Hyatt Regency

Pittsburgh International Airport

Pittsburgh, PA

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The Pennsylvania State University

Department of Architectural Engineering

Hyatt Regency – Hotel and Conference Center

Pittsburgh International Airport
1111 Airport Boulevard
Pittsburgh, PA 15231

PROJECT TEAM

- *Owner* – Dauphin Co. General Authority
- *Architects* – L. Robert Kimball & Associates
- *Structural Engineers* – DeSimone Consulting Engineers
- *MEP Engineers* – L. Robert Kimball & Associates
- *Electrical Engineers* – L. Robert Kimball & Associates
- *General Contractor* – Dick Corporation

ARCHITECTURE AND OVERVIEW

- Designed to compliment the adjacent terminals at Pittsburgh International Airport
- 275,000 square feet
- 11-story main tower featuring 336 guest rooms including 11 suites
- 1-story conference center featuring 20,000 square feet of function space
- Combination of pre-cast concrete panels and a glass / aluminum curtain wall

STRUCTURAL

- Foundation consists of a combination of piles, spread footings, and grade beams
- 6-inch slab on grade
- Concrete cast-in-place moment frame, 8-inch filigree slab construction for tower
- Steel framing over conference center with average bay size of 25-foot by 25-foot

CONSTRUCTION

- $35 million estimated cost
- Construction manager at risk
- Construction dates: November 1998 – May 2000
- Follows design constraints of FAA for proximity to the airport

LIGHTING/ELECTRICAL

- 2500kVA transformer provides 3-phase, 4-wire 480/277V supply to building
- Each floor has a transformer to step power down to 208/120V for general use
- Lighting – 150 W incandescent in guest areas, 32W fluorescent for service areas
- Electrical system backed up by a 600A, 400kW, 480Y/227V emergency generator

MECHANICAL

- 4 gas boilers in mechanical room supply hot water to the building
- 2 rooftop cooling towers provide chilled water supply
- 13 air-handling-units with supply fans provide 410 cfm of fresh air to each floor

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EXECUTIVE SUMMARY

The Hyatt Regency Hotel and Conference Center is located adjacent to the Pittsburgh International Airport in Pittsburgh, PA. The 275,000 square foot Hyatt consists of a 12 story tower with guest rooms and a 1 story conference center. It is also the only hotel located on airport property.

The original design of the Hyatt Regency tower is a system of concrete moment frames and filigree floor slabs. The conference center is constructed of steel framing, typically employing wide flange shapes. The tower resists lateral loads through its concrete moment frames, while the conference center relies on steel braced frames.

The seismic loading on the existing tower control the lateral resisting frame design in the East-West direction over the wind loading that would typically control in the local area. The seismic loads are very large due to the weight of the concrete structure. Analysis has been carried out to compare more lightweight steel framing to the original concrete framing to determine if the steel framing is a more viable alternative. There are a number of design constraints on the project. Foremost is a height limitation due to its proximity to the airport. The new steel design will attempt to stay within all architectural constraints, while reducing the building weight and overall seismic loads.

Hand calculations were performed to generate initial member sizing and to iterate a floor layout. After a suitable initial design was reached, a computer model was created in RAM Structural Systems to assist in member design and load calculations. Chevron braced frames were added in locations that did not interfere with the architectural layout to resist the lateral loads on the structure. From the calculations, it was found that the building weight and seismic loadings were greatly reduced in the steel framing as compared to the concrete framing. However, even with small member sizes with minimal depth, the building height was still impacted slightly. Additional vibration, mechanical/fire protection, and construction management analyses were also performed to determine the viability of the new steel framed design.

The new steel framing was found to support the proposed reduction in weight and seismic loading. Based on other conditions such as the possible vibration problems, the increased cost, and the increased building height, the alternative framing does not seem to be the best choice. In other situations, where the height limit is not a major controlling factor, or where seismic loads need to be decreased, the steel framing seems to be the best selection.

INTRODUCTION

Project Team

Owner:		Dauphin County General Authority www.thegeneralauthority.com
Architects:	*Primary* -	L. Robert Kimball & Associates www.lrkimball.com
	Associate -	Thompson, Ventulett, Stainback, and Associates www.tvsa.com
Structural Engineers:		DeSimone Consulting Engineers www.de-simone.com
MEP Engineers:		L. Robert Kimball & Associates www.lrkimball.com
Electrical Engineers:		L. Robert Kimball & Associates www.lrkimball.com
General Contractor:		Dick Corporation www.dickcorp.com

General Project Information

Building Name:	Hyatt Regency Hotel and Conference Center	
Location and Site:	Pittsburgh International Airport 1111 Airport Boulevard Pittsburgh, PA 15231	
Building Occupant Name:	Global Hyatt Corporation	
Occupancy or Function Types:	Primary Occupancy:	Hotel
	Secondary Occupancy:	Conference Center
Size:	275,000 Sq. Ft.	
Number of Stories Above Grade:	11 Story Main Tower (+1 level partially below grade) 1 Story Conference Center (+1 level partially below grade)	
Dates of Construction:	Planned	– November 1998 - July 2000
	Actual	– November 1998 - May 2000
Costs:	Building Cost	– approx. $ 30 million
	Soft Costs	– approx. $ 3 million
Project Delivery Method:	Construction Manager at Risk	

Building Overview

The Hyatt Regency is located adjacent to the Pittsburgh International Airport's Landside Terminal and is the only hotel on airport property. The 12-story tower houses 336 guest rooms, including 11 suites, designed make guests' stay comfortable and convenient. The Hyatt features a coffee bar, health club, indoor pool, sauna, and Mediterranean restaurant among other amenities. The conference center features 20,000 sq. ft. of function and 7,400 sq. ft. of pre-function space and the largest hotel ballroom outside of downtown Pittsburgh. The hotel is approximately 17,000 sq. ft. per floor with typical floor to floor heights of 10'-0". The main level has a 20'-0" height to accommodate a spacious lobby.

Figure 1. – Exterior of Building Tower (left) and Conference Center (right)

Figure 2. – Interior of Typical Hotel Guest Room

General Existing Conditions

Architecture:

The building's architecture is designed to compliment that of the terminal buildings. The tower's exterior is a combination of pre-cast concrete panels and a glass and aluminum curtain wall. The conference center is clad with pre-cast concrete, spandrel glass, and metal paneling. At entrance level, around the main lobby, a curved glass curtain wall welcomes guests to the Hyatt.

Structural:

The structural system is divided into two independent parts: the main tower and the low-rise conference center. At ground level, both parts of the building use 6" slab on grade with 6x6 W2.0x2.0 WWF reinforcement. Concrete reinforcement is specified by ACI 318-89. All bolted steel connections are A325 or A490 slip critical, ¾" long bolts. Lateral resistance is typically provided with steel braced moment frames in the conference center and concrete moment frames in the main tower.

> <u>Main Tower:</u> The main tower is primarily a cast-in-place concrete structure with an exterior curtain wall. The tower's foundation consists of piles spaced on an approximately 27' x 20' grid. The pile caps extend from the top of shale to between 1' to 21' below the main level based on changes in grade. The concrete columns are typically sized at 22" x 28" or 22" x 32". Typical floor composition is an 8" filigree floor system consisting of a 2¼" precast slab, 3½" voids, and 2¼" cast-in-place concrete. Typical column strips are 6'-0" with no voids. The perimeter of the building has 18" deep drop-beams with varying widths.
>
> <u>Low-rise Conference Center:</u> The conference center is a steel structure consisting of average bay sizes of approximately 25' x 25'. The ballroom adds a large bay size of approximately 72' x 130'. The conference center's foundation consists of various sized spread footings and grade beams. Spread footings range in size between 5'x5'x12" to 14'x14'x27". Grade beams range in size from 18"x24" to 26"x40". Typical column sizes range from W10x33 to W10x49. Beams are typically W21x44 to W24x76, depending on span. The roof is a standard 3"-18 GA. roof decking.

Construction:

The Hyatt was constructed in accordance to the FAA regulations to building height in proximity to an airport. The design phase of the building started in March 1998 and Dick Corporation, the General Contractor, was permitted to move on site in

November 1998. Construction was completed in May 2000. The project was delivered via. construction manager at risk. The total building cost totaled $32 million.

Fire Protection:

The building is classified as 'fire-resistive' based on the Pennsylvania L & I Fire and Panic Code. It has been designed with 2 hour fire rated walls around the elevators and stairs. The other rooms including guest, mechanical, and storage rooms have 1 hour fire rated walls. All areas have automatic sprinkler systems installed. The automatic sprinklers and 2½-inch fire department hose valves are supplied by a 1250 gpm diesel fire pump located in the North-West corner of the low-rise section.

Mechanical:

A total of 13 main air handling units (AHU) provide air throughout the building. Each AHU is routed to a VAV box, supplying approximately uniform cfm to various spaces. Additionally, there is a make up air unit supplying 11,500 cfm to the kitchen. The tower incorporates 3 rooftop heat recovery fresh air units providing around 12,000 cfm of fresh air each, approximately 1200 cfm to each floor. In addition, supply fans provide 4510 cfm (410 cfm per floor) to pressurize and ventilate the stairwells.

4 gas-boilers in the mechanical room supply hot water to the building, while 2 cooling towers located on the roof of the low-rise section provide chilled water. The boilers are set atop 4" concrete pads, below the AHUs in the mechanical penthouse located above. Each water supply (condensed, chilled, and heated) is distributed through 2 pumps with 1 additional stand-by pump for backup.

Lighting/Electrical:

The primary electrical supply is an exterior 2500kVA, 480/277V transformer. From the main transformer, feeders distribute the supply to various transformers through feeders ranging from: 4-wire, 3 inch down to 3-wire, 1¼ inch. The electrical system is backed up by a 600A, 400kW, 480Y/227V emergency generator. Each floor of the tower has a transformer to step down from 480Y/227V to 208/120V to meet lighting and receptacle power requirements. Power is then distributed to 3 switchboards per floor with a 4 wire, 2½ inch feeder.

The conference rooms, lobbies, and ballroom are typically illuminated with 150W incandescent lighting; while the ballroom also has multiple series of 7 cable suspended pendant luminaries totaling 330W each. The guest rooms and hallways in the main tower also have incandescent lighting. Service areas including housekeeping, mechanical rooms, electrical rooms, and offices use fluorescent lighting.

DEPTH WORK – STRUCTURAL ANALYSIS AND DESIGN

Original Structural System

The Hyatt Regency at the Pittsburgh International Airport is a 275,000 square foot hotel and conference center. The building consists of a 12-story tower with a 2-story attached conference center; each has 1 level partially below grade. The building has a combination of structural steel and cast-in-place concrete framing. The conference center is primarily structural steel framing while the hotel tower is cast-in-place concrete.

Foundation Systems (Spread Footings, Pile Caps, and Piles)

Spread Footings

- Spread footings are used under the conference center.
- Bottom of footings is -3'-6" below grade.
- 15 different sizes of spread footings
    - o 11 different square footings range from: 5' x 5' to 14' x 14'
      Footings are from 12" to 27" deep with rebar sizes from #4 to #8.
    - o 4 different rectangular footings ranging from: 10' x 14' to 12' x 26'
      Footings are from 23" to 28" deep with rebar sized from #7 to #9.

Pile Caps

- Piles and pile caps are used under the main hotel tower.
- 3 sizes of pile caps are used, as follows:
    - o Exterior pile caps are roughly triangular, see Pile Cap 1. They are 48" deep and have #11 rebar in 3 directions.
    - o Interior pile caps are square, see Pile Cap 2. They are 43" deep and have #8 rebar in both directions.
    - o Pile caps at stair wells are rectangular, see Pile Cap 3. They are 40" deep and have #11 rebar in the long direction and #4 rebar in the short.

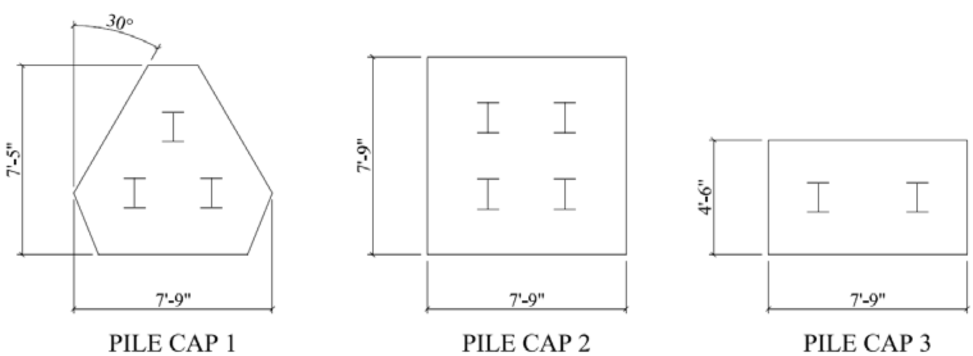


Figure 3. – Pile Cap Dimensions

Piles

- Allowable gross bearing capacity of soil is 2ksf, requiring piles below the tower.
- Piles are typically HP14x117 Bearing Piles.
- Piles are driven to depths of around 90-100 feet until shale is reached.

Floor Systems

Tower (Guest Rooms)

The tower is framed as cast-in place concrete. The concrete moment frames act as a lateral resisting system for the building as well as providing the primary gravity system. Each floor of the tower is approximately 17,000 square feet.

The tower is a system of concrete columns and a one-way slab system. There are 44 columns in the typical tower floor plan, 22”x28” or 22”x32”, with 4 smaller columns, 12”x18” or 16”x24” columns around each of the two stair towers. Typical bay sizes are: 27’-0”x18’-0” and 27’-0”x23’-0”. *(See plans, next page.)*

6’ wide, 8” deep column strips are oriented N-S on the typical tower plans. The floor slab consists of an 8” thick slab with polystyrene voids in a typical layout between column strips (see plan and section views below).

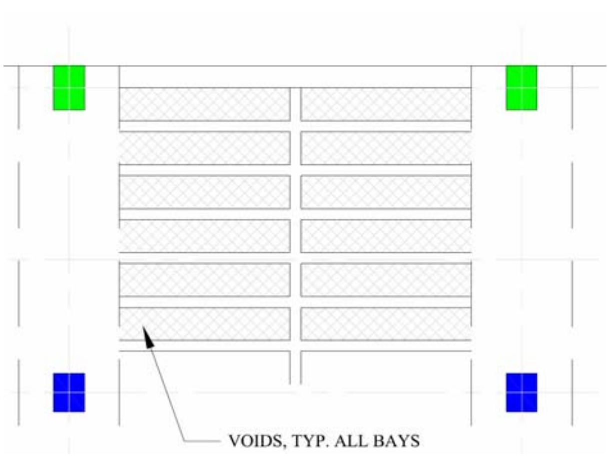


Figure 4. – Plan of Typical Bay Void Layout

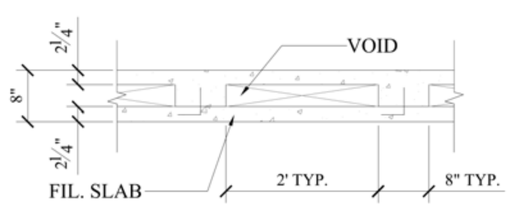
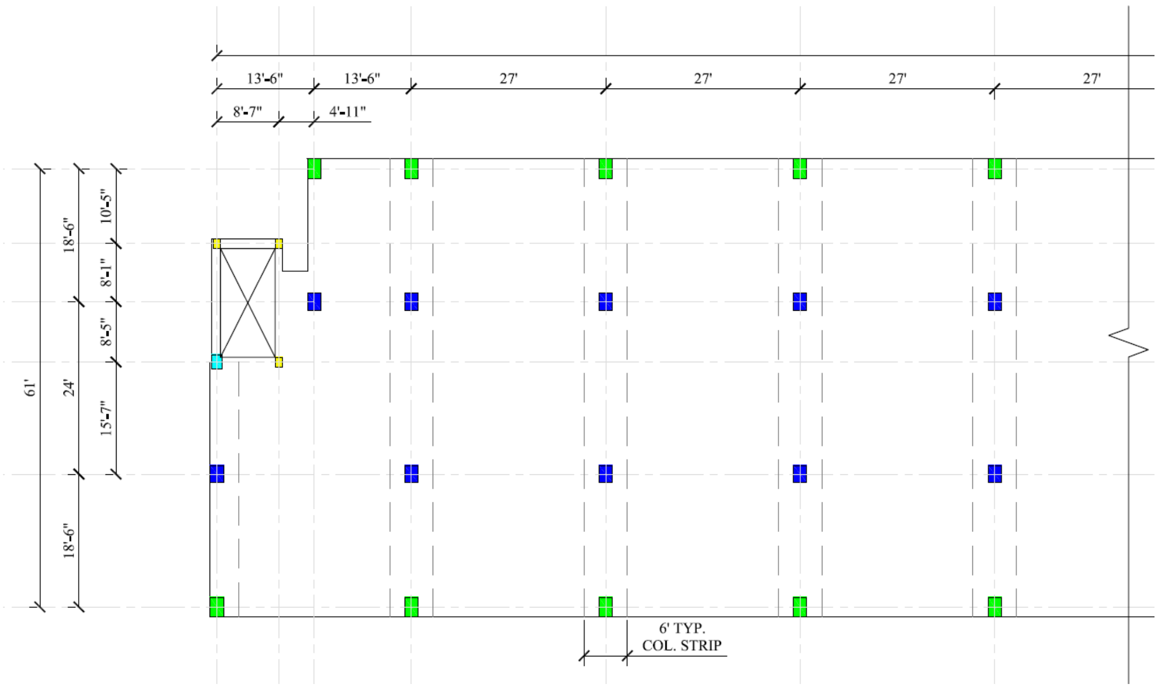
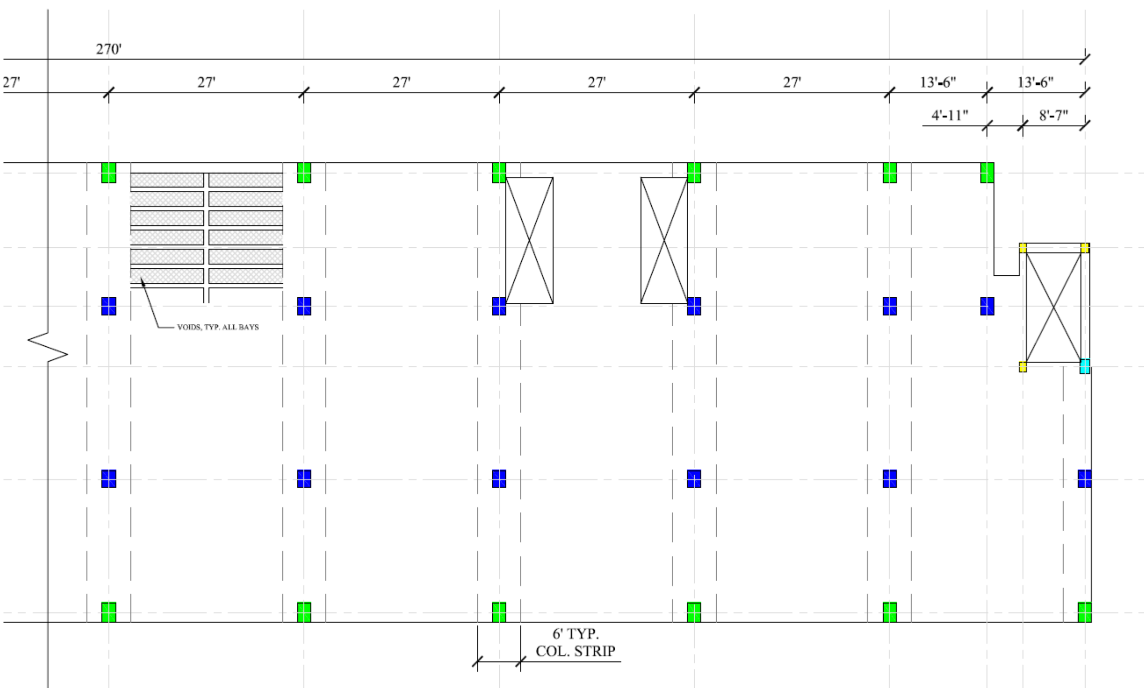


Figure 5. – Section Through Voids



TOWER FRAMING PLAN – WEST END



TOWER FRAMING PLAN – EAST END

Figure 6. – Layout of Existing Floor Plan (Tower)

Conference Center - As the tower is the primary structural system investigated in this report, information on the conference center will be of a more general nature.

The conference center portion of the building is framed with structural steel; approximately 17,000 square feet on the ground floor (partially below grade) and approximately 50,000 square feet on the first floor. Steel connections are made with standard A325 or A490, ¾" bolts and welds are specified on the structural drawings as being no smaller than ¼". At moment connections, the connections are designed for the full moment capacity of the beams.

- <u>Ground Level:</u> The ground level framing supports a composite steel deck and concrete floor slab. A continuous 14" concrete foundation wall contains embedded plates to attach to the steel framing. The wall also acts as a retaining wall for soils around the section that is below grade. The steel framing for the first floor is typically W12X19 for 12'-16' spans, W14X22 for 25' spans, and W21X44 for 35' spans. Column sizes range from W10X33 to W10X49.

- <u>First Floor:</u> The first floor is the top level for the conference center. The framing for the roof consists of both W-sections and joists. There is a large size difference in all steel beams, ranging from W12X14 to W21X50 and girders ranging from W24X55 to W27X94. Long-span joists are used over the large span of the main conference center located in the middle of the conference center. 68DLH17(s) frame over the 73' span, with diagonal bracing between joists for stability. Framing supports 3"-18 gage steel roof deck.

Codes and Material Requirements

Codes
- BOCA 1996 – adopted by the Township of Findlay, PA.
- AISC 1989 – "Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design" (Note: new load checks performed use LRFD design)
- ACI 318-89 – "Building Code Requirements for Reinforced Concrete"

Structural Steel

• Rolled Shapes	ASTM A572, Grade 50
• Plates, Angles, Channels, Connection Materials	ASTM A36
• Tube Sections	ASTM A500, Grade B
• Pipe Sections	ASTM A53, Grade B
• Anchor Bolts	ASTM A307

Cast-in-place Concrete (Normal weight)

• Pile Caps and Basement Slab-on-Grade	3000 psi
• Columns	5000 psi

• Walls, Grade Beams, Structural Slab-on-Grade	4000 psi
• Slabs on Metal Deck	3500 psi
• Tower Slabs and Beams	4500 psi

Reinforcement

• Deformed Rebar	ASTM A615, Grade 60
• Welded Wire Fabric	ASTM A185, Grade 60

Bolts and Welds

• Welding Electrodes	E70XX Low-Hydrogen
• Bolting Materials	ASTM A325 or A490

Gravity Loads

Design loads with updates from ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures.

Live Loads

• Basement – Slab-on-grade	75 psf
• First Floor – Structural Slab	100 psf
• Lobby	100 psf
• Conference Center Roof	12 psf
• Tower Roof	20 psf
• Guest Rooms	40 psf
• Tower Corridors	100 psf

Dead Loads

• Basement – Slab-on-grade	75 psf
• First Floor – Structural Slab	125 psf
• Lobby	60 psf
• Conference Center Roof	30 psf
• Tower Roof	80 psf
• Guest Rooms	80 psf
• Tower Corridors	80 psf

Superimposed Dead Loads

• Basement – Slab-on-grade	20 psf
• First Floor – Structural Slab	20 psf
• Lobby	40 psf
• Conference Center Roof	30 psf
• Tower Roof	20 psf
• Guest Rooms	20 psf
• Tower Corridors	20 psf

Snow Load

• Roof Snow Load	25 psf

Lateral Loads

Typical tower frames shown in figures 7 and 8, below. Orientation of columns is shown with respect to the elevation. Column sizes are color coordinated per the column legend.

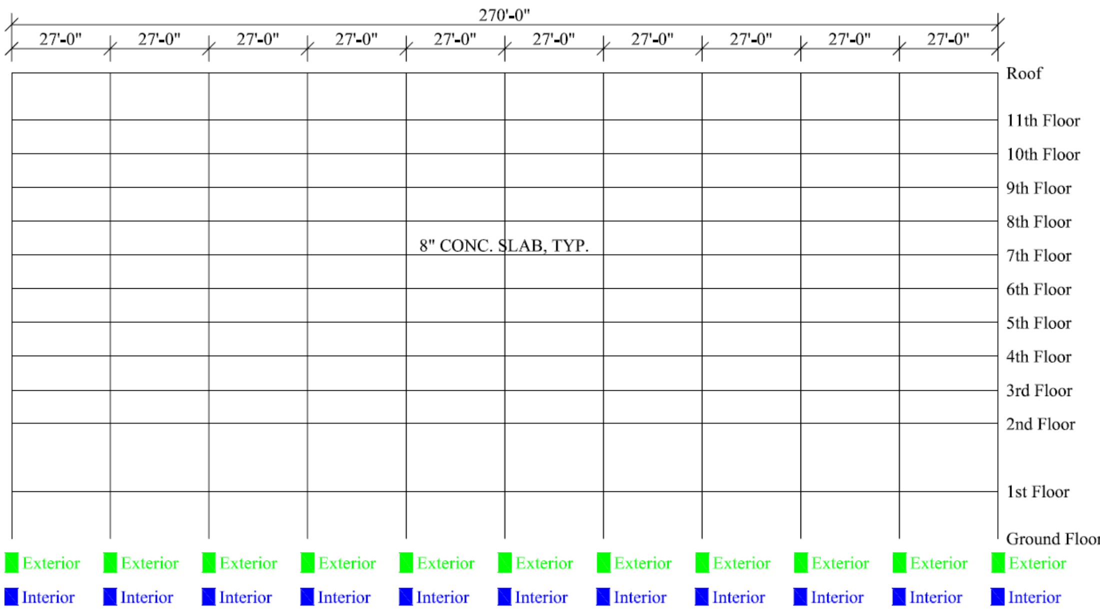


Figure 7. – Typical frame resisting lateral loads in E/W direction (See column legend for sizes)

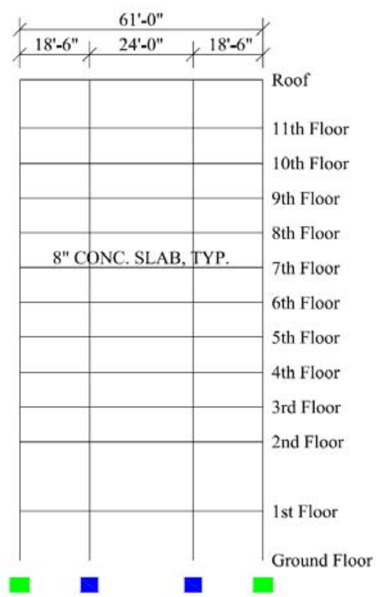


Figure 8. – Typical frame resisting lateral loads in N/S direction (See column legend for sizes)

Wind Loads

The design wind loads have been determined in accordance to IBC 2003 and ASCE 7-02. Wind loads have been calculated based on the 12-story, 140-foot tower of the building. The main building factors for determining the wind loads are the basic wind speed of 90mph, exposure C, importance category II. The calculations assume that the building behaves as a rigid, rectangular structure. There is some variation between the calculated loads and those in the design documents; however, this is most likely due to code changes, and the values are not significantly different. Story Shears have been determined from the tributary area to each story. See Appendix A for calculations.

Figure 9. – Original E/W Wind Story Shears and Base Shear Value

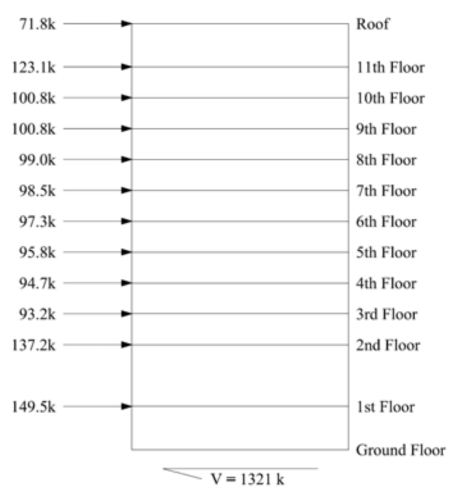


Figure 10. – Original N/S Wind Story Shears and Base Shear Value

Seismic Loads

Seismic calculations have been calculated using IBC 2003 and ASCE 7-02. The loads have been calculated based on the existing tower conditions. The original design of the building did not include seismic requirements, so these loadings were most likely not considered during the design of the concrete moment framing that serves as the lateral resisting system for the building. The calculations were made using the Equivalent Lateral Force Procedure. The building weight was approximated for the calculations based on a typical tower floor plan, so the value may vary slightly from the actual weight, but this should not change the loading significantly. Seismic loads are the same from each direction. See Appendix B for calculations.

Figure 11. – Original E/W Seismic Story Shears and Base Shear Value

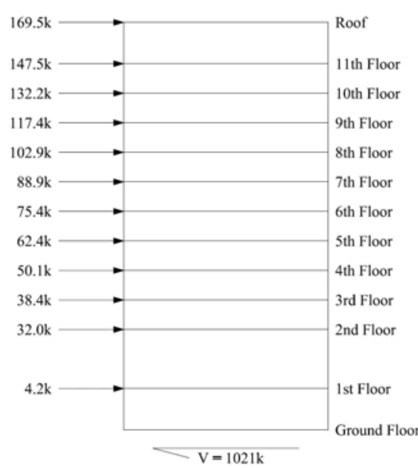


Figure 12. – Original N/S Seismic Story Shears and Base Shear Value

Problem Statement

The actual design for the Hyatt is very suitable for the design conditions given. Analyzing alternative floor systems for the tower revealed that the current design was likely the best solution to the design problem. However, while the current design may be the best solution to the problem as it was stated there are many other viable solutions to the problem.

In the case of the Hyatt, at the time of its design, the codes used did not require seismic loading analysis to be performed. In the lateral analyses of the existing structure, it was found that the large self-weight greatly increases the seismic loading to the building. While the system was found to be adequate to resist the lateral loadings, there may be alternate designs that can better resist these lateral loads or decrease the building weight to in turn decrease the seismic loading on the building. The main area of concern for these seismic loadings is the concrete tower.

With its location in Pittsburgh, PA, the site has a 0.2 second spectral response acceleration of 0.127g and a 1.0 second spectral response acceleration of 0.054g. These values are very low in comparison to critical locations in the United States such as California with 0.2 second spectral response accelerations up to 2.5g and 1.0 second spectral response accelerations up to 1.5g. Based on the location of the Hyatt, seismic loading should not be a great consideration; however, based on the original design, the weight of the structure greatly increases the seismic loading on the building.

Proposal

Research and calculations will be performed to design the tower as a steel framed system. There are multiple types of steel framing that can be used, so preliminary research has looked into the most feasible and best alternatives. From previous analysis, a non-composite steel floor system was analyzed, which warranted further investigation. In addition, composite steel framing will be considered, which will likely be the best alternative due to increased strength and stiffness based on composite action. To select preliminary beam and column sizes, hand calculations will be performed.

The lateral resisting system will also require a re-design with the change from concrete moment frames to steel. Both braced and moment frames will be considered with research to determine which would be the most viable solution. Consideration will be taken to place frames so that they do not interfere with architectural room layouts.

Once a preliminary design is established, computer modeling will be performed using RAM, which is a commonly used structural engineering software package for structural steel design. Using the software, models can be created for the tower framing.

If a 3-dimensional model is created, the building weight can be determined and compared to the calculated weight of the concrete tower. New lateral loading analysis will be performed to determine the decrease in loading due to the decrease in building weight. Analysis will also look at the impact the changes can make on the foundation of the building. With decreased building weight, it is believed that the foundation size can be decreased as well. Calculations from the design software will be verified with hand calculations.

Once a new structural system has been designed, it will be compared with the current system to see if it meets all of the requirements and design criteria set for the project. Upon comparison to the design criteria, it will be determined whether the steel framing is a more viable option for the tower or if the existing concrete structure is the best choice for the design.

The new design will update to the IBC 2003 code requirements. Design loads will be determined using the ASCE 7-02: Minimum Design Loads for Buildings and Other Structures. Steel design procedures by hand and computer calculations will be performed in accordance to the AISC Manual of Steel Construction, 3rd Edition LRFD. It will utilize A992 wide-flange structural steel. The columns will be selected from a trial group of W14 sections spliced every 3 levels or as needed. The new design will adhere to the floor plan laid out by the architects; this will prevent columns from interfering with guest rooms.

Design Criteria

A major design criterion for the project was the building height, which is critical because of FAA regulations for buildings in close proximity to airports. The restrictions imposed on the architectural design were based on limitations in the FAA Advisory Circular AC 150/5190-4A which basically states that within a 5,000 ft radii from runways designated utility and 10,000 ft radii from other runways, the Horizontal Zone is established that is 150 feet above airport elevation. Another restriction on height, FAR Part 77, Objects Affecting Navigable Airspace, sets a zone sloping 7 ft horizontal to 1 ft vertical from a 'primary surface,' 1000 ft wide, centered on each runway. (The hotel is virtually centered between the northern Runway 10L-28R and the southern Runway 10R-28L.)

In conversation with the design architects, the Hyatt was within these set limits; however, greatly increasing the building height would not be possible for the conditions set by the regulations. Thus, the height of the building is a major restriction to be followed. While it may not be possible to stay completely within the original architectural constraints, any deviation will be considered in the resulting conclusions of the new design and compared to the original design.

Another design criterion that must be followed is the layout of the floor plans. In similar conversation with the design architects, the Global Hyatt Corporation has set criteria for general building design. In order to prevent architectural conflict, the new design will try to constrain to architectural layout with any variations noted in the conclusions.

Overview of New Design

The new design for the Hyatt tower is structural steel framing with symmetrically placed chevron braced frames to resist lateral loads. The preliminary design compared similar systems to determine the most viable option for a full new design. Various non-composite and composite steel floor systems were compared to select the primary system used in typical bays in the new design.

Non-composite systems were analyzed with 4'-0", 4'-6", and 5'-0" beam spacing in both typical bay sizes: 24'-0" x 27'-0" and 18'-6" x 27'-0". All non-composite designs resulted in deep sections (compared to the existing 8" filigree slab). Composite systems were then analyzed to determine the feasibility of composite steel framing. With the use of W8x48 beams and composite action, the total floor depth was increased to 12", significantly smaller than the 14" depth of typical non-composite configurations.

Using the composite floor framing, preliminary sizes were found for beams and columns. A computer model was created in RAM Structural Systems to assist in calculations and distribution of loads. Using code specified loads and load cases; the new structure was designed. Braced frames were selected to prevent greatly increasing member sizes through the use of moment frames.

Member sizes of beams, columns, and braces were edited to reduce moment-axial interaction to levels below 95% of allowable interaction. In addition, member sizes were standardized throughout the design to minimize the number of different sections used and create more typical framing. Column splices were placed every 3 levels (main level counted as 2 levels due to increased height).

New seismic and wind loads are compared to the original loads on the original design to compare the effects of the new design on the loading. The seismic loading decreased significantly with the significant decrease in building weight, while wind loads increase slightly based on the small increase of building height.

A vibration analysis has been calculated to determine the impact of walking induced vibrations in guest rooms based on the excitation force from the corridors. This check determines whether or not the lighter framing could cause serviceability issues for guests.

In accordance with IBC 2003 LRFD design, the following factored load cases were considered for the new design:

- 1.4 D
- 1.2 D + 1.6 L
- 1.2 D ± 1.6 W + 0.5 L
- 1.2 D ± 1.0 E + 0.5 L
- 0.9 D ± (1.6 W or 1.0 E)

D = dead load or dead load effects
L = live load or live load effects
W = wind load or wind load effects
E = seismic load or seismic load effects

Gravity Load Design

The new gravity design was achieved through a process of comparing various alternate floor framing systems and choosing the system that would best fit the criteria for the new design. As the floor to floor height would greatly impact the overall height of the building, which is a major design criterion to be met, the new gravity system would need to be designed to provide the necessary strength to resist the gravity loads as well as having a relatively small depth. The initial framing choices were compared to the initial filigree system to determine which would be the best choice for the new design.

The major three steel framing systems investigated were: open-web steel joists with steel beams, non-composite steel beams and girders, and composite steel beams and girders. Through investigation of various spacing of the members, the depths of the systems were roughly determined. Through open-web steel joists with 16K5 at 2’-0” on center, with 2.5” slab thickness; total floor depth was 18.5”. The non-composite W10X49 at 6’-0” on center, with 3.5” slab thickness; total floor depth was 13.5”. The composite W8X48 at 8’-0” on center, with 3.5” slab thickness; total floor depth 12”. See Appendix C for calculations.

Although the 12” depth is still greatly increased from the filigree slab thickness of 8”, the composite system was the best choice for the framing. Although it may be possible to decrease the floor depth slightly by decreasing the beam spacing, it would be a less efficient floor system. In addition, based on deck spans and beam depths, it is unlikely that a composite system could be found that would decrease the depth of the floor much more than the 12” depth found. In addition, AISC shear connections require a minimum 2-bolt connection; with a 3” bolt spacing and ¾” A325-N bolts, this would require a web depth (minus flange thickness) of at least 5.5”, which is within the range of

W8 sections. Smaller members would prove both inefficient and may not even be feasible under standard construction practices. While the composite system may increase the labor costs for the installation of the shear studs, the more critical design criterion of preventing the building height from significantly increasing is a more critical factor in the consideration of the system.

Bay sizes are typical in the tower with 27'-0" x 18'-6" exterior bays and 27'-0" x 24'-0" interior bays. Beams were selected to span 27' in both bays with girders running perpendicular. The selection was made so that the girders would be spanning shorter distances than the beams that they support. This selection was made so that the girders would be located above the partition walls between guest rooms, where the depth of the members was less critical. In exterior bays, beam spacing was selected to have 2 equal spaces, or 9'-3" center-to-center spacing of the beams; exterior girders span 18'-6" between columns. In interior bays, beam spacing was selected to have 3 equal spaces, or 8'-0" center-to-center spacing of the beams; interior girders span 24'-0" between columns.

Gravity column sizes were initially selected from hand calculations (See Appendix D) by determining accumulated gravity loads below levels 2, 8, and 12 of typical interior and exterior columns. This allowed an approximate design size to be compared to initial computer design sizes to ensure that loads were being accounted for properly. Live load reduction on the non-roof levels was taken into account as the live loads were not greater than 100 psf and met other criteria for reduction according to ASCE 7-02. W14 sections were chosen for the initial selection as they are easy to stack and are commonly used for columns in professional practice. Initial member sizes selected for the columns below level 2 are W14X48; below level 8 are W14X53 exterior and W14X109 interior; and below level 12 are W14X74 exterior and W14X145 interior.

Once the initial sizes were determined from hand calculations, a computer model was created in RAM Structural Systems for analysis and further member design. Since member depth was of great concern, after initial computer sizing calculations, member sizes were manually overridden to match those that were determined by hand calculations. Computer checks of the updated model showed that the member sizes selected through hand calculations were suitable for design, although they were not the optimized member sizes selected through the RAM calculations. Since the members selected have a smaller depth than the optimized sections, some of the members require a camber to meet deflection criteria of $^{\ell}/_{360}$ for live loads and $^{\ell}/_{240}$ for dead plus live loads.

The following page contains floor plans for the West (Figure 13) and East (Figure 14) ends of the tower with new steel framing member sizes shown. In addition, required cambers and shear studs requirements are detailed on the plan.

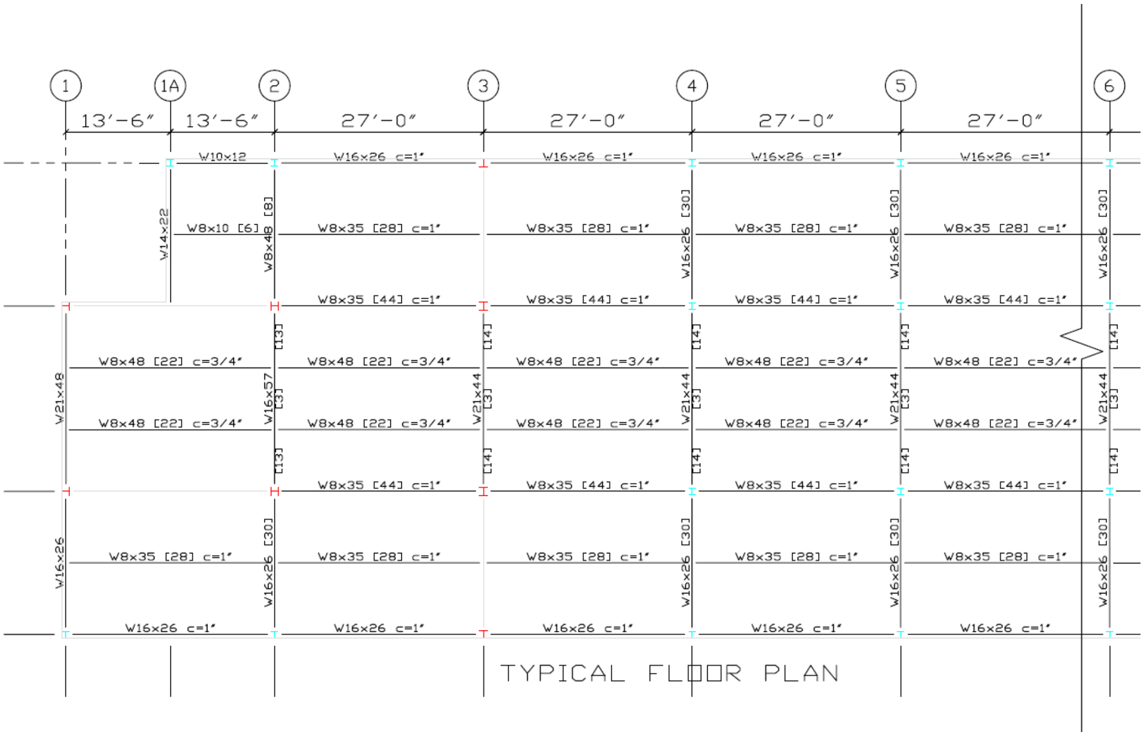


Figure 13. – Layout of New Floor Plan (Tower – West End)

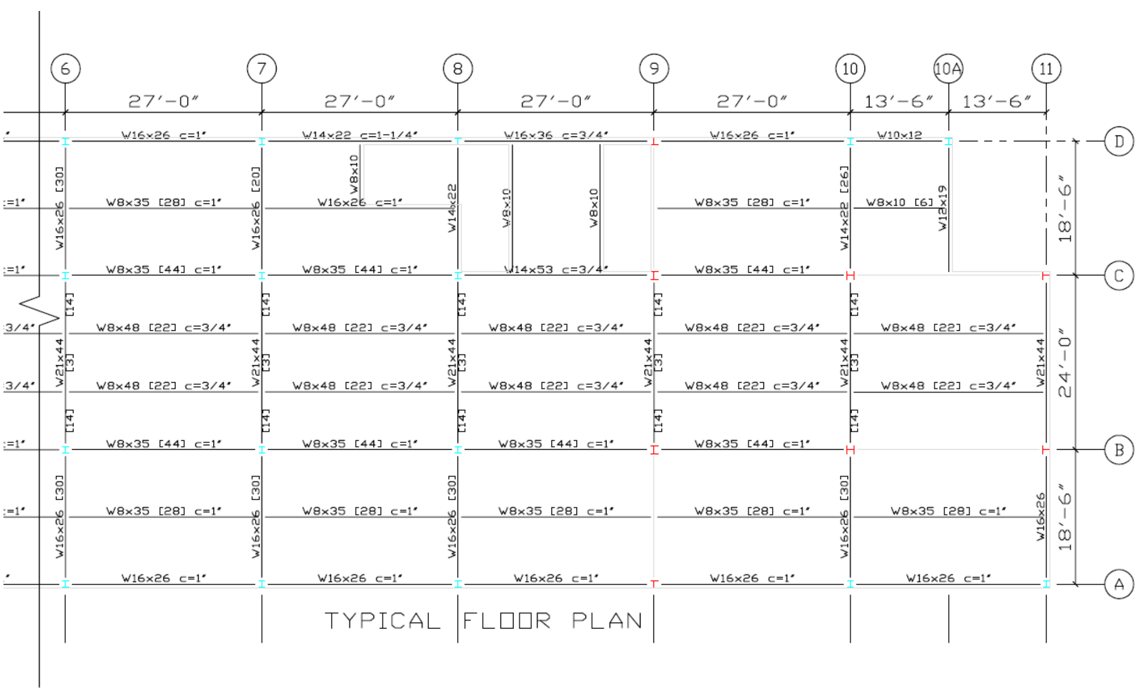


Figure 14. – Layout of New Floor Plan (Tower – East End)

With the increased depth, the floor-to-floor story height was changed from 10' to 11' in levels 2 through the roof to accommodate the increased member depth as well as allow for an architectural ceiling to be added below the new steel framing. This would have the effect of adding 10' to the total building height. While this is a significant increase, with larger member sizing (even only 13.5" floor depth), over 10 stories, the building height increases 1 foot per 1.2" of depth added to the section. The 10' increase in height was determined to be acceptable, as it is less than a 10% increase in the total height of the building. (While this may not be acceptable for the airport restrictions, for the purpose of analysis and comparison, the change will be considered acceptable) If the building height would be required to be decreased based on the new design, the height of the main level that houses the lobby could be decreased to accommodate the changes, or the building could be adjusted to have the ground level start at a lower overall elevation. With the minimal change in height, any problems arising could easily be resolved with minor architectural changes that would not affect the guest rooms in the tower.

Columns splices were set every 3 stories. Column schedule and plans below:

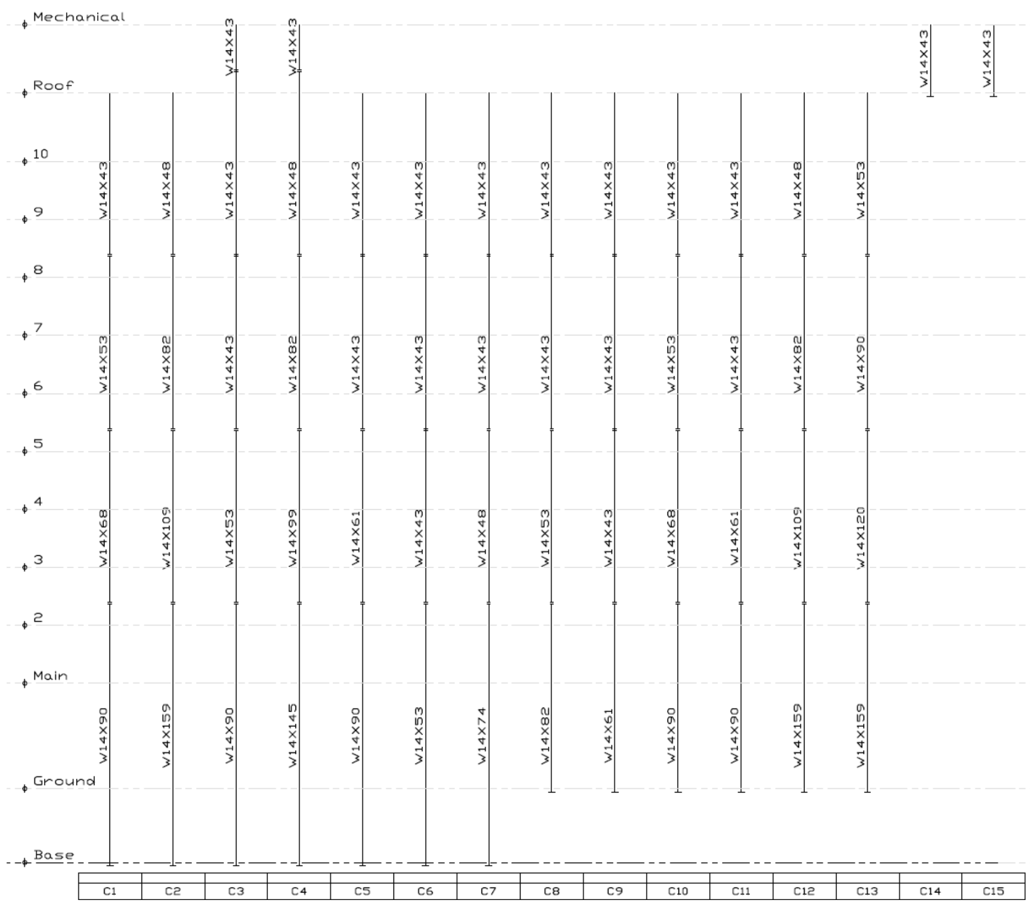


Figure 15. – Gravity Column Schedule (See plans, next page, for location)

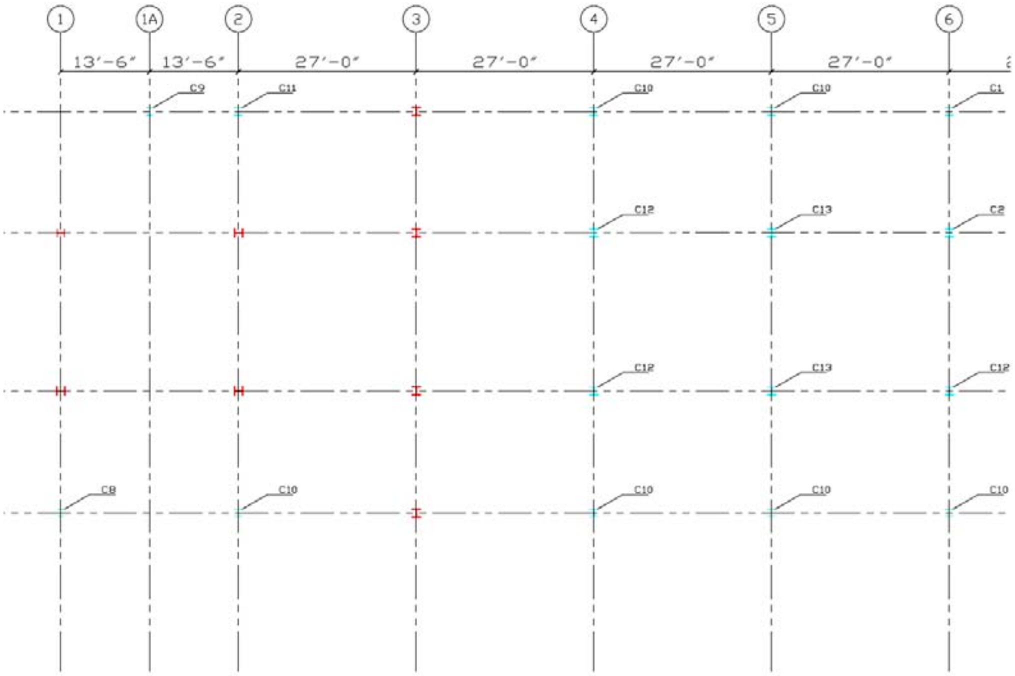


Figure 16. – Layout of Columns (Tower – East End)

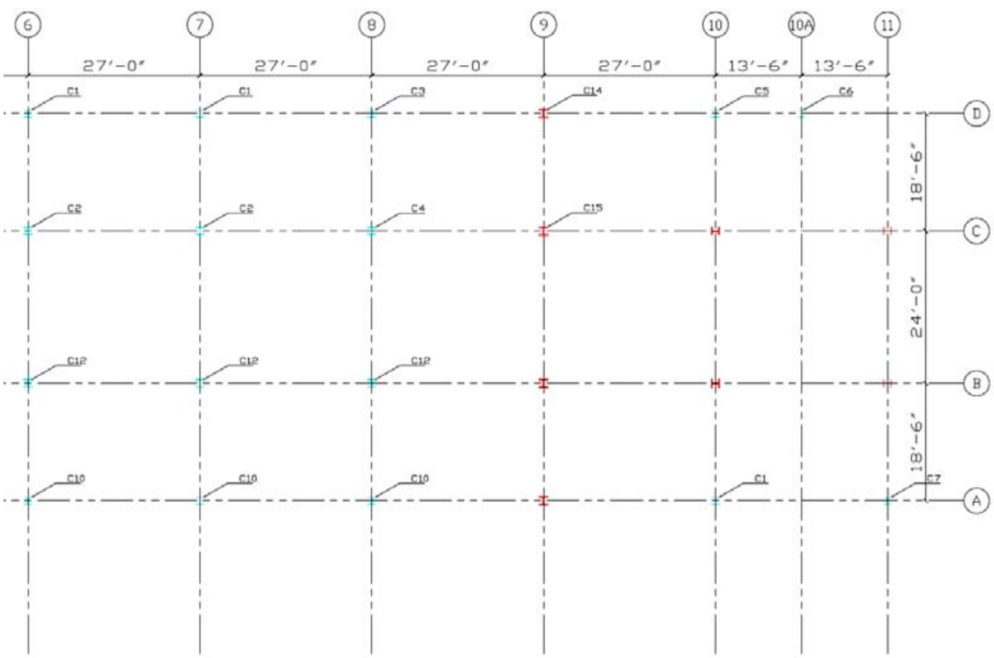


Figure 17. – Layout of Columns (Tower – East End)

Lateral Load Design

The lateral load resisting system for the new design must be adequate to resist the design load combinations. The loads have been calculated for wind design based on the ASCE 7-02 Method 2 – Analytical Procedure. For the seismic loading, ASCE 7-02 Equivalent Lateral Force System method was used.

In selecting the type of lateral load resisting system to use, there were a number of considerations for the design. The primary systems to compare were fully restrained rigid moment frames, simple partially restrained frames, or semi-rigid partially restrained frames. Each system had advantages and disadvantages that were considered. The semi-rigid partially restrained frame was not selected because it involves a complex design process and there was no primary design reason to select this system over simple partially restrained frames. Fully restrained moment frames would best suit the architectural requirements for the floor plan layout; however, they are typically more costly than simple braced frames. In addition, fully restrained moment frames are more difficult to fabricate and more inefficient in resisting lateral loads. Partially restrained braced frames were chosen because they have the advantages of: being very stiff, simple shear connections, and a determinate analysis. If consideration is given to the bracing layout, there should be minimal architectural impact.

To additionally minimize architectural impact, chevron braces were selected. Based on the configuration, openings can still be oriented near the middle of the bracing configuration. Based on the Hyatt floor plan, 4 frames were placed in each direction (see Figure 18). The frames are located where openings would occur near the middle of the bracing configuration.

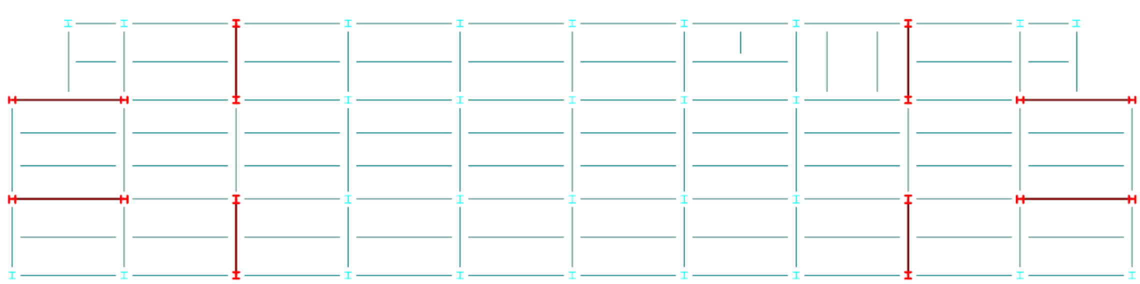


Figure 18. – Layout of Braced Frame Locations

Figure 19, below, shows an isometric view of the frame configuration. Originally, additional frames were placed in the North-South orientation, but through iterative analysis, it was found that the configuration shown was adequate to resist the lateral loads applied.

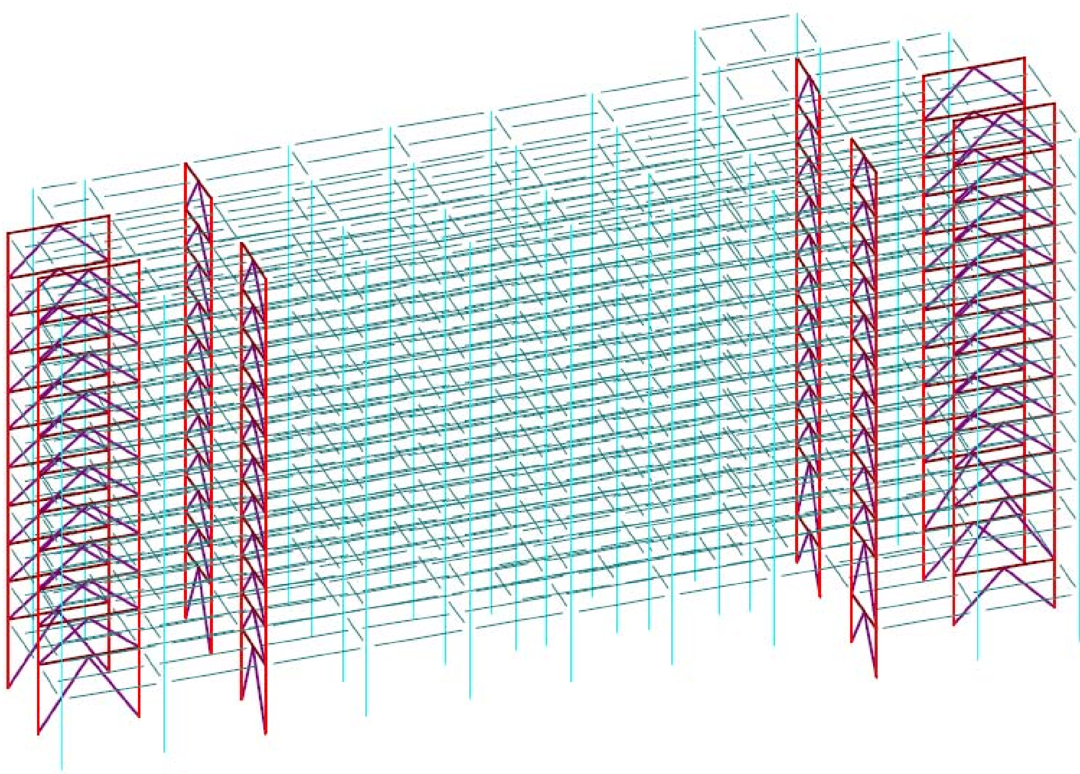


Figure 19. – Isometric View of Braced Frame Configuration

The frames have been oriented so that they have minimal impact on the floor plan layout while also being laid out symmetrically to prevent building torsion. The chevron brace members are typically W8X40 in the North-South frames and W12X53 in the East West Frames. Brace slenderness in compression controlled the members in the East-West frames as well as the braces below level 2, which has a larger story height. Below level 2, typical braces are W12X65 in both directions. Figures 20 and 21 below show the member sizes for the frames: beam and column sizes on the left of each figure and brace sizes on the right (separated for clarity).

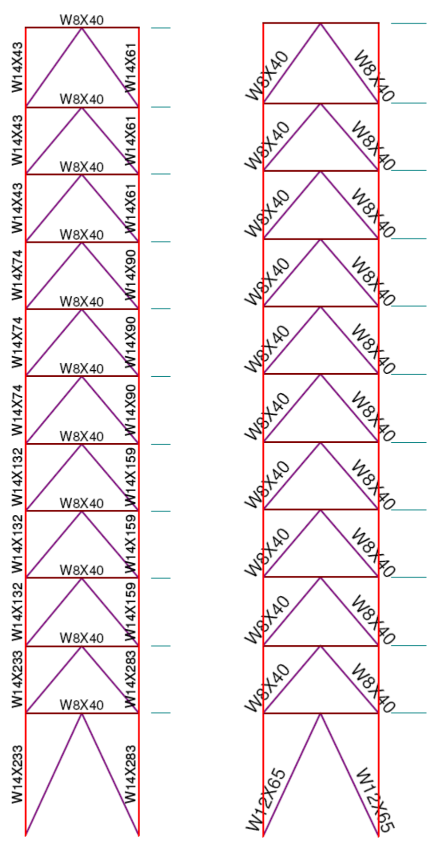


Figure 20. – North-South Braced Frame Members

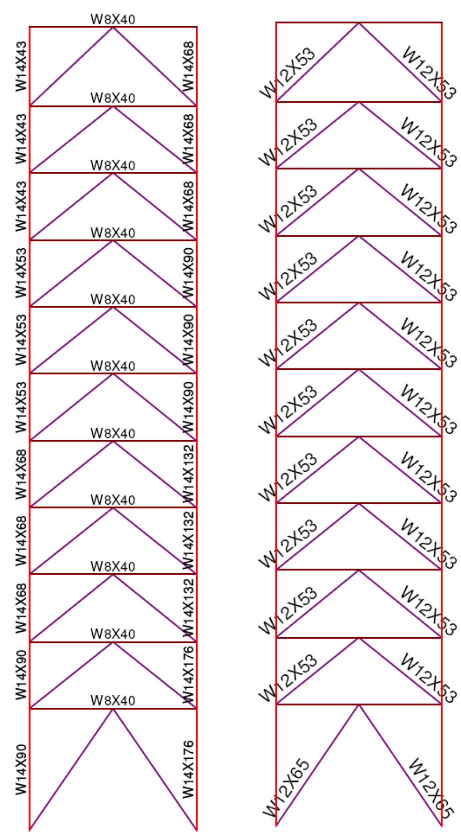


Figure 21. – East-West Braced Frame Members

Wind Loads

With a change in building height, even a minimal adjustment, the wind loading calculated for the original design needed revision. The height adjustment not only increased the velocity pressure with increased roof elevation, it also increased the area that the wind pressure acted over.

Similar to the original calculations, ASCE 7-02 Method 2 – Analytical Procedure was used to determine the lateral loading from wind pressures on the tower. The design factors for the wind loading remains the same, with only the building dimensions changing. From this calculation, the new base shear values are slightly increased from those in the original design. In the North-South direction, the original base shear value was 1321 kips; it is now increased to 1355 kips. In the East-West direction, the original base shear value was 269 kips, increased to 273 kips. While these increases do not significantly impact the design of the lateral force resisting system, it is worth noting that there is a slight increase.

Updated story shears have been calculated based on the tributary area of each floor. The new values and new base shears can be seen in Figures 22 and 23. The new loading is slightly conservative as the loading for each story only takes into account a maximum pressure on the tributary area. If the change in pressure with change in height were completely accounted for, the loading would decrease slightly. Below are listed the updated windward and leeward. See Appendix A for calculations.

Windward Wind Pressures:

$p_{0\text{-}15}$ = 10.2 psf ± 2.7 psf
p_{20} = 10.8 psf ± 2.9 psf
p_{25} = 11.3 psf ± 3.0 psf
p_{30} = 11.8 psf ± 3.1 psf
p_{40} = 12.4 psf ± 3.3 psf
p_{50} = 13.1 psf ± 3.5 psf
p_{60} = 13.5 psf ± 3.6 psf
p_{70} = 14.0 psf ± 3.7 psf
p_{80} = 14.5 psf ± 3.8 psf
p_{90} = 14.9 psf ± 3.9 psf
p_{100} = 15.1 psf ± 4.0 psf
p_{120} = 15.7 psf ± 4.2 psf
p_{140} = 16.3 psf ± 4.3 psf
p_{150} = 16.5 psf ± 4.4 psf

Leeward Wind Pressures:

$p_{N/S}$ = -10.3 psf ± 4.4 psf
$p_{E/W}$ = -4.1 psf ± 4.4 psf

Figure 22. – New E/W Wind Story Shears and Base Shear Value

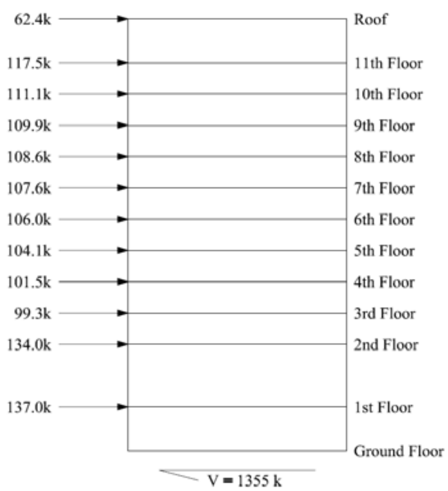


Figure 23. – New N/S Wind Story Shears and Base Shear Value

Seismic Loads

New seismic loads were computed using the ASCE 7-02 Equivalent Lateral Force Procedure. Story weights have been calculated based on tributary weight to each floor from RAM model data. The RAM model takeoffs of the gravity beams, gravity columns, and frame members provided the necessary weights to accurately calculate the framing weight. From the new weight and new factors, new story and base shears were calculated and can be seen in Figures 24 and 25. See Appendix B for calculations.

Figure 24. – New E/W Seismic Story Shears and Base Shear Value

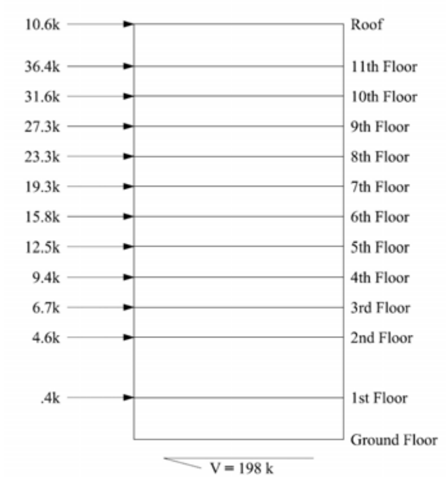


Figure 25. – New N/S Seismic Story Shears and Base Shear Value

Impact of New Design on Seismic Loading

Calculation of the seismic loading for the new design changes the applied loads to each floor and base shear values. Based on the new building design, there are a number of design factors that impact the loading.

Building Weight:

Based on the building weight and C_s, the seismic response coefficient, the base shear value is calculated. As this calculation is linearly dependent on the weight of the building, the weight has a large effect on the base shear value. For the original structure design, the weight of each floor was calculated to be approximately 1950 kips. For the new structure, typical floor weight was calculated to be around 680 kips (lower floors have slightly larger weight based on increasing column sizes near ground level). This is a decrease of 1270 kips per floor. On an overall building scale, the total building weight calculated for the concrete design was 22,700 kips, whereas the steel framing has a total weight of 7350 kips; this results in a change in total building weight of over 15,000 kips, thus having a large impact on the seismic loading by decreasing the total seismic base shear.

Seismic Response Coefficient, C_s:

The seismic response coefficient also impacts the seismic base shear. The factor is multiplied by the building weight to determine the total base shear. It is a factor of the design spectral response acceleration, S_{DS}, the importance factor, I, and the response modification factor, R. With the change in framing, the design spectral response acceleration and importance factor do not change, but the response modification factor is changed based on the building frame systems. The original concrete moment frame system has a response modification factor of 3. The new steel braced frames have a response modification factor of 5. In the equation to calculate the seismic response coefficient, the formula is: $C_s = S_{DS}/(R/I)$. As the response modification factor is in the denominator of the equation, the increased value in the new steel braced frames has the effect of decreasing the response coefficient and therefore also decreasing the total base shear value. The response coefficient has been therefore decreased from a value of 0.045 to a value of 0.027, which is still greater than the code minimum of 0.006 (as calculated for this case).

Vertical Distribution Factors, C_{vx}:

To distribute the total base shear vertically into story forces, vertical distribution factors, C_{vx}, are calculated for each floor. The factor is a function of the floor weight, floor height, and the approximate fundamental period of the structure, T_a. The floor

weight does not have a great impact on the distribution unless a specific level has a much larger weight than other levels. The height of the levels has increased slightly, so the change is factored into the equation. The period of the structure has changed with the change from concrete moment frames to steel braced frames. For the case of concrete moment frames, the fundamental period is allowed to be approximated as 0.1 times the number of stories (not exceeding 12). This provides a period of 1.2 seconds. With the new design, the period is calculated from parameters based on the structure. The new calculated period is 1.26 seconds. This changes the distribution to each story by increasing the exponent, *k*, on the height of the floor, *h*. From the distribution, it is found that the loads to the upper levels are significantly larger than those at the lower levels. Table 1, below, shows the distribution of the base shear (198 kips) to each story.

Table 1. - New Seismic Story Force Distribution

Story	w_x	h_x	$w_x h_x^k$	C_{vx}	Story Force
1	233.2	14	15905	0.002	0.35
2	755.0	34	212967	0.023	4.63
3	702.4	45	310259	0.034	6.74
4	690.6	56	432840	0.047	9.40
5	690.6	67	576695	0.063	12.53
6	683.7	78	728143	0.080	15.82
7	676.7	89	890062	0.098	19.34
8	676.7	100	1072497	0.118	23.30
9	671.9	111	1258408	0.138	27.34
10	667.0	122	1453120	0.159	31.57
11	669.1	133	1673608	0.184	36.36
Roof	168.7	146	489869	0.054	10.64
		∑	9114374	1.000	198.00

Braced Frame Design

The controlling factor in the design of the members was the slenderness of the member in compression. The tension slenderness ratio of kl/r ≤ 300 was typically met by all initially sized members; however, the kl/r ≤ 200 for members in compression controlled the design and required member sizes to be increased. To meet the slenderness requirements and for constructability, member sizes for braces were selected to be the same through similar frames. This makes the design of the frames in the N-S direction all the same and the frames in the E-W direction are also all the same. Braces are subject to tension and compression forces (axial), so the interaction of moment and axial forces was not significant in these cases.

Frame members have been standardized to keep the stiffness of all frames in one direction equal. With the concrete slab in place, the floor will act as a rigid diaphragm and distribute loads based on the stiffness of the frame and its distance from the center of rigidity. As the stiffness of the North-South frames are equal and the stiffness of the East-West frames are equal, the spacing of the frames has the only impact on the forces each frame takes. Similarly to the stiffness, all frames in either orthogonal direction are spaced equal distances from the center of rigidity. Therefore, all North-South frames resist an equal portion of the story forces in the North-South direction and all East-West frames resist an equal portion of the story forces in the East-West direction.

The standardization of the bracing members not only simplifies the distribution of the lateral forces but also helps prevent building torsion. Large torsional forces occur when the applied location of the load has a significant eccentricity to the center of rigidity. With the symmetrical layout of frames, the torsional impact on the building is insignificant.

Drifts were calculated in the RAM model, and compared to an $\ell/400$ value for the building. The calculated value of $\ell/400$ for the new building height is 4.8 inches. From the model, calculated drift from the controlling load case of 1.2 D + 0.5 L + 1.6 W was determined to be 4.8 inches in the N/S direction and 0.7 in the E/W direction.

Column and beam sizes were checked to meet the combined force equations H1-1a and H1-1b, in the Specifications Chapter H in the AISC 3rd Edition LRFD Design Manual. Initial member sizes selected did not meet the combined loading criteria, so were resized to limit all interactions to a value less than 0.95 or 95% of the allowable combined loading. The load case of 1.2 D + 0.5 L + 1.6 W controlled the design of all members in the frames. Most column members remain well below 90% of the allowable combined loading due to the location of the column splices. Members were sized based on the lowest point at a splice, so in levels above (before a new splice), the strength of the member becomes conservative.

Vibration Analysis of New Design

With the large decrease in weight of the new structural design, a vibration analysis was checked to determine if the walking excitations of hotel guests in the corridors would result in unfavorable conditions for guests in the adjacent rooms. Calculations of a typical bay were carried out in accordance to the criterion set in Chapter 4 of AISC Design Guide 11 – Floor Vibrations Due to Human Activity.

In the calculations (see Appendix E), the floor acceleration is calculated and compared to a recommended limit. For the hotel, the acceleration limit, a_o/g, of 0.5% gravity was used, which is the recommended limit value for offices, residences, and churches. From the same general building category, the constant force, Po, was taken to be 65 lb. and the damping ratio, β, was taken as 0.05 (for full height partitions between floors).

The value of the peak acceleration as a fraction of gravity, a_p/g was calculated and compared to the acceleration limit. The peak acceleration is a function of the constant force, the damping ratio, the effective weight supported, and the fundamental natural frequency, f_n, of the combined panel. The fundamental natural frequency of the beam panel was calculated to be 5.3 Hz. The fundamental natural frequency of the girder panel was calculated to be 9.3 Hz. When the two values are combined into a bay panel frequency, the result was 4.6 Hz. The effective weight supported was calculated to be 41351 lbs. which resulted in a peak acceleration value of 0.006 or 0.6% gravity.

From the calculations performed, the floor does not meet the recommended criteria set forth by the design guide. While this may produce unfavorable conditions, it is still possible that guests will not be affected by the vibrations caused by walking in the corridors. Since the building is a hotel, there are typically more partitions than a normal office building or similar structure of this size. As this is the case, it may result that the damping ratio from the increased number of partitions will prevent problems from arising. It can be noted that an increase in the damping ratio from 0.05 to 0.06 (or a 1% increase) results in the peak acceleration calculated to be 0.005 or 0.5% gravity, which is equal to the recommended limit.

Remedial measures could also be taken to reduce the effects of the floor vibration. The simplest way of fixing the problem would be to stiffen the beams. As the girder panel frequency is much larger than the beam panel frequency, by increasing the stiffness of the beams, the frequency is increased and this results in a larger combined panel frequency. Typically in a design of this type, larger and stiffer members would be selected in preliminary design, which would increase the frequency of the beam panel and effectively decrease the peak acceleration; however, with the floor-to-floor height

criterion set, the members selected do not allow for the recommended acceleration limit to be met.

Overall Impact of the New Design

When the foundations are evaluated based on the allowable soil bearing capacity, 2ksf, it is evident that even with the decrease in building size, the deep foundations are the most practical for the situation. Without piles, the required footing size for a typical gravity column would be approximately 19'x19' and 3 feet deep. While this is not impossible to construct, it results in almost the equivalent of a mat foundation. Mat foundations are difficult to construct, require methods to dissipate heat generated and delay the schedule until the foundation has cured. While the reduced building weight may allow smaller pile configurations, it does not reduce the foundations to a shallow system.

The new steel design has also impacted a number of aspects of the tower that can be discussed and used to evaluate whether or not the new design would be recommended as a good alternative to the original design. The advantages and disadvantages can be compared below.

<u>Existing system</u>: Concrete moment frames with 8" filigree slab.

Advantages:

- Filigree slab allows short floor to floor story heights.
- Concrete moment frames have little impact on floor layout except at column locations.
- Filigree slab allows faster construction than typical cast-in-place flat slab or similar concrete construction.

Disadvantages:

- Large building self weight.
- Increased seismic loads due to self weight.
- Longer construction time for cast-in-place sections.
- Large column sizes have some impact on floor plan.

New system: Steel framing with braced frames and 3.5” composite slab.

Advantages:

- Lightweight framing decreases loads to columns and foundations.
- Decrease in building seismic loads.
- Smaller column sizes.

Disadvantages:

- Slightly increased floor to floor story heights.
- Slightly increased wind loading due to height increase.
- Braced frames restrict openings through frame location.
- Possible vibration issues based on lightweight framing.
- Requires additional fireproofing.

Conclusions of Depth Work

The analysis of the building has proven to show that the seismic loads can greatly be reduced in lightweight structures; in this case the conversion from concrete framing to steel framing. There other issues that are impacted by the new design must be accounted for when determining what the best choice is for the given project.

With the redesign to steel framing, the architectural constraints that were set as design criterion were adhered to as closely as possible. The use of the chevron bracing allowed for the openings in the wall to remain where the architects had laid them out in the original design.

The new design has greatly decreased the total weight of the structure. As noted, the original structure weight was calculated to be 22,700 kips, whereas the new structural design has a self-weight of only 7,350 kips. This has reduced the seismic base shear to be reduced from 1021 kips to 198 kips. This changes the East-West controlling load case from seismic to wind. The controlling wind load causes a base shear of 269 kips, much less than the previous controlling seismic load.

While the increase in building height does have some impact on the architectural constraints of the building, the building height has increased less than 10%. The FAA regulations on building height in proximity to airports would still need to be enforced; however, it would be possible to reduce the main story height or the overall elevation of the building by adjusting the ground floor level if necessary. In cases where the building was not located adjacent to an airport, the height limitation would widely increase the viability of steel framing over concrete framing.

The analysis of walking induced vibrations has also shown that lightweight framing can also have disadvantages for serviceability. While there might not be any complaints based on these vibrations, it is known from the analysis that this case is more susceptible to vibrations than other systems with beams with higher stiffness.

From the combination of these analyses, I believe that the original floor system would be the most viable option for the building. Although the steel framing could be adapted for use in this situation, the architectural requirements (including the height limit) in this case would enforce the fact that the low floor-to-floor heights are preferable in this case. The new design has supported the proposal that the seismic loading could be reduced so that wind forces would control in both directions. It has also shown that structural steel framing is an alternative option and will greatly decrease the overall weight of the structure without greatly increasing the floor-to-floor height or overall height of the structure. In this particular building case, the disadvantages of the new system seem to outweigh the advantages that it provides.

BREADTH WORK – MECHANICAL/FIRE PROTECTION AND CONSTRUCTION MANAGEMENT

Mechanical/Fire Protection

With the layout of the braced frames, there was insignificant impact on the plans of the Hyatt. Spot checks of the plans were performed to determine if any mechanical changes needed to be evaluated based on the new layout. There were no significant changes to be made; however, in regards to fire protection, the new steel framing would need to be reviewed based on code specified fire ratings and new measures may need to be taken, such as the addition of spray on fireproofing. With the addition of heat, particularly in the case of a fire, the strength of the steel members is reduced, leading to possible failure.

Based on the classifications, the IBC 2003 selected occupancy group is R-1 for hotels with occupants that are not primarily permanent. The construction classification used in this case is type 1A. This classification requires a 3 hour fire rating for the structural frame, a 2 hour fire rating for the floor construction and a 1.5 hour fire rating for the roof construction. While the selected construction classification may not be the exact classification for the project, it is a conservative analysis and also provides a safer building for the occupants.

Grace Construction Products produces a spray applied cementitious fireproofing spray that is commonly used on steel beams, columns, and concrete/steel decking. Their product, Monokote® MK-6®, is mixed with water on the job site and applied as a cohesive slurry.

Product requirements were taken from the Underwriter's Laboratory Online Certifications Directory. From the Grace Construction Products website the steel beams can use UL Design No. N779 to provide a 3 hour fire rating, a 1" thick spray is required. For the columns, UL Design No. X772 is specified. For a 3 hour fire rating on an average sized column, a 2½" thick application is required. For the floor systems, primarily the composite decking, a 2 hour fire rating can be achieved with UL Design No. D780, with an application of 5/8" thickness.

The addition of the fireproofing has an impact in increasing the overall cost of the steel framing. A rough approximation of the increase in cost is based on the total linear feet of columns and beams and a square footage of decking. From calculation in R. S. Means, the approximate increase in cost is $500,000.

Construction Management

When comparing alternative framing systems, other considerations are the impacts on the construction management of the project. In many cases the cost or time schedule of a project dominates the design rather than what might be the best choice in the opinion of the structural engineers or architects.

Cost Estimate

For the Hyatt project, a cost estimate has been calculated for the existing structure as well as the new structural design. Using the R. S. Means Building Construction Cost Data 2006, the cost of the framing members has been calculated. In the case of the concrete framing, the estimate was calculated based on the number of cubic yards of cast-in-place concrete for the columns and slab and based on a total square foot calculation of the filigree precast plank. Final results are shown in Table 2. Additional calculations can be found in Appendix F.

Table 2. - Concrete Cost Estimate

Total Building Cost

Precast Plank	$952,680
C.I.P. Slab on Plank	$52,662
C.I.P. Columns	$1,117,282
Total Cost =	**$2,100,000**

The cost per cubic yard for the columns includes a crew of 25, including a foreman, carpenters, rodmen (reinforcement placement), laborers, a cement finisher, and an equipment operator for the concrete pump. It also includes the cost of a gas engine vibrator and the use of a concrete pump. In addition to the crew, the cost for the columns also includes forms, concrete, placement of the concrete, reinforcing steel, and finishing costs. The cost per cubic yard of the slab includes a crew of 9, including a foreman, laborers, cement finishers, and equipment operators. It also includes the cost of 2 gas engine vibrators, a concrete bucket, and a crane for placement. The cost for the filigree precast plank was taken as the cost for a lightweight concrete precast plank from the Cementitious Decks & Underlayments section of R. S. Means. While this may not exactly represent the actual cost of the filigree slabs, it does provide an estimate to the cost of similar construction. The cost includes a crew of 6, including a foreman, carpenters, a laborer, and an equipment operator. Also included is the use of a crane for plank placement.

For the cost of the new design, the cost was calculated primarily on the basis of total linear feet of each W-section used. To determine the linear feet of steel used, takeoffs from RAM were used to summarize the sections used and compute the actual lengths to be used for the cost estimate. For the slab and decking, total square footage was used; for the shear studs, the value was based on the total number of studs. Final results are shown in Table 3. Additional calculations can be found in Appendix F.

Table 3. - Steel Cost Estimate

Total Building Cost

Gravity Beams	$1,185,164
Gravity Columns	$410,046
Frame Beams and Braces	$249,257
Frame Columns	$256,027
Shear Studs	$33,010
Decking and Slab	$734,400
Total Cost =	**$2,900,000**

The cost per linear foot for the various members was in some cases estimated from the values in R. S. Means. As some W-sections are not included for estimates, similar shapes and sizes were used to calculate some member costs. The cost for the structural steel includes a crew of between 6 and 10, depending on the sizes of the members, including foremen, steel workers, welders, and equipment operators. The costs also include the use of a crane and welding machine. For the decking, the cost includes a crew of 4, including a foreman and steel workers; it also includes the use of a welding machine. For the shear studs, the cost includes a crew of a foreman and a welder as well as the use of a welding machine. The slab is calculated similar to the slab for the concrete design including a similar crew.

Based on the cost estimates calculated, the steel framing would cost $800,000 more than the existing concrete framing. Note that this does not include the approximated cost of $500,000 in the fire protection breath study. It can also be noted, however, that the cost does not factor in any savings that may be involved with reduced number of piles at each pile cap. In the scope of the whole project, this only represents around a 5% increase in cost. For the purpose of this comparison, the increase in cost is unfavorable.

Time Estimate

In addition to a cost comparison, the approximate time for construction has been calculated based on the same units that the cost estimate was based on. The daily output value represents the number of units (cubic yards, square feet, linear feet, etc.) that can be constructed in a day. By dividing the units required by the output per day of the given crew, the resulting time for construction is estimated.

In the case of this comparison, the existing concrete framing was given a time estimate of 220 labor days. The new steel framing was calculated to take only 205 labor days. This time savings of 15 labor days would result in 3 weeks (5 labor days per week) time savings with the steel framing. It should be noted however, that with increased number of crews, it may be possible for the cost of the concrete framing may be able to reduce this difference with minimal additional cost. The daily outputs were simply calculated based on the given crew size that is assumed in R. S. Means. Calculations can be found in Appendix G.

Conclusions from Breadth Work

Overall, the breadth work seemed to show more disadvantages of the steel framing. With the upgraded fire protection, the cost of the project would increase significantly. In addition, the construction management cost analysis showed that the time savings were not very significant for the increase in cost.

The spray fireproofing ratings showed that steel can easily meet the code requirements for structural members. Only a 1" thick spray on beams, 2 ½" thick spray on columns, and a 5/8" thick spray on the decking provided the fire rating required.

Although it was not fully developed due to poor soil reports, the number of piles required may have decreased and in turn decreased the difference in cost from concrete to steel framing. The use of filigree precast slabs in the original design greatly increases the speed of that type of concrete construction. With cast-in-place slabs, the steel framing would typically have provided a much faster construction time.

FINAL RECOMMENDATIONS AND CONCLUSIONS

Through comparing and contrasting the original concrete design of the Hyatt Regency tower with the new steel design, a number of conclusions can be made with what the best choice of a system is and what has been learned through this research. Based on the design constraints of the architectural requirements of the building, in particular the building height based on proximity to the Pittsburgh International Airport, the new structural steel framing does not seem to be the best solution for the building. If the architectural constraints were not present, the selection of beams and overall design would be much freer to be optimized for member efficiency and cost efficiency. However, with the previous limitations set, the member sizes are much more limited and efficiency is removed from the project. In addition, the stiffness of the floor system has also given rise to additional vibrational problems.

The research did support the initial proposal that the change in framing from concrete to steel would reduce the seismic loads so that wind loadings controlled in both orthogonal directions. The combination of the reduction in building weight along with the change in framing type contributed to a large decrease in the seismic base shears. Also, it should be noted that the additional height did have an impact, even if relatively insignificant, on the wind loadings on the building.

The breadth analyses also supported the original design for the design conditions set forth for the new design. The requirement for additional fireproofing measures in addition to increased cost would typically not be in the best interest of the owners. Although the steel framing solution is still viable, in many business situations, it would not be selected due to increased cost. In addition, the time savings compared to the filigree construction does not substantially justify the use of the steel framing.

Overall, the new steel framing design does support the proposed reduction in weight and seismic loading. Based on other conditions such as the possible vibration problems, the increased cost, and the increased building height, the alternative framing does not seem to be the best choice. In other situations, where the height limit is not a major controlling factor, or where seismic loads need to be decreased, the steel framing seems to be the best selection.

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APPENDIX

Appendix A – Wind Loadings

Appendix B – Seismic Loadings

Appendix C – Preliminary Beam Design Hand Calculations

Appendix D – Preliminary Column Design Hand Calculations

Appendix E –Vibration Analysis Hand Calculations

Appendix F – Cost Estimates

Appendix G – Time Estimates

APPENDIX A

<u>Wind Loading Calculations:</u>
(using ASCE 7-02 Method 2 – Analytical Procedure)

V = 90 mph
Exposure C
I = 1.0
K_{zt} = 1.0 (no topographic features)
K_d = 0.85 (main lateral system)
G = 0.85 (for rigid structures - assumed)
$GC_{pi} = \pm 0.18$ (for enclosed buildings)

Velocity Pressure, q_z

z (ft)	K_z	$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$ *(lb/ft²)*
15	0.85	15.0
20	0.90	15.9
25	0.94	16.6
30	0.98	17.3
40	1.04	18.3
50	1.09	19.2
60	1.13	19.9
70	1.17	20.6
80	1.21	21.3
90	1.24	21.9
100	1.26	22.2
120	1.31	23.1
140	1.36	24.0
150	1.38	24.3

$q_h = 24.3\ \text{lb/ft}^2$

Wall Pressure Coefficients, C_p

Surface	*Direction*	*L (ft)*	*B (ft)*	*L/B*	C_p
LEEWARD	N/S	65	273	0.2	-0.5
	E/W	273	65	4.2	-0.2
WINDWARD	N/S, E/W	All Values			0.8

WIND PRESSURE CALCULATIONS:

$p = qGC_p - q_i(GC_{pi})$ *(lb/ft²)*

WINDWARD WIND PRESSURES:

$p_{0\text{-}15}$ = 10.2 psf ± 2.7 psf

p_{20} = 10.8 psf ± 2.9 psf

p_{25} = 11.3 psf ± 3.0 psf

p_{30} = 11.8 psf ± 3.1 psf

p_{40} = 12.4 psf ± 3.3 psf

p_{50} = 13.1 psf ± 3.5 psf

p_{60} = 13.5 psf ± 3.6 psf

p_{70} = 14.0 psf ± 3.7 psf

p_{80} = 14.5 psf ± 3.8 psf

p_{90} = 14.9 psf ± 3.9 psf

p_{100} = 15.1 psf ± 4.0 psf

p_{120} = 15.7 psf ± 4.2 psf

p_{140} = 16.3 psf ± 4.3 psf

p_{150} = 16.5 psf ± 4.4 psf

LEEWARD WIND PRESSURES:

$p_{N/S}$ = -10.3 psf ± 4.4 psf

$p_{E/W}$ = -4.1 psf ± 4.4 psf

CONCRETE FRAMING

STORY SHEARS AND BASE SHEAR CALCULATIONS:

East-West Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	7	73	14.5	8.4	11.7
1	1131	14	7	24	17	73	15.6	8.4	29.8
2	1151	34	24	39	15	73	16.8	8.4	27.6
3	1161	44	39	49	10	73	17.4	8.4	18.8
4	1171	54	49	59	10	73	17.9	8.4	19.2
5	1181	64	59	69	10	73	18.3	8.4	19.5
6	1191	74	69	79	10	73	18.8	8.4	19.9
7	1201	84	79	89	10	73	19.2	8.4	20.1
8	1211	94	89	99	10	73	19.4	8.4	20.3
9	1221	104	99	109	10	73	20.0	8.4	20.7
10	1231	114	109	119	10	73	20.0	8.4	20.7
11	1241	124	119	131	12	73	20.6	8.4	25.4
Roof	1255	138	131	138	7	73	20.6	8.4	14.8

E-W Base Shear = 269 kips

North-South Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	7	292	14.5	14.52	59.3
1	1131	14	7	24	17	292	15.6	14.52	149.5
2	1151	34	24	39	15	292	16.8	14.52	137.2
3	1161	44	39	49	10	292	17.4	14.52	93.2
4	1171	54	49	59	10	292	17.9	14.52	94.7
5	1181	64	59	69	10	292	18.3	14.52	95.8
6	1191	74	69	79	10	292	18.8	14.52	97.3
7	1201	84	79	89	10	292	19.2	14.52	98.5
8	1211	94	89	99	10	292	19.4	14.52	99.0
9	1221	104	99	109	10	292	20.0	14.52	100.8
10	1231	114	109	119	10	292	20.0	14.52	100.8
11	1241	124	119	131	12	292	20.6	14.52	123.1
Roof	1255	138	131	138	7	292	20.6	14.52	71.8

N-S Base Shear = 1321 kips

STEEL FRAMING

STORY SHEARS AND BASE SHEAR CALCULATIONS:

North-South Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	7	292	12.9	14.7	56.4
1	1131	14	7	24	17	292	12.9	14.7	137.0
2	1151	34	24	39.5	15.5	292	14.9	14.7	134.0
3	1162	45	39.5	50.5	11	292	16.2	14.7	99.3
4	1173	56	50.5	61.5	11	292	16.9	14.7	101.5
5	1184	67	61.5	72.5	11	292	17.7	14.7	104.1
6	1195	78	72.5	83.5	11	292	18.3	14.7	106.0
7	1206	89	83.5	94.5	11	292	18.8	14.7	107.6
8	1217	100	94.5	105.5	11	292	19.1	14.7	108.6
9	1228	111	105.5	116.5	11	292	19.5	14.7	109.9
10	1239	122	116.5	127.5	11	292	19.9	14.7	111.1
11	1250	133	127.5	139	11.5	292	20.3	14.7	117.5
Roof	1262	145	139	145	6	292	20.9	14.7	62.4

E-W Base Shear = 1355 kips

East-West Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	7	73	12.9	8.5	10.9
1	1131	14	7	24	17	73	12.9	8.5	26.6
2	1151	34	24	39.5	15.5	73	14.9	8.5	26.5
3	1162	45	39.5	50.5	11	73	16.2	8.5	19.8
4	1173	56	50.5	61.5	11	73	16.9	8.5	20.4
5	1184	67	61.5	72.5	11	73	17.7	8.5	21.0
6	1195	78	72.5	83.5	11	73	18.3	8.5	21.5
7	1206	89	83.5	94.5	11	73	18.8	8.5	21.9
8	1217	100	94.5	105.5	11	73	19.1	8.5	22.2
9	1228	111	105.5	116.5	11	73	19.5	8.5	22.5
10	1239	122	116.5	127.5	11	73	19.9	8.5	22.8
11	1250	133	127.5	139	11.5	73	20.3	8.5	24.2
Roof	1262	145	139	145	6	73	20.9	8.5	12.9

N-S Base Shear = 273 kips

APPENDIX B

CONCRETE FRAMING

<u>Seismic Loading Calculations:</u>
(using ASCE 7-02 Equivalent Lateral Force System)

For Pittsburgh, PA

$S_s = 0.127g$
$S_1 = 0.054g$

Occupancy II
Seismic Use Group I
$I_E = 1.0$

Site Class: D *(without sufficient detail to determine a Site Class, class D shall be used. As found in the geotechnical report prepared by L. Robert Kimball & Associates, the samples have a plasticity index (PI) ranging from 8-20. Site Class E is not used since the PI indicates that it is not a soft clay (PI>20)*

Based on site class, S_s, and S_1,

$F_a = 1.6$ $F_v = 2.4$

$S_{DS} = {}^2/_3 S_{MS} = {}^2/_3 F_a Ss = {}^2/_3(1.6)(0.127) = 0.135$
$S_{D1} = {}^2/_3 S_{M1} = {}^2/_3 F_v S_1 = {}^2/_3(2.4)(0.054) = 0.086$

S_{DS}, S_{D1}, and Seismic Use Group I, yields:

Seismic Design Category B

Based on this Seismic Design Category, the Equivalent Lateral Force System is permissible.

Base Shear Calculation:

$V_{BASE} = C_S W$

$C_S = S_{DS}/(R/I_E) \geq 0.044 S_{DS} I_E$

R = 3.0 (for ordinary reinforced concrete moment frames)

$C_S = 0.135/(3.0/1.0) \geq 0.044(0.135)(1.0)$
$= 0.045 \geq 0.006$ *(OK)*

W (Total weight is calculated from the typical floor plan for the tower, adjusted for the heights and floor plans of the main and ground levels)

Weight of Typ. Floor

Columns							
	L	W	H	lb/ft3	#/floor		Wt.
	22"	32"	10'	150 pcf	44/floor	=	**322.7k**
Col. Strip							
	61'	72"	8"	150 pcf	11/floor	=	**402.6k**
Slab							
	t	*SF*		*lb/ft^3*			
	8"	16302.5 sf		150 pcf	.75	=	**1222.7k**
					TOTAL	**=**	**1948k**

Weight ofMain Floor

Columns							
	L	W	H	lb/ft3	#/floor		Wt.
	22"	32"	20'	150 pcf	44/floor	=	**645.3k**
Col. Strip							
	61'	72"	8"	150 pcf	11/floor	=	**402.6k**
Slab							
	t	*SF*		*lb/ft^3*			
	8"	16302.5 sf		150 pcf	.75	=	**1222.7k**
					TOTAL	**=**	**2270.6k**

Weight of Ground Floor

Columns							
	L	W	H	lb/ft3	#/floor		Wt.
	22"	32"	14'	150 pcf	18/floor	=	**184.8k**
Col. Strip							
	61'	72"	8"	150 pcf	11/floor	=	**402.6k**
Slab							
	t	*SF*		*lb/ft³*			
	8"	4739.8 sf		150 pcf	.75	=	**355.5k**
					TOTAL	**=**	**942.9k**

$W_{TOT} = 10(1948) + 2270.6 + 942.9 = 22694k$
$V_{BASE} = 0.045(22694k) = 1021\ k$

Distribution to Floors:

The building does not exceed 12 stories, the lateral resisting system is entirely concrete, and the story height is at least 10ft, therefore the following assumption is valid:

$T_a = 0.1N = 0.1(12) = 1.2$ sec

$k = 1.3$ *(linear interpolation between 1 and 2 for a value of $T_a = 1.2$ sec)*

$F_x = C_{vx}V$ *(force at story x)*

$C_{vx} = w_x h_x^k/(\Sigma w_i h_i^k)$

Story Forces

Story	w_x	h_x	$w_x h_x^k$	C_{vx}	Story Force
1	942.9	14	29136	0.004	4.2
2	2270.6	34	222363	0.031	32.0
3	1948	44	266733	0.038	38.4
4	1948	54	348097	0.049	50.1
5	1948	64	434133	0.061	62.4
6	1948	74	524312	0.074	75.4
7	1948	84	618233	0.087	88.9
8	1948	94	715575	0.101	102.9
9	1948	104	816079	0.115	117.4
10	1948	114	919529	0.130	132.3
11	1948	124	1025740	0.144	147.5
Roof	1948	138	1178777	0.166	169.5
		Σ	7098709	1	1021.0

STEEL FRAMING

<u>Seismic Loading Calculations:</u>
(using ASCE 7-02 Equivalent Lateral Force System)

For Pittsburgh, PA

$S_s = 0.127g$
$S_1 = 0.054g$

Occupancy II
Seismic Use Group I
$I_E = 1.0$

Site Class: D *(without sufficient detail to determine a Site Class, class D shall be used. As found in the geotechnical report prepared by L. Robert Kimball & Associates, the samples have a plasticity index (PI) ranging from 8-20. Site Class E is not used since the PI indicates that it is not a soft clay (PI>20)*

Based on site class, S_s, and S_1,

$F_a = 1.6$ $F_v = 2.4$

$S_{DS} = {}^2/_3 S_{MS} = {}^2/_3 F_a Ss = {}^2/_3(1.6)(0.127) = 0.135$
$S_{D1} = {}^2/_3 S_{M1} = {}^2/_3 F_v S_1 = {}^2/_3(2.4)(0.054) = 0.086$

S_{DS}, S_{D1}, and Seismic Use Group I, yields:

Seismic Design Category B

Based on this Seismic Design Category, the Equivalent Lateral Force System is permissible.

Base Shear Calculation:

$V_{BASE} = C_S W$

$C_S = S_{DS}/(R/I_E) \geq \mathit{0.044S_{DS}I_E}$

R = 5.0 (for ordinary steel concentrically braced frames)

$C_S = 0.135/(5.0/1.0) \geq \mathit{0.044(0.135)(1.0)}$
$= 0.027 \geq \mathit{0.006}$ *(OK)*

W = (Weights taken from members designed in RAM Model.)

Story	Framing Weight
Roof	168.7
10	669.1
9	667.0
8	671.9
7	676.7
6	676.7
5	683.7
4	690.6
3	690.6
2	702.4
Main	755.0
Ground	233.2

$\sum W = 7350$

$V_{BASE} = 0.027(7350k) = 198$ k

Distribution to Floors:

$T_a = C_t h_n^x = 0.03(146)^{0.75} = 1.26$

$k = 1.6$ *(linear interpolation between 1 and 2 for a value of $T_a = 1.26$ sec)*

$F_x = C_{vx}V$ *(force at story x)*

$C_{vx} = w_x h_x^k / (\Sigma w_i h_i^k)$

Story Forces

Story	w_x	h_x	$w_x h_x^k$	C_{vx}	Story Force
1	233.2	14	15905	0.002	0.35
2	755.0	34	212967	0.023	4.63
3	702.4	45	310259	0.034	6.74
4	690.6	56	432840	0.047	9.40
5	690.6	67	576695	0.063	12.53
6	683.7	78	728143	0.080	15.82
7	676.7	89	890062	0.098	19.34
8	676.7	100	1072497	0.118	23.30
9	671.9	111	1258408	0.138	27.34
10	667.0	122	1453120	0.159	31.57
11	669.1	133	1673608	0.184	36.36
Roof	168.7	146	489869	0.054	10.64
		∑	9114374	1.000	198.00

APPENDIX C

PROJECT: HYATT REGENCY | Hiro McNulty | PG 1 OF 11

NON-COMPOSITE PRELIMINARY ANALYSIS/DESIGN

TYPICAL BAY SIZES: 27' x 18'-6" (ROOMS) 27' x 24' (CORRIDOR)

f'c = 3000 psi (NORMAL WEIGHT

A992 STEEL BEAMS & COLUMNS

SERVICE LOADS
20 psf DL
80 psf LL (CORRIDOR)
40 psf LL (ROOMS)

3-SPAN

27'x24' BAY SIZE

1.0 C(CSV) DECKING - SUPERIMPOSED UNIFORM LOADING = 80+20 = 100 psf

4'-0" CLEARSPAN REQUIRES 3" TOTAL SLAB DEPTH
w 6x6 W2.1xW2.1 W.W.F.
1.0 C26 = 31 psf

5'-0" CLEARSPAN REQUIRES 3.5" TOTAL SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.0 C24 = 37 psf

4'-6" CLEARSPAN REQUIRED 3.5" TOTAL SLAB DEPTH
w 6x6 W2.1xW2.1 W.W.F.
1.0 C24 = 37 psf

1.5 C DECKING - SUPERIMPOSED UNIFORM LOADING = 100 psf

4'-0" CLEARSPAN REQUIRES 3.5" TOTAL SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.5 C24 = 37 psf

5'-0" CLEARSPAN REQUIRES 4" TOTAL SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.5 C24 = 43 psf

4'-6" CLEARSPAN REQUIRES 3.5" TOTAL SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.5 C24 = 37 psf

27' x 18'-6" BAY SIZE

1.0 C (CSV) DECKING - SUPERIMPOSED UNIFORM LOADING = 40 + 40
= 80 psf

3'-6" CLEARSPAN REQUIRES 2½" SLAB DEPTH
w 6x6 W2.1xW2.1 W.W.F.
1.0 C26 = 25 psf

4'-0" CLEARSPAN REQUIRES 3" SLAB DEPTH
w 6x6 W2.1xW2.1 W.W.F.
1.0 C26 = 31 psf

4'-6" CLEARSPAN REQUIRES 3" SLAB DEPTH
w 6x6 W2.1xW2.1 W.W.F.
1.0 C26 = 31 psf

5'-0" CLEARSPAN REQUIRES 3½" SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.0 C24 = 37 psf

1.5 C DECKING - SUPERIMPOSED UNIFORM LOADING = 80 psf

4'-0" CLEARSPAN REQUIRES 3½" SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.5 C24 = 37 psf

4'-6" CLEARSPAN REQUIRES 3½" SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.5 C24 = 37 psf

5'-0" CLEARSPAN REQUIRES 4" SLAB DEPTH
w 6x6 W2.9xW2.9 W.W.F.
1.5 C24 = 43 psf

27' x 24' BAY (COMPOSITE DECKING)

1.5 VLR DECKING SUPERIMPOSED UNIFORM LOADING = 80+20 = 100 psf

9'-0" CLEAR SPAN REQUIRES 3½"

1.5VLR18 DECK, 38 psf

2.0 VLR DECKING SUPERIMPOSED LOAD = 100 psf

9'-0" CLEAR SPAN REQUIRES 4"

2VLI22 DECK, 39 psf

27' x 18'-6" BAY SUPERIMPOSED DEAD LOAD = 40+20 = 60 psf

1.5 VLR DECK 9'-6" CLEARSPAN REQUIRES 3½"

1.5VLR 22 DECK, 38 psf

2VLR DECK 9'-6" CLEAR SPAN REQUIRES 4"

2VLI 22 DECK, 39 psf

27'x24' BAY SIZE

4 of 11

1.0C DECKING WITH 4'-0" CLEARSPAN

$$w_u = [1.2(31+20)+1.6(80)]4' = 756.8 \text{ plf} = 0.757 \text{ klf}$$

$$M_u = \frac{w_u l^2}{8} = \frac{(0.757\text{klf})(27')^2}{8} = 69.0 \text{ ft-k}$$

$$V_u = \frac{w_u l}{2} = \frac{0.757(27')}{2} = 10.2 \text{ k}$$

1.0C DECKING WITH 4'-6" CLEARSPAN

$$w_u = [1.2(37+20)+1.6(80)]4.5' = 883.8 \text{ plf} = 0.884 \text{ klf}$$

$$M_u = \frac{w_u l^2}{8} = \frac{(0.884\text{klf})(27')^2}{8} = 80.6 \text{ ft-k}$$

$$V_u = \frac{w_u l}{2} = \frac{(0.884)(27')}{2} = 12.0 \text{ k}$$

1.0 C DECKING WITH 5'-0" CLEARSPAN

$$w_u = [1.2(37+20)+1.6(80)]5' = 982 \text{ plf} = 0.982 \text{ klf}$$

$$M_u = \frac{w_u l^2}{8} = \frac{(0.982\text{klf})(27')^2}{8} = 89.5 \text{ ft-k}$$

$$V_u = \frac{w_u l}{2} = \frac{(0.982)(27')}{2} = 13.3 \text{ k}$$

27' x 24' Bay Size

1.5 C Decking with 4'-0" clearspan

$$w_u = [1.2(37+20) + 1.6(80)]\,4' = 785.6\ plf = 0.786\ klf$$

$$M_u = \frac{w_u l^2}{8} = \frac{(0.786)(27')^2}{8} = 71.6\ ft\text{-}k$$

$$V_u = \frac{w_u l}{2} = \frac{(0.786)(27')}{2} = 10.6\ k$$

1.5 C Decking with 4'-6" clearspan

$$w_u = [1.2(37+20) + 1.6(80)]\,4.5' = 883.8\ plf = 0.884\ klf$$

$$M_u = \frac{w_u l^2}{8} = \frac{(0.884)(27')^2}{8} = 80.6\ ft\text{-}k$$

$$V_u = \frac{w_u l}{2} = \frac{(0.884)(27')}{2} = 11.9\ k$$

1.5 C Decking with 5'-0" clearspan

$$w_u = [1.2(43+20) + 1.6(80)]\,5' = 1018\ plf = 1.02\ klf$$

$$M_u = \frac{w_u l^2}{8} = \frac{(1.02)(27')^2}{8} = 92.9\ ft\text{-}k$$

$$V_u = \frac{w_u l}{2} = \frac{(1.02)(27')}{2} = 13.8\ k$$

27' x 18'-6" BAY SIZE

1.0C DECKING WITH 3'-6" CLEARSPAN

$w_u = [1.2(25+20) + 1.6(80)]\, 3.5' = 655.2 \text{ plf} = 0.655 \text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.655)(27')^2}{8} = 59.7 \text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{0.655(27')}{2} = 8.8 \text{ k}$

1.0C DECKING WITH 4'-0" CLEARSPAN

$w_u = [1.2(31+20) + 1.6(80)]\, 4' = 756.8 \text{ plf} = 0.757 \text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.757)(27')^2}{8} = 69.0 \text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{0.757(27')}{2} = 10.2 \text{ k}$

1.0C DECKING WITH 4'-6" CLEARSPAN

$w_u = [1.2(31+20) + 1.6(80)]\, 4.5' = 823.5 \text{ plf} = 0.824 \text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.824)(27')^2}{8} = 75.1 \text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{(0.824)(27')}{2} = 11.1 \text{ k}$

1.0C DECKING WITH 5'-0" CLEARSPAN

$w_u = [1.2(37+20) + 1.6(80)]\, 5' = 982 \text{ plf} = 0.982 \text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.982)(27')^2}{8} = 89.5 \text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{(0.982)(27')}{2} = 13.3 \text{ k}$

27' x 18'-6" BAY SIZE

1.5C DECKING WITH 4'-0" CLEARSPAN

$w_u = [1.2(37+20) + 1.6(40)]\,4' = 529.1\text{ plf} = 0.530\text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.53)(27')^2}{8} = 48.3\text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{(0.53)(27')}{2} = 7.2\text{ k}$

1.5C DECKING WITH 4'-6" CLEARSPAN

$w_u = [1.2(37+20) + 1.6(40)]\,4.5' = 595.8\text{ plf} = 0.596\text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.596)(27')^2}{8} = 54.3\text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{(0.596)(27')}{2} = 8.0\text{ k}$

1.5C DECKING WITH 5'-0" CLEARSPAN

$w_u = [1.2(43+20) + 1.6(40)]\,5' = 698\text{ plf} = 0.698\text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{(0.698)(27')^2}{8} = 63.6\text{ ft-k}$

$V_u = \frac{w_u l}{2} = \frac{(0.698)(27')}{2} = 9.4\text{ k}$

1.5 VLR 18

$$w_u = \left[1.2(38+20) + 1.6(80)\right] 9' = 1778\,plf = 1.8\,klf$$

$$M_u = \frac{(1.8\,klf)(27')^2}{8} = 164'^k$$

$$V_u = \frac{(1.8)(27)}{2} = 24.3^k$$

2VLI22

$$w_u = \left[1.2(39+20) + 1.6(80)\right] 9' = 1789 = 1.8\,klf$$

$$M_u = 164'^k$$

$$V_u = 24.3^k$$

[AS PREVIOUS]

1.5VLR22

$$w_u = \left[1.2(38+20) + 1.6(40)\right] 9.5' = 1269\,plf = 1.3\,klf$$

$$M_u = \frac{(1.3\,klf)(27')^2}{8} = 118.5'^k$$

$$V_u = \frac{(1.3)(27')}{2} = 17.6^k$$

2.0VLI22

$$w_u = \left[1.2(39+20) + 1.6(40)\right] 9.5 = 1281\,plf \; 1.3\,klf$$

$$M_u = 118.5'^k$$

$$V_u = 17.6^k$$

COMPOSITE DECK YIELDS FAR FEWER BEAMS (9'-0"-9'-6" SPACING RATHER THAN 5'-0" SPACING)

FOR $L_b = 27'\text{-}0"$ & $M_u = 164^{k}$ FOR 27'x24' BAY

USE W10x49 BEAMS

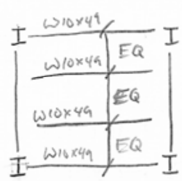
- LOW DEPTH
- GOOD CAPACITY

BEST OPTION

27' x 24' BAY

W10x49 BEAMS EQ SPACING

WITH 1.5 VLR 18 DECK & CONC (3½" THICKNESS)

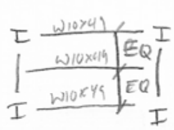


27' x 18'-6" BAY

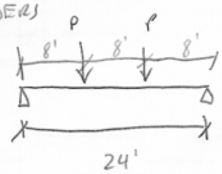
W10x49 BEAMS EQ SPACING

WITH 1.5 VLR 22 (USE 1.5 VLR 18 FOR SIMPLICITY IN LABOR)

1.5 VLR 18 DECK & CONC
3½" THICKNESS



GIRDERS



$w_t = 8'$

$w_u = \left[\left[1.2(20+38)+1.6(80)\right]8' + 1.2(49)\right]$

$= 1639.6\ plf = 1.64\ klf$

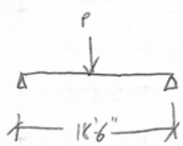
$M_u = \frac{(1.64)(27')^2}{8} = 149.4'^k \le \phi_b M_r = 164'^k$

$V_u = \frac{1.64(27')}{2} = 22.1^k \le \phi_b V_n = 91.6^k$

$P = V_u = 22.1^k$

$M_u = 22.1^k(8') = 176.8'^k$

$V_n = \frac{P}{l}(L-a+b) = \frac{22.1^k}{24'}(24'-8'+8') = 22.1^k$



$w_t = 9'\text{-}3''$

$w_u = \left[1.2(20+38)+1.6(40)\right]9.25' + 1.2(49)$

$= 1295\ plf = 1.30\ klf$

$M_u = \frac{(1.31k)(27')^2}{8} = 118.5'^k \le \phi_b M_r = 164'^k$

$V_u = \frac{1.3(27)}{2} = 17.6^k$

$P = V_u = 17.6$

$M_u = \frac{Pl}{4} = \frac{17.6(18.5)}{4} = 81.4'^k$

$V_u = \frac{P}{2} = \frac{17.6}{2} = 8.8'^k$

11 of 11

$M_u = 177^{'k}$ $\quad l_b = 24'$ $\quad$ TRY W10x49

$M_u = 176.8^{'k}$ + SELF WT

$$M_s = \frac{\frac{1.2(49)}{1000}(24)^2}{8} = 4.2^{'k}$$

$M_u = 176.8 + 4.2 = 181^{'k} \geq \phi_b M_r = 164^{'k}$ X

BRACING REQUIRED

$M_u = 81.4^{'k}$ $\quad L_b = 18.5'$ $\quad$ TRY W10x33

$M_u = 81.4^{'k}$ + SELF WT

$$M_s = \frac{\frac{1.2(33)}{1000}(18.5)^2}{8} = 1.7^{'k}$$

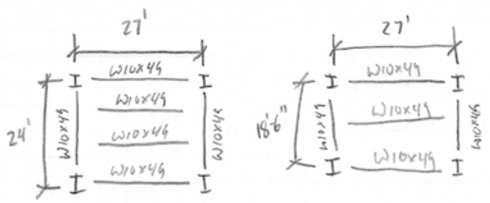
$M_u = 81.4 + 1.7 = 83.1^{'k} \leq \phi_b M_r = 105^{'k}$

COULD USE W10x49 FOR CONTINUITY

$$M_s = \frac{\frac{1.2(49)}{1000}(18.5)^2}{8} = 2.52^{'k}$$

$M_u = 81.4 + 2.52 = 84^{'k} \leq \phi_b M_r = 164^{'k}$

OK

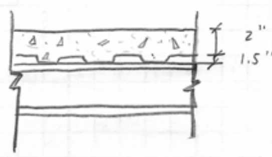


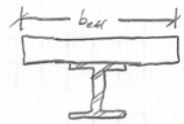
WITH 3½" 1.5ULR18 DECK & CONC

PROJECT: HYATT REGENCY — HIRO MCNULTY — PG 1 OF 5

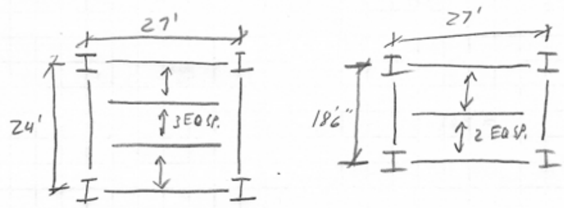
COMPOSITE STEEL PRELIMINARY ANALYSIS/DESIGN

FROM NON-COMPOSITE ANALYSIS, 1.5 VLR18 DECK & CONCRETE REQUIRED FOR 8' & 9'3" BEAM SPACING





TYPICAL BAYS



BEAMS SPACED AT 8' & 9'3"

$$b_{eff} = \frac{1}{4}(\text{SPAN}) = \frac{1}{4}\left(27'\left(12''/_{1'}\right)\right) = 81''$$

$$= \text{SPACING} = 9'3\left(12''/_{1'}\right) = 111''$$

$$8'\left(12''/_{1'}\right) = 96''$$

$$\therefore\ b_{eff} = 81''$$

$f'_c = 3\text{ksi}$

$f_y = 50\text{ksi}$

For 27' × 18'6" BAY

$W_{SD} = 20_{psf}$
$W_{LL} = 40_{psf}$
$W_D = 33_{psf}$ + BEAM SELF WT. (WILL CHECK)

$W_u = 1.2(33+20) + 1.6(40) = 127.6_{psf}$

$M_u = \frac{[(127_{psf})(9'3")](27')^2}{8} = 107.0 ft\text{-}k$

$V_u = \frac{127(9'3")(27)}{2} = 15.9k$

FROM NON-COMPOSITE, W10×49 WORKS, TRY

A W8 SECTION TO REDUCE DEPTH OF SECTION
(ASSUME FULLY BRACED) W8×48 $A_s = 14.1 in^2$

$C_c = 0.85(3ksi)(81")(2") = 413.1^k$

$T_s = A_s f_y = (14.1 in^2)(50ksi) = 705k$

$C_q = 0$: fully composite

$T_s - C_c = 705^k - 413.1^k = 291.9^k$

$291.9^k = A_{sc}(50)(2)$

$A_{sc} = 2.919 in^2$

$t_f = 0.685 in$ $b_f = 8.11 in$

$x = \frac{2.919 in^2}{8.11 in} = 0.36 in < t_f$ ✓

$M_n = 413.1(3.5-1) + 705\left(\frac{8.5}{2}\right) - 291.9\left(\frac{0.36}{2}\right) = 3976.5^{in\text{-}k}$

$= 331.4 ft\text{-}k$

SHEAR STUD STR RED. FACTOR (3/4"ø, 3" STUDS)

$\frac{0.85}{\sqrt{2}}(3/1.5")\left[(3"/1.5") - 1\right] \leq 1.0$ (FOR 2 STUD/RIB)

$1.2 \leq 1.0$

NO RED. FACTOR

PG 3 OF 5

$C = Q = 413.1^k$ TO TRANSFER STRESS

for $f'_c = 3$ ksi, NWC $\frac{3}{4}$" STUD

$Q_n = 21.0^k$

$\#_{studs} = \frac{2(413.1^k)}{21^k} = 39.3 \rightarrow 40$ SHEAR STUDS

$b_f = 8.11$ in $t_f = 0.685$ in

$\phi_{stud} \leq 2.5(0.685\text{ in}) = 1.71$ in ✓

SPACING LONGITUDINALLY $\geq 6d_s = 6(\frac{3}{4}) = 4\frac{1}{2}$"

SPACING TRANSVERSE $\geq 4d_s = 4(\frac{3}{4}) = 3$"

CHECK WITH SELF WT. $w_u = 1.2[(33+20)(9'3") + 48] + 1.6[(40)(9'5")]$

$w_u = 1.2$ klf

$M_u = \frac{1.2(27)^2}{8} = 109.4$ ft-k

$\phi M_n = 0.85(331.4\text{ ft-k}) = 281.7$ ft-k $\geq M_u$ ✓

$V_u = \frac{1.2(27)}{2} = 16.2^k$

$h/t_w = 15.9 \leq 2.45\sqrt{\frac{29000}{50}} = 59$ ✓

$\phi V_n = 0.9(50)(8.5)(0.4) = 153^k \geq V_u$ ✓

USE COMPOSITE W8X48 WITH 1.5 VLR 18 w/
40, $\frac{3}{4}$" ϕ SHEAR STUDS

FOR 27' x 24' BAY

$w_{SDL} = 20\,psf$
$W_{LL} = 80\,psf$
$W_D = 33\,psf$ + BEAM SELF WT. (TO CHECK)

$w_u = 1.2(33+20) + 1.6(80) = 191.6\,psf$

$M_u = \frac{(0.1916\,ksf)(8')(27')^2}{8} = 139.7\,ft\text{-}k$

$V_u = \frac{(0.1916\,ksf)(8')(27')}{2} = 20.7^k$

FROM NON-COMPOSITE, W10x45 WAS USED
TRY W8 x48 AS WITH THE OTHER BAY SIZE

$w_u = 1.2[(33+20)(8) + 48_{plf}] + 1.6(80)(8) = 1.59\,klf$

$M_u = \frac{(1.59)(27)^2}{8} = 144\,ft\text{-}k$

$\phi M_n = 281.7\,ft\text{-}k \geq M_u$ ✓

$V_u = \frac{1.59(27)}{2} = 21.5^k$

$\phi V_n = 153^k \geq V_u$ ✓

USE COMPOSITE W8x48 W
1.5VLR 18 W 40 3/4" Ø SHEAR STUDS

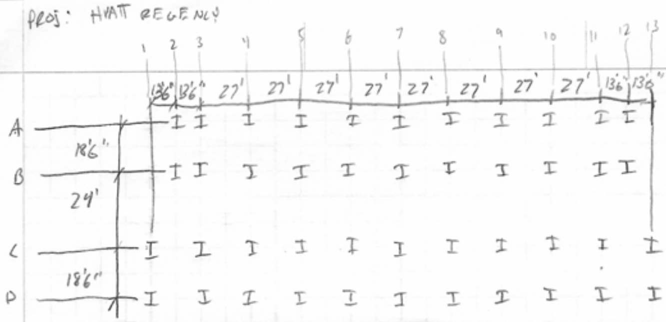
22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

REDUCTION IN THE NUMBER OF SHEAR
STUDS CAN BE DETERMINED IN FURTHER CALCULATIONS

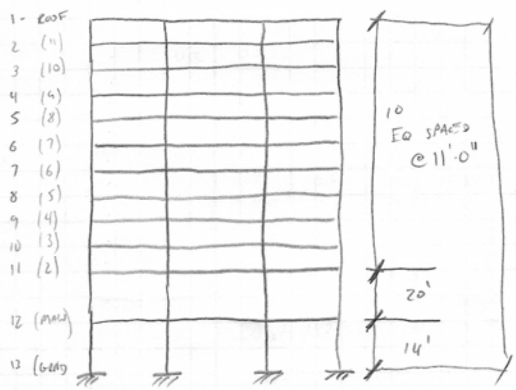
APPENDIX D

PROJ: HYATT REGENCY

PG. 1 OF 6



COLUMN LINE 3, 7, 11



* WITH ADJUSTMENTS DUE TO RE-FRAMING DEPTH

FRAME 7

LOOK AT COLUMNS UNDER 2, 8, 12

INTERIOR COLUMNS

$$A_T = \left(\frac{18.5' + 24'}{2}\right) 27' = 573.75\ ft^2$$

INTERIOR

LIVE LOAD REDUCTION (≤ 100 psf & NON ROOF)

$A_I = 4A_T = 2295\,ft^2$

BELOW 2

$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = 0.563\,L_o \geq 0.5\,L_o$

BELOW 8

$L = L_o\left(0.25 + \frac{15}{\sqrt{7(2295)}}\right) = \cancel{0.368} \geq 0.4\,L_o$

BELOW 12

USE $0.4\,L_o$

COLUMN BELOW	A_I	LLR(%)	A_T	LL	DL	ROOF LL	ROOF DL
2	2295	0.563	573.75	32.3	57.4	14.3	57.4
8	16065	0.4	4016.25	160.7	401.6	14.3	57.4
12	25245	0.4	6311.25	252.5	631.1	14.3	57.4

ROOF LL $= 25\,psf\,(573.75\,ft^2) = 14.3^k$
ROOF DL $= 100\,psf\,(573.75) = 57.4^k$

BELOW 2 LL $= 0.563(100)(573.75) = 32.3^k$
BELOW 2 DL $= 100(573.75) = 57.4^k$

BELOW 8 LL $= 0.4(100)(4016.25) = 160.7^k$
BELOW 8 DL $= 100(4016.25) = 401.6^k$

BELOW 12 LL $= 0.4(100)(6311.25) = 252.5^k$
BELOW 12 DL $= (100)(6311.25) = 631.1^k$

BEAM LL RED

$A_T = (24')(27) = 648\,ft^2$
$(18.5)(27) = 499.5 = 500\,ft^2$

$A_I = 2A_T = 1296\,ft^2$
$= 1000\,ft^2$

$L = L_o\left(0.25 + \frac{15}{\sqrt{1296}}\right) = 0.67\,L_o$
(MIDDLE)

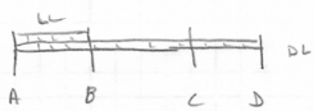
$L = L_o\left(0.25 + \frac{15}{\sqrt{1000}}\right) = 0.72\,L_o$
(ENDS)

PG. 3 OF 6

BEAMS

END $w_u = 1.2(100) + 0.72(1.6)(40) = 166.1$ psf
MIDDLE $w_u = 1.2(100) + 0.67(1.6)(100) = 227.2$ psf
$w_D = 1.2(100) = 120$ psf

END SPANS
$w_u = 166.1(18.5) = 3.1$ klf $w_D = 120(18.5) = 2.2^k$
MIDDLE SPAN
$w_u = 227.2(24) = 5.5$ klf $w_D = 120(24) = 2.9^k$



$FEM_{BA} = \frac{3.1(18.5)^2}{12} = 88.4^{'k}$

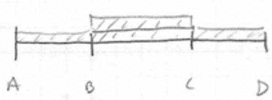
$FEM_{BC} = \frac{2.9(24)^2}{12} = 139.2^{'k}$

$P_u = 1.2(57.4 + 57.4) + 1.6(32.3) + 0.5(14.3) = 196.6^k$

JOINT B2

88.4 (+) 139.2 $\Delta FEM = 139.2 - 88.4 = 50.8^{'k}$

for DESIGN $P_u = 196.6^k$
$M_u = \frac{\Delta FEM}{2} = \frac{50.8}{2} = 25.4^{'k}$



$FEM_{AB} = \frac{2.2(18.5)^2}{12} = 62.7^{'k}$ 62.7 (+) 264

$FEM_{BC} = \frac{5.5(24)^2}{12} = 264^{'k}$

$P_u = 196.6^k$

$\Delta FEM = 264^{'k} - 62.7^{'k} = 201.3^{'k}$

for DESIGN $P_u = 196.6^k$
$M_u = 100.7^{'k}$

TRY W14 $P_u = 196.6^k$

$M_u = 100.7^{'k}$

$\ell = 11'$

$P_{eq} = P_u + \alpha M_u = 196.6 + \frac{24}{14}(100.7) = 369.2^k$

(1.71)

WITH $k\ell = 11'$ $P_{u\,eq} = 369.2^k$

USE W14x48 , $\phi P_n = 423^k$

EXTERIOR

ROOF LL = $25\left(\frac{18.5}{2}\right)(27) = 6.2^k$

ROOF DL = $100\left(\frac{18.5}{2}\right)(27) = 25.0$

$L = L_o\left(0.25 + \frac{15}{\sqrt{4\left(\frac{18.5}{2}\right)(27)}}\right) = 0.72L_o$

LL = $0.72(40)\left(\frac{18.5}{2}\right)(27) = 7.2^k$

DL = $100\left(\frac{18.5}{2}\right)(27) = 25.0^k$

WALL = $(110psf)\left(\frac{4''}{12}\right)(11')(27') = 10.9^k$

$FEM_{AB} = \frac{3.1(18.5)^2}{12} = 88.4^k$

ASSUME 1/3 INTO COL

$M_u = \frac{88.4}{3} = 29.5^{'k}$

$P_u = 1.2(25 + 25 + 10.9) + 1.6(7.2) + 0.5(6.2) = 87.7^k$

$P_{eq} = 87.7 + 1.71(29.5) = 138.1^k$

USE W14x48 FOR SYMMETRY & STACKING PURPOSES

BELOW 8

INTERIOR

$P_u = 1.2(401.6 + 57.4) + 1.6(160.7) + 0.5(14.3) = 815.1^k$

$M_{u\ DESIGN} = 100.7^k$

$P_{eq} = 815.1 + 1.71(100.7) = 987.3^k$

USE W 14 × 109 $\quad \phi P_n = 1240$

EXTERIOR

$P_u = 1.2(7(25) + 25 + 7(10.4)) + 1.6(7(7.2)) + 0.5(6.2) = 415.3^k$

$M_{u\ DESIGN} = 29.5'^k$

$P_{eq} = 415.3 + 1.71(29.5) = 465.7^k$

USE W 14 × 53 $\quad \phi P_n = 469^k$

OR W14 × 61 $\quad \phi P_n = 615^k$

BELOW 12

INTERIOR

$P_u = 1.2(631.1 + 57.4) + 1.6(252.5) + 0.5(14.3) = 1237.4^k$

$M_{u\ DESIGN} = 100.7^k$

$P_{eq} = 1237.4 + 1.71(100.7) = 1409.6^k$

USE W14×145 $\quad \phi P_n = 1530^k$

EXTERIOR

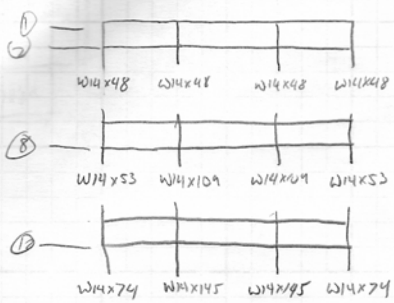
$P_u = 1.2(11(25) + 25 + 110psf(\frac{11}{12})(23)(120')) + 1.6(11(7.2)) + 0.5(6.2) = 644.3^k$

$M_{u\ DESIGN} = 29.5'^k$

$P_{eq} = 644.3 + 1.71(29.5) = 694.7^k$

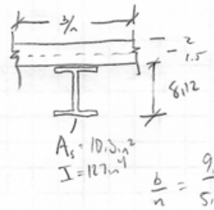
USE W14 × 74 $\quad \phi P_n = 753^k$

SUMMARY OF COLUMNS BELOW 2, 8, 12



APPENDIX E

HYATT REDESIGN | VIBRATION ANALYSIS | 1 OF 4



$$n = \frac{E_s}{1.35E_c} = \frac{29000000}{1.35\left(57000\sqrt{4000}\right)} = 5.96$$

$$\frac{b}{n} = \frac{9.25'(12)}{5.96} = 18.62$$

$$y_{comp} = \frac{(18.62'')(2'')(1'') + 10.3\,in^2\left(2''+1.5+\frac{8.12}{2}\right)}{\left(10.3\,in^2 + (18.62'')(2'')\right)} = 2.42\,in \text{ from top of conc}$$

$$I_{comp} = 127\,in^4 + \frac{(18.62)(2)^3}{12} + 10.3\left(\frac{8.12}{2}+2+1.5-2.42\right)^2 + 18.62(2)(2.42-1)^2$$

$$= 127 + 12.4 + 272.1 + 75.1$$

$$I_{comp} = 486.6\,in^4$$

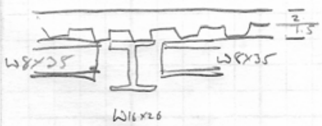
$$f_j = 0.18\sqrt{\frac{g}{\Delta}}$$

$$\Delta = \frac{5 w_j \ell^4}{384EI} \qquad w_j = (38+4+11)(9.25) + 35\,plf = 525.3\,plf = 0.525\,klf$$

$$\Delta = \frac{5(0.525)(27)^4(12)^3}{384(29000)(486.6)} = 0.445\,in$$

$$f_j = 0.18\sqrt{\frac{32.2(12)}{0.445}} = 5.3\,Hz$$

HYATT REDESIGN | VIBRATION ANALYSIS | 2 OF 4



W16X26
$A = 7.68 in^2$
$I = 301 in^4$
$d = 15.7 in$

$b = 0.4 l_b = 0.4 (18.5'(12''/ft)) = 88.8 in$

$n = 5.96$

$b/n = \frac{88.8}{5.96} = 14.9$

$\frac{b/2}{n} = \frac{44.4}{5.96} = 7.4$

$$\bar{y}_{comp} = \frac{14.9(2)(1) + 7.4(1.5)(2+\frac{1.5}{2}) + 7.68(2+1.5+\frac{15.7}{2})}{14.9(2) + 7.5(1.5) + 7.68}$$

$$= 3.03 in$$

$$I_{comp} = 301 + \frac{14.9(2)^3}{12} + \frac{7.4(1.5)^3}{12} + 7.68(2+1.5+\frac{15.7}{2} - 3.03)^2$$
$$+ 14.9(2)(1-3.03)^2 + 7.4(1.5)(2+\frac{1.5}{2}-3.03)^2$$

$$= 301 + 9.93 + 2.08 + 531.6 + 122.8 + 2.5$$

$$= 969.9 in^4$$

$$w_j = \left(\frac{525.3 plf}{9.25'}\right)(27') + 26 plf = 1559.3 plf = 1.56 klf$$

$$\Delta_j = \frac{5(1.56)(18.5)^4(12)^3}{384(29000)(969.9)} = 0.146 in$$

$$f_j = 0.18\sqrt{\frac{32.2(12)}{0.146}} = 9.26 Hz$$

$$f_s = \sqrt{\frac{1}{\left(\frac{1}{9.26^2}\right) + \left(\frac{1}{5.3^2}\right)}} = 4.6 Hz$$

$P_o = 65^{lb}$

$\beta = 0.05$

$\frac{a_p}{g} \le \frac{a_o}{g} = 0.006$

Joist Panel

$D_s = \frac{12 d_e^3}{(12 \cdot n)} = \frac{12\left(2 + \frac{1.5}{2}\right)^3}{12(5.96)} = 3.49\ in^4/ft$

$D_j = \frac{486.6 in^4}{(9.25')} = 52.6\ in^4/ft$

$C_j = 2.0$

$B_j = C_j\left(D_s / D_j\right)^{1/4} L_j$

$= 2.0\left(3.49/52.6\right)^{0.25}(27')$

$= 27.4\ ft$

$\le \frac{2}{3}$ floor width ✓

$W_j = w B_j L_j$

$= \frac{525.3 plf}{9.25'}(27.4)(27) = 42013\ lb$

Girder Panel

$C_g = 1.8$

$D_g = \frac{I_g}{L_j} = \frac{969.9 in^4}{27'} = 35.92\ in^4/ft$

$B_g = 1.8\left(52.6/35.92\right)^{0.25}(18.5) = 36.6 \le 2/3$ width $= \frac{2}{3}(61) = 40.67$ ✓

$W_g = \frac{1560}{27}(36.6)(18.6) = 39333\ lb$

$W = \frac{\Delta_j}{\Delta_g + \Delta_j} W_j + \frac{\Delta_g}{\Delta_g + \Delta_j} W_g$

$W = \frac{0.445}{\underbrace{0.445 + 0.146}_{0.591}}(42013) + \frac{0.146}{0.445 + 0.146}(39333) = 41351\ lb$

HYATT REDESIGN | VIBRATION ANALYSIS | 4 OF 4

$$\frac{a_p}{g} = \frac{P_o\, e^{(-0.35 f_n)}}{\beta W} = \frac{65\, e^{(-0.35(4.6))}}{0.05\,(41351)} = 0.006$$

$0.6\%\, g$

$0.6\%\, g > 0.5\%\, g$

NOT WITHIN LIMITS OF
AISC D.G. 11 LIMITS

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

APPENDIX F

Concrete Framing Cost Estimate

Filigree Precast Plank

	Size (Sq. Ft./ Floor)	*Floors*	*Cost ($/sq.ft.)*	*Total Cost ($)*
Precast Plank	17,000	12	4.67	**$952,680**

C.I.P. Slab on Plank

	Volume (C.Y./ Floor)	*Floors*	*Cost ($/C.Y.)*	*Total Cost ($)*
Slab	131	12	33.5	**$52,662**

C.I.P. Columns

	Volume (C.Y./ Column)	*Columns*	*Cost ($/C.Y.)*	*Total Cost ($)*
22"x28"	21.9	22	1141	$549,734
22"x32"	25	22	1141	$627,550
				$1,177,284

Total Building Cost

Precast Plank	$952,680
C.I.P. Slab on Plank	$52,662
C.I.P. Columns	$1,117,282
Total Cost =	**$2,100,000**

Steel Framing Cost Estimate

Gravity Beam Cost

	Total Length (ft)	*Cost ($/ft)*	*Total Cost ($)*
W8X10	966	16.46	$15,900
W8X35	8370	43.05	$360,329
W8X48	5585	56.55	$315,832
W10X12	311	18.56	$5,772
W10X19	37	21.71	$803
W12X14	162	18.75	$3,038
W12X19	681	27.10	$18,455
W14X22	898	30.64	$27,515
W14X53	270	60.00	$16,200
W16X26	8164	30.61	$249,900
W16X31	251	36.51	$9,164
W16X36	318	46.50	$14,787
W16X57	240	57.00	$13,680
W18X35	94	41.36	$3,888
W18X40	54	46.86	$2,530
W18X50	259	57.62	$14,924
W21X44	2232	50.38	$112,448
			$1,185,164

Gravity Column Cost

	Total Length (ft)	*Cost ($/ft)*	*Total Cost ($)*
W14X74	2341	81.16	$189,996
W14X120	1195	128.78	$153,892
W14X176	352	187.95	$66,158
			$410,046

Frame Beam and Brace Cost

	Total Length (ft)	*Cost ($/ft)*	*Total Cost ($)*
W8X40	3323	39.05	$129,763
W12X53	1482	80.63	$119,494
			$249,257

Frame Column Cost

	Total Length (ft)	*Cost ($/ft)*	*Total Cost ($)*
W14X43	175	81.16	$14,203
W14X48	70	81.16	$5,681
W14X53	68	81.16	$5,519
W14X61	311	81.16	$25,241
W14X68	68	81.16	$5,519
W14X74	66	81.16	$5,357
W14X82	132	81.16	$10,713
W14X90	328	81.16	$26,620
W14X109	90	128.76	$11,588
W14X120	66	128.76	$8,498
W14X132	198	128.76	$25,494
W14X159	185	128.76	$23,821
W14X176	107	187.95	$20,111
W14X233	208	187.95	$39,094
W14X283	152	187.95	$28,568
			$256,027

Shear Stud Cost

	Number	*Cost (per stud)*	*Total Cost ($)*
Studs	24095	1.37	**$33,010**

Decking and Slab Cost

	Size (Sq. Ft.)	*Cost ($/sq.ft.)*	*Total Cost ($)*
Steel Decking	204000	1.93	$393,720
Slab	204000	1.67	$340,680
			$734,400

Total Building Cost

Gravity Beams	$1,185,164
Gravity Columns	$410,046
Frame Beams and Braces	$249,257
Frame Columns	$256,027
Shear Studs	$33,010
Decking and Slab	$734,400
Total Cost =	$2,900,000

APPENDIX G

Concrete Framing Time Estimate

Filigree Precast Plank

	Size (Sq. Ft./ Floor)	*Floors*	*Daily Output*	*Days Required*
Precast Plank	17,000	12	1575	**129.52**

C.I.P. Slab on Plank

	Volume (C.Y./ Floor)	*Floors*	*Daily Output*	*Days Required*
Slab	131	12	95	**16.55**

C.I.P. Columns

	Volume (C.Y./ Column)	*Columns*	*Daily Output*	*Days Required*
22"x28"	21.9	22	14.15	34.05
22"x32"	25	22	14.15	38.87
				72.92

Total Building Time

Precast Plank	129.52
C.I.P. Slab on Plank	16.55
C.I.P. Columns	72.92
Total No. of Days =	**220**

Steel Framing Time Estimate

Gravity Beam Time

	Total Length (ft)	*Daily Output*	*Days Required*
W8X10	966	600	1.61
W8X35	8370	550	15.22
W8X48	5585	550	10.15
W10X12	311	600	0.52
W10X19	37	600	0.06
W12X14	162	880	0.18
W12X19	681	880	0.77
W14X22	898	990	0.91
W14X53	270	720	0.38
W16X26	8164	1000	8.16
W16X31	251	900	0.28
W16X36	318	800	0.40
W16X57	240	800	0.30
W18X35	94	960	0.10
W18X40	54	960	0.06
W18X50	259	912	0.28
W21X44	2232	1064	2.10
			41.48

Gravity Column Time

	Total Length (ft)	*Daily Output*	*Days Required*
W14X74	2341	984	2.38
W14X120	1195	960	1.24
W14X176	352	912	0.39
			4.01

Frame Beam and Brace Time

	Total Length (ft)	*Daily Output*	*Days Required*
W8X40	3323	550	6.04
W12X53	1482	640	2.32
			8.36

Frame Column Time

	Total Length (ft)	Daily Output	Days Required
W14X43	175	984	0.18
W14X48	70	984	0.07
W14X53	68	984	0.07
W14X61	311	984	0.32
W14X68	68	984	0.07
W14X74	66	984	0.07
W14X82	132	984	0.13
W14X90	328	984	0.33
W14X109	90	960	0.09
W14X120	66	960	0.07
W14X132	198	960	0.21
W14X159	185	960	0.19
W14X176	107	912	0.12
W14X233	208	912	0.23
W14X283	152	912	0.17
			2.31

Shear Stud Time

	Number	Daily Output	Days Required
Studs	24095	960	**25.10**

Decking and Slab Time

	Size (Sq. Ft.)	Daily Output	Days Required
Steel Decking	204000	4300	47.44
Slab	204000	2685	75.98
			123.42

Total Building Time

Gravity Beams	41.48
Gravity Columns	4.01
Frame Beams and Braces	8.36
Frame Columns	2.30
Shear Studs	25.10
Decking and Slab	123.42
Total No. of Days =	**205**