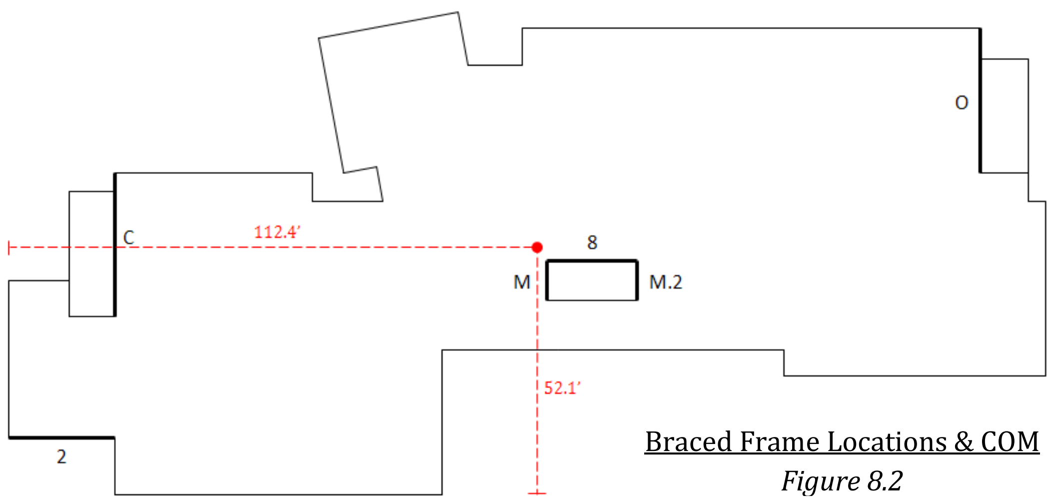
$\Delta_{wind} = H/400$ (Allowable Building Drift)

$\Delta_{seismic} = 0.02H_{sx}$ (Allowable Story Drift)

Design Process

Braced Frame Layout/Design

Since the structural redesign consists of all steel framing members, it was chosen to use concentric braced frames for the lateral force resisting system. Braced frames were selected also because of their lightweight, simple connections, ease of construction, and are more economical. With the removal of the masonry shear walls of the existing lateral system, placement of the braced frames will be crucial to resist all lateral forces. To have the least impact on the architectural layout of the building, braced frames were placed is key areas, such as along staircases, the elevator shafts, and exterior walls with no windows. This resulted in four braced frames in the North/South direction, and two braced frames in the East/West direction. Figure 8.2 below shows the locations of the braced frames in plan view.



Braced Frame Locations & COM
Figure 8.2

The design of the concentric braced frames involved a process that utilized ETABS computer modeling software. ETABS was used because the program effectively determines the relative stiffness of the braced frames, center of mass and center of rigidity of each story, the controlling ASCE 7-05 load combinations, story displacements, story drifts, and the effects of torsion. The ETABS model was simplified to represent lateral members and floor diaphragms only. Diaphragms were considered to be rigid and were modeled as area elements. Gravity loads were then applied to the diaphragms as additional area masses. Initially, the sizes of the lateral frame members were based on the gravity load analysis from RAM, choosing HSS members for the braces. The lateral members were then analyzed in ETABS by manually inputting the wind and seismic loads which were determined using Method 2 – Analytical Procedure from ASCE 7-05. Since the braces were released of end fixity, they do not carry moment and can be evaluated as axial members. Axial loads determined from the ETABS output, as well as Table 4-4 of the Steel Manual were used to evaluate the strength of the braces. The designs of the columns in the braced frames were evaluated as beam-columns. After applying the necessary factors, modified interaction equations taken from Chapter H - *Design of Members for Combined Forces and* Torsion, as well as Part 6 of the Steel Specification were used to evaluate the data collected. Since the diaphragms of the ETABS model were defined as rigid, the beams do not carry axial loads. Therefore, beams were evaluated as simple flexural members by obtaining maximum moments determined from the ETABS output and using Table 3-2 of the Steel Specification to evaluate the strength of the flexural members. Lateral force members were then resized if necessary for strength requirements. Finally, a check was performed to make sure drift limitations were met in accordance to H/400. Please refer to Appendix D summarizing the results of the lateral force resisting system.

Load Path and Distribution

Lateral force resisting systems transfer all lateral loads (wind and/or seismic) to the building's foundation where the loads dissipate. In the case of Cambria Suites Hotel, the hollow-core concrete plank serves as the rigid diaphragm which transfers the lateral loads to the lateral force resisting system. As previously discussed, the lateral force resisting system consists of concentrically braced frames that are located near the building's core and near the exterior of the building. The HSS cross braces transfer the lateral loads from the diaphragm to the wide-flange steel columns of the lateral system. The wide-flange steel columns then transfer the lateral loads down through the building, until transferring the loads to the grade beam foundation. Finally, the loads are transferred from the grade beam to the concrete caissons which transfer the loads into bedrock.

The distribution of the lateral loads is dependent on the relative stiffness of each braced frame. Braced frames with higher relative stiffness resist more of the lateral load. In determining the relative stiffness of each braced frame, an arbitrary force of 100 kips was applied in the respective direction to each individual frame. Story displacements were then determined from the ETABS output, and were applied to calculate the story stiffness. Once story stiffness was calculated, the relative story stiffness was calculated for each individual frame to determine the distributed lateral loads. Relative story stiffness in both the North/South and East/West direction are shown in Tables 5 and 6.

Table 5: Relative Story Stiffness, R_{ix}

	North-South Frames (X-Direction) Displacement, Δ_p (in.)		Arbitrary Unit Load,P (kips)	Story Stiffness, K_i $K_{ix} = P/\Delta_p$		Total Story Stiffness $K_{ix,total}$	Relative Story Stiffness, R_i $R_{ix} = K_{ix}/K_{ix,total}$	
	Frame 2	Frame 8		Frame 2	Frame 8		Frame 2	Frame 8
Roof	14.928	22.874	100	6.70	4.37	11.07	0.605	0.395
7	13.030	19.698	100	7.67	5.08	12.75	0.602	0.398
6	11.086	16.492	100	9.02	6.06	15.08	0.598	0.402
5	9.092	13.274	100	11.00	7.53	18.53	0.593	0.407
4	7.102	10.132	100	14.08	9.87	23.95	0.588	0.412
3	5.168	7.168	100	19.35	13.95	33.30	0.581	0.419
2	3.374	4.522	100	29.64	22.11	51.75	0.573	0.427
Hotel Level	1.621	2.005	100	61.69	49.88	111.57	0.553	0.447

Table 6: Relative Story Stiffness, R_{iy}

Level	North-South Frames (Y-Direction) Displacement, Δ_p (in.)				Arbitrary Unit Load, P (kips)	Story Stiffness, K_i $K_{iy} = P/\Delta_p$				Total Story Stiffness $K_{iy,total}$	Relative Story Stiffness, R_i $R_{iy} = K_{iy}/K_{iy,total}$			
	Frame C	Frame M	Frame M.2	Frame O		Frame C	Frame M	Frame M.2	Frame O		Frame C	Frame M	Frame M.2	Frame O
Roof	7.35	108.98	85.07	8.99	100	13.60	0.92	1.18	11.12	26.81	0.507	0.034	0.044	0.415
7	6.39	92.15	71.45	7.80	100	15.65	1.09	1.40	12.82	30.96	0.506	0.035	0.045	0.414
6	5.42	75.45	57.96	6.62	100	18.44	1.33	1.73	15.11	36.61	0.504	0.036	0.047	0.413
5	4.44	59.13	44.84	5.42	100	22.51	1.69	2.23	18.46	44.90	0.501	0.038	0.050	0.411
4	3.47	43.58	32.50	4.23	100	28.82	2.29	3.08	23.65	57.84	0.498	0.040	0.053	0.409
3	2.55	29.33	21.47	3.09	100	39.26	3.41	4.66	32.40	79.73	0.492	0.043	0.058	0.406
2	1.72	17.04	12.37	2.04	100	58.31	5.87	8.08	49.00	121.26	0.481	0.048	0.067	0.404
Hotel Level	0.87	5.96	4.43	0.99	100	115.47	16.78	22.57	100.70	255.53	0.452	0.066	0.088	0.394

As Figures 6 and 7 above demonstrate, Frame 2 resists an average of 58.7% of the lateral loads acting in the East/West direction as opposed to an average of 41.3% resisted by Frame 8. In the North/South direction, Frame C and O resist the majority of the lateral loads for an average of 49.25% and 40.8% respectively. The final 9.95% is distributed evenly between Frames M and M.2.

Upon calculating the relative story stiffness of each frame, the Center of Rigidity (COR) was determined for each story level and compared to the ETABS output. However, a slight difference exists between the ETABS output and the hand calculated values because when determining rigidity, ETABS takes into account the stiffness of the floor diaphragms and frames whereas the calculated COR values only consider the stiffness of the frames. Appendix C summarizes the calculations performed in determining the COR for each story level. For consistency throughout this report, the COR values obtained from the ETABS model will be used for any other necessary calculations.

Direct Shear

Upon determining the governing load combination for the North/South and East/West direction, as well as the relative story stiffness for each frame, direct forces were computed and applied to the ETABS computer model. In calculating the direct shear in each frame, its relative stiffness at a given story level is multiplied by the factored lateral force acting at the respective story level. Tables 7 and 8 summarize the distributed forces to each lateral force resisting frame.

Table 7: East/West Direct Shear Due To Seismic

0.9D+1.0E

Level	Force (k)	Factored Force (k)	Relative Story Stiffness		Distributed Force (k)	
			Frame 2	Frame 8	Frame 2	Frame 8
Roof	48.28	48.28	0.605	0.395	29.21	19.07
7	46.72	46.72	0.602	0.398	28.12	18.60
6	39.01	39.01	0.598	0.402	23.33	15.68
5	31.62	31.62	0.593	0.407	18.77	12.85
4	24.63	24.63	0.588	0.412	14.48	10.15
3	18.04	18.04	0.581	0.419	10.48	7.56
2	11.97	11.97	0.573	0.427	6.86	5.12
Plaza	5.62	5.62	0.553	0.447	3.11	2.51

Table 8: North/South Direct Shear Due To Wind

$1.2D+1.6W+1.0L+0.5L_R$

Level	Force (k)	Factored Force (k)	Relative Story Stiffness				Distributed Force (k)			
			Frame C	Frame M	Frame M.2	Frame O	Frame C	Frame M	Frame M.2	Frame O
Roof	40.86	65.37	0.507	0.034	0.044	0.415	33.15	2.24	2.87	27.12
7	40.20	64.32	0.506	0.035	0.045	0.414	32.52	2.25	2.91	26.64
6	39.10	62.56	0.504	0.036	0.047	0.413	31.52	2.27	2.95	25.83
5	38.00	60.80	0.501	0.038	0.050	0.411	30.49	2.29	3.02	25.01
4	36.73	58.77	0.498	0.040	0.053	0.409	29.28	2.33	3.13	24.03
3	35.39	56.62	0.492	0.043	0.058	0.406	27.88	2.42	3.31	23.01
2	32.56	52.09	0.481	0.048	0.067	0.404	25.05	2.52	3.47	21.05
Plaza	36.85	58.96	0.452	0.066	0.088	0.394	26.65	3.87	5.21	23.24

Torsional Shear

In addition to direct shear, a torsional shear force is present on the building due to the torsional moments produced on each floor caused by the eccentricity. Thus, each concentrically braced frame will have to resist this additional force. Depending on the location of the lateral frame with respect to the center of rigidity, the following equation will be used to calculate the total shear which is resisted by each frame.

$$F_{total} = F_{direct} \pm F_{torsional}$$

Table 9 shows a comparison of the total shear obtained from the ETABS output to the calculated direct shear, at the 4th level. After evaluating the results in this table, it is clear that the total shear resisted by the lateral system is significantly affected by the torsional shear forces. In order to fully understand the extent to which the torsional shear affects the behavior of the lateral force resisting system, a more in-depth analysis would be required.

Table 9: Total Shear/Direct Shear Comparison at Level 4

Frame	ETABS Total Shear (k)	Calculated Direct Shear (k)	Direction	Controlling Load Case
C	154.62	29.28	North/South	$1.2D+1.6W+1.0L+0.5L_R$
M	33.86	2.33		
M.2	29.98	3.13		
O	99	24.03		
2	27.42	10.15	East/West	0.9D+1.0E
8	35.66	14.48		

Torsion

Torsion is present when the center of rigidity and the center of mass do not occur at the same location. Eccentricity (the distance between the center of rigidity and center of mass) induces a moment, which creates an additional force on the building called torsional shear. When determining the torsional effects on the building, two different types of torsional moment need to be taken into account.

According to ASCE 7-05, torsion for rigid diaphragms is the sum of the inherent torsional moment and the accidental torsional moment. The inherent torsional moment, M_t, is a result from the eccentricity between the locations of the center of rigidity and center of mass. This eccentricity times the lateral force at the specified floor level will give the inherent torsional moment. The accidental torsional moment, M_{ta}, is caused by an assumed displacement of the center of mass. This displacement is 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 produced can be seen in Table 10. Appendix C shows detailed calculations for building torsion.

Table 10: Seismic Torsional Effects										
Level	East-West (X-Direction)					North-South (Y-Direction)				
	Factored Story Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	M_{total} (ft-k)	Factored Story Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	M_{total} (ft-k)
Roof	48.28	-5.3	-255.89	530.29	274.40	48.28	-11.9	-574.54	238.80	-335.74
7	46.72	-5.7	-266.33	513.20	246.87	46.72	-12.2	-570.04	231.10	-338.94
6	39.01	-6.1	-237.93	428.42	190.48	39.01	-12.5	-487.57	192.92	-294.65
5	31.62	-6.3	-199.21	347.31	148.10	31.62	-13.0	-411.07	156.40	-254.67
4	24.63	-6.0	-147.75	270.47	122.72	24.63	-13.7	-337.37	121.80	-215.57
3	18.04	-4.9	-88.41	198.17	109.76	18.04	-14.9	-268.83	89.24	-179.59
2	11.97	-2.4	-28.73	131.49	102.76	11.97	-16.6	-198.72	59.21	-139.51
Plaza	5.62	1.9	10.68	61.71	72.39	5.62	-19.2	-107.88	27.79	-80.09
				Total:	1267.48				**Total:**	-1838.77

Drift and Displacement

The overall drift is a concern for nonstructural members and should be limited as much as possible. Building drift and deformation becomes a larger factor as the height of the building increases. According to IBC 2006, wind load drift is limited to an allowable drift of $\Delta = \ell/400$, whereas the seismic drift is limited to an allowable drift of $\Delta = 0.02h_{sx}$. Wind controls the drift in the North/South direction of the building and the seismic forces control the drift in the East/West direction. The allowable building drift limit for Cambria Suites Hotel will be:

$$\Delta limit = 1042"/400 = 2.605"$$

Wind drifts were computed by ETABS and were evaluated against the allowable drift acceptable by industry standard. Tables 11 and 12 were created to provide a summary of the wind drifts in both the North/South and East/West direction. As seen in the tables, the total building drift at the roof level, in both wind directions, is acceptable based on the industry standard.

Table 11: Controlling Wind Drift (X-Direction)				
Level	Height Above Ground, h (ft)	Allowable Drift $\Delta_{allowable} = h/400$	Total Drift (ETABS)	Adequate?
Roof	86.833	2.60	1.37	OK
7	76.833	2.30	1.17	OK
6	66.833	2.00	0.99	OK
5	56.833	1.70	0.82	OK
4	46.833	1.40	0.64	OK
3	36.833	1.10	0.47	OK
2	26.833	0.80	0.32	OK
Plaza	14.833	0.44	0.14	OK

Table 12: Controlling Wind Drift (Y-Direction)				
Level	Height Above Ground, h (ft)	Allowable Drift $\Delta_{allowable} = h/400$	Total Drift (ETABS)	Adequate?
Roof	86.833	2.60	1.91	OK
7	76.833	2.30	1.56	OK
6	66.833	2.00	1.36	OK
5	56.833	1.70	1.1	OK
4	46.833	1.40	0.86	OK
3	36.833	1.10	0.62	OK
2	26.833	0.80	0.48	OK
Plaza	14.833	0.44	0.06	OK

Seismic drifts were computed by ETABS and were evaluated against the allowable story drifts using Table 12.12-1 in ASCE 7-05. This table specifies for an Occupancy Category II, an allowable drift of $0.02h_{sx}$ is acceptable, where h_{sx} is the story height below the considered floor level. Tables 13 and 14 were created to show a summary of the seismic drifts for both the North/South and East/West direction. As seen in the tables, the seismic story drifts computed by ETABS do not exceed the allowable drifts.

Table 13: Controlling Seismic Drift (X-Direction)				
Level	Height of Story, h_{sx} (ft)	Allowable Story Drift $\Delta_{allowable} = 0.02h_{sx}$	Total Drift (ETABS)	Adequate?
Roof	10	0.20	0.005	OK
7	10	0.20	0.005	OK
6	10	0.20	0.004	OK
5	10	0.20	0.004	OK
4	10	0.20	0.004	OK
3	10	0.20	0.003	OK
2	12	0.24	0.002	OK
Plaza	14.83	0.30	0.001	OK

Table 14: Controlling Seismic Drift (Y-Direction)				
Level	Height of Story, h_{sx} (ft)	Allowable Story Drift $\Delta_{allowable} = 0.02h_{sx}$	Total Drift (ETABS)	Adequate?
Roof	10	0.20	0.015	OK
7	10	0.20	0.014	OK
6	10	0.20	0.013	OK
5	10	0.20	0.012	OK
4	10	0.20	0.01	OK
3	10	0.20	0.007	OK
2	12	0.24	0.007	OK
Plaza	14.83	0.30	0.002	OK

Impact on Foundation

Overturning Moment

Since lateral forces and moments are exerted on the building, overturning effects must be considered. These overturning moments are a concern due to the impact that they could potentially have on the foundation system. Therefore, a calculation must be conducted to determine if the dead load of the building will be sufficient enough to resist the impact of the overturning moments. As shown in table 15, total overturning moments are provided due to wind and seismic loads. Note that the wind loads controlled in the North/South direction, whereas the seismic loads controlled in the East/West direction. In order to verify that the dead load was adequate to resist these overturning moments due to wind and seismic loads, the stresses due to the lateral loads were compared to the stresses due to the self-weight of the building. It was concluded that the stresses due to the lateral loads were such a small fraction of the stresses due to the dead loads; thus the foundation will experience minimal overturning affects. However, a force will be present along the perimeter of the building due to the moment exerted on the structure. Detailed calculations for overturning moments can be found in Appendix E.

Table 15: Overturning Moments

Floor	Height Above Ground Z (ft)	Story Height (ft)	N/S Wind Forces		E/W Seismic Forces	
			Lateral Force F_x (k)	Total Moment M_x (ft-k)	Lateral Force F_x (k)	Total Moment M_x (ft-k)
PH Roof	102.167	15.333	8.88	839.16	2.27	214.58
Roof	86.833	10	40.86	3343.58	48.28	3950.93
7	76.833	10	40.20	2887.66	46.72	3356.39
6	66.833	10	39.10	2417.75	39.01	2411.84
5	56.833	10	38.00	1969.80	31.62	1639.00
4	46.833	10	36.73	1536.48	24.63	1030.15
3	36.833	10	35.39	1126.53	18.04	574.34
2	26.833	10	32.56	710.78	11.97	249.40
1	14.833	12	36.85	325.51	5.62	41.67
Plaza	0	14.833	0	0	0	0
		Total =	308.57	15157.26	228.16	13468.29

Foundation Caissons

To evaluate the foundation impact due to the redesigned structural system, the number of concrete caissons to support the existing structural system will be compared to the required number of caissons for the redesigned steel structural system.

The existing foundation utilizes a combination of concrete caissons which tie into grade beams. Typical caissons are 30", 36", or 42" in diameter and are embedded in bedrock with an allowable end bearing pressure of 15 ton/SF. Typical grade beams span along the exterior of the building and are sized at 24"x36" or 30"x36". Table 16 below summarizes the number of caissons required for each structural system.

Table 16: Number of Caissons	
Structural System	No. of Caissons
Existing CMU System	74
Redesigned Steel System	76

After evaluating Table 16, it is clear that both systems require roughly the same amount of concrete caissons. However, since the redesigned steel structure is lighter in weight, it can be expected that the caissons will be redesigned with a smaller diameter and require less reinforcement due to the reduced loads being transferred to them. This will ultimately reduce the cubic yards of concrete and steel reinforcement used for the foundation, which reduces the overall cost of the foundation.

Breadth Study 1: Architectural/Façade Study

In the existing façade design, the load bearing concrete masonry walls did not allow for many design options. The typical exterior wall construction consisted of brick veneer against a CMU wall. Converting the structural system from a concrete masonry structure to a steel structure will change the architectural features of the building, especially the exterior façade design. For this study, research was performed to find a new architectural system which could possibly create a more flexible layout and aesthetical look compared to the existing design. With the use of steel along the exterior of the structure, façade systems such as curtain walls and non-load bearing masonry walls will be researched. For the purpose of this study, the façade alterations will only occur along the north façade of the building. Heat loss calculations, as well as cost and schedule of each alternate facade will be compared to the existing façade.

Thermal Gradient Comparison

In order to determine the overall heat transfer through each wall system, it was necessary to determine the thermal resistance (R-value) for each material with each wall system. The ASHRAE Fundamentals Handbook was used to determine the R-values for the brick veneer system, as well as the existing CMU/Masonry system. The curtain wall system is a Kawneer 7500 Wall System and the R-values were determined from the product specifications. Upon determining the R-values, the change in temperature through each material of each system was calculated using the following equation:

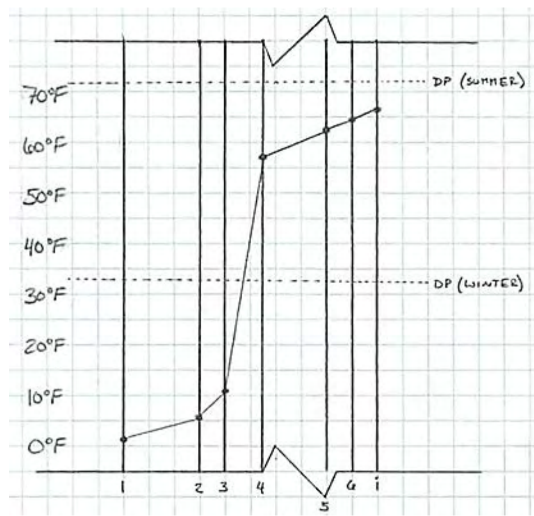
$$T_x = T_{outdoor} + (T_{indoor} - T_{outdoor})(\Sigma R_{o\text{-}x}/\Sigma R_{o\text{-}i})$$

The following assumptions were made for these calculations:

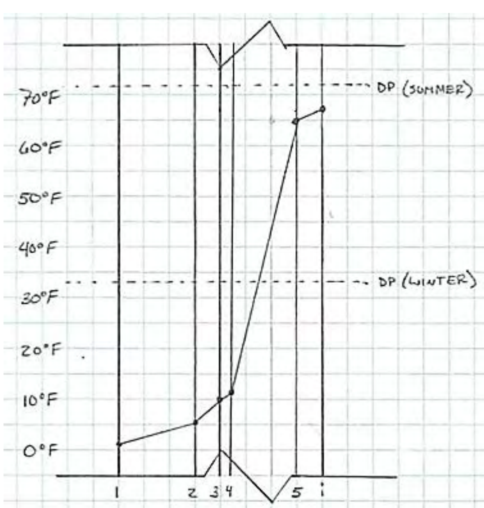
1. The outdoor air temperature ($T_{outdoor}$) was taken as 2°F
2. The indoor air temperature (T_{indoor}) was taken as 70°F
3. The relative humidity and dew point values were taken for Pittsburgh, PA

Figures 9.1, 9.2, and 9.3 were created to show the thermal gradients for the original CMU/masonry façade, as well as the alternate curtain wall and brick veneer wall systems. Detailed calculations for how the thermal gradients were determined can be found in Appendix F.

Existing CMU/Masonry System		
Between Material	ΣR_{o-x} (°F-ft^2-h/BTU)	Temperature (°F)
0-1	0.17	2
1-2	0.81	5.75
2-3	1.79	10.3
3-4	12.06	57.9
4-5	13.09	62.7
5-6	13.55	64.9
6-i	14.01	66.98
Total	14.66	70
U-Value = 0.0682 (BTU/°F-ft^2-h)		



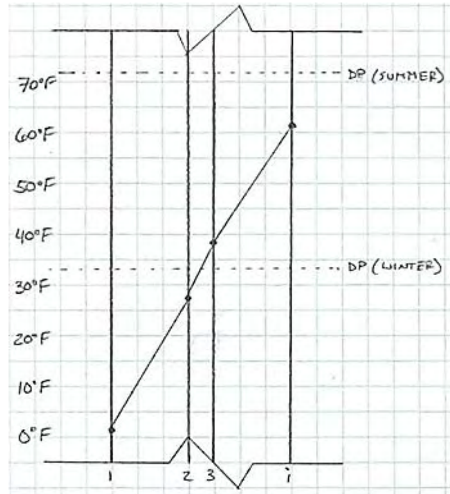
Existing Façade Thermal Gradient
Figure 9.1



Brick Veneer Thermal Gradient
Figure 9.2

Brick Vaneer System		
Between Material	ΣR_{o-x} (°F-ft^2-h/BTU)	Temperature (°F)
0-1	0.17	2
1-2	0.81	5.62
2-3	1.79	10
3-4	1.91	10.54
4-5	14.1	65.08
5-i	14.56	67.14
Total	15.2	70
U-Value = 0.0658 (BTU/°F-ft2-h)		

Curtain Wall System		
Between Material	ΣR_{o-x} (°F-ft^2-h/BTU)	Temperature (°F)
0-1	0.17	2
1-2	2.045	27.62
2-3	0.98	38.95
3-i	2.045	62.6
Total	0.64	70
U-Value = 0.17 (BTU/°F-ft2-h)		



Curtain Wall Thermal Gradient

Figure 9.3

Cost and Construction Time Comparison

A rough estimate was performed using RS Means to compare the cost and construction time of the existing wall system versus the alternative wall systems. The estimate for each wall system is based on the square footage of just the north façade of the building. The construction time for the CMU/Brick system is the time it takes to build five stories of CMU wall and then begin the masonry veneer until completion. In addition, both masonry systems consider scaffolding into the estimate. The estimate for each wall system is summarized in Table 17 below.

Table 17: Façade Comparisons							
Façade of Existing System							
Wall System	S.F.	Crew Size	Material Cost/SF	Labor Cost/SF	Total Cost	Daily Output	Construction Time
CMU/Brick System	19,016	3 Bricklayers, 3 Bricklayer Helpers	$9.40	$21.00	$578,096	159	120 days
Façade Systems for Redesigned System							
Wall System	S.F.	Crew Size	Material Cost	Labor Cost	Total Cost	Daily Output	Construction Time
Curtain Wall System	19,016	2 Glaziers, 2 Structural Steel Workers	$33.50	$7.05	$771,099	205	93 days
Brick Veneer System/Metal Stud Backup	19,016	3 Bricklayers, 2 Bricklayer Helpers	$6.95	$15.55	$427,860	230	83 days

Conclusions

Upon evaluating the heat transfer for each wall system, it is clear that the CMU/Brick system and the Brick Veneer system are more efficient than that of the Curtain Wall system. The Curtain Wall system transfers approximately 38.7% more BTU/hr than the CMU/Brick or Brick Veneer systems. Therefore, it can be concluded that utilizing either the existing or Brick Veneer system would minimize the heat loss through the exterior façade of the hotel. Although not using the Curtain Wall system will eliminate the possibility of an aesthetical look to the hotel's exterior enclosure, it will allow for optimum comfort for the hotel guests.

With respect to the construction schedule, the time it takes to construct the brick veneer façade is much quicker than the CMU/Brick system. This is due to the additional time to construct the CMU backup wall before the brick veneer construction can begin. In addition, the total cost to build the Brick Veneer system is cheaper than both the CMU/Brick and Curtain Wall system, which makes the Brick Veneer system the most efficient system for the redesigned building.

Breadth Study II: Construction Management

To further determine which structural system would be most practical for the Cambria Suites Hotel, a cost and schedule comparison was performed between the existing CMU bearing walls and the steel framing structure. In modifying the existing structure to steel framing, the erection time should be faster, resulting in a reduced construction schedule. The elimination of concrete shear walls for the lateral system will also speed up the overall construction schedule. Since the original opening time for Cambria Suites Hotel is half way through the Pittsburgh Penguins season, a reduced construction schedule would allow for a sooner opening date.

The structural redesign of Cambria Suites Hotel did not have a significant impact on the foundation. Therefore for this study, it is assumed that the foundation is complete before comparing the impact of the construction schedule and cost for both systems.

Construction Schedule Comparison

Construction Schedule of Existing Structural System

The existing structural system of the Cambria Suites Hotel was scheduled to begin on January 6, 2010. The CMU/Brick system was estimated to take approximately seven months, being completed on August 10, 2010.

A schedule for the construction of the structural system coordinates the erection of CMU bearing walls, steel members, precast concrete plank, and masonry veneer. A summary of the construction time is provided in Table 18. A detailed construction schedule of the existing construction schedule is provided in Appendix G.

Construction Schedule of Redesigned Structural System

The redesigned structural system will have the same start date of January 6, 2010. The Steel system was estimated to take approximately five months, being completed on May 31, 2010.

By modifying the structural system to steel, a substantial amount of the construction time was saved. Ignoring the construction of the masonry façade, it took 177 days to erect the existing CMU/Plank system, as opposed to 75 days to erect the redesigned Steel/Plank system. A mock construction schedule for the redesigned structural system was created which coordinates the erection of steel members and pre-cast concrete plank. Please refer to Appendix G for a detailed construction schedule of the structural system.

Table 18 was created to show a side by side comparison of the construction time of the existing system versus the redesigned system. As shown, the redesigned structural system reduced the construction time by 102 days.

Table 18: Construction Time Comparison			
Component	Existing System (days)	Redesigned System (days)	Savings
CMU Walls	139	0	+139
Steel Frame	10	50	-40
Pre-Cast Plank	28	25	+3
Total	177	75	**+102**

Cost Comparison

A simplified cost estimate was created to compare the materials used in the existing structural system and the redesigned structural system. Material, labor, and equipment costs were taken from the RS Means Cost Data 2011 and were used to create a cost estimate summaries for both systems (As shown in Tables 19 & 20). Detailed material takeoffs which support all cost estimate calculations can be found in Appendix G. Note that similar materials present in both systems were omitted in the cost estimate. Other assumptions are as follows:

- The foundation system was not modified, therefore not included in estimate
- Cost of steel connections was not included in estimate

Cost Estimate of Existing System

Table 19: Cost Estimate of Existing System								
Shearwalls	Amount	Unit	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
8" CMU, reinforced	59904	SF	2.62	4.03	0	6.65	9.35	560102.40
12" CMU, reinforced	12339	SF	3.65	6.25	0	9.9	14	172746.00
Steel	Amount	Unit	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
Columns	1224	LF	84	2.7	1.65	88.35	99	121176.00
Baseplates	52.2	SF	46	0	0	46	0	2401.20
Beams	2888	LF	68	3.45	1.56	73.01	83	239704.00
Fireproofing	27180	SF	1.31	0.29	0.04	1.64	1.95	53001.00
Concrete	Amount	Unit	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
10" P.C. Plank	120000	SF	7.5	0.95	0.53	8.98	10.55	1266000.00
							Total Cost of Existing System:	2415130.60

Cost Estimate of Redesigned System

Table 20: Cost Estimate of Redesigned System								
Steel	Amount	Unit	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
Columns	4986	LF	84	2.7	1.65	88.35	99	493614.00
Baseplates	119.2	SF	46	0	0	46	0	5483.20
Beams	9435	LF	62	3.99	1.8	55.29	63.5	599122.50
Braces	2368	LF	47.14	3.79	2.32	53.25	n/a	126096.00
Fireproofing	95400	SF	1.31	0.29	0.04	1.64	1.95	186030.00
Concrete	Amount	Unit	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
8" P.C. Plank	120000	SF	7.2	1.07	0.6	8.87	10.45	1254000.00
							Total Cost of Redesigned System:	2664345.70

Table 21 was created to show a side by side comparison of the cost estimate of the existing system versus the redesigned system.

Table 21: Overall Cost Comparison			
Component	Existing System	Redesigned System	Additional Cost
CMU Walls	$732,848.40	$0.00	-$732,848.4
Steel Bracing	$0.00	$126,096.00	$126,096
Steel Framing	$416,282.20	$1,284,249.70	$994,063.5
Pre-Cast Plank	$1,266,000	$1,254,000	-$12,000
Total	**$2,415,130.60**	**$2,664,345.70**	**$249,215.1**

Conclusions

Upon evaluating the construction schedule of the existing and redesigned structural systems, it is clearly evident that the use of the steel structural system significantly reduced the construction schedule. Since steel structures can be constructed at a much faster rate compared to CMU systems, the construction time was able to be reduced by 57.6%. In regards to the construction cost of the structural systems, the cost to build either system was fairly similar. To build the steel system as opposed to the CMU system, it only increased cost by 9.4%. This result is mainly due to the elimination of CMU walls and increased amount of steel members for the redesigned structural system. In the event that the building owner wanted the hotel to be completed by an earlier deadline and has a slightly higher budget, the use of the Girder-Slab composite steel and pre-cast plank system could be an efficient option.

Conclusion and Recommendations

The overall focus of this final thesis report is to reduce the total building weight by optimizing the gravity system, as well as the lateral force resisting system. Since the existing masonry wall structure is a heavier system by nature, it was necessary at the time of design to utilize a steel structural system which would decrease the total building weight and reduce the loads being transferred to the foundation.

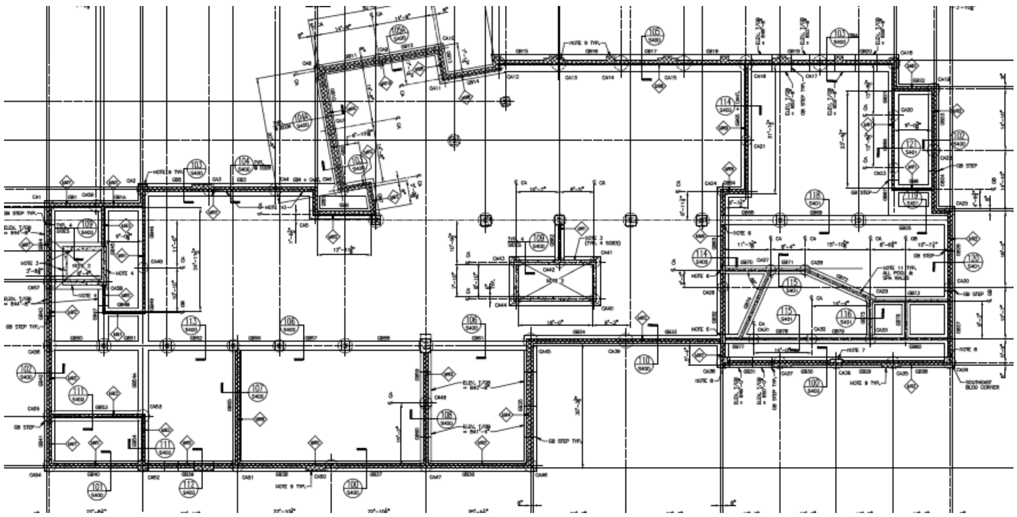
The gravity system consists of the Girder-Slab system which uses specially designed D-Beams for which the precast plank can rest on. Modifying the existing structural system to the Girder-Slab system proved to significantly reduce the overall building weight by 29.5%. This reduced building weight also resulted in a 55.1% reduction in base shear and a 54.3% reduction in total moment. The design of the framing plan conformed easily to the existing hotel floor layout while maintaining the floor-to-ceiling height. However, an increase in columns needed for the Girder-Slab system will have an effect on the open layout of the Hotel Floor Level. In relation to construction schedule and cost, the redesigned structural system reduced the construction schedule by 57.6%, but slightly increasing cost by 9.4%.

To maintain a common building material, the lateral force resisting system was comprised of concentrically braced frames. Compared to the existing shear wall lateral system, braced frames are lighter in weight, are quickly constructed, and are economical. The lateral frames were easily laid out in the East and West stair cases, the core elevator shaft, and along an exterior South-West wall. This configuration, along with the design of the lateral members, proved to be a sufficient system while maintaining an overall building drift within code limitations.

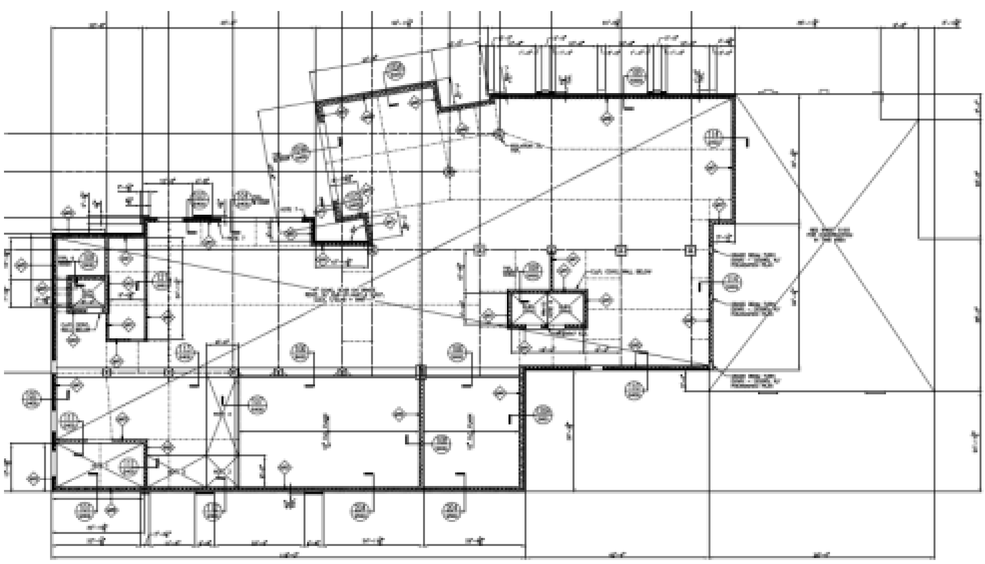
The façade breadth was conducted to indicate the architectural effects due to the removal of the exterior masonry walls. The overall focus was to improve guest comfort pertaining to natural daylight against the heat transfer through a particular wall system. Although implementing the brick veneer system will eliminate the possibility of natural daylight and an aesthetical exterior look, it provides an efficient wall system with a lower heat transfer rate which will ultimately create a comfortable interior environment.

The goals of this thesis report were to design an equally effective and efficient structural system for the Cambria Suites Hotel. Based on the data and results throughout the report, it is clear that these goals are met. If a minimal cost increase and minor floor layout changes on the Hotel Floor level were not an issue to the building owner, the alternative steel structural system could be implemented as the final design as each study impacts the building in a positive way.

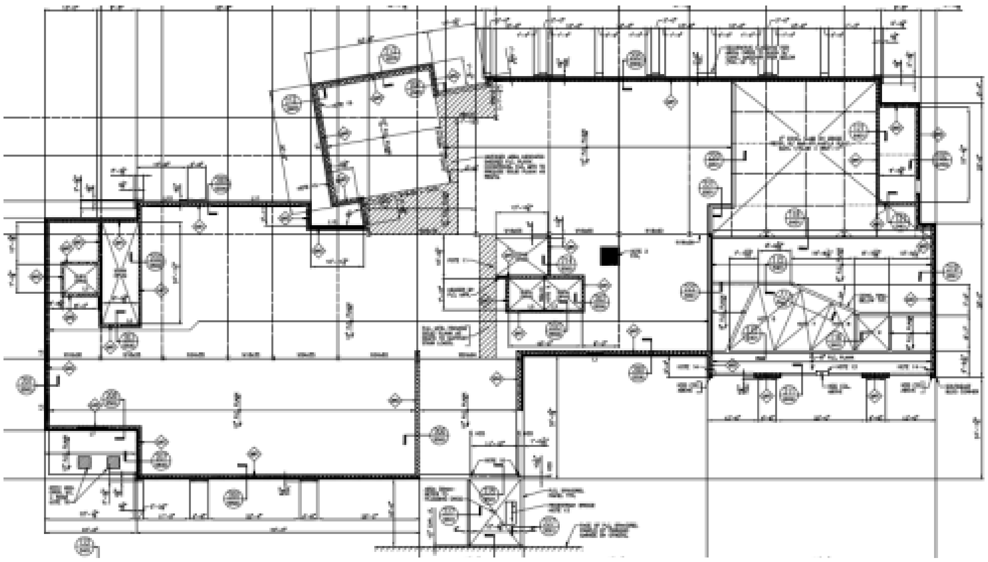
Appendix A: Existing Floor Plans



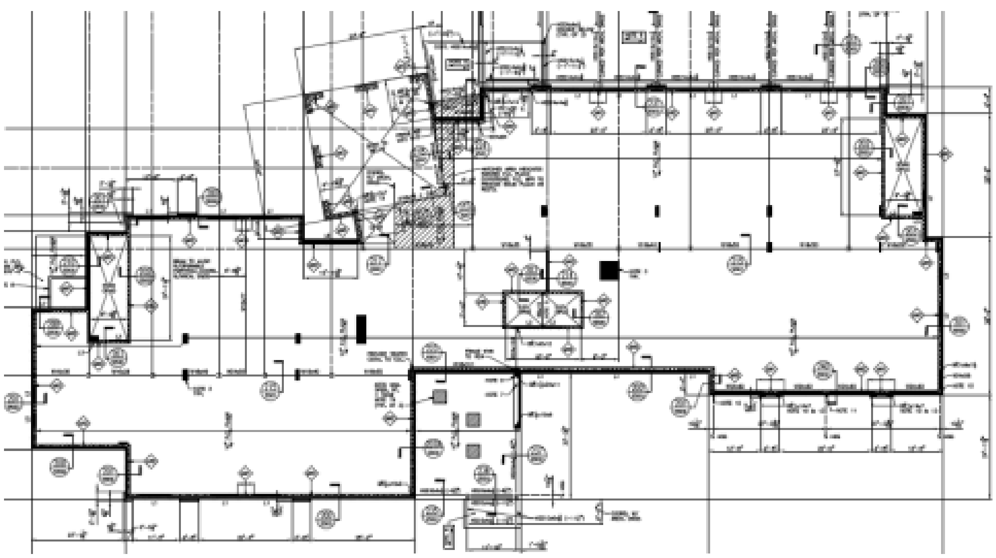
Foundation Plan



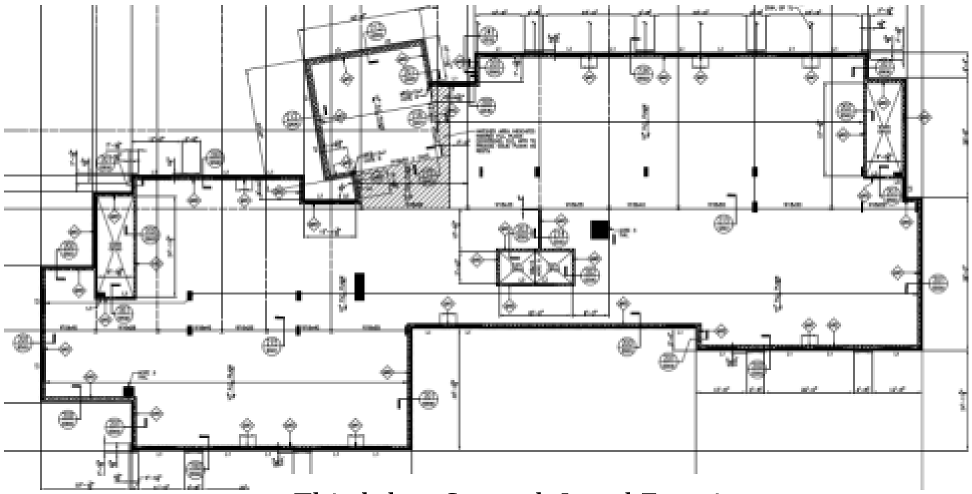
Plaza Level Framing Plan



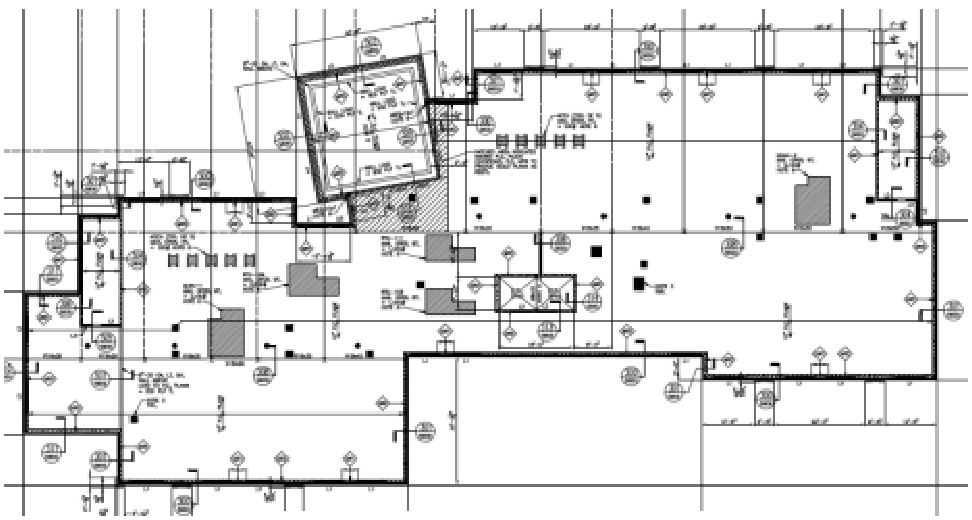
Hotel Level Framing Plan



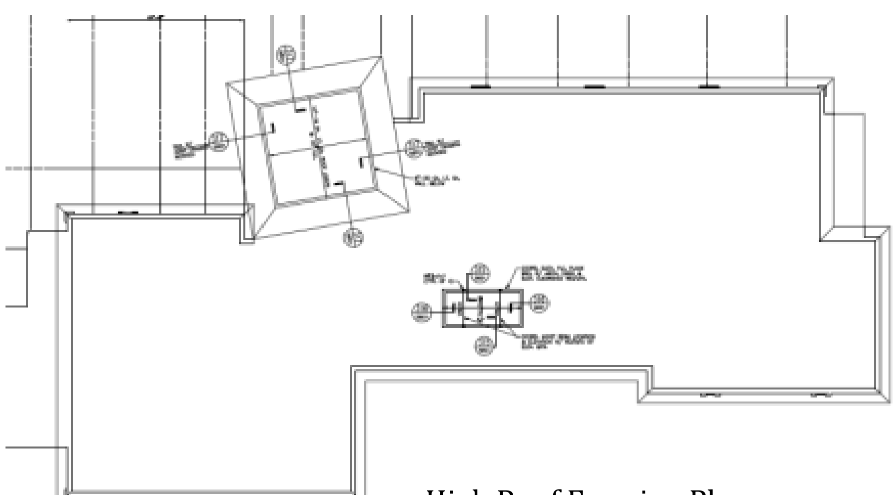
Second Level Framing Plan



Third thru Seventh Level Framing



Roof Framing Plan



High Roof Framing Plan

Appendix B: Gravity System Redesign

PRECAST HOLLOW-CORE CONCRETE PLANK
ON STEEL FRAMING

- LOADS

LL = 40 PSF (HOTEL ROOMS)
SDL = 25 PSF (MEP, PART., FINISHES)
DL = 15 PSF (W/ TOPPING ⇒ PCI HANDBOOK)

TOTAL LOAD = 80 PSF

f'_c = 5000 PSI
f_{pu} = 270,000 PSI
SPAN = 27'-6"

- DESIGNED FOR 8" W/ TOPPING
  4'-0" × 8" NWC (4HCB + 2)

- FROM PCI HANDBOOK
  76-S carrying 88 PSF capacity @ 28' span
  0.2" estimated camber @ erection
  -0.4" estimated long term camber
  7 strands @ 6/16" ϕ - straight
  Self weight of slab = 81 PSF

- GIRDERS (where applicable)

$$w_u = 1.2(25+81) + 1.6(40) = 191.2 \text{ PSF}$$

$$M_u = \frac{191.2(18')(27.5)^2}{8} = \underline{\underline{325.3 \text{ ft.k}}}$$

- USE W14 × 61 (less economical, but decreases system depth)

$\phi M_n = 383 \text{ ft·k} > 325.3 \text{ ft·k} = M_u \quad \therefore \text{OK}$

$\Delta_{LL} = \ell/360 = \dfrac{18'(12)}{360} = 0.267''$

$0.267'' = \dfrac{5\,(40)(27.5)(18)^4(1728)}{384(29000)\,I_x\,(1000)} \Rightarrow I_x = 335.6 \text{ in}^4 < 640 \text{ in}^4$ for W14 × 62 $\therefore$ OK

$\Delta_{TL} = \dfrac{5(40+25+81)(27.5)(18)^4(1728)}{384(29000)(640)(1000)} = 0.511''$

$\Delta_{TL} = 0.511'' < \ell/240 = \dfrac{18(12)}{240} = 0.9'' \quad \therefore \text{OK}$

BEAM / GIRDER DESIGN

- DESIGN LOADS

$DL = 81$ PSF (PLANK)
$SDL = 25$ PSF (PART., MEP, FINISHES)
$LL = 40$ PSF (HOTEL ROOMS)

- INTERIOR BEAM

Factored Load: $1.2(81+25) + 1.6(40) = 191.2$ PSF

Trib. Width: $27.5'$

$w_u = 191.2 \text{ PSF} (27.5')/1000 = 5.26 \text{ k/ft.}$

$M_u = \frac{w_u l^2}{8} = \frac{5.26(18)^2}{8} = 213.03$ ft.k

W14×61 $\phi M_n = 383$ ft.k $> M_u = 213.03$ ft.k ∴ OK

Uniform Dist. Load:
$191.2(27.5)(18)/1000 = 94.64^k$ w/ $KL = 18'$

W14×61 total capacity $= 170^k > 94.64^k$ ∴ OK

- DEFLECTION CHECK

$\Delta_{LL} = l/360 = \frac{18'(12)}{360} = 0.267''$

$0.267 = \frac{5\left(\frac{1.74 \text{ k/ft}}{12}\right)(18\times12)^4}{384(29000) I_x} \Rightarrow I_x = 536.8 \text{ in}^4 < 640 \text{ in}^4$ for W14×61 ∴ OK

$\Delta_{TL} = l/240 = \frac{18(12)}{240} = 0.9''$

$0.9 = \frac{5\left(5.26/12\right)(18\times12)^4}{384(29000) I_x} \Rightarrow I_x = 476 \text{ in}^4 < 640 \text{ in}^4$ for W14×61 ∴ OK

• EXTERIOR GIRDER

$W_u = 1.2(81+25) + 1.6(40) = 191.2$ PSF

Trib. Width $= \frac{27.5'}{2} = 13.75'$
Beam Length $= 18'$

$W_u = 191.2(13.75)/1000 = 2.63$ k/ft.

$M_u = \frac{2.63 \text{ k/ft } (18')^2}{8} = 106.5$ ft·k

W14×61 $\phi M_n = 383$ ft·k > 106.5 ft·k $= M_u$ ∴ OK

Uniform Dist. Load:
191.2 PSF $(18')(13.75')/1000 = 47.32^k$ w/ KL = 18'

W14×61 total capacity $= 170^k > 47.32^k$ ∴ OK

• DEFLECTION CHECK

$\Delta_{LL} = \ell/360 = \frac{18'(12)}{360} = 0.267''$

$0.267'' = \frac{5\left(\frac{0.88}{12}\right)(18'\times 12)^4}{384(29000)I_x} \Rightarrow I_x = 268.4 \text{ in}^4 < 640 \text{ in}^4$
for W14×61 ∴ OK

$\Delta_{TL} = \ell/240 = \frac{18(12)}{240} = 0.9''$

$0.9'' = \frac{5\left(\frac{2.63}{12}\right)(18\times 12)^4}{384(29000)I_x} \Rightarrow I_x = 238 \text{ in}^4 < 640 \text{ in}^4$
for W14×61 ∴ OK

GIRDER-SLAB D-BEAM DESIGN (where applicable)

- LOADS

Plank DL, untopped = 56 PSF
Partition load = 15 PSF
Live Load = 40 PSF
Topping = 25 PSF
Plank f'_c = 5000 ksi
Grout f'_c = 5000 ksi
Plank Span = 27.5'
DB Span = 18'

$$\text{Allowable } \Delta_{LL} = \ell/360 = \frac{18(12)}{360} = 0.6''$$

- DB 9x46 Properties

Steel Section	Transformed Section
$I_s = 195 \text{ in}^4$	$I_t = 356 \text{ in}^4$
$S_t = 33.7 \text{ in}^3$	$S_t = 68.6 \text{ in}^3$
$S_b = 50.8 \text{ in}^3$	$S_b = 80.6 \text{ in}^3$
$M_{All.} = 84$ k.ft.	$b = 5.75$ in
$t_w = 0.375$ in.	

- INITIAL LOAD – PRECOMPOSITE

$$M_{DL} = (27.5')(0.056\text{ ksf})(18')^2 / 8 = 62.37\text{ kft} < 84\text{ kft} = M_{ALL.}$$

$\therefore$ OK

$$\Delta_{DL} = \frac{5(27.5')(0.056\text{ ksf})(18)^4(1728)}{384(195\text{ in}^4)(29000)} = 0.64\text{ in.}$$

- TOTAL LOAD – COMPOSITE

$$M_{SDP} = (27.5')(0.015 + 0.04 + 0.025)(18')^2/8 = 89.1\text{ kft.}$$

$$M_{TL} = 62.37 + 89.1 = 151.5\text{ kft}$$

$S_{req.} = 151.5 \text{ kft } (12 \text{ in/ft}) \,/\, (0.6)(50 \text{ ksi}) = 60.6 \text{ in}^3 < 68.6 \text{ in}^3 \quad \therefore \text{ OK}$

$$\Delta_{SUP} = \frac{5(27.5)(0.015 + 0.04 + 0.025)(18)^4(1728)}{384(356)(29000)}$$

$= 0.50 \text{ in.} < 0.6 \text{ in}$

- CHECK COMPRESSIVE STRESS ON CONCRETE

$$N \text{ Value} = \frac{E_{steel}}{E_{conc.}} = \frac{29000 \text{ ksi}}{57000 (5000 \text{ psi})^{1/2}} = \frac{29000 \text{ ksi}}{4030} = 7.20$$

$S_{tc} = 7.20(68.6) = 494 \text{ in}^3$

$f_c = 89.1 \text{ kft } (12 \text{ in/ft}) \,/\, 494 \text{ in}^3 = 2.16 \text{ ksi}$

$F_c = 0.45(5 \text{ ksi}) = 2.25 \text{ ksi} > 2.16 \text{ ksi} \quad \therefore \text{ OK}$

- CHECK BOTTOM FLANGE TENSION STRESS (TOTAL LOAD)

$$f_b = \frac{(62.37 \text{ kft})(12 \text{ in/ft})}{50.8 \text{ in}^3} + \frac{(89.1 \text{ kft})(12 \text{ in/ft})}{80.6 \text{ in}^3}$$

$= 14.7 + 13.3$

$= 28 \text{ ksi}$

$F_b = 0.9(50 \text{ ksi}) = 45 \text{ ksi} > 28 \text{ ksi} \quad \therefore \text{ OK}$

- CHECK SHEAR

Total Load $= 56 + 15 + 40 + 25 = 136$ PSF

$w = 0.136 \text{ ksf } (27.5') = 3.74 \text{ k/ft.}$

$R = 3.74 \text{ k/ft } (18') / 2 = 33.7^{k}$

$f_v = 33.7^{k} \,/\, (0.375 \text{ in})(5.75) = 15.6 \text{ ksi}$

$F_v = 0.4(50 \text{ ksi}) = 20 \text{ ksi} > 15.6 \text{ ksi} \quad \therefore \text{ OK}$

COLUMN SPOT CHECKS

- SUMMARY OF LOADS

ROOF : LL = 20 PSF
DL = 81 PSF

FLOOR : LL = 100 PSF or 40 PSF
DL = 106 PSF

EXTERIOR WALL = DL = 45 PSF (assumed)

- AXIAL LOAD ON INTERIOR COLUMNS (PLAZA LEVEL : ROOF + 7 FLOORS)

$A_T = (27.5' \times 18') = 495 \text{ ft}^2$

$LL_{red} = 0.25 + \frac{15}{\sqrt{4(7)(495)}} = 0.377 > 0.4 \therefore \text{use } 0.4$

$P_{LL} = 0.4[(100)(1)(495) + (40)(6)(495)] = 67.32^k$

$P_{SL} = 20(495) = 9.9^k$

$P_{DL} = 81(495) + 106(7)(495) = 367.9^k$

$P_u = 1.2(367.9) + 1.6(67.32) + 0.5(9.9) = 554.1^k$

$@\ KL = 14'$ $\underline{\underline{W10 \times 60}}$ w/ $\phi P_n = 581^k > 554.1^k = P_u$

$\therefore$ OK

- AXIAL LOAD ON EXTERIOR COLUMNS FOR INTERIOR FRAMES (PLAZA LEVEL: ROOF + 7 FLOORS)

$A_T = (27.5' \times 9') = 247.5\ ft^2$

$LL_{red} = 0.25 + \dfrac{15}{\sqrt{4(7)(247.5)}} = 0.43$

$P_{LL} = 0.43\left[(100)(1)(247.5) + (40)(6)(247.5)\right] = 36.2^k$

$P_{SL} = 20(247.5) = 4.95^k$

$P_{DL} = 81(247.5) + 106(7)(247.5) + 450(7)(27.5) = 290.3^k$

$P_u = 1.2(290.3) + 1.6(36.2) + 0.5(4.95) = 408.8^k$

@ $KL = 14'$ W10x49 w/ $\phi P_n = 471^k > 408.8^k = P_u$

$\therefore$ OK

- AXIAL LOAD ON EXTERIOR CORNER COLUMNS (PLAZA LEVEL: ROOF + 7 FLOORS)

$A_T = (13.75' \times 9') = 123.75\ ft^2$

$LL_{red} = 0.25 + \dfrac{15}{\sqrt{4(7)(123.75)}} = 0.51$

$P_{LL} = 0.51\left[(100)(1)(123.75) + (40)(6)(123.75)\right] = 21.6^k$

$P_{SL} = 20(123.75) = 2.48^k$

$P_{DL} = 81(123.75) + 106(7)(123.75) + 450(7)(22.75) = 173.5^k$

$P_u = 1.2(173.5) + 1.6(21.6) + 0.5(2.48) = 244^k$

@ $KL = 14'$ W10x33 w/ $\phi P_n = 253^k > 244^k = P_u$

$\therefore$ OK

Appendix C: Wind & Seismic Load Analysis

Wind Loads

WIND LOADS

METHOD 2 : ANALYTICAL PROCEDURE

- WIND VARIABLES

$V = 90$ mph
$K_d = 0.85$
$I = 1.0$
EXPOSURE : B
$K_{zt} = 1.0$

(TABLE 6-3)
CASE 2

NOTE : INTERPOLATE
K_z VALUES

LEVEL	HEIGHT	K_z
B	0'	0'
1	14'-10"	0.56
2	26'-10"	0.63
3	36'-10"	0.74
4	46'-10"	0.79
5	56'-10"	0.84
6	66'-10"	0.88
7	76'-10"	0.92
ROOF	86'-10"	0.95
HIGH ROOF	102'-2"	1.00

$$q_z = 0.00256\, \underbrace{\boxed{K_z}}_{\text{VARIES BY LEVEL}}\, K_{zt}\, K_d\, V^2\, I \qquad \text{(Eq. 6-15)}$$

$$q_z = 0.00256\, K_z (1.0)(0.85)(90^2)(1.0)$$

* THIS IS COMPLETED FOR ALL LEVELS
AND PUT IN TABLE

EXAMPLE @ LEVEL 1 : $q_z = 0.00256(0.56)(1.0)(0.85)(90^2)(1.0)$
$= \underline{\underline{9.87}}$ PSF

WIND LOADS (CONT.)

q_h @ MEAN ROOF HEIGHT : $\bar{z} = \dfrac{86.833' + 102.167'}{2} = 94.5'$

$\Downarrow$

$K_{zt} = 0.97$

$\bar{z} = 0.6h = 0.6(94.5') = 56.7' > \bar{z}_{min} = 30'$

$q_h = 0.00256(0.97)(1.0)(0.85)(90^2)(1.0) = \underline{\underline{17.10 \text{ PSF}}}$

- C_p - EXTERNAL PRESSURE COEFFICIETS

NORTH / SOUTH	EAST / WEST
WINDWARD = 0.8	WINDWARD = 0.8
LEEWARD = -0.5	LEEWARD = -0.2
$L/B = 0.45$	$L/B = 2.22$
$L = 98.92'$ $B = 219.67'$	$L = 219.67'$ $B = 98.92'$

- WIND PRESSURE

$p_z = q_z G C_p - q_h G C_{pi}$ (WINDWARD)

$p_h = q_h G C_p - q_h G C_{pi}$ (LEEWARD)

$GC_{pi} = \pm 0.18$
FOR ENCLOSED BUILDINGS

NORTH / SOUTH EXAMPLE : @ LEVEL 1

$p_z = 9.87(0.85)(0.8)$
$= \underline{\underline{6.71 \text{ PSF}}}$

$p_h = 17.10(0.85)(-0.5)$
$= \underline{\underline{-7.27 \text{ PSF}}}$

WIND LOADS (CONT.)

EAST/WEST EXAMPLE : @ LEVEL 1

$P_z = 9.87(0.85)(0.8)$
$= \underline{\underline{6.71 \text{ PSF}}}$

$P_h = 17.10(0.85)(-0.2)$
$= \underline{\underline{-2.91 \text{ PSF}}}$

* WIND PRESSURES CALCULATED FOR EACH STORY AND PUT IN TABLE

- FORCE OF WINDWARD ONLY

$F_w = B\,(\text{story height})\,p_z$

N/S EXAMPLE : @ LEVEL 1 $\quad F_w = (219.67')(12')(6.71) = \underline{\underline{17.69 \text{ k}}}$

- FORCE OF TOTAL PRESSURE

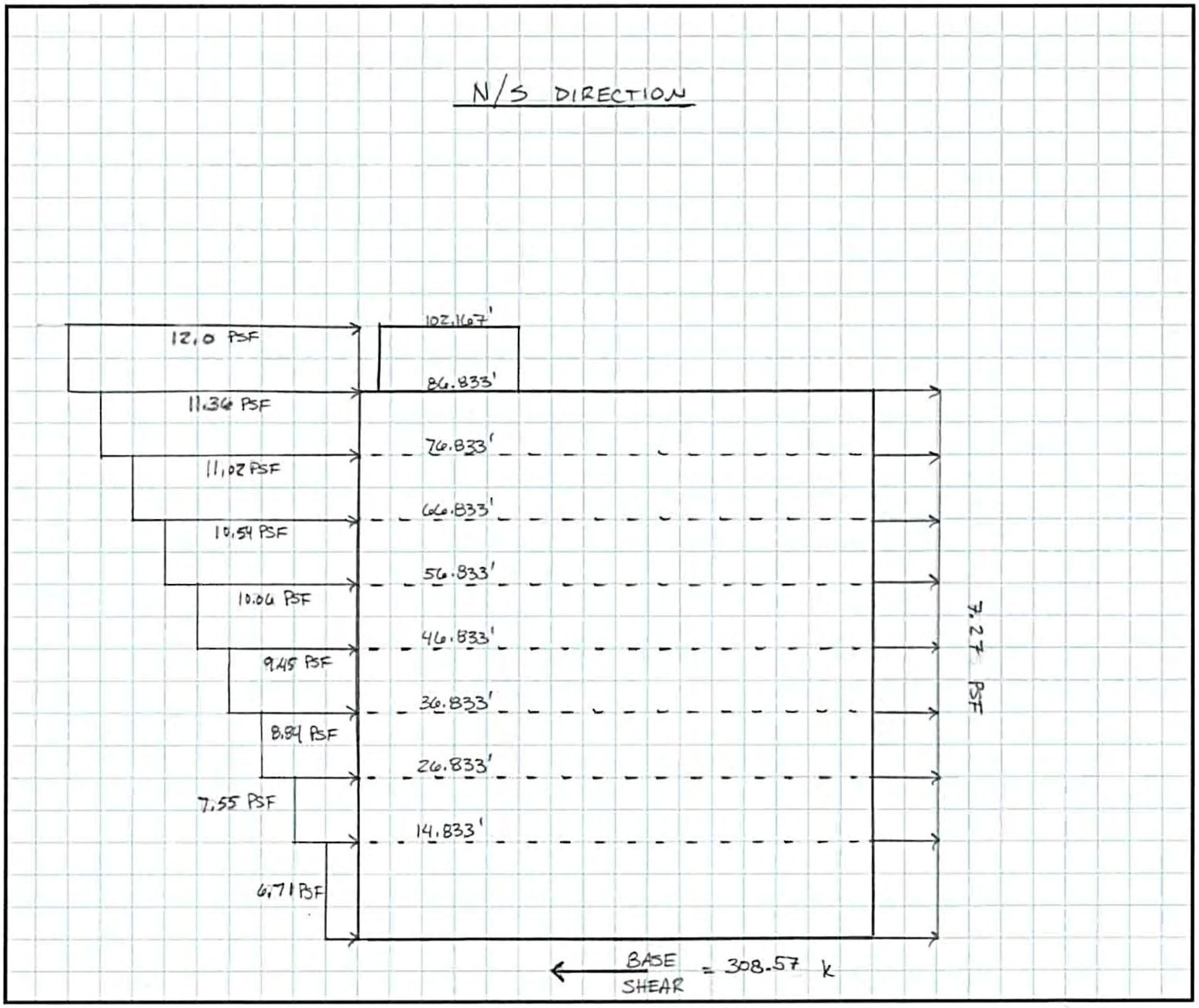
N/S EXAMPLE : @ LEVEL 1 $\quad F_T = (219.67')(12')(14.0 \text{ PSF}) = \underline{\underline{36.85 \text{ k}}}$

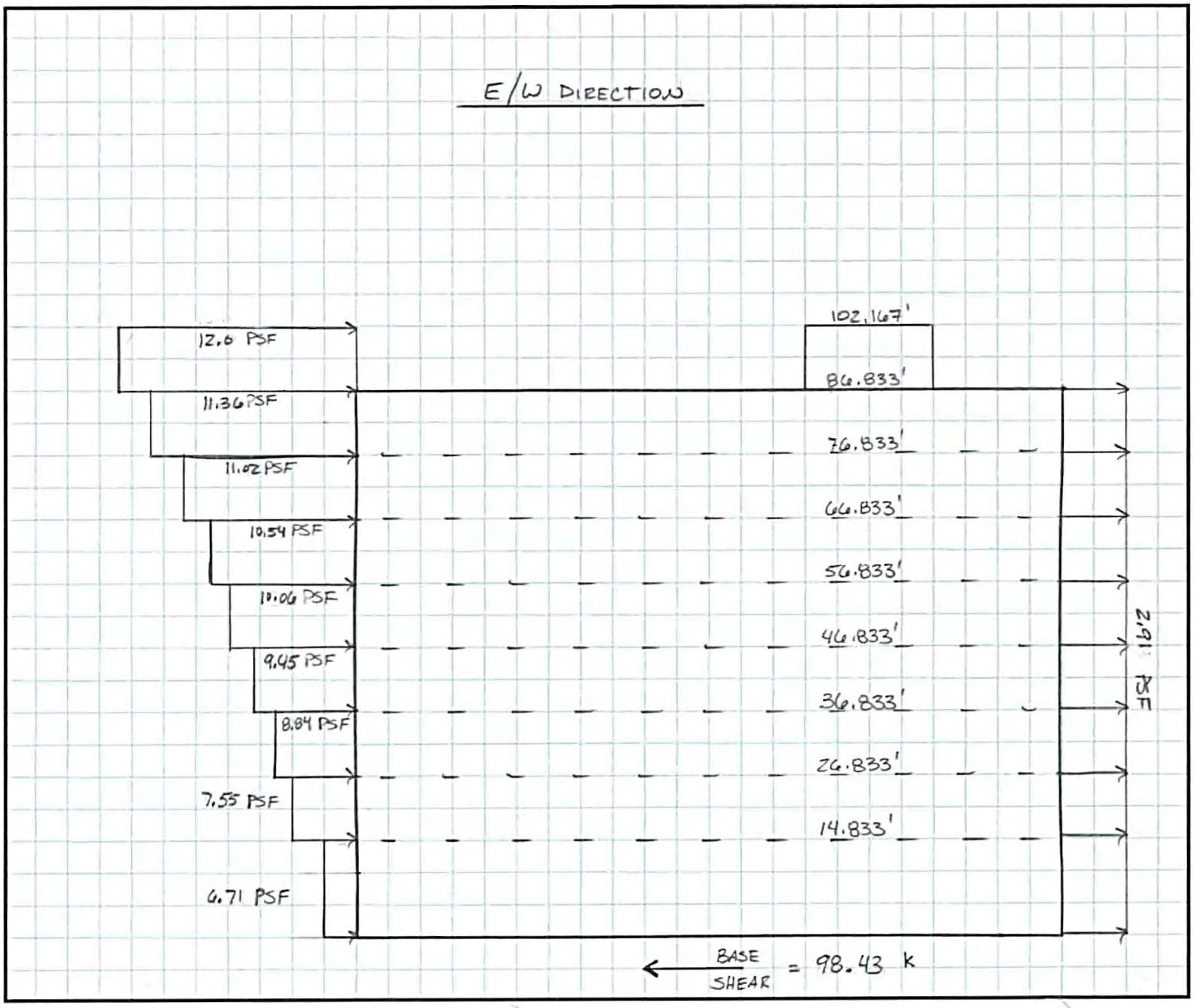
- WINDWARD SHEAR STORY

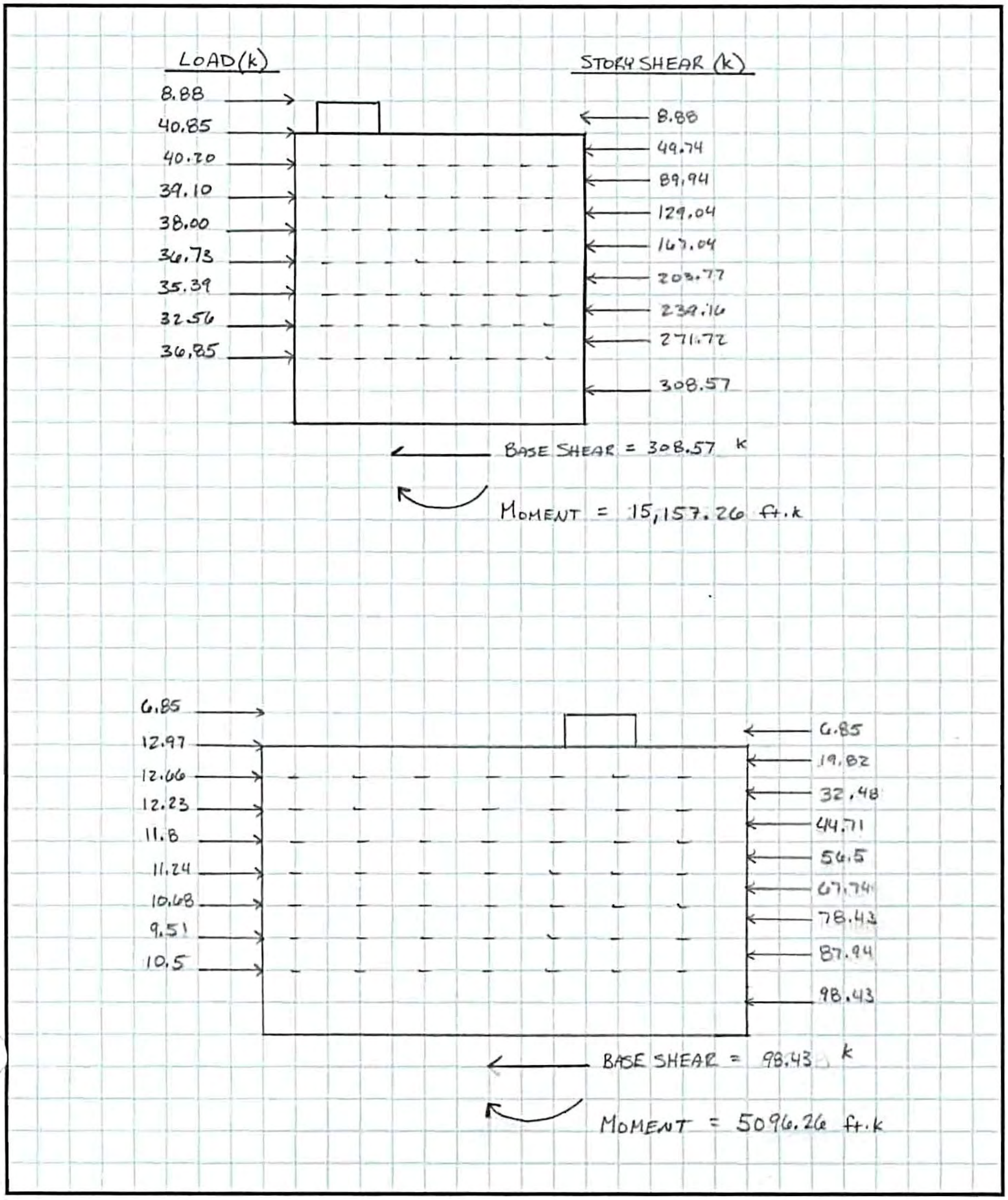
N/S EXAMPLE : @ LEVEL 7 $\quad F = F_w$ @ (HIGH ROOF + ROOF + 7)
$= 5.52 + 24.95 + 24.21 = \underline{\underline{54.68 \text{ k}}}$

- TOTAL STORY SHEAR

N/S EXAMPLE : @ LEVEL 7 $\quad F = F_T$ @ (HIGH ROOF + ROOF + 7)
$= 8.88 + 40.86 + 40.20 = \underline{\underline{89.94 \text{ k}}}$


N/S DIRECTION
102.167'
12.0 PSF
86.833'
11.36 PSF
76.833'
11.02 PSF
66.833'
10.54 PSF
56.833'
10.06 PSF
46.833'
9.45 PSF
36.833'
8.84 PSF
26.833'
7.55 PSF
14.833'
6.71 PSF
7.27 PSF
BASE SHEAR = 308.57 k


E/W DIRECTION
102.167'
86.833'
76.833'
66.833'
56.833'
46.833'
36.833'
26.833'
14.833'
12.6 PSF
11.36 PSF
11.02 PSF
10.54 PSF
10.06 PSF
9.45 PSF
8.84 PSF
7.55 PSF
6.71 PSF
2.91 PSF
BASE SHEAR = 98.43 k


LOAD (k)
STORY SHEAR (k)
8.88
40.85
40.20
39.10
38.00
36.73
35.39
32.56
36.85
8.88
49.74
89.94
129.04
167.04
203.77
239.16
271.72
308.57
BASE SHEAR = 308.57 k
MOMENT = 15,157.26 ft.k
6.85
12.97
12.66
12.23
11.8
11.24
10.68
9.51
10.5
6.85
19.82
32.48
44.71
56.5
67.74
78.43
87.94
98.43
BASE SHEAR = 98.43 k
MOMENT = 5096.26 ft.k

Wind Loads (East/West Direction)													
B = 98'-11" L = 219'-8"													
Level	Height Above Ground, z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (PSF)		Total Pressure (PSF)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft.-k)	Total Moment (ft.-k)
					Windward	Leeward							
High Roof	102.167	15.333	1.00	17.6	12.0	-2.91	14.9	5.52	6.850	5.52	6.850	521.64	647.33
Roof	86.833	10	0.95	16.7	11.36	-2.91	14.3	10.32	12.97	15.84	19.820	844.89	1061.39
7	76.833	10	0.92	16.2	11.02	-2.91	13.9	10.02	12.66	25.86	32.481	719.44	909.48
6	66.833	10	0.88	15.5	10.54	-2.91	13.5	9.58	12.23	35.44	44.710	592.52	756.11
5	56.833	10	0.84	14.8	10.06	-2.91	13.0	9.15	11.80	44.59	56.5	474.27	611.40
4	46.833	10	0.79	13.9	9.45	-2.91	12.4	8.59	11.24	53.19	67.744	359.49	470.17
3	36.833	10	0.74	13.0	8.84	-2.91	11.8	8.04	10.68	61.22	78.427	255.84	340.06
2	26.833	10	0.63	11.1	7.55	-2.91	10.5	6.86	9.51	68.09	87.935	149.83	207.59
1	14.833	12	0.56	9.87	6.71	-2.91	9.6	7.32	10.50	75.41	98.432	64.68	92.72
B	0	14.833	0	0	0	0	0	0	0	75.41	98.43	0	0

Σ Windward Story Shear =	75.41	kips
Σ Total Story Shear =	98.43	kips
Σ Windward Moment =	3982.60	ft-k
Σ Total Moment =	5096.26	ft-k

Wind Loads (North/South Direction)													
B = 219'-8" L = 98'-11"													
Level	Height Above Ground, z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (PSF)		Total Pressure (PSF)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft.-k)	Total Moment (ft.-k)
					Windward	Leeward							
High Roof	102.167	15.333	1.00	17.6	12.0	-7.27	19.3	5.52	8.88	5.52	8.88	521.64	839.16
Roof	86.833	10	0.95	16.7	11.36	-7.27	18.6	24.95	40.86	30.47	49.74	2042.10	3343.58
7	76.833	10	0.92	16.2	11.02	-7.27	18.3	24.21	40.20	54.68	89.94	1738.91	2887.66
6	66.833	10	0.88	15.5	10.54	-7.27	17.8	23.15	39.10	77.84	129.04	1431.63	2417.75
5	56.833	10	0.84	14.8	10.06	-7.27	17.3	22.10	38.00	99.93	167.04	1145.45	1969.80
4	46.833	10	0.79	13.9	9.45	-7.27	16.7	20.76	36.73	155.62	203.77	868.40	1536.48
3	36.833	10	0.74	13.0	8.84	-7.27	16.1	19.42	35.39	140.11	239.16	618.16	1126.53
2	26.833	10	0.63	11.1	7.55	-7.27	14.8	16.59	32.56	156.70	271.72	362.10	710.78
1	14.833	12	0.56	9.87	6.71	-7.27	14.0	17.69	36.85	174.38	308.57	156.24	325.51
B	0	14.833	0	0	0	0	0	0	0	174.38	308.57	0	0

Σ Windward Story Shear =	174.38	kips
Σ Total Story Shear =	308.57	kips
Σ Windward Moment =	8884.63	ft-k
Σ Total Moment =	15157.26	ft-k

Seismic Loads

Seismic Force Resisting System: Floor Weights					
Plaza Level					
Approximate Area:		15,113	SF		
Floor to Floor Height:		14	ft.		
Walls:			**Superimposed:**		
Perimeter:	0	ft.	Partitions:	15	PSF
Height:	0	ft.	MEP:	10	PSF
Unit Weight:	0	PSF	Finishes:	5	PSF
Weight =	**0.00**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	0	in.			
Unit Weight:	0	PSF			
Weight =	**0**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x33	28	33	14	12.94	
W10x45	8	45	14	5.04	
W10x49	6	49	14	4.12	
W10x39	4	39	14	2.18	
W10x68	4	68	14	3.81	
W10x77	5	77	14	5.39	
W12x65	2	65	14	1.82	
W10x60	4	60	14	3.36	
W12x87	1	87	14	1.22	
W10x54	1	54	14	0.76	
			Weight =	**40.63**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Total Beam Length (ft)	Total Weight (k)	
W10x12	34	12	546.96	6.56	
W12x14	3	14	52.5	0.74	
W12x16	1	16	12.5	0.20	
W14x22	7	22	80.5	1.77	
W16x26	2	26	33	0.86	
W14x26	1	26	14	0.36	
W14x30	7	30	95	2.85	
W16x31	2	31	36	1.12	
W18x35	2	35	41.5	1.45	
W14x38	1	38	10	0.38	
W14x43	1	43	18	0.77	
W14x48	6	48	91.5	4.39	
W14x53	1	53	18	0.95	
W14x61	13	61	228.5	13.94	
			Weight =	**36.35**	**k**
		Total Weight of Floor =	**530.37**	**k**	
			35.09	PSF	

Seismic Force Resisting System: Floor Weights					
Hotel Level					
Approximate Area:		15,113	SF		
Floor to Floor Height:		12	ft.		
Walls:			**Superimposed:**		
Perimeter:	0	ft.	Partitions:	15	PSF
Height:	0	ft.	MEP:	10	PSF
Unit Weight:	0	PSF	Finishes:	5	PSF
Weight =	**0**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	8	in.			
Unit Weight:	81	PSF			
Weight =	**1224.153**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x33	26	33	12	10.30	
W10x45	8	45	12	4.32	
W10x49	5	49	12	2.94	
W10x39	5	39	12	2.34	
W10x68	4	68	12	3.26	
W10x77	5	77	12	4.62	
W12x65	1	65	12	0.78	
W10x60	4	60	12	2.88	
W12x87	1	87	12	1.04	
W10x54	1	54	12	0.65	
			Weight =	**33.13**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Total Beam Length (ft)	Total Weight (k)	
W8x10	3	10	73	0.73	
W10x12	25	12	410.3	4.92	
W12x14	1	14	28.5	0.40	
W16x26	1	26	18.5	0.48	
W18x35	1	35	23	0.81	
DB 9x46	45	46	647	29.76	
W40x183	1	183	24	4.39	
			Weight =	**41.49**	**k**
		Total Weight of Floor =	**1752.17**	**k**	
			115.94	PSF	

Seismic Force Resisting System: Floor Weights					
Floor Levels 2-4					
Approximate Area:		15,113	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	0	ft.			
Height:	0	ft.	Partitions:	15	PSF
			MEP:	10	
Unit Weight:	0	PSF	Finishes:	5	PSF
Weight =	**0**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	8	in.			
Unit Weight:	81	PCF			
Weight =	**1224.153**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x33	39	33	10	12.87	
W10x45	7	45	10	3.15	
W10x49	2	49	10	0.98	
W12x50	2	50	10	1	
W10x39	5	39	10	1.95	
			Weight =	**19.95**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Total Beam Length (ft)	Total Weight (k)	
W10x12	28	12	506.7	6.08	
W16x26	1	26	18.5	0.48	
W18x35	1	35	23	0.81	
DB 9x46	41	46	602.5	27.72	
			Weight =	**35.08**	**k**
		Total Weight of Floor =	**1732.57**	**k**	
			114.64	PSF	

Seismic Force Resisting System: Floor Weights					
Floor Levels 5-7					
Approximate Area:		15,113	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	0	ft.			
Height:	0	ft.	Partitions:	15	PSF
			MEP:	10	
Unit Weight:	0	PSF	Finishes:	5	PSF
Weight =	**0**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	8	in.			
Unit Weight:	81	PCF			
Weight =	**1224.153**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x33	55	33	10	18.15	
W12x40	2	40	10	0.8	
			Weight =	**18.95**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Total Beam Length (ft)	Total Weight (k)	
W10x12	28	12	506.7	6.08	
W16x26	1	26	18.5	0.48	
W18x35	1	35	23	0.81	
DB 9x46	41	46	602.5	27.72	
			Weight =	**35.08**	**k**
Total Weight of Floor =			**1731.57**	**k**	
			114.58	PSF	

Seismic Force Resisting System: Floor Weights					
Roof Level					
Approximate Area:		15,113	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	0	ft.		0	PSF
Height:	0	ft.	MEP:	10	PSF
Unit Weight:	0	PSF	Roof Mat:	10	PSF
Weight =	**0.00**	**k**	**Weight =**	**302.26**	**k**
Slab:					
Thickness:	8	in.			
Unit Weight:	81	PCF			
Weight =	**1224.153**	**k**			
Columns:					
			Weight =		**k**
Beams:					
Shape	Quantity	Weight (PLF)	Total Beam Length (ft)	Total Weight (k)	
W10x12	4	12	52	0.62	
			Weight =	**0.62**	**k**
		Total Weight of Floor =	**1527.04**	**k**	
			101.04	PSF	

Seismic Force Resisting System: Floor Weights					
High Roof Level					
Approximate Area:		576	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	0	ft.		0	PSF
Height:	0	ft.	MEP:	10	PSF
Unit Weight:	0	PSF	Roof Mat:	10	PSF
Weight =	**0.00**	**k**	**Weight =**	**11.52**	**k**
Slab:					
Thickness:	8	in.			
Unit Weight:	81	PCF			
Weight =	**46.656**	**k**			
		Total Weight of Floor =	**58.18**	**k**	
			101.00	PSF	

SEISMIC LOADS

- SITE CLASS C – Very Dense Soil & Soft Rock (TABLE 20.3-1)
- OCCUPANCY CATEGORY II (TABLE 1-1)
- IMPORTANCE FACTOR = 1.0 (TABLE 11.5-1)
- SPECTRAL RESPONSE ACCELERATION, SHORT (S_S) (FIG. 22-1 thru 22-14)
  &
  SPECTRAL RESPONSE ACCELERATION, 1s (S_1)

  $S_S = 0.125$
  $S_1 = 0.049$

- SITE COEFFICIENTS (F_a & F_v) (TABLE 11.4-1 & 11.4-2)

  $F_a = 1.2$
  $F_v = 1.7$

- $S_{MS} = F_a S_s = 1.2(0.125)$ $\boxed{S_{MS} = 0.15}$ (Eq 11.4-1)
- $S_{DS} = 2/3(S_{MS}) = 2/3(0.15)$ $\boxed{S_{DS} = 0.10}$
- $S_{M1} = F_v S_1 = 1.7(0.049)$ $\boxed{S_{M1} = 0.0833}$ (Eq. 11.4-2)
- $S_{D1} = 2/3\, S_{M1} = 2/3(0.0833)$ $\boxed{S_{D1} = 0.055}$ (Eq. 11.4-4)

- $T_a = C_t h_n^x$ (Eq. 12.8-7)
  $= 0.02(102.167)^{0.75}$ $\boxed{T_a = 0.643\ s}$

- $C_u = 1.7$ (TABLE 12.8-1)

- $T = T_a C_u$
  $= 0.643(1.7)$ $\boxed{T = 1.09\ s}$ (SEC. 12.8.2)

- $C_s = $ MIN.
  $$\frac{S_{D1}}{T(R/I)} = \frac{0.055}{1.09(3.25/1.0)} = \boxed{0.016} \geq 0.01$$
  $$\frac{S_{DS}}{(R/I)} = \frac{0.10}{(3.25/1.0)} = 0.031 \geq 0.01$$
  $$\frac{S_{D1}T_L}{T^2(R/I)} = \frac{0.055(12)}{(1.09)^2(3.25/1.0)} = 0.171 \geq 0.01$$

  WHERE: $R = 3.25$ (TABLE 12.2-1)
  $I = 1.0$ (TABLE 11.5-1)
  $T_L = 12$ (FIG. 22-15)

- $k = 0.75 + 0.5(T)$
  $= 0.75 + 0.5(1.09)$ $\boxed{k = 1.295}$ (SEC. 12.8.3)

- SEE EXCEL SPREADSHEETS FOR FLOOR WEIGHTS

FLOOR	APPROX. FLOOR AREA	TOTAL WEIGHT
B	15,113 SF	35.09 PSF
1	15,113 SF	115.94 PSF
2-4	15,113 SF	114.64 PSF
5-7	15,113 SF	114.58 PSF
ROOF	15,113 SF	101.04 PSF
HIGH ROOF	576 SF	101 PSF

- TOTAL BUILDING WEIGHT (W_T)

$$\boxed{W_T = 14,260 \text{ k}}$$

- BASE SHEAR (V)

$$V = C_s W_T = 0.016(14260)$$

$$\boxed{V = 228.16^k}$$

- $w_x h_x^k$ (varies @ height)

Example @ Level 1 : $w_x = 1752.17^k$, $h_x = 14.833'$, $k = 1.295$

$$w_x h_x^k = 1752.17(14.833)^{1.295} = \boxed{57,586 \text{ ft} \cdot \text{k}}$$

- $\Sigma w_i h_i^k = \boxed{2338382 \text{ ft k}}$

- $C_{vx} = \dfrac{w_x h_x^k}{\Sigma w_i h_i^k}$ (varies @ height) (Eq. 12.8-12)

  Example @ Level 1 : $C_{vx} = \dfrac{57586}{2338382} = \boxed{0.025}$

- $F_x = C_{vx}(V)$

  Example @ Level 1 : $F_x = 0.025(228.16) = \boxed{5.62^k}$

- STORY SHEAR (V_x)

  $V_x = F_x$ (@ Level) + F_x (@ all levels above)

  Example @ Level 7 : $V_x = F_x(HR) + F_x(Roof) + F_x(7)$
  $= 2.27 + 48.28 + 46.72$
  $= \boxed{97.28^k}$

- MOMENTS

  M_x = (Trib. Floor Area Height)(F_x)

  Example @ Level 7 : $M_x = ((76.833 + 66.833)/2)(46.72)$
  $= \boxed{3356.39 \text{ ft.k}}$

Redesign Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	M_x (ft-k)
High Roof	102.167	58.18	23272	0.010	2.27	2.27	214.58
Roof	86.833	1527.04	494820	0.212	48.28	50.55	3950.93
7	76.833	1731.57	478878	0.205	46.72	97.28	3356.39
6	66.833	1731.57	399764	0.171	39.01	136.28	2411.84
5	56.833	1731.57	324077	0.139	31.62	167.90	1639.00
4	46.833	1732.57	252380	0.108	24.63	192.53	1030.15
3	36.833	1732.57	184913	0.079	18.04	208.30	574.34
2	26.833	1732.57	122692	0.052	11.97	222.54	249.40
1	14.833	1752.17	57586	0.025	5.62	228.16	41.67
B	0	530.37	0	0	0	228.16	0
			2338382				

Total Building Weight =	14260	k
Base Shear =	228.16	k
Total Moment =	13468.29	ft-k

Existing Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	M_x (ft-k)
High Roof	102.167	7.92	3168	0.001	0.52	0.52	48.89
Roof	86.833	1864.55	604186	0.194	98.67	99.18	8074.14
7	76.833	2372.19	656045	0.211	107.13	206.32	7695.82
6	66.833	2372.19	547662	0.176	89.44	295.75	5530.06
5	56.833	2372.19	443974	0.143	72.50	368.26	3758.03
4	46.833	2372.19	345553	0.111	56.43	424.69	2360.64
3	36.833	2372.19	253178	0.081	41.35	465.51	1316.14
2	26.833	2372.19	167987	0.054	27.43	493.46	571.51
1	14.833	2712.91	89161	0.029	14.56	508.03	107.99
B	0	1404.82	0	0	0	508.03	0
			3110915				

Total Building Weight =	20223	k
Base Shear =	508.03	k
Total Moment =	29463.22	ft-k

COR and COM Calculations

The following equations were used to calculate the Center of Rigidity for both the X and Y direction for each level.

$$XCR = \frac{\sum kiyxi}{\sum Kiy,total} \qquad YCR=\frac{\sum kixyi}{\sum Kix,total}$$

Center of Rigidity (XCR)										
Level	K_{iy}				x_i (ft)				K_{iy},total	XCR (ft)
	Frame C	Frame M	Frame M.2	Frame O	Frame C	Frame M	Frame M.2	Frame O		
Roof	13.60	0.92	1.18	11.12	22.6	118.5	136.5	205	26.81	106.53
7	15.65	1.09	1.40	12.82	22.6	118.5	136.5	205	30.96	106.64
6	18.44	1.33	1.73	15.11	22.6	118.5	136.5	205	36.61	106.76
5	22.51	1.69	2.23	18.46	22.6	118.5	136.5	205	44.90	106.88
4	28.82	2.29	3.08	23.65	22.6	118.5	136.5	205	57.84	107.05
3	39.26	3.41	4.66	32.40	22.6	118.5	136.5	205	79.73	107.48
2	58.31	5.87	8.08	49.00	22.6	118.5	136.5	205	121.26	108.54
Hotel Level	115.47	16.78	22.57	100.70	22.6	118.5	136.5	205	255.53	110.84

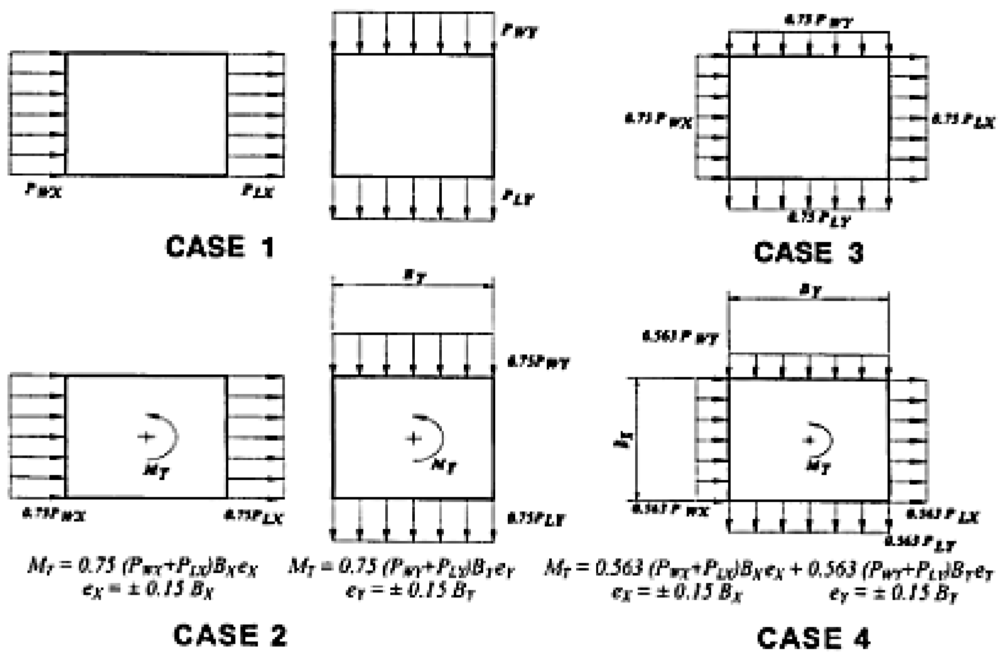
Center of Rigidity (YCR)						
Level	K_{ix}		y_i (ft)		K_{ix},total	YCR (ft)
	Frame 2	Frame 8	Frame 2	Frame 8		
Roof	6.70	4.37	12.5	51	11.07	37.70
7	7.67	5.08	12.5	51	12.75	37.84
6	9.02	6.06	12.5	51	15.08	37.97
5	11.00	7.53	12.5	51	18.53	38.15
4	14.08	9.87	12.5	51	23.95	38.37
3	19.35	13.95	12.5	51	33.30	38.63
2	29.64	22.11	12.5	51	51.75	38.95
Hotel Level	61.69	49.88	12.5	51	111.57	39.71

Center of Rigidity & Center of Mass								
Level	ETABS COR		Calculated COR		Difference in COR		ETABS COM	
	XCR (ft)	YCR (ft)	XCR (ft)	YCR (ft)	XCR (ft)	YCR (ft)	XCM (ft)	YCM (ft)
Roof	107.1	40.2	106.46	37.70	0.6	2.5	112.4	52.1
7	106.7	39.9	106.56	37.84	0.1	2.1	112.4	52.1
6	106.3	39.6	106.66	37.97	-0.4	1.6	112.4	52.1
5	106.1	39.1	106.76	38.15	-0.7	1.0	112.4	52.1
4	106.4	38.4	106.90	38.37	-0.5	0.0	112.4	52.1
3	107.5	37.2	107.28	38.63	0.2	-1.4	112.4	52.1
2	110.0	35.5	108.23	38.95	1.8	-3.4	112.4	52.1
Hotel Level	114.3	32.9	110.25	39.71	4.1	-6.8	112.4	52.1

Design Wind Load Cases

Figure 6-9: Design Wind Load Cases from ASCE 7-05

Main Wind Force Resisting System– Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

The following tables summarize the design wind load cases analyzed in ETABS when considering ASCE 7-05 load combinations. Data is based on the wind load cases defined in Figure 6-9 of ASCE 7-05 (pictured above).

Case 1X		
Level	Factored P_x (k)	P_Y (k)
Roof	20.75	0
7	20.26	0
6	12.23	0
5	18.87	0
4	17.98	0
3	17.09	0
2	15.21	0
Hotel Level	16.80	0

Case 1Y		
Level	P_X (k)	Factored P_X (k)
Roof	0	65.37
7	0	64.32
6	0	62.56
5	0	60.80
4	0	58.77
3	0	56.62
2	0	52.09
Hotel Level	0	58.96

Case 2X					
Level	Factored P_X (k)	$0.75P_X$ (k)	B_x (ft)	e_x (ft)	M_T (ft-k)
Roof	20.75	15.56	98.92	14.84	230.94
7	20.26	15.19	98.92	14.84	225.44
6	12.23	9.17	98.92	14.84	136.08
5	18.87	14.15	98.92	14.84	210.03
4	17.98	13.49	98.92	14.84	200.12
3	17.09	12.82	98.92	14.84	190.21
2	15.21	11.41	98.92	14.84	169.30
Hotel Level	16.80	12.60	98.92	14.84	186.91

Case 2Y					
Level	Factored P_Y (k)	$0.75P_Y$ (k)	B_y (ft)	e_y (ft)	M_T (ft-k)
Roof	65.37	49.03	219.67	32.95	1615.57
7	64.32	48.24	219.67	32.95	1589.52
6	62.56	46.92	219.67	32.95	1546.09
5	60.80	45.60	219.67	32.95	1502.66
4	58.77	44.07	219.67	32.95	1452.28
3	56.62	42.47	219.67	32.95	1399.30
2	52.09	39.07	219.67	32.95	1287.25
Hotel Level	58.96	44.22	219.67	32.95	1457.14

Case 3				
Level	Factored P_X (k)	Factored P_Y (k)	$0.75P_X$ (k)	$0.75P_Y$ (k)
Roof	20.75	65.37	15.56	49.03
7	20.26	64.32	15.19	48.24
6	12.23	62.56	9.17	46.92
5	18.87	60.80	14.15	45.60
4	17.98	58.77	13.49	44.07
3	17.09	56.62	12.82	42.47
2	15.21	52.09	11.41	39.07
Hotel Level	16.80	58.96	12.60	44.22

Case 4									
Level	Factored P_X (k)	$0.563P_X$ (k)	Factored P_Y (k)	$0.563P_Y$ (k)	B_x (ft)	e_x (ft)	B_y (ft)	e_y (ft)	M_T (ft-k)
Roof	20.75	11.68	65.37	36.81	98.92	14.84	219.67	32.95	1386.12
7	20.26	11.41	64.32	36.21	98.92	14.84	219.67	32.95	1362.43
6	12.23	6.88	62.56	35.22	98.92	14.84	219.67	32.95	1262.75
5	18.87	10.63	60.80	34.23	98.92	14.84	219.67	32.95	1285.66
4	17.98	10.12	58.77	33.09	98.92	14.84	219.67	32.95	1240.40
3	17.09	9.62	56.62	31.88	98.92	14.84	219.67	32.95	1193.19
2	15.21	8.56	52.09	29.33	98.92	14.84	219.67	32.95	1093.38
Hotel Level	16.80	9.46	58.96	33.20	98.92	14.84	219.67	32.95	1234.14

The following tables are summaries of the seismic data considering ASCE 7-05 wind load combinations. All data considers inherent and accidental torsion, as defined in Section 12.8.4.1 and 12.8.4.2 of ASCE 7-05.

Accidental Torsion, M_{ta} (X-Direction)				
Level	Structural Width (ft)	5% of Width (ft)	Story Force	Moment, M_{ta} (ft-k)
Roof	219.67	11.0	48.28	530.3
7	219.67	11.0	46.72	513.2
6	219.67	11.0	39.01	428.4
5	219.67	11.0	31.62	347.3
4	219.67	11.0	24.63	270.5
3	219.67	11.0	18.04	198.2
2	219.67	11.0	11.97	131.5
Plaza	219.67	11.0	5.62	61.7

Accidental Torsion, M_{ta} (Y-Direction)				
Level	Structural Width (ft)	5% of Width (ft)	Story Force	Moment, M_{ta} (ft-k)
Roof	98.92	4.9	48.28	238.8
7	98.92	4.9	46.72	231.1
6	98.92	4.9	39.01	192.9
5	98.92	4.9	31.62	156.4
4	98.92	4.9	24.63	121.8
3	98.92	4.9	18.04	89.2
2	98.92	4.9	11.97	59.2
Plaza	98.92	4.9	5.62	27.8

Seismic Torsional Effects										
Level	East-West (X-Direction)					North-South (Y-Direction)				
	Factored Story Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	M_{total} (ft-k)	Factored Story Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	M_{total} (ft-k)
Roof	48.28	-5.3	-255.89	530.29	274.40	48.28	-11.9	-574.54	238.80	-335.74
7	46.72	-5.7	-266.33	513.20	246.87	46.72	-12.2	-570.04	231.10	-338.94
6	39.01	-6.1	-237.93	428.42	190.48	39.01	-12.5	-487.57	192.92	-294.65
5	31.62	-6.3	-199.21	347.31	148.10	31.62	-13.0	-411.07	156.40	-254.67
4	24.63	-6.0	-147.75	270.47	122.72	24.63	-13.7	-337.37	121.80	-215.57
3	18.04	-4.9	-88.41	198.17	109.76	18.04	-14.9	-268.83	89.24	-179.59
2	11.97	-2.4	-28.73	131.49	102.76	11.97	-16.6	-198.72	59.21	-139.51
Plaza	5.62	1.9	10.68	61.71	72.39	5.62	-19.2	-107.88	27.79	-80.09
				Total:	1267.48				**Total:**	-1838.77

The following table is a summation of the base shears and overturning moments produced by ETABS in the analysis of the ASCE 7-05 design wind load cases. It is confirmed that wind loads control in the North/South direction (Case 1Y) and seismic loads control in the East/West direction (Case EX).

Design Wind Load Cases: Controlling Base Shears and Overturning Moments								
Story	Point	Load	FX	FY	FZ	MX	MY	MZ
Summartion	0,0,Base	Case 1X	-139.20	0.00	0.00	0.00	-7377.10	7034.70
Summartion	0,0,Base	Case 1Y	0.00	-479.50	0.00	25364.80	0.00	-52504.20
Summartion	0,0,Base	Case 2X	-104.39	0.00	0.00	0.00	-5532.30	3726.84
Summartion	0,0,Base	Case 2Y	0.00	-359.62	0.00	19023.74	0.00	-51228.20
Summartion	0,0,Base	Case 3	-104.39	-359.62	0.00	19023.74	-5532.30	-34102.52
Summartion	0,0,Base	Case 4	-78.36	-269.97	0.00	14281.11	-4153.11	-35659.47
Summartion	0,0,Base	Case EX	-225.89	0.00	0.00	0.00	-14408.67	11764.63
Summartion	0,0,Base	Case EY	0.00	-225.89	0.00	14408.67	0.00	-25392.93

Appendix D: Lateral System Design

The following tables summarize the results of an analysis performed in determining the adequacy of the lateral brace designs for the proposed braced frame lateral system.

Frame	Level	Brace	Factored Axial Load (k)	Pu/ϕPn < 1.0
C	7	HSS8x8x5/8	13.35	0.03
			20.39	0.046
			23.88	0.044
			11.54	0.021
	6	HSS8x8x5/8	31.98	0.072
			43.9	0.098
			52.34	0.096
			32.33	0.059
	5	HSS8x8x5/8	54.24	0.121
			69.25	0.155
			80.88	0.148
			58.85	0.108
	4	HSS8x8x5/8	68.58	0.153
			96.16	0.215
			107.64	0.197
			76.58	0.14
	3	HSS8x8x5/8	80.24	0.18
			132.15	0.296
			144.98	0.266
			94.06	0.173
	2	HSS8x8x5/8	86.69	0.201
			139.41	0.312
			184.16	0.338
			120.04	0.22
	Hotel	HSS8x8x5/8	115.99	0.224
			212	0.41
			180.93	0.426
			94.91	0.224
	Plaza	HSS8x8x5/8	141.74	0.291
			250.1	0.514
			200.2	0.502
			107.08	0.268

Frame	Level	Brace	Factored Axial Load (k)	Pu/ϕPn < 1.0
O	7	HSS8x8x5/8	9.12	0.02
			17.93	0.04
			18.05	0.033
			4.52	0.008
	6	HSS8x8x5/8	21.36	0.048
			35.22	0.079
			38.9	0.071
			16.95	0.031
	5	HSS8x8x5/8	35.17	0.079
			55.87	0.125
			62.68	0.115
			29.29	0.054
	4	HSS8x8x5/8	48.34	0.108
			78.91	0.177
			89.66	0.165
			43.66	0.08
	3	HSS8x8x5/8	59.36	0.133
			102.9	0.23
			117.31	0.215
			61.22	0.112
	2	HSS8x8x5/8	71.88	0.161
			127.92	0.286
			142.61	0.262
			83.41	0.153
	Hotel	HSS8x8x5/8	73.63	0.174
			147.29	0.347
			194.26	0.375
			96.01	0.186
	Plaza	HSS8x8x5/8	170.69	0.428
			220.72	0.454
			108.27	0.223
			73.02	0.183

Frame	Level	Brace	Factored Axial Load (k)	Pu/ϕPn < 1.0
M	7	HSS8x8x5/8	23.17	0.038
			14.62	0.024
	6	HSS8x8x5/8	42.03	0.069
			7.81	0.013
	5	HSS8x8x5/8	68.64	0.113
			21.07	0.035
	4	HSS8x8x5/8	102.17	0.168
			40.9	0.067
	3	HSS8x8x5/8	140.37	0.231
			67.52	0.111
	2	HSS8x8x5/8	181.58	0.299
			98.74	0.162
	Hotel	HSS8x8x5/8	224.88	0.39
			150.83	0.261
	Plaza	HSS8x8x5/8	210.86	0.389
			241.25	0.445

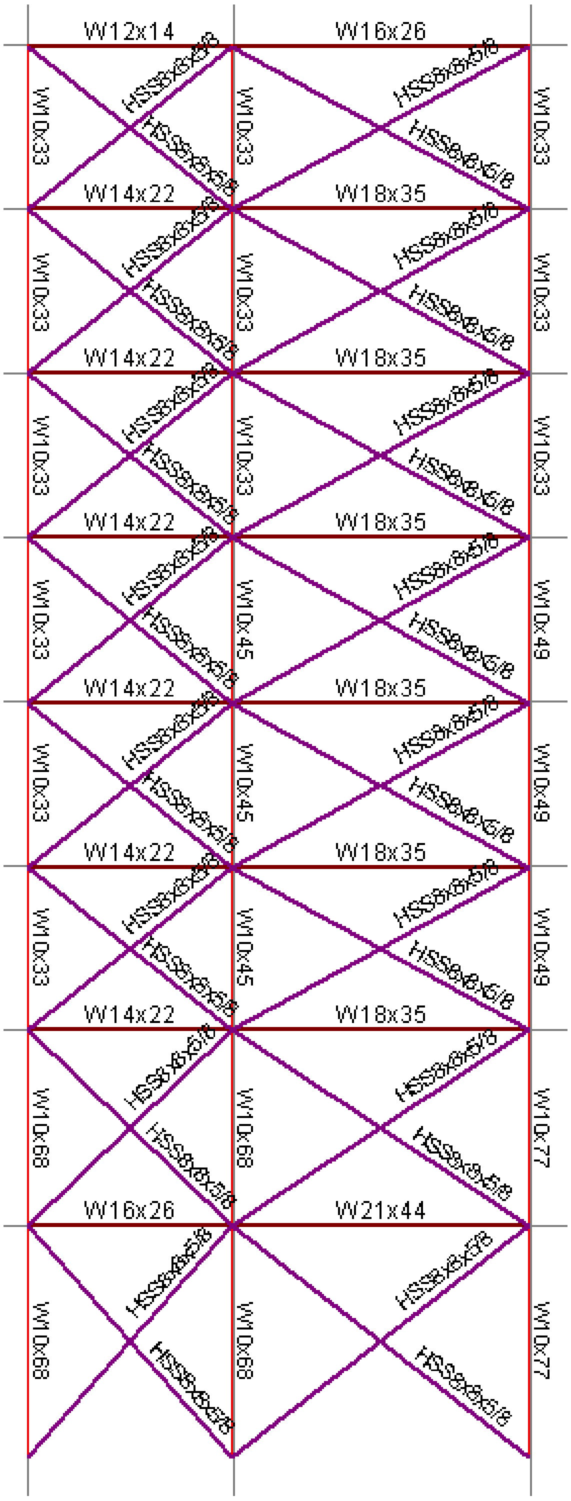
Frame	Level	Brace	Factored Axial Load (k)	Pu/ϕPn < 1.0
M.2	7	HSS8x8x5/8	36.2	0.06
			28.53	0.047
	6	HSS8x8x5/8	72.24	0.119
			47.97	0.079
	5	HSS8x8x5/8	115.17	0.189
			73.45	0.121
	4	HSS8x8x5/8	163.95	0.27
			109.43	0.18
	3	HSS8x8x5/8	216.63	0.356
			151.72	0.25
	2	HSS8x8x5/8	275.06	0.452
			194.62	0.32
	Hotel	HSS8x8x5/8	277.84	0.481
			222.54	0.386
	Plaza	HSS8x8x5/8	276.98	0.51
			292.14	0.538

Frame	Level	Brace	Factored Axial Load (k)	Pu/ϕPn < 1.0
2	7	HSS8x8x5/8	21.69	0.061
			16.39	0.046
	6	HSS8x8x5/8	38.93	0.109
			36.06	0.101
	5	HSS8x8x5/8	62.13	0.174
			64.25	0.18
	4	HSS8x8x5/8	86.05	0.241
			92.4	0.259
	3	HSS8x8x5/8	111.64	0.313
			121.78	0.341
	2	HSS8x8x5/8	140.38	0.393
			129.65	0.363
	Hotel	HSS8x8x5/8	167.75	0.495
			154.33	0.455
	Plaza	HSS8x8x5/8	179.07	0.562
			207.31	0.65

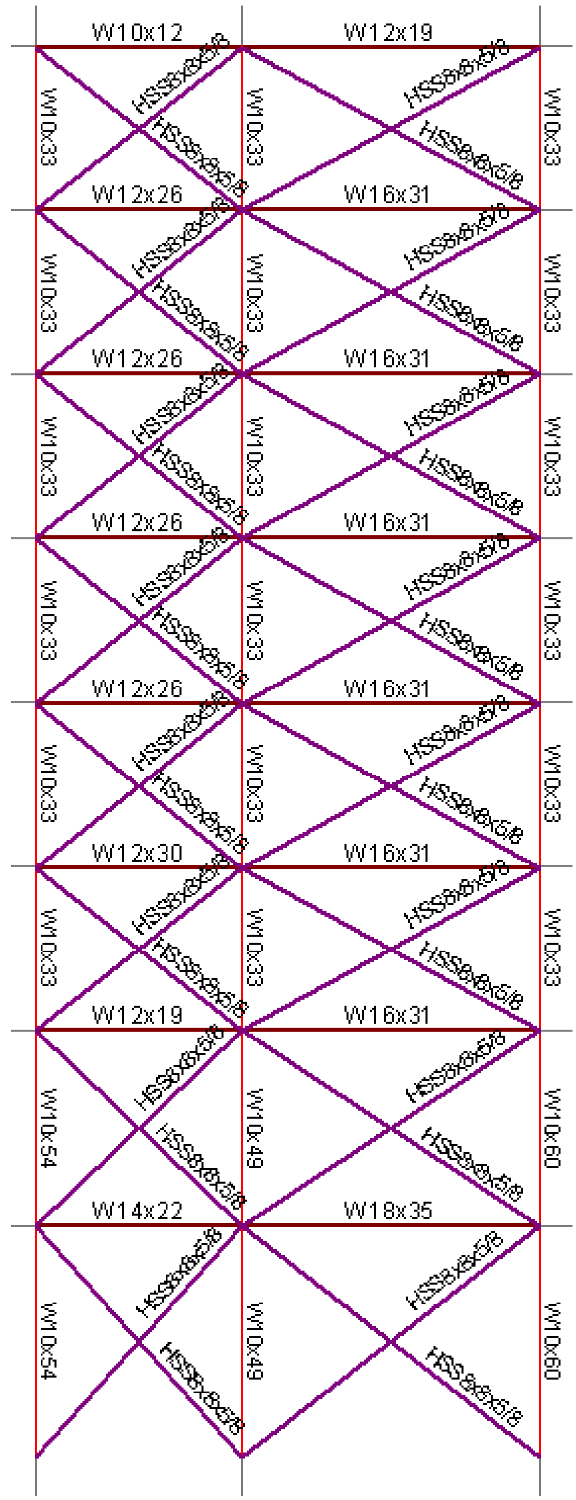
Frame	Level	Brace	Factored Axial Load (k)	Pu/ϕPn < 1.0
8	7	HSS8x8x5/8	45.55	0.102
			52.14	0.117
	6	HSS8x8x5/8	64.09	0.143
			68.12	0.152
	5	HSS8x8x5/8	91.48	0.205
			90.31	0.202
	4	HSS8x8x5/8	119.27	0.267
			115.59	0.259
	3	HSS8x8x5/8	140.72	0.315
			130.49	0.292
	2	HSS8x8x5/8	154.61	0.346
			144.63	0.324
	Hotel	HSS8x8x5/8	177.79	0.419
			163.6	0.386
	Plaza	HSS8x8x5/8	184.28	0.462
			208.52	0.523

The following table summarizes the results of a braced frame column check for Frame 2. This spreadsheet was developed thoroughly to determine the adequacy of the member designs for the braced frame lateral force resisting system.

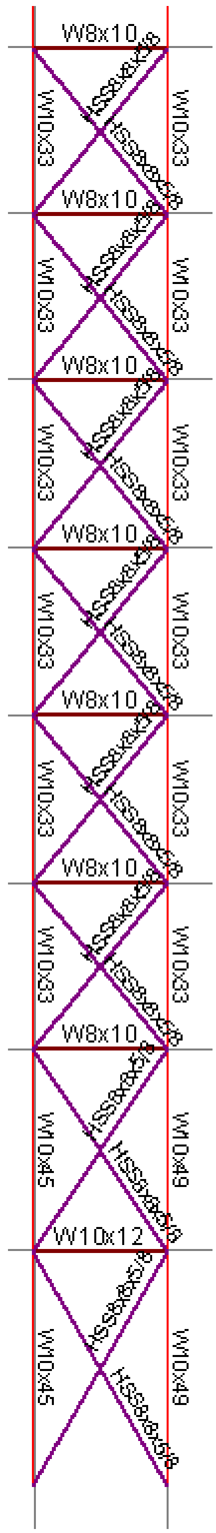
Column	Level	Factored Axial Load, P_u	Moment, M_{ux}	Moment, M_{uy}	Designed Member	pPr	Equation H1-1a or H1-1b?	< 1.0
A.2	7	18.3	0.26	0	W10x33	0.055	H1-1b	0.03
	6	50.66	0.32	0.01		0.153	H1-1b	0.079
	5	93.51	0.31	0.01		0.283	H1-1a	0.285
	4	155.6	-0.24	-0.1	W10x49	0.282	H1-1a	0.284
	3	229.63	-0.66	-0.14		0.417	H1-1a	0.421
	2	319.51	-1.77	-0.16		0.58	H1-1a	0.588
	Hotel	438.94	-7.01	-0.46	W12x96	0.405	H1-1a	0.418
	Plaza	601.85	-7.01	-0.46		0.588	H1-1a	0.601
C.2	7	48.61	0.18	0	W10x33	0.147	H1-1b	0.075
	6	105.29	0.31	0.01		0.319	H1-1a	0.321
	5	182.68	0.7	0		0.553	H1-1a	0.557
	4	287.16	2.17	0.01	W10x68	0.373	H1-1a	0.379
	3	403.75	3.15	0.07		0.525	H1-1a	0.534
	2	533.53	5.31	0.13		0.694	H1-1a	0.709
	Hotel	687.89	16.2	0.44	W12x120	0.506	H1-1a	0.528
	Plaza	897.33	16.2	0.44		0.698	H1-1a	0.72



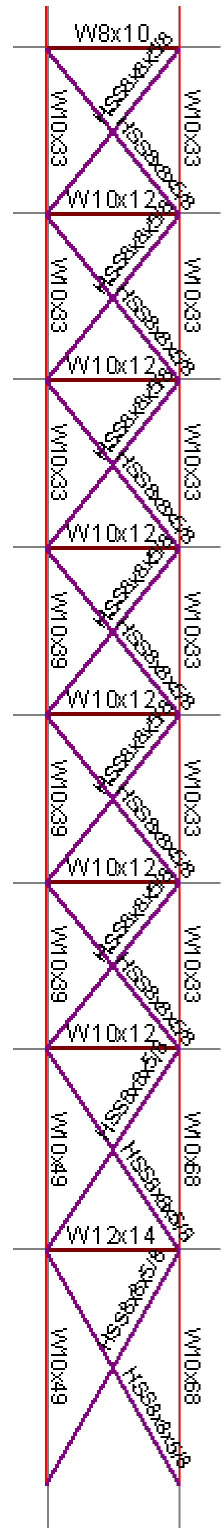
Braced Frame C



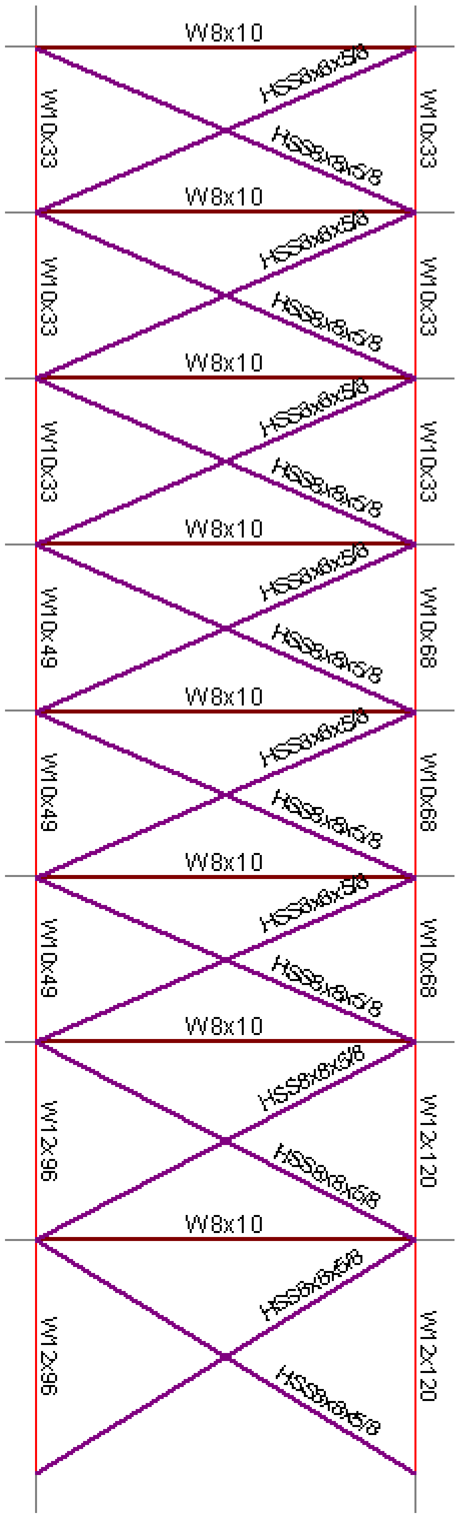
Braced Frame O



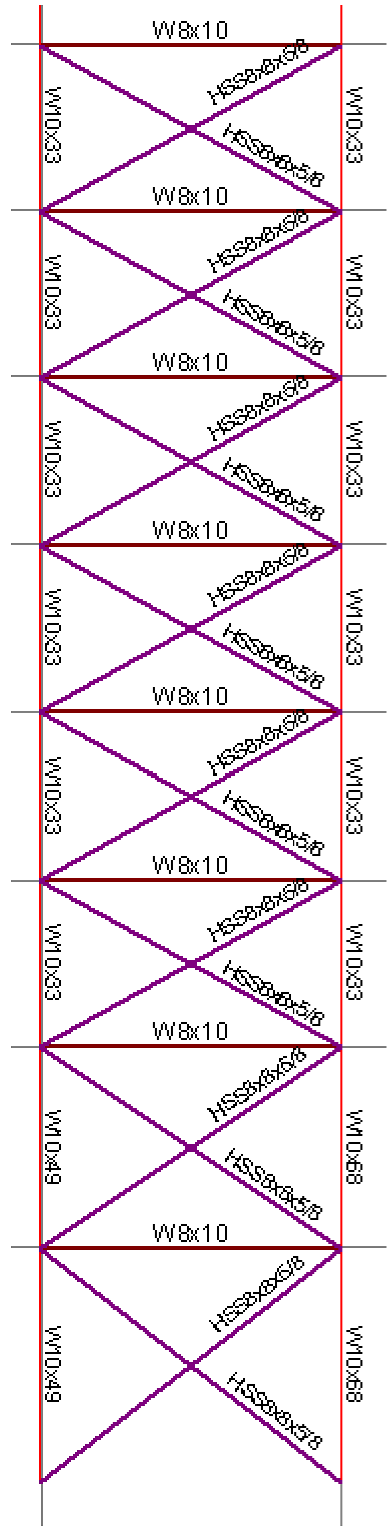
Braced Frame M



Braced Frame M.2



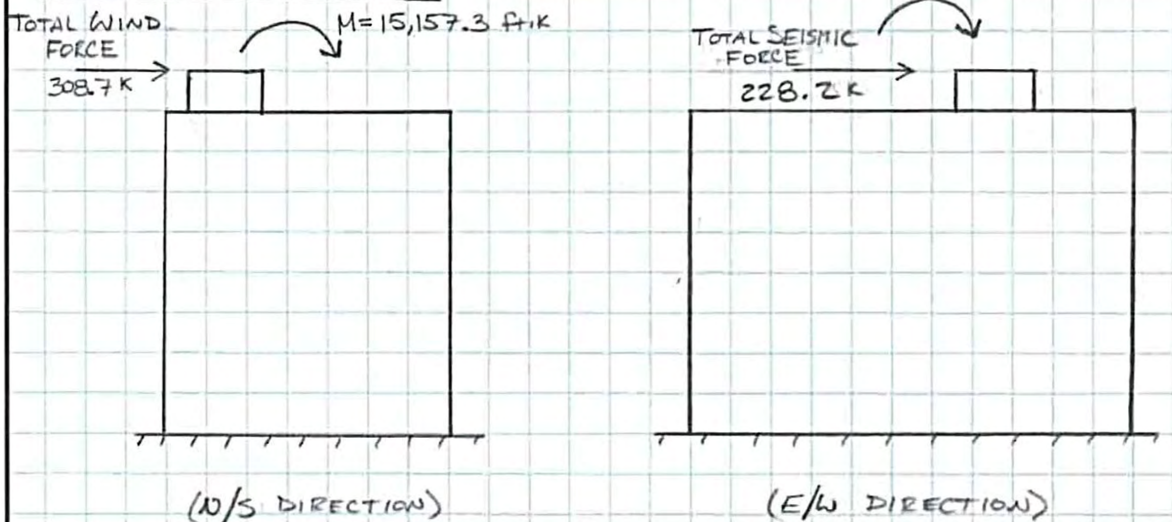
Braced Frame 2



Braced Frame 8

Appendix E: Foundation Check

OVERTURNING MOMENT



- Lateral loads will create overturning moment, while the gravity loads will try to resist the overturning moment.
- To determine if the gravity loads will exceed the lateral loads, the stress due to seismic and lateral loads will be calculated.

STRESS DUE TO DEAD LOADS

$$\Rightarrow \frac{\text{Weight of building}}{\text{Square Footage of Foundation}} = \frac{14{,}260^{K}}{12{,}808\ SF} \times 1000\ lb. = 1113.4\ PSF$$

STRESS DUE TO E/W SEISMIC LOADS

$$\Rightarrow \frac{228.2^{k}\,(1000\text{ lb})}{12{,}808\text{ SF}} = 17.82\text{ PSF}$$

$$\Rightarrow \frac{17.82}{1113.4} \times 100\% = 1.6\%\text{ of Dead Load}$$

STRESS DUE TO N/S LATERAL LOADS

$$\Rightarrow \frac{308.7^{k}\,(1000\text{ lb})}{12{,}808\text{ SF}} = 24.1\text{ PSF}$$

$$\Rightarrow \frac{24.1}{1586.6} \times 100\% = 1.5\%\text{ of Dead Load}$$

- Since the stresses of the lateral and seismic loads are a much smaller percentage of the gravity loads, overturning is not a concern for the design of Cambria Suites Hotel.

Appendix F: Architectural/Façade Study Calculations

THERMAL GRADIENT CALCULATIONS

EXISTING CMU / BRICK SYSTEM :
① Brick (TTW), 4"
② Cavity, 1"
③ Rigid Ins., 2"
④ CMU Block, 8"
⑤ Steel Furrings, 1"
⑥ GWB, 5/8"

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	2	75	88
RH (%)	25	80	50	59
DPT (°F)	33	-3	56	72

$R_o = 0.17$
$R_1 = 0.64$
$R_2 = 0.98$
$R_3 = 10.27$
$R_4 = 1.03$
$R_5 = 0.46$
$R_6 = 0.46$
$R_i = 0.64$

$\Sigma R_{o-i} = 14.66$
$u = 0.0682$

$$T_x = T_o + (T_i - T_o)\left(\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}\right)$$

$$T_1 = 2 + (70-2)\left(\frac{0.81}{14.66}\right) = 5.75\,°F$$

$$T_2 = 2 + (70-2)\left(\frac{1.79}{14.66}\right) = 10.3\,°F$$

$$T_3 = 2 + (70-2)\left(\frac{12.06}{14.66}\right) = 57.9\,°F$$

$$T_4 = 2 + (70-2)\left(\frac{13.09}{14.66}\right) = 62.7\,°F$$

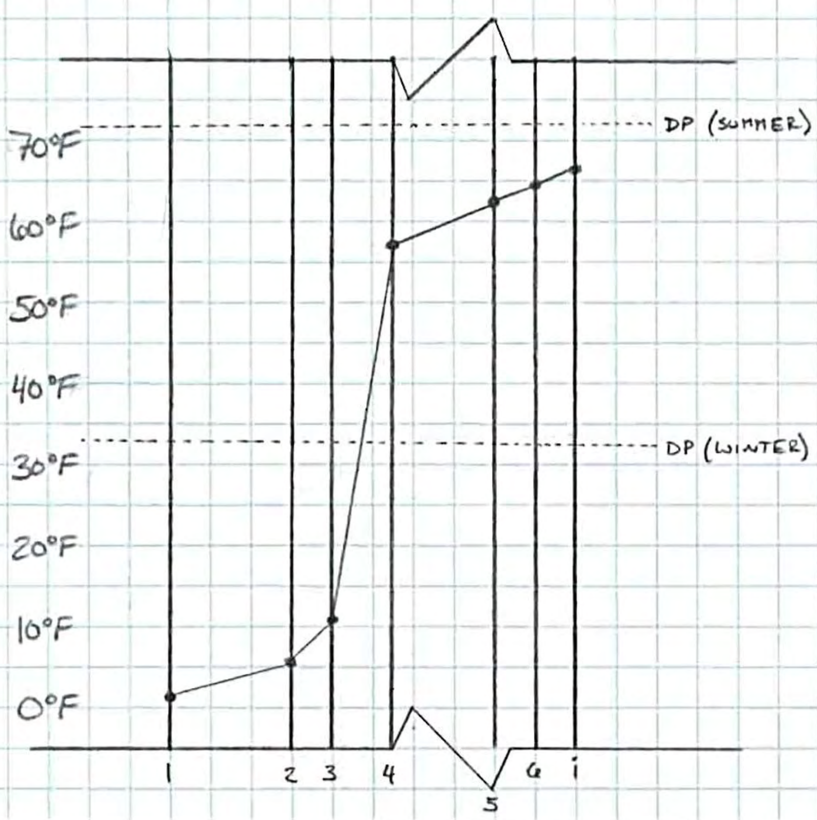
$$T_5 = 2 + (70-2)\left(\frac{13.55}{14.66}\right) = 64.9\,°F$$

$$T_6 = 2 + (70-2)\left(\frac{14.01}{14.66}\right) = 66.98\,°F$$

$$T_i = 2 + (70-2)\left(\frac{14.66}{14.66}\right) = 70\,°F$$

THERMAL GRADIENT

BTW	ΣR_{0-x}	Temp. (°F)
0 - 1	0.17	2
1 - 2	0.81	5.75
2 - 3	1.79	10.3
3 - 4	12.06	57.9
4 - 5	13.09	62.7
5 - 6	13.55	64.9
6 - i	14.01	66.98
	14.66	70° F



OPTION 1 - BRICK VENEER SYSTEM :
① Brick (TTW), 4"
② Cavity, 1"
③ Poly Film, 4 MIL
④ Batt. Ins., 4"
⑤ GWB, 5/8"

	Winter		Summer	
	Int.	Ext.	Int.	Ex.
Temp. (°F)	70	2	75	88
RH (%)	25	80	50	59
DPT (°F)	33	-3	56	72

$R_o = 0.17$
$R_1 = 0.64$
$R_2 = 0.98$
$R_3 = 0.12$
$R_4 = 12.19$
$R_5 = 0.46$
$R_i = 0.64$

$\Sigma R_{o-i} = 15.2$
$u = 0.0658$

$$T_x = T_o + (T_i - T_o)\left(\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}\right)$$

$$T_1 = 2 + (70-2)\left(\frac{0.81}{15.2}\right) = 5.62\ °F$$

$$T_2 = 2 + (70-2)\left(\frac{1.79}{15.2}\right) = 10\ °F$$

$$T_3 = 2 + (70-2)\left(\frac{1.91}{15.2}\right) = 10.54\ °F$$

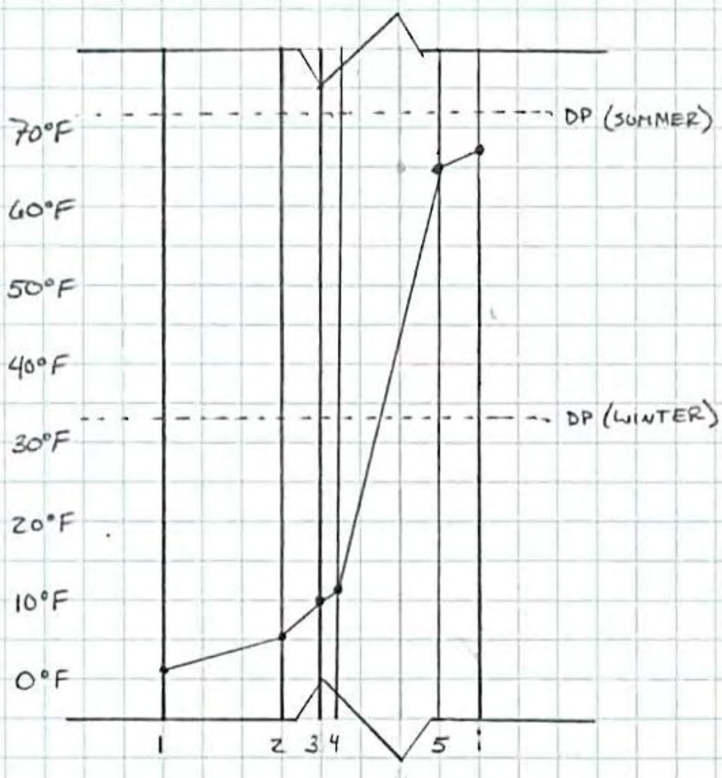
$$T_4 = 2 + (70-2)\left(\frac{14.1}{15.2}\right) = 65.08\ °F$$

$$T_5 = 2 + (70-2)\left(\frac{14.56}{15.2}\right) = 67.14\ °F$$

$$T_i = 2 + (70-2)\left(\frac{15.2}{15.2}\right) = 70\ °F$$

THERMAL GRADIENT

BTW	ΣR_{0-x}	Temp. (°F)
0 - 1	0.17	2
1 - 2	0.81	5.62
2 - 3	1.79	.10
3 - 4	1.91	10.54
4 - 5	14.1	65.08
5 - i	14.56	67.14
	15.2	70°F



OPTION 2 - CURTAIN WALL SYSTEM :
① Glass
② Cavity, 1/2"
③ Glass

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp. (°F)	70	2	75	88
RH (%)	25	80	50	59
DPT (°F)	33	-3	56	72

$R_o = 0.17$
$R_1 = 2.045$
$R_2 = 0.98$
$R_3 = 2.045$
$R_i = 0.64$

$\Sigma R_{o-i} = 5.88$
$U = 0.17$

$$T_x = T_o + (T_i - T_o)\left(\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}\right)$$

$$T_1 = 2 + (70-2)\left(\frac{2.215}{5.88}\right) = 27.62\,°F$$

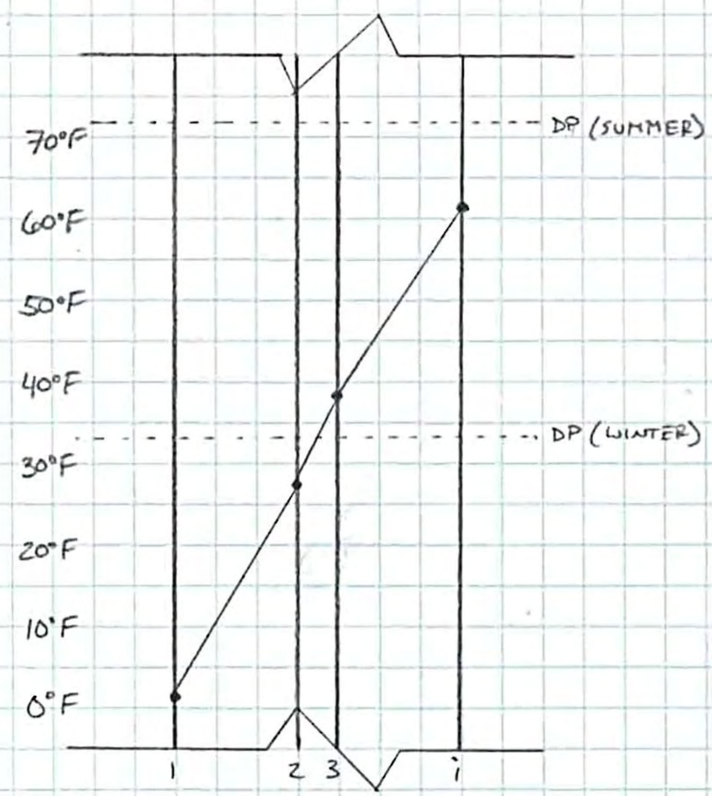
$$T_2 = 2 + (70-2)\left(\frac{3.195}{5.88}\right) = 38.95\,°F$$

$$T_3 = 2 + (70-2)\left(\frac{5.24}{5.88}\right) = 62.6\,°F$$

$$T_i = 2 + (70-2)\left(\frac{5.88}{5.88}\right) = 70°\,F$$

THERMAL GRADIENT

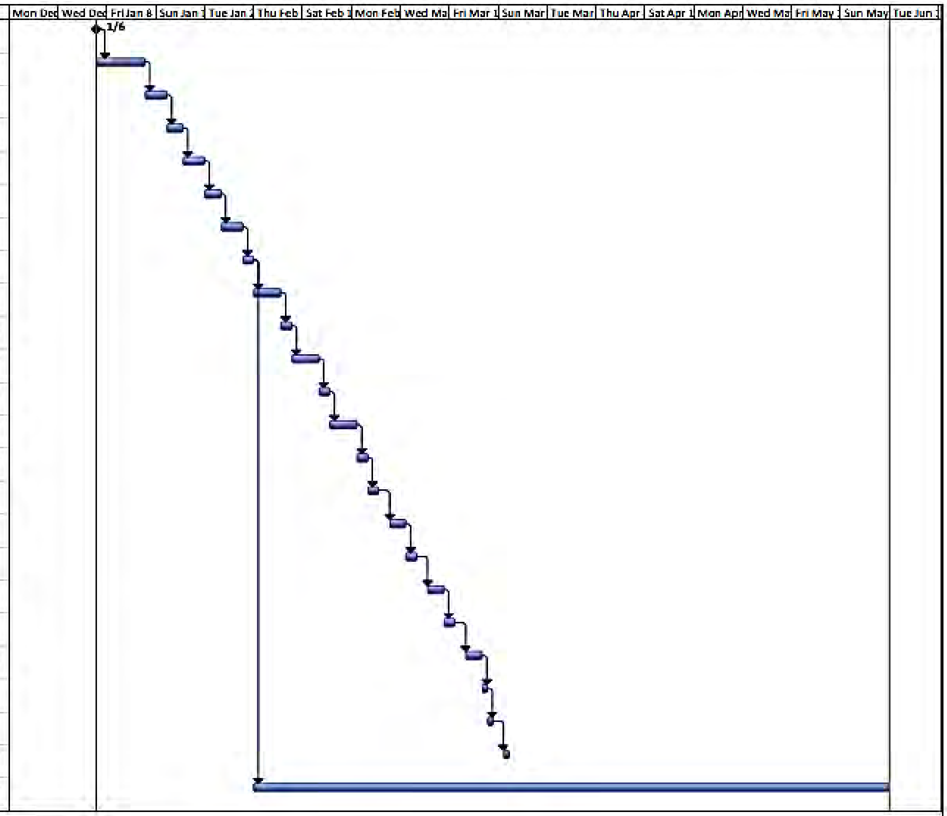
BTW	ΣRo-x	Temp. (°F)
0 - 1	0.17	2
1 - 2	2.215	27.62
2 - 3	3.195	38.95
3 - i	5.24	62.6
	5.88	70°F



Appendix G: Construction Schedule & Cost Calculations

Redesigned Construction Schedule

ID		Task	Task Name	Duration	Start	Finish
1			Start Date	0 days	Wed 1/6/10	Wed 1/6/10
2			Erect Steel Columns: Levels Plaza-Hotel	7 days	Wed 1/6/10	Thu 1/14/10
3			Erect Steel Girders & Lateral Bracing: Hotel Level	2 days	Fri 1/15/10	Mon 1/18/10
4			Pre-Cast Concrete Plank: Hotel Level	3 days	Tue 1/19/10	Thu 1/21/10
5			Erect Steel Girders & Lateral Bracing: 2nd Level	2 days	Fri 1/22/10	Mon 1/25/10
6			Pre-Cast Concrete Plank: 2nd Level	3 days	Tue 1/26/10	Thu 1/28/10
7			Erect Steel Columns: Levels 2nd-4th	2 days	Fri 1/29/10	Mon 2/1/10
8			Erect Steel Girders & Lateral Bracing: 3rd Level	2 days	Tue 2/2/10	Wed 2/3/10
9			Pre-Cast Concrete Plank: 3rd Level	3 days	Thu 2/4/10	Mon 2/8/10
10			Erect Steel Girders & Lateral Bracing: 4th Level	2 days	Tue 2/9/10	Wed 2/10/10
11			Pre-Cast Concrete Plank: 4th Level	3 days	Thu 2/11/10	Mon 2/15/10
12			Erect Steel Girders & Lateral Bracing: 5th Level	2 days	Tue 2/16/10	Wed 2/17/10
13			Pre-Cast Concrete Plank: 5th Level	3 days	Thu 2/18/10	Mon 2/22/10
14			Erect Steel Columns: Levels 5th-7th	2 days	Tue 2/23/10	Wed 2/24/10
15			Erect Steel Girders & Lateral Bracing: 6th Level	2 days	Thu 2/25/10	Fri 2/26/10
16			Pre-Cast Concrete Plank: 6th Level	3 days	Mon 3/1/10	Wed 3/3/10
17			Erect Steel Girders & Lateral Bracing: 7th Level	2 days	Thu 3/4/10	Fri 3/5/10
18			Pre-Cast Concrete Plank: 7th Level	3 days	Mon 3/8/10	Wed 3/10/10
19			Erect Steel Girders & Lateral Bracing: Roof Level	2 days	Thu 3/11/10	Fri 3/12/10
20			Pre-Cast Concrete Plank: Roof Level	3 days	Mon 3/15/10	Wed 3/17/10
21			Erect Steel Columns: Roof Level	1 day	Thu 3/18/10	Thu 3/18/10
22			Erect Steel Girders: High Roof level	1 day	Fri 3/19/10	Fri 3/19/10
23			Pre-Cast Concrete Plank: High Roof level	1 day	Mon 3/22/10	Mon 3/22/10
24			Masonry Veneer	83 days	Thu 2/4/10	Mon 5/31/10



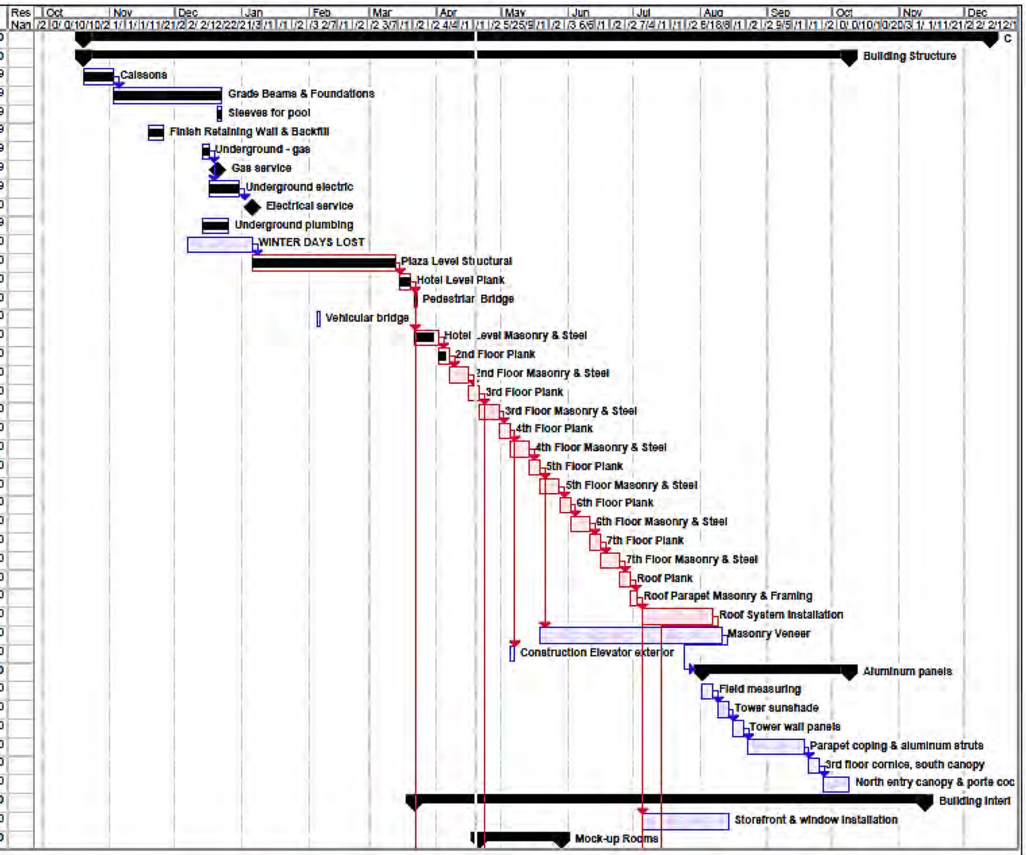
Project: Cambria Suites Schedule_
Date: Mon 3/21/11

Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | Deadline | Progress

Page 1

Existing Construction Schedule

ID	Task Name	Duration	Start	Finish	Res Nan
1	**CAMBRIA SUITES - NEW ARENA HOTEL**	**298 days**	**Tue 10/20/09**	**Mon 12/13/10**	
2	**Building Structure**	**253 days**	**Tue 10/20/09**	**Fri 10/8/10**	
3	Caissons	10 days	Tue 10/20/09	Mon 11/2/09	
4	Grade Beams & Foundations	7 wks	Tue 11/3/09	Tue 12/22/09	
5	Sleeves for pool	2 days	Mon 12/21/09	Tue 12/22/09	
6	Finish Retaining Wall & Backfill	5 days	Thu 11/19/09	Wed 11/25/09	
7	Underground - gas	3 days	Mon 12/14/09	Wed 12/16/09	
8	Gas service	0 days	Mon 12/21/09	Mon 12/21/09	
9	Underground electric	10 days	Thu 12/17/09	Wed 12/30/09	
10	Electrical service	0 days	Wed 1/6/10	Wed 1/6/10	
11	Underground plumbing	10 days	Mon 12/14/09	Fri 12/25/09	
12	WINTER DAYS LOST	22 days	Mon 12/7/09	Tue 1/5/10	
13	Plaza Level Structural	48 days	Wed 1/6/10	Fri 3/12/10	
14	Hotel Level Plank	1 wk	Mon 3/15/10	Fri 3/19/10	
15	Pedestrian Bridge	1 day	Mon 3/22/10	Mon 3/22/10	
16	Vehicular bridge	1 day	Fri 2/5/10	Fri 2/5/10	
17	Hotel Level Masonry & Steel	9 days	Mon 3/22/10	Thu 4/1/10	
18	2nd Floor Plank	3 days	Fri 4/2/10	Tue 4/6/10	
19	2nd Floor Masonry & Steel	7 days	Wed 4/7/10	Thu 4/15/10	
20	3rd Floor Plank	3 days	Fri 4/16/10	Tue 4/20/10	
21	3rd Floor Masonry & Steel	7 days	Wed 4/21/10	Thu 4/29/10	
22	4th Floor Plank	3 days	Fri 4/30/10	Tue 5/4/10	
23	4th Floor Masonry & Steel	7 days	Wed 5/5/10	Thu 5/13/10	
24	5th Floor Plank	3 days	Fri 5/14/10	Tue 5/18/10	
25	5th Floor Masonry & Steel	7 days	Wed 5/19/10	Thu 5/27/10	
26	6th Floor Plank	3 days	Fri 5/28/10	Tue 6/1/10	
27	6th Floor Masonry & Steel	7 days	Wed 6/2/10	Thu 6/10/10	
28	7th Floor Plank	3 days	Fri 6/11/10	Tue 6/15/10	
29	7th Floor Masonry & Steel	7 days	Wed 6/16/10	Thu 6/24/10	
30	Roof Plank	3 days	Fri 6/25/10	Tue 6/29/10	
31	Roof Parapet Masonry & Framing	3 days	Wed 6/30/10	Fri 7/2/10	
32	Roof System Installation	5 wks	Mon 7/5/10	Fri 8/6/10	
33	Masonry Veneer	12 wks	Wed 5/19/10	Tue 8/10/10	
34	Construction Elevator exterior	2 days	Wed 5/5/10	Thu 5/6/10	
35	**Aluminum panels**	**50 days**	**Mon 8/2/10**	**Fri 10/8/10**	
36	Field measuring	5 days	Mon 8/2/10	Fri 8/6/10	
37	Tower sunshade	5 days	Mon 8/9/10	Fri 8/13/10	
38	Tower wall panels	5 days	Mon 8/16/10	Fri 8/20/10	
39	Parapet coping & aluminum struts	20 days	Mon 8/23/10	Fri 9/17/10	
40	3rd floor cornice, south canopy	5 days	Mon 9/20/10	Fri 9/24/10	
41	North entry canopy & porte cochere	10 days	Mon 9/27/10	Fri 10/8/10	
42	**Building Interior**	**170 days**	**Mon 3/22/10**	**Fri 11/12/10**	
43	Storefront & window installation	30 days	Mon 7/5/10	Fri 8/13/10	
44	**Mock-up Rooms**	**28 days**	**Wed 4/21/10**	**Fri 5/28/10**	



Project: Cambria suites (update 2)
Date: Mon 4/19/10

Task | Critical Task | Progress | Milestone | Summary | Rolled Up Task | Rolled Up Critical Task | Rolled Up Milestone | Rolled Up Progress | Split | External Tasks | Project Summary | Group By Summary | Deadline

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Cost Estimate of Redesigned System											
Steel	Amount	Unit	Crew	Daily Output	Labor Hours/Unit	Labor Hours	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
Columns	4986	LF	E-2	984	0.057	284	84	2.7	1.65	88.35	99	493614.00
Baseplates	119.2	SF	E-2	60	0.061	7	46	0	0	46	n/a	5483.20
Beams	9435	LF	E-5	912	0.088	830	62	3.99	1.8	55.29	63.5	599122.50
Braces	2368	LF	E-5	n/a	n/a	n/a	47.14	3.79	2.32	53.25	n/a	126096.00
Fireproofing	95400	SF	G-2	3000	0.008	763	1.31	0.29	0.04	1.64	1.95	186030.00
Concrete	Amount	Unit	Crew	Daily Output	Labor Hours/Unit	Labor Hours	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
8" P.C. Plank	120000	SF	C-11	3200	0.023	2760	7.2	1.07	0.6	8.87	10.45	1254000.00
											Total Cost of Redesigned System:	2664345.70

Cost Estimate of Existing System												
Shearwalls	Amount	Unit	Crew	Daily Output	Labor Hours/Unit	Labor Hours	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
8" CMU, reinforced	59904	SF	D-8	395	0.101	6050	2.62	4.03	0	6.65	9.35	560102.40
12" CMU, reinforced	12339	SF	D-9	300	0.16	1974	3.65	6.25	0	9.9	14	172746.00
Steel	Amount	Unit					Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
Columns	1224	LF	E-2	984	0.057	70	84	2.7	1.65	88.35	99	121176.00
Baseplates	52.2	SF	E-2	60	0.061	3	46	0	0	46	0	2401.20
Beams	2888	LF	E-5	1110	0.072	208	68	3.45	1.56	73.01	83	239704.00
Fireproofing	27180	SF	G-2	3000	0.008	217	1.31	0.29	0.04	1.64	1.95	53001.00
Concrete	Amount	Unit	Crew	Daily Output	Labor Hours/Unit	Labor Hours	Material Cost/Unit	Labor Cost/Unit	Equipment Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	Total Cost
10" P.C. Plank	120000	SF	C-11	3600	0.02	2400	7.5	0.95	0.53	8.98	10.55	1266000.00
											Total Cost of Existing System:	2415130.60