Architectural Engineering Senior Thesis 2006

PENNSTATE

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Renaissance Schaumburg Hotel & Convention Center
Schaumburg, Illinois

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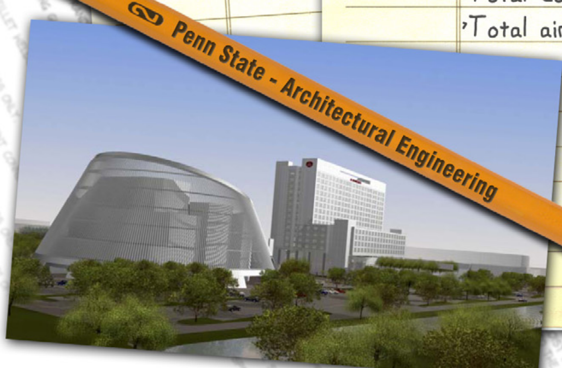
http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/ejy112

Building: Renaissance Schaumburg Hotel and Convention Center
Schaumburg Illinois
Hotel Size: 465,885, ft^2 with 17 stories
Total Cost: $207.1 Million
Project Team: Owner - Village of Schaumburg
General Contractor - Walsh Construction
Architect - John Portman & Associates
Program Manager - HDC International
Structural Consultant - Halvorson Kaye Structural Engineers

Electric/Lighting
- Distribution at 480/277V and 208/120V
- Hotel Transformers are 500 KVA @ 480V and 3 phase
- 3 phase-4 Wire primary voltage
- Service is supplied at 200A to 1600A per panel

Structural
- Post-Tensioned slab for multifunction floors
- Lobby consists of reinforced 42" concrete circular columns
- 12" and 18" Shear Walls for lateral support
- Driven steel piles for foundation support

Mechanical
- 2 packaged rooftop air conditioning units
- Total Cooling capacity of 189MBH
- Total air quantity of 6,000 CFM



Architecture
The project includes a restaurant, 500 guest rooms, dance ballrooms, health club, indoor swimming pool, and a large atrium area in the lobby. The exterior has concrete paneling and aluminum cladding and is accented with a waterfall feature that leads to a reflecting pond.

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Executive Summary

This report will detail the description and analysis of not only the current structural system of the Renaissance Schaumburg Hotel and Convention Center but also will investigate a steel frame alternative.

As the building is currently designed, it is a 17 story hotel (including two mechanical floors) that is composed of cast-in-place concrete columns and beams, and utilizes a 10" flat plate post-tensioned flooring system. In order to investigate if a suitable substitute could be found a few studies were complete on possible alternate systems. It was found that steel framing may lend itself well to this assignment. After analysis of the steel system, shear walls had to be replaced with braced frames in order obtain reasonable story drift. The system allows for comparable deflection (5.1" of deflection over a new height of 203' compared to 4.4" deflection over the original height of 188' – corresponding to an L/470 and L/510 respectively.

After the initial structural change a cost and time schedule study was completed in order to see which system would carry the most advantage. The steel system was estimated to save 16% over the use of the concrete system. The steel system costs about $16.40 per square foot while the current concrete system is about $19.50 per square foot. When comparing construction schedules of both buildings it was found that the steel system (assuming procurement of the steel was complete at the same time the concrete system would begin construction) was considerably shorter in erection time (approximately a 7 week difference).

The last study preformed included a detailed look into the lighting and design of the guest room spaces. Luminares were selected online, modeled in 3D and the space was rendered and analyzed using Autodesk Viz 2006. The point of this exercise was to get a firm grasp of light space requirements and was an attempt to accurately depict how a finished space may look. An exercise that could also prove useful in the advertisement of the building prior to opening and would give the owners an accurate detail of the space they are attempting to create.

After thorough investigation, comparing the performance of the structural systems, their relative costs, and construction timelines, it is recommended that since the steel system is comparable in performance and saves considerably in terms of overall cost and scheduling it should be considered as an efficient alternative to the current system.

This report is limited to analysis based on the most current design documents made available for the Renaissance Schaumburg Hotel and Convention Center by the lead structural engineer and architecture firm. Its function is to provide a detailed description and analysis of the systems currently in use, and the system proposed through the document. Simplified sketches have been included to further explain system layouts and details. Please see the appendix for other figures.

Introduction – *Building Background*

General Background Information

This study will examine many aspects of The Renaissance Schaumburg Hotel and Convention Center located in a northwest suburb of Chicago, Illinois. The building is a hotel structure that consists of a 465,885 square foot hotel and the area's largest convention center which is approximately 260,000 square feet. The 17 story (188 foot) structure is primarily constructed of cast-in-place concrete and utilizes shear walls for lateral support, the structural systems will be described in more detail later.


Exterior Photo of Hotel's North Face

The cost, as reported by the Village of Schaumburg, was $99 million for the hotel building, $104 million for the convention center, and the entire package (including parking and landscaping) was estimated to be around $207 million. The project was first discussed in the mid-1980's, but the Village of Schaumburg did not acquire the 45 acre building site until March 2000. The hotel's ground breaking ceremony was held in July 2004, and the topping out of the hotel was complete on May 26th 2005. The official opening date is slated for later this summer (July 2006). Below is an outline of companies and personnel responsible for the buildings construction.

Owner:
Village of Schaumburg
Robert O. Atcher Municipal Center
101 Schaumburg Court
Schaumburg, IL 60193
847.895.4500

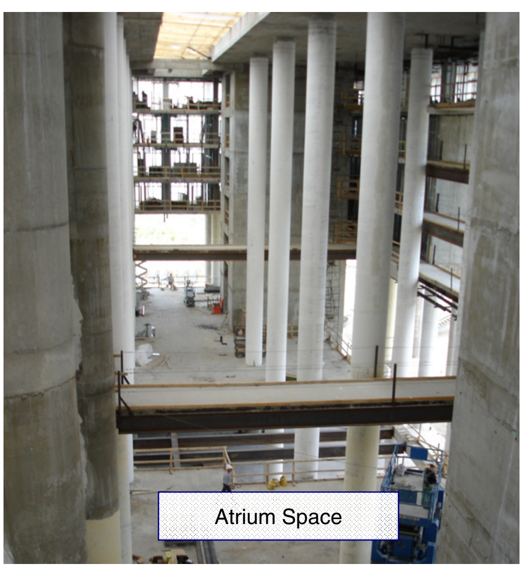
General Contractor:
Walsh Construction
929 West Adams Street
Chicago, Illinois 60607
312.563.5400

Program Manager:
HDC International
945 Linkside Terrace
Alpharetta, GA 30202
770.664.0101

Architects:
John Portman & Associates
303 Peachtree Street, NE, Suite 4600
Atlanta, GA 30308
404.614.5555

Daniel P Coffey & Associates
233 South Wacker Drive
Sears Tower Suite #5750
Chicago, Illinois 60606
312.382.9898

Structural Consultant:
Halvorson Partners
600 West Chicago Avenue Suite 650
Chicago, Illinois 60610
312.274.2400


Atrium Space

The complex will not have any problems attracting visitors to its 4-star quality, 500 room hotel. You are greeted to site from Thoreau Drive into an embrace of a circular terrace which directs you to the hotel's main entry. The driveway which runs to the building wraps in front of the hotel where you encounter a large waterfall which empties into one of many of the reflection ponds located on the building site. The entrance promises to be a grand way of welcoming hotel guests to one of the largest attractions in Schaumburg.

The hotel is technically a separate structure from the convention center and offers a vast array of amenities including a health club with pool, business meeting rooms, internet-ready guest rooms, a restaurant, a 28,000 sq. ft. Grand Ballroom and a winter garden. The large atrium lobby space is a flattering contrast to the modern yet intimate seating areas located throughout the rest of the building. To compliment the hotel, the Village of Schaumburg decided to add a 100,000 sq. ft. exhibition/convention center to the east end of the project site. The hotel has metal panels and opaque glass for most of the exterior shell, relying on a small amount of precast concrete paneling. The main lobby area includes a 5 story atrium that draws in a small amount sky-lighting from above.

The Renaissance Schaumburg Hotel and Convention Center (RSHCC) utilizes 480/277V and 208/120V distribution panels for electrical service throughout the building. The hotel transformers are typically 500KVA @ 480V, with primary voltage supplied as 3 phase - 4 wire. Each panel is supplies between 200 to 1600 amp service per panel.

The mechanical systems for the RSHCC include 2 rooftop packaged air conditioning units with contain 189MBH of cooling capacity. This system also maintains a total air quantity transfer of 6000 cubic feet per minute.

Structural Systems Description

The Renaissance Schaumburg Hotel's primary structural system is constructed almost entirely of concrete. The Convention Center is quite the opposite; it relies on the use of large steel joists due its large roof spans providing 100,000 square feet of support-free space. The Hotel uses a large amount of 42" circular concrete columns throughout the footprint, which add to the architecture of the atrium space, making it appear that each floor is almost floating. Primarily, a post-tensioned concrete slab system is used for most of the floors. The only time steel is used is in the span of the hotel's restaurant at the north end of the building. Floors 1 through 3 employ the use of steel for large open spaces, and also transfer the gravity load of the above stories using space more efficiently on the lower floors.

Floors 8 through 14 are highly repetitive and consist of post-tensioned slab and use typical 18" x 28" concrete columns. The roof level is constructed of two way slab and post-tensioned concrete slab to support the mechanical systems on the 17th and 18th stories. As the figures below will show, the building has a rather regular spacing of framing elements and is almost perfectly symmetric about the north-south direction. This lends well in designing the lateral force resisting elements since areas that would expect large stress concentrations can be easily predicted.

Shear walls

The main lateral force resisting system includes 9 concrete shear walls, the location and shape of which are detailed in the figures 1 and 2 below.

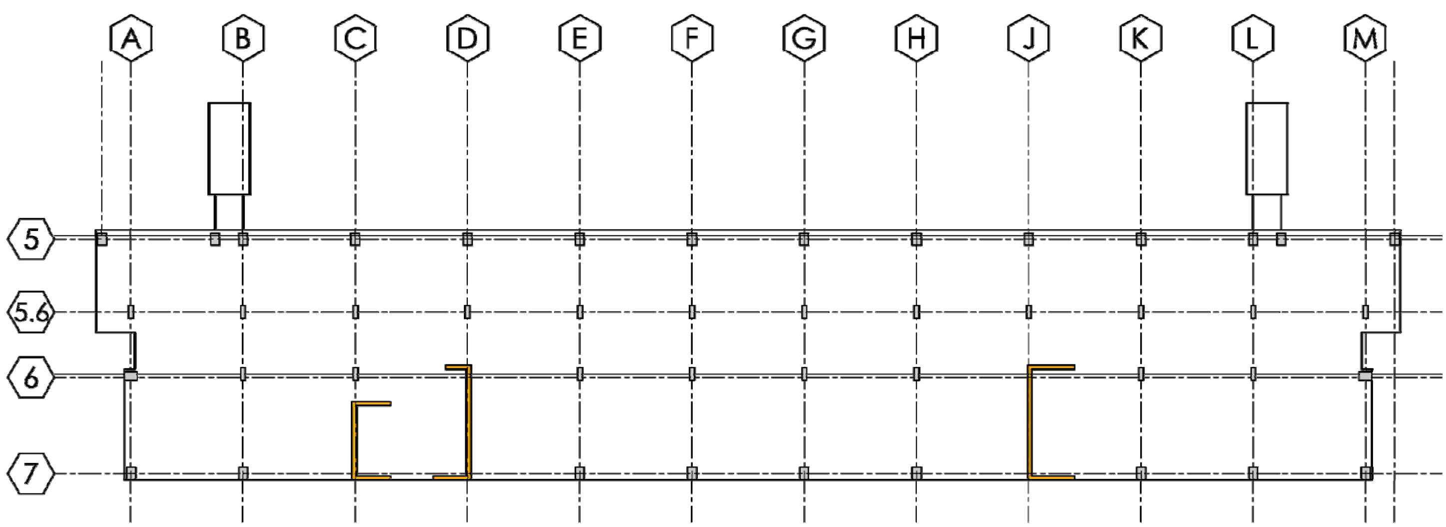


Figure 1 - Shear walls on typical floor (Floors 6 through the top mechanical level)

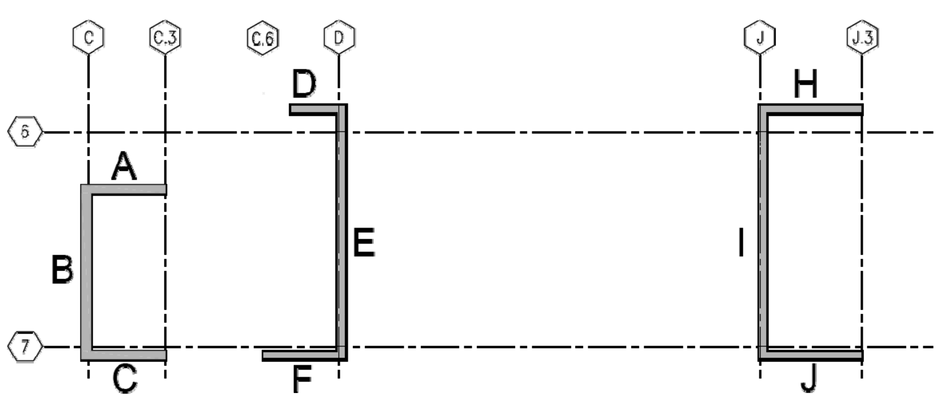
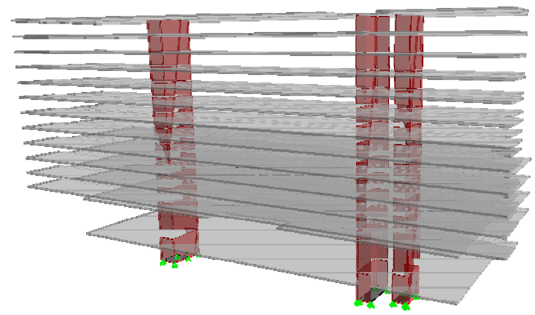


Figure 2 - Shear wall naming convention



These shear walls are to be constructed of 8,000 psi concrete on lower floors (up to floor 6) and 6,000 psi concrete on the upper levels, this is also when they change from a maximum of 18" thick at the bottom, to no less than 11". Reinforcement for the shear walls is typical ASTM A615 Grade 60 steel varying from #4's at 12" as a minimum, to #8's at 8". As one can see from the previous figure, there are 9 shear walls that create 3 C-shaped patterns. The naming convention used throughout the rest of this paper will refer to each wall with a letter as shown in figure 2. The shear wall system was evaluated in ETABS earlier this semester with 75% of the lateral load (it was assumed that 25% could be resisted by other concrete framing elements in the building). The ETABS model included a modified modulus of elasticity since the program does not assume cracking under deflection calculations. The elastic modulus was changed although in the real world this reduction is due to a cracked concrete section which reduces the moment of inertia and since the deflection is analogous to the M/EI diagram, reducing the modulus should properly help to predict the actual behavior of the system.

Framing

The frame skeleton of the RSHCC is rather unique. The architect called for large atrium spaces and designed the floor systems above the main lobby area to appear as though they almost float (see accompanying image). To accomplish this, a typical 42" diameter concrete column spans the first 3 to 6 levels of the hotel, which supports the slab. Typical slab thickness is 7.5" and on most floors uses a post-tension slab system which helps to reduce the amount of concrete needed. Steel is also utilized on lower floors (usually as a gravity load transfer from upper levels of concrete columns) which typical include beam and girder sizes of W16x26 and W24x55 respectively. The column grid for the main hotel structure is laid out in the east-west direction to 27' on center for 5 spans. However, there is a rather non-regular spacing of north-south column lines which also have 5 spans totaling 117 feet. Each of the two stair cases on the front exterior of the building are constructed out of steel and use moment resisting connections.

"Floating" Slab-to-Column Connection

Slab Systems

Multiple types of concrete slab systems are used in this project including one-way, two-way (with drop-panels), and post-tensioned slabs. Stud-rails are also used near column supports in order to minimize punching failure, eliminate excess drop-panels, and allow for the possibility of smaller column sizes. These stud-rails are typically used on column lines K, L and M, or the south-east side of the building, this is most likely due to the column line's adjacency to a change in slab elevations‡.

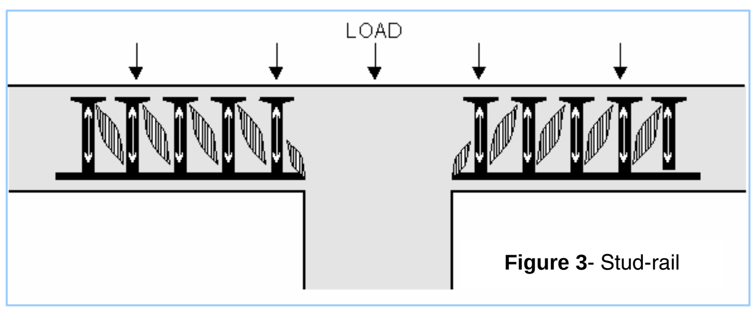


Figure 3- Stud-rail

The post-tensioned concrete slab is the most prevalent type of floor system used through the 17 stories of the building. Typical effective stresses in the post-tensioned tendons are typically around 20 kips per foot. This type of slab is useful due to its efficient use of concrete. In some systems, it results in a 30% savings of concrete material when compared to typically reinforced concrete slabs.

Structural Depth

Proposal Summary

For the proposed design of the hotel a focus on the performance of the concrete gravity system with shearwalls compared to a steel system that employs the use of braced framing was completed. Earlier a couple of different flooring systems were explored to determine if any could possibly out-weigh the benefits of the entirely concrete system. The system with most potential appeared to be a steel alternative. Steel allows for faster construction, longer spans, and

‡ Figure 3 – Stud-rail image courtesy www.studrail.com

significantly smaller member size. A reduction of foundation size could also accompany such a change in structural systems, but the design documentation of the original system was not given and due to time constraints each system's foundation requirements and designs were not accounted for.

Switching to the proposed system offers a couple of advantages that will be described. First of all, the preliminary reasons to switch to a steel system included the ability to remove a row of concrete columns (along line 5.6), as shown on a typical upper level floor plan as shown below in figure 4. This removal of columns could allow for more flexibility in floor plan layout of each room. Since the corridor runs between lines 5.6 and 6 more room could be utilized here, or each room can have additional space allocated to it.

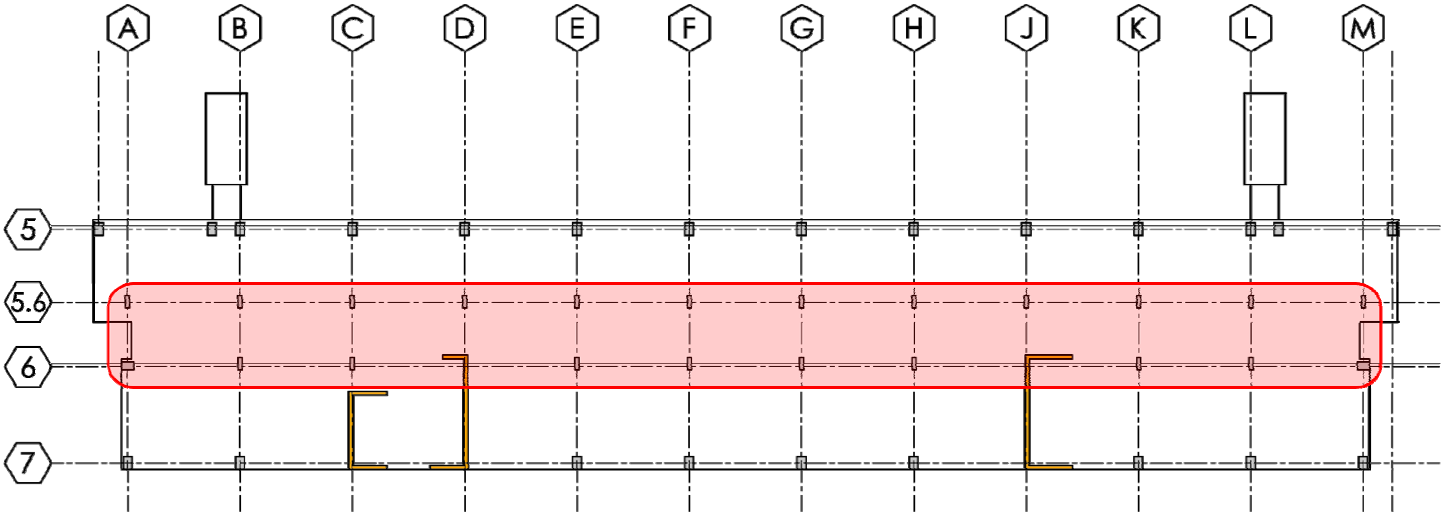


Figure 4 - This figure shows the two interior rows of columns in the current system, the advantage of using a steel system would be the ability to remove the columns along line 5.6

This would result in a floor plan that created more support-free space on the main guest room levels of the hotel, a floor plan of the proposed system can also be seen below in figure 5.

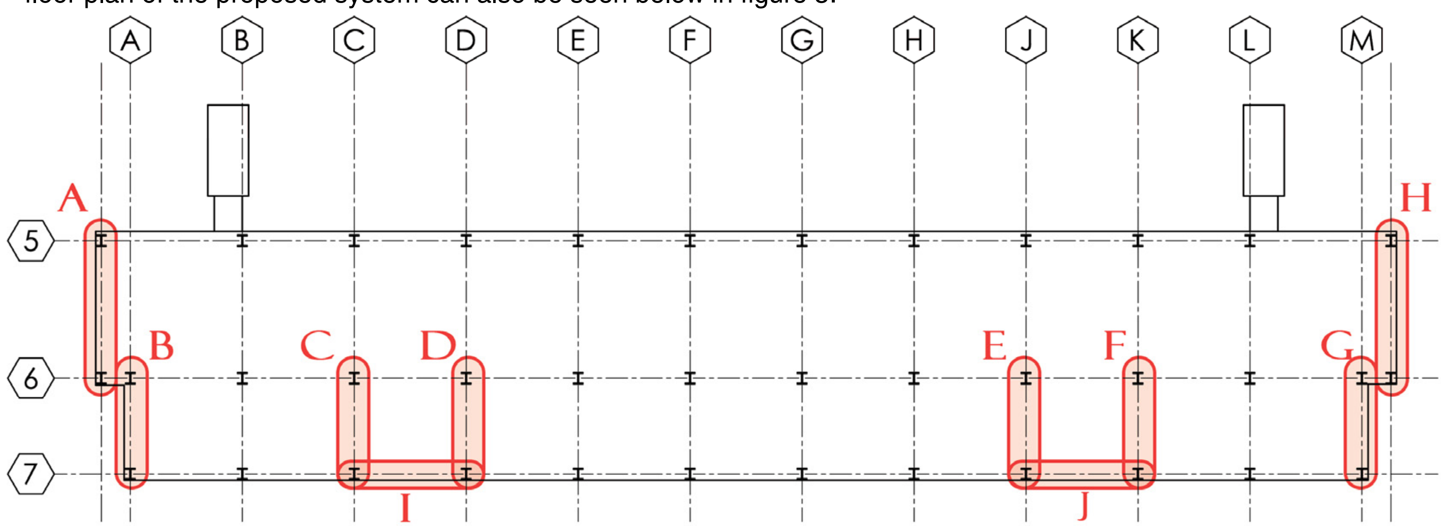


Figure 5 – Shown above is the steel framing plan, including braced frames as highlighted

The steel system would also help to reduce the total building weight, the major contributor to seismic lateral forces. Upon the design of the proposed structure it was found that wind (as expected) would control the lateral system design. Originally it was believed that the shear wall system currently used would be able to remain as the lateral force resisting system in the steel frame model, however, after analysis in ETABS it was determined that the shear wall system would not be sufficient in resisting building drift.

The original design was analyzed to take 75% of the lateral load, the rest being resisted by the other concrete framing elements (columns and beams). Under the implementation of the shear walls into the steel frame structure the floor to floor heights increased (by 17" per floor) and the entire 100% of the lateral load was distributed to the shear wall. Since the steel gravity framing would resist much less lateral force than the concrete gravity framing they replaced, this assumption (along with the increased height) resulted in unsatisfactory shear wall performance. Since the shear walls were already pretty significantly reinforced and in most places 18" thick at the bottom floors a design that incorporates steel braced frames, instead of a concrete shear wall, was used. A brace system will help in many different aspects of the design. First, it creates quite a reduction in weight and provides much more lateral resistance. Changing to steel bracing members also helps to ease construction since steel framing to concrete shear walls can be troublesome to install and detail.

With the exception of a few hand calculations RAM Structural Systems and ETABS were used for most structural analysis, following the procedures outlined in the IBC 2003, ASCE 7-02, ACI 318-02, and 3rd Edition (LRFD) AISC Manual of Steel Construction steel design codes (see Appendix A for load determination, load combinations, and other preliminary design information).

Solution Summary

The largest effort of this study was focused on the comparison of the current and proposed lateral systems. The current system uses shear walls as described in the background information above, and the proposed system changes not only the gravity elements to steel, but also the lateral system is replaced with steel braced frames. Further explanation of the lateral systems can be found in the next section, but for now, the gravity system will be detailed.

Typical framing sizes for an upper level floor type (levels 8+) in this building can be seen below, while the rest can be found in Appendix B.

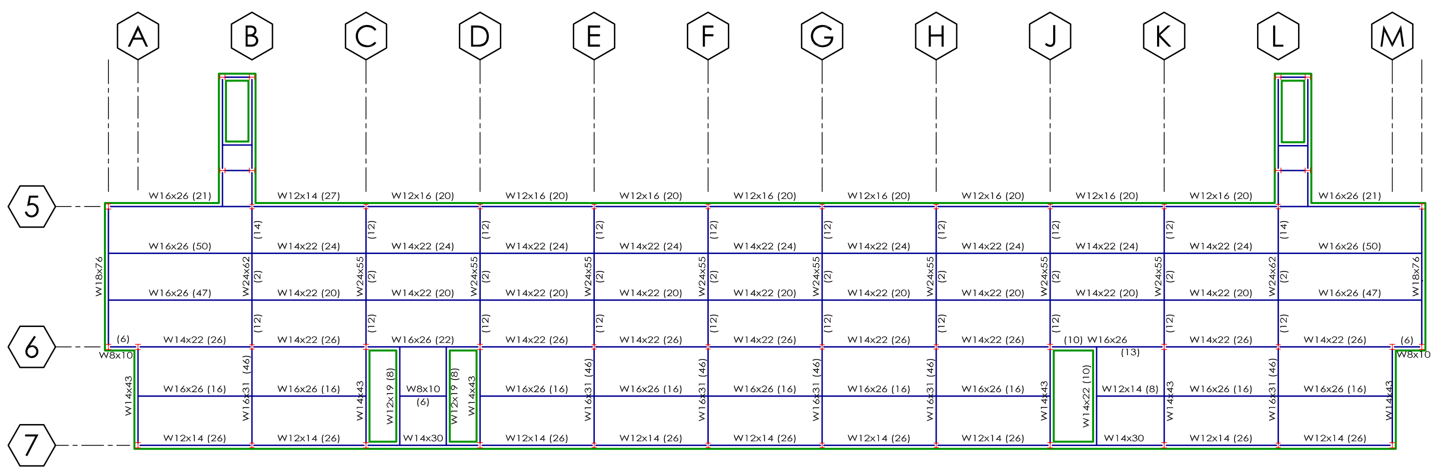


Figure 6 – Shown above is the steel framing plan, including braced frames as highlighted

Each floor was modeled in RAM Structural Systems, and random member checks (see Appendix A) were completed to ensure a consistent (and accurate) design. Since the steel beams with the composite slab are significantly deeper than the 10" flat plate system they replace- a modification of story height was necessary. Since the deepest floor members were ~23" (W24x55) and the 4" slab system was used (2003 International Fire Code demands a minimum of 3.6" slab for the required 2 hour fire rating), the average floor to floor height of 9'-8" was increased by 17" for the preliminary design.

Steel column members (the concrete columns in the atrium area were retained in order to limit the change in architectural design) were designed using a similar process as the beam and girders described above (again, see Appendix A for hand checks and Appendix B for column schedule).

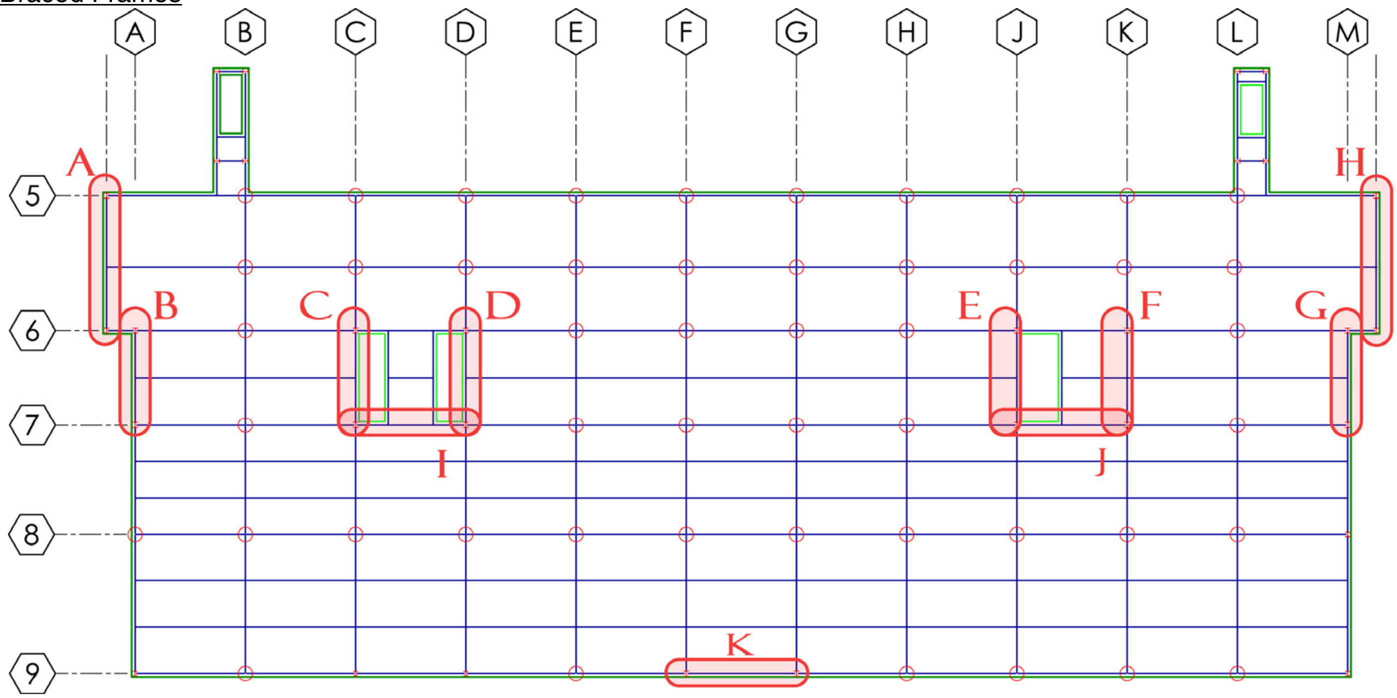
The original slab system was removed and in its place a 4" composite slab was analyzed. The post-tensioned slab added slightly over $1 million to the project and is typically a chore to install. Specifically, a specialized contractor needs to be used for installation and the stressing of each tendon must be closely watched. A composite system is much simpler to install, and does not require specialized consultation. For these reasons, and the ability for the system to be easy implemented into the steel frame model a composite slab was used in the analysis of the proposed structure. A hand check of the composite floor slab can be seen in Appendix A.

Changing the lateral system design was the last consideration of this study, although it is noted that since the building is being changed from concrete to steel the foundation design should reflect the reduction in dead weight of the structure. Originally the shear walls were going to be included as the lateral force resisting system, however, since the entire building was concrete in the current design, 25% of the lateral forces were allowed to be distributed to the concrete framing members, meaning the shear walls only had to resist 75% of the lateral load. Upon changing the system to steel the shear walls would have to resist all of the lateral force, and thus either had to be re-designed or an alternate system had to be implemented.

»Shear Walls

The shear wall system incorporated into the steel framing design, upon being loaded with 100% of the lateral load, was not able to be successfully designed. The shear wall had excessive deflection (well over 6 inches) and reinforcement would have been particularly large. Therefore, without increasing the width of the walls, which were already 18" through the bottom 4 stories, the shear wall system was determined to be insufficient and a steel braced frame was then implemented.

»Braced Frames



Since steel braced frames were the next obvious alternative, the structure was retrofitted with 10 braces throughout the floor plan, and an 11th frame (labeled K above) was included on the lower 6 floors to help with resistance in the east-west direction of the building. A major concern of using the brace layout as shown in figure 6 above is that the columns that are apart of two frames that support orthogonal directions have to resist significant weak axis bending, and bi-axial bending when considering diagonal wind forces. In order to account for this both direct and torsional effects were considered including all 4 applicable load cases for wind loading under ASCE 7-02. The controlling case for deflection was wind applied directly to the north or south face of a building (due to symmetry) and blowing directly north or south.

After initially modeling the steel braced frames C, D, E, I and J it was determined that in order to minimize member sizes of the frames that additional support would be needed. For symmetry and to minimize the impact on the architectural aspects of the building frames F and J were added around the east elevator shaft. Also, a total of four frames were added on the east and west face of the building. Placing these braced frames proved most interesting since the frames on the exterior faces of the building had to allow for the window openings of the hall way. Fortunately, frames B and G do not have any windows located on their elevation since each suite only has windows in the north of south face of the building, however, frames A and H do. The figure below (figure 7) shows a composite of the window opening requirements and the placement of the braced frame. Due to the occupied space taken by the frame, the original 5' window needed to be replaced with a 3' wide window (of 4' in height), while it is recommended that the top two floors remove these windows, or allow for alternate placement. Complete calculations and member schedules can be located in Appendix B.

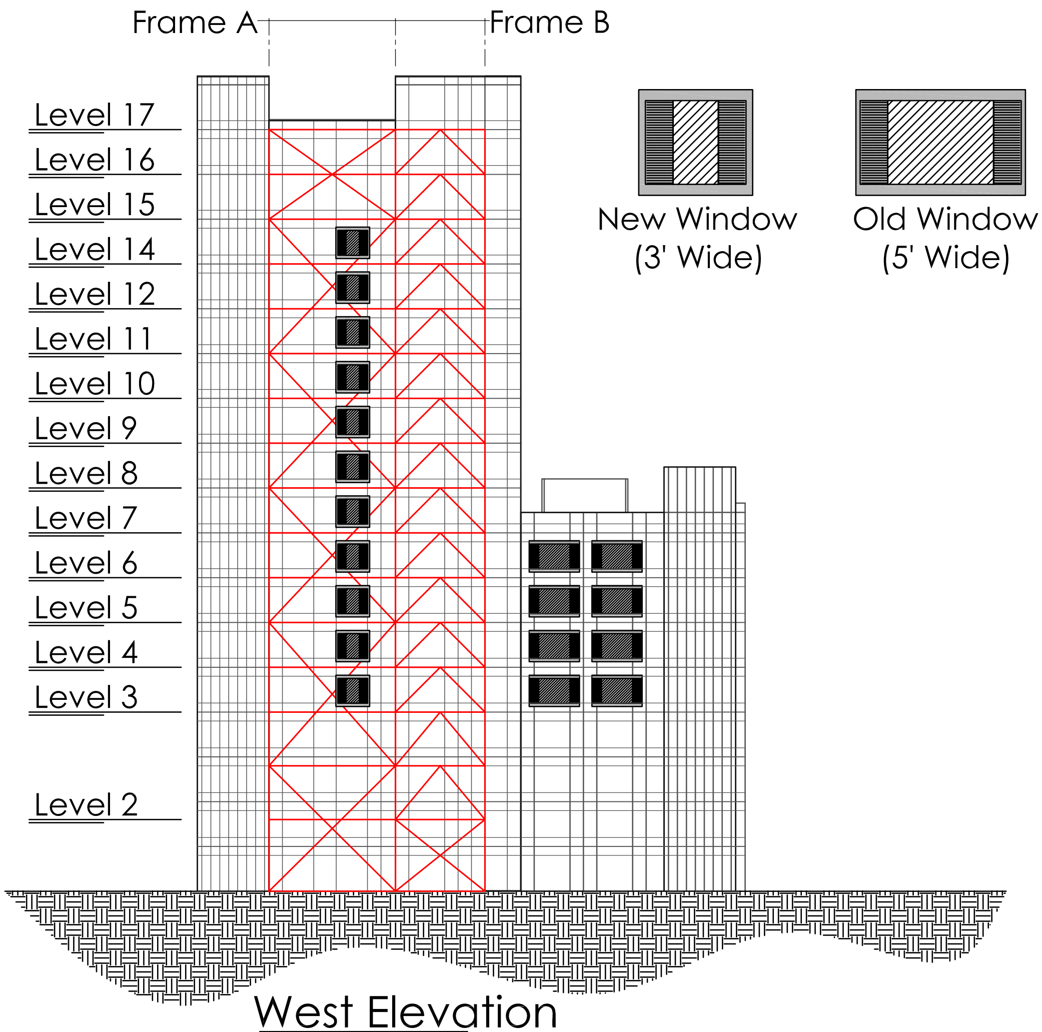


Figure 7 – Shown above is the elevation of the exterior face steel framing plan, including braced frames as highlighted. The bracing members require that the windows in the corridor space of the building be reduced to 3' from their original 5' width.

Structural Depth Summary

Considering the change from concrete to steel support, and the change from shear walls to braced frames a major overhaul of the system was undertaken. The impact of the placement of interior frames was very minimal, with the largest consideration of impact being due to the frames added along the east and west faces of the building. The floor system was changed to a composite decking, which including beam/girder member size, totaled to 27" thick, a full 17" larger than the flat plate post-tensioned concrete it would be substituting.

After taking a look at how each gravity and lateral system performs one may draw a few conclusions regarding the analysis. First of all there is a large reduction of weight when the steel system was used in place of concrete, this reduces axial load on lower columns and could reduce the size of foundations (given a longer work schedule further investigation (including overturning moment) and sizing of foundation systems would be considered). The major disadvantage to changing from concrete to steel is the required fireproofing associated with steel buildings, as explored in the breath study to come the fireproofing process adds a significant time to the steel construction schedule and cost.

Overall the braced frames in conjunction with the steel gravity framing and composite slab perform very well for their extended height, comparatively the concrete system had a total building drift of 4.41" corresponding to and average story drift of L/513 and the steel system had a total building drift of 5.17" providing an average floor drift of L/470. Even though the concrete system appears to have resisted the lateral much better, remember that it is only taking 75% of the load that the steel frame is resisting and that both of these deflection values would be considered to be acceptable. In conclusion of this study, although drift performance was lost, the steel system incorporated many other advantages including the removal of a row of columns, minimal architectural impact, and a much easier erection method.

Breadth Concerns— Construction Issues and Lighting Analysis

Construction Issues

When changing the structural framing from cast-in-place concrete to steel significant impacts were made on the final building cost and construction timeline.

First, I investigated the cost differences in material and construction for each system. RS Means was used to compare the system costs with as little variability as possible (meaning most of the same assumptions held true for both estimates- as highlighted in Appendix C).

Below you can see a final price summary of each system:

Systems Comparison

System	Total Cost
Current	$ 8,486,680.32
Proposed	$ 7,136,813.04

Result of proposed switch	
Savings	$ 1,349,867.28
% Difference	15.91%

Per Square Foot	Total Cost
Current	$19.4659 /sq.ft.
Proposed	$16.3697 /sq.ft.

Total square footage
435977.7

The second issue with changing the framing system occurs when constructing the building. There is inherently large difference in changing from a concrete system to a steel frame, so analysis of construction times was necessary in order to compare which system had the largest construction advantage.

The figure below (Figure 8) shows the estimated construction times of each system, as though they shared the same start time. As you may note, the concrete systems onsite construction time is approximate 7 months in duration while the steel system comes in at just under 5 months. A summary of specific tasks, durations, and task precedence can be found in Appendix C.

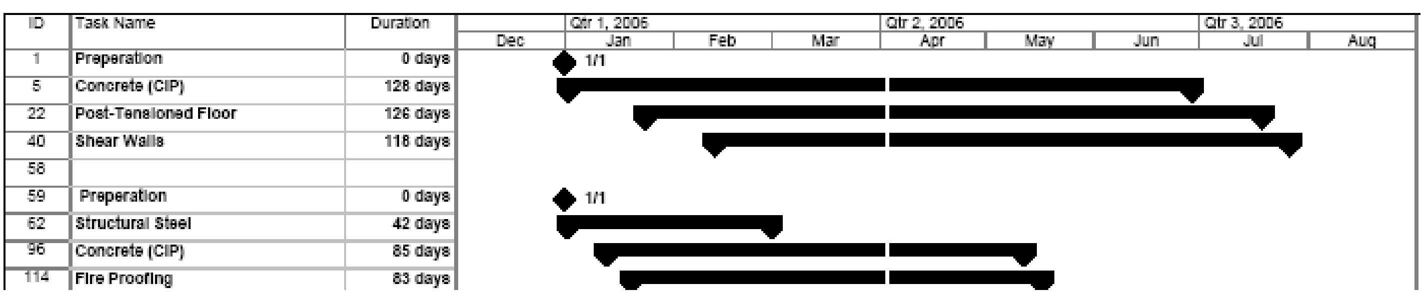


Figure 8 – Schedule Comparison

Project Schedule Comparison	Days
Steel Duration	97
Concrete Duration	148

There are a few assumptions that need to be state before taking these numbers to heart. First of all procurement times for steel members and concrete were not factored into the above estimation; these were assumed to be taken care of as construction starts and (with luck) would not impact the time-line negatively. This assumption is an advantage to the steel system since all of the steel members would have to be designed, go through the shop-drawing procedure, have a steel order placed, complete pre-construction details, and be shipped to site. The concrete must also be procured (which would be much shorter than the described steel process), but since the design calls for the structure to be cast-in-place there would be a longer construction time for the allotment of time for the concrete formwork, pouring, and curing time. Again, the schedule above only estimates on-site construction of the framing system and there would be a significant impact on lead times based on material types.

Lighting Analysis

After specializing in structural analysis and its construction ramifications I decided to take a look at another out of depth topic. Originally I had planned to address concerns with the HVAC system, specifically the plenum spaces that could change with the framing type. This step proved to be rather difficult because most of the horizontal duct work is on the lower 2 floors (which were effectively unchanged since there story heights where much larger than the rest of the building), while the guest suites are provided air through vertical plenum spaces (which were not affected by a change in the structural framing). Since the proposed design did not significantly impact this part of the system (the guest suites still had vertical supply and lower floors had very large floor to floor heights) I decided to take a look at how the lighting system for a typical guest suite would be implemented.

I was able to model the given architecture plans into 3D spaces that allowed for me to use lighting profiles of luminares currently offered by various manufactures, in this case Erco was chosen. Three specific lighting fixtures were chosen for the space- a down-light for typical lighting of the room, a wall wash that highlights some of the architectural aspects of the space, and finally for the bathroom vanity, a list of which can be found below.

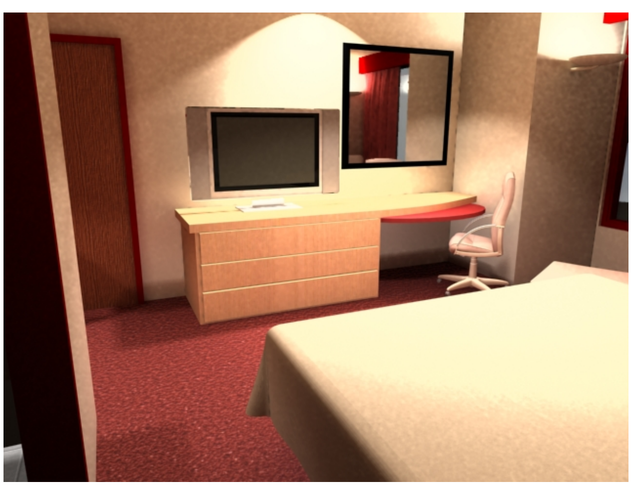
Type	#	Wattage/fixture	Ballast	Location
Erco	3	33W		Mirror (bath)
Erco	1	68W		Downlights
Erco	2	34W		Wall wash
Erco	4	34W		Single Down

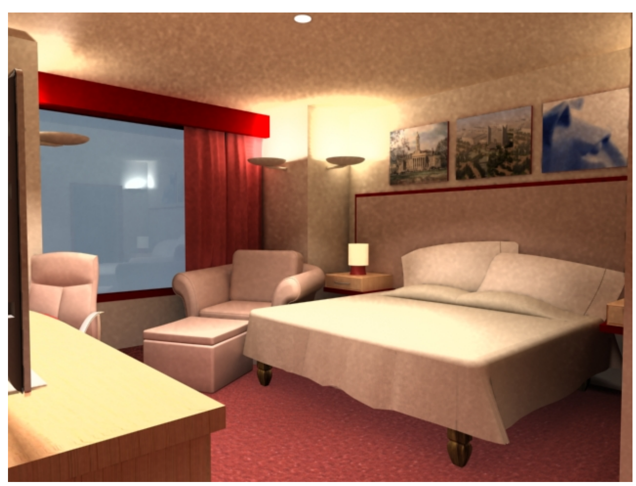
Total Room Wattage
371W

Illuminance Levels to Reach		
Area	fc	lux
Floor	10-20	100-200
Desk	50	500
Shower	20	200
Face level in bathroom	50	500
Architectural Elements	30-50	300-500

As-Rendered Illuminace Levels			
Area	Approx.		Outcome
	fc	lux	
Floor	20	200	Sufficent
Desk	50	500	Sufficent
Architectural Elements	40-50	400-500	Sufficent

The down-lights were an Erco Lightcast size 8 fixture, the wall washers were and Erco atrium with CFL's and the lights above the mirror are Erco mirror fixtures the ballast, fixture and light information can be found in Appendix D.





Figures 9 and 10 - Render of typical guest suites, see Appendix D for pseudo-color images.

Breadth Summary

After considering the lighting requirements and a critical investigation of the finished guest suite spaces I believe that (although previous lighting conditions were unknown) the exercise helped to form a better of the finished space the structure was framing. This investigation helped to form a different perspective of how the building was to function.

The construction cost and timeline investigation gave a further understanding to the advantages of the change in framing systems. The steel system is considerably cheaper, by 16% of the original system cost and also takes approximately 7 weeks less to erect. After completing this study, based on constructability and the cost analysis completed (and presented in Appendix C) it would be recommended to switch to the steel system as the main structural framing system.

Conclusions and Recommendations

Concluding Summary

After a lengthy inspection of the current design and analysis of suitable replacement systems (comparing not only structural performance but also the impact on the architectural design and construction costs/time) it was concluded that the proposed steel system had many benefits not to be overlooked, and overall is the system that I would recommended. The steel system ended up with an acceptable drift over the entire building height and even though the original system appeared to perform better with deflections, the new proposal eliminates an entire row of columns (while keeping the resulting structure relatively shallow) and has major advantages when considering both construction time, ease of installation, and estimated building cost.

This project considered many aspects of the design process and, as in most journeys, could not have been completed with the help of faculty or the other students responsible for completing their own studies, yet always available to answer my questions. This exercise helped to gain a very profound knowledge of the building described above and familiarized me with many different analysis methods, and computer software packages in order to complete the study. The completion of this assignment undoubtedly prepared me for future projects that will be completed in the professional world and has instilled skills and problem solving strategies that may not be fully recognized until well after my formal education has finished.[§]

[§] End of Final Report

Acknowledgements

Professional

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Personal

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