Architectural Engineering Senior Thesis 2006

PENNSTATE 1855

Eric Yanovich
Structural Option
Advisor: Kevin Parfitt

Renaissance Schaumburg Hotel & Convention Center
Schaumburg, Illinois

Eric J. Yanovich - Structural Option - ejyl12@psu.edu

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

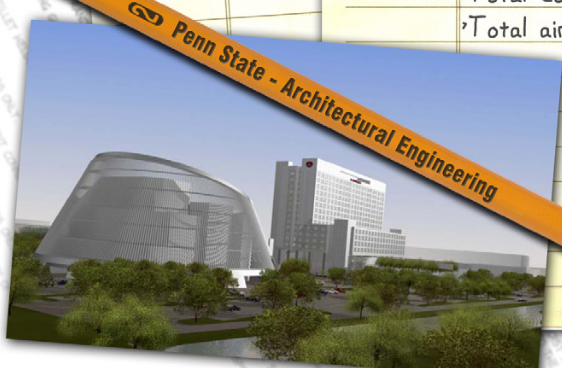
- Post-Tensioned slab for multifunction floors
- Lobby consists of reinforced 42" concrete circular columns
- 12" and 18" Shear Walls for lateral support
- Driven steel piles for foundation support

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.



Penn State - Architectural Engineering

PENNSTATE 1855

Contents

Page

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

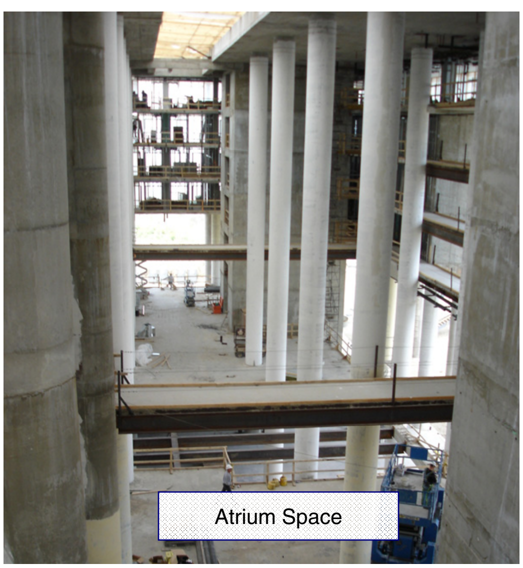
General Contractor:
Walsh Construction
929 West Adams Street
Chicago, Illinois 60607
312.563.5400

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.

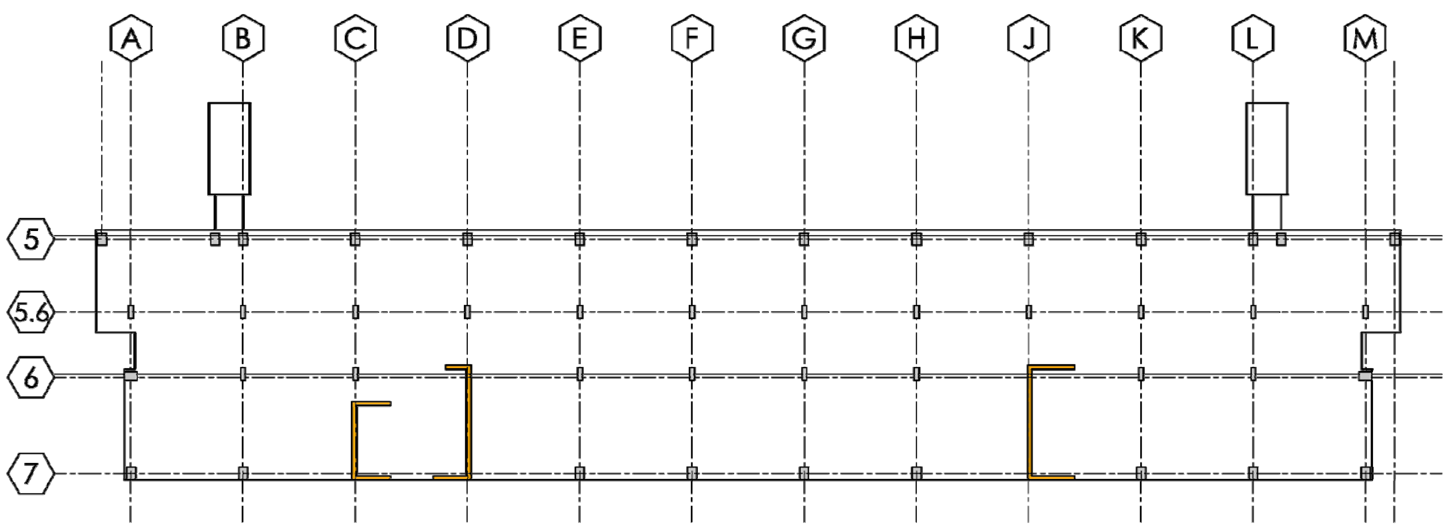


Figure 1 - Shear walls on typical floor (Floors 6 through the top mechanical level)

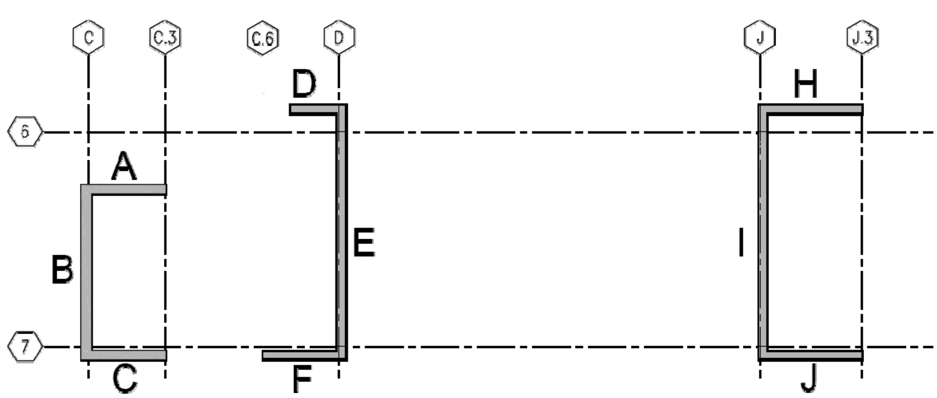
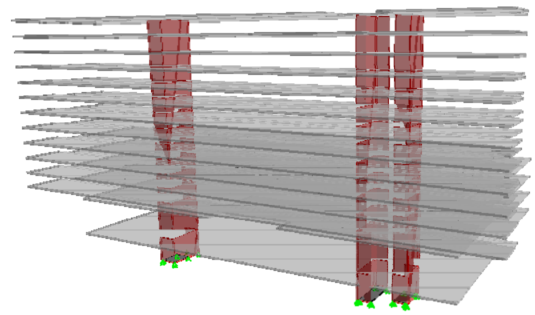


Figure 2 - Shear wall naming convention



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[‡].

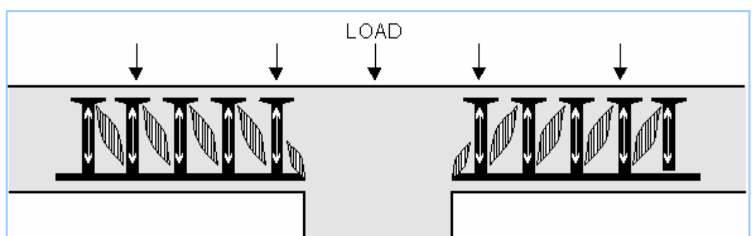


Figure 3- Stud-rail

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.

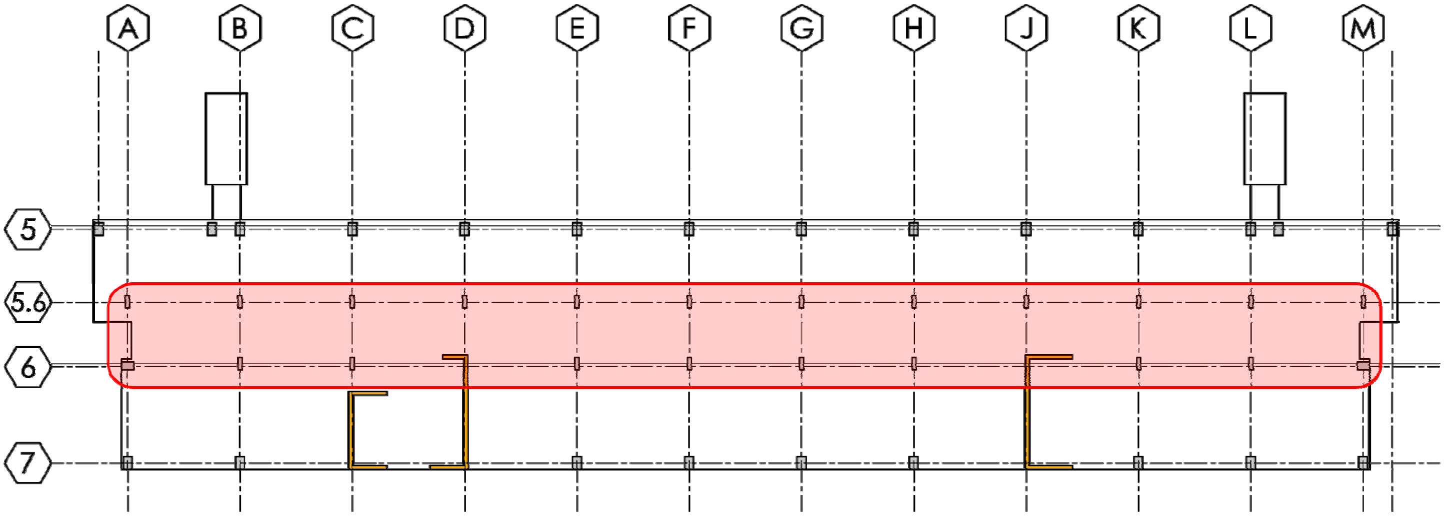


Figure 4 - This figure shows the two interior rows of columns in the current system, the advantage of using a steel system would be the ability to remove the columns along line 5.6

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.

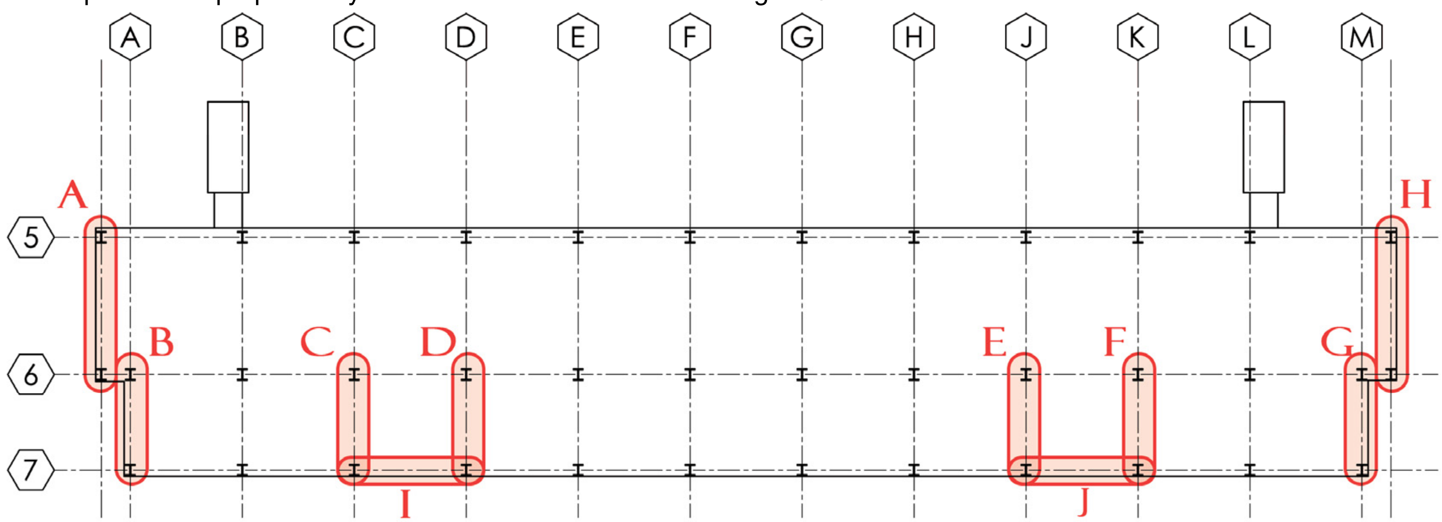


Figure 5 – Shown above is the steel framing plan, including braced frames as highlighted

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.

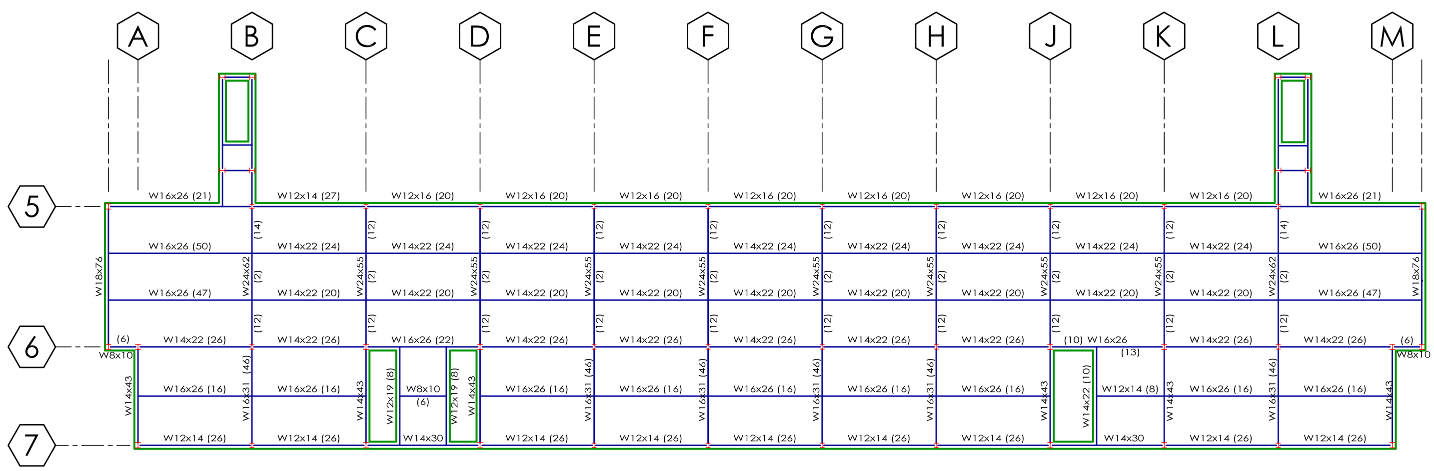


Figure 6 – Shown above is the steel framing plan, including braced frames as highlighted

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).

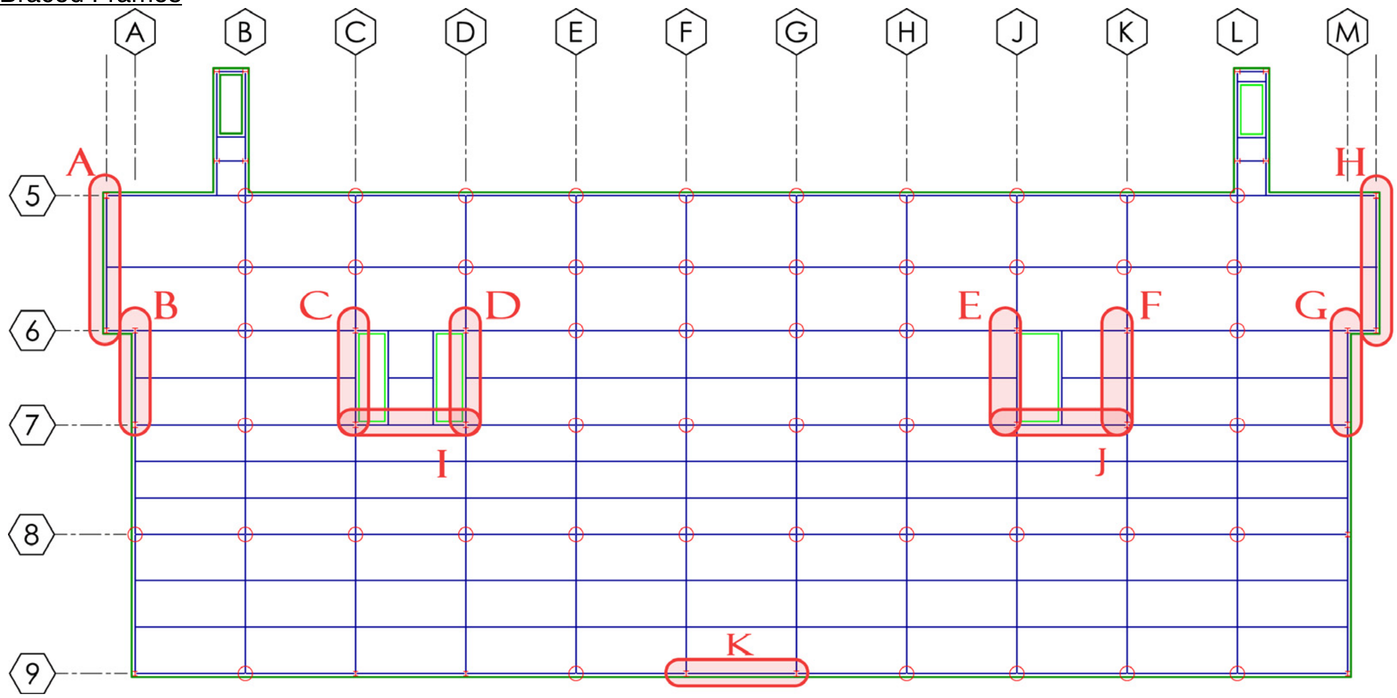
The original slab system was removed and in its place a 4" composite slab was analyzed. The post-tensioned slab added slightly over $1 million to the project and is typically a chore to install. Specifically, a specialized contractor needs to be used for installation and the stressing of each tendon 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.

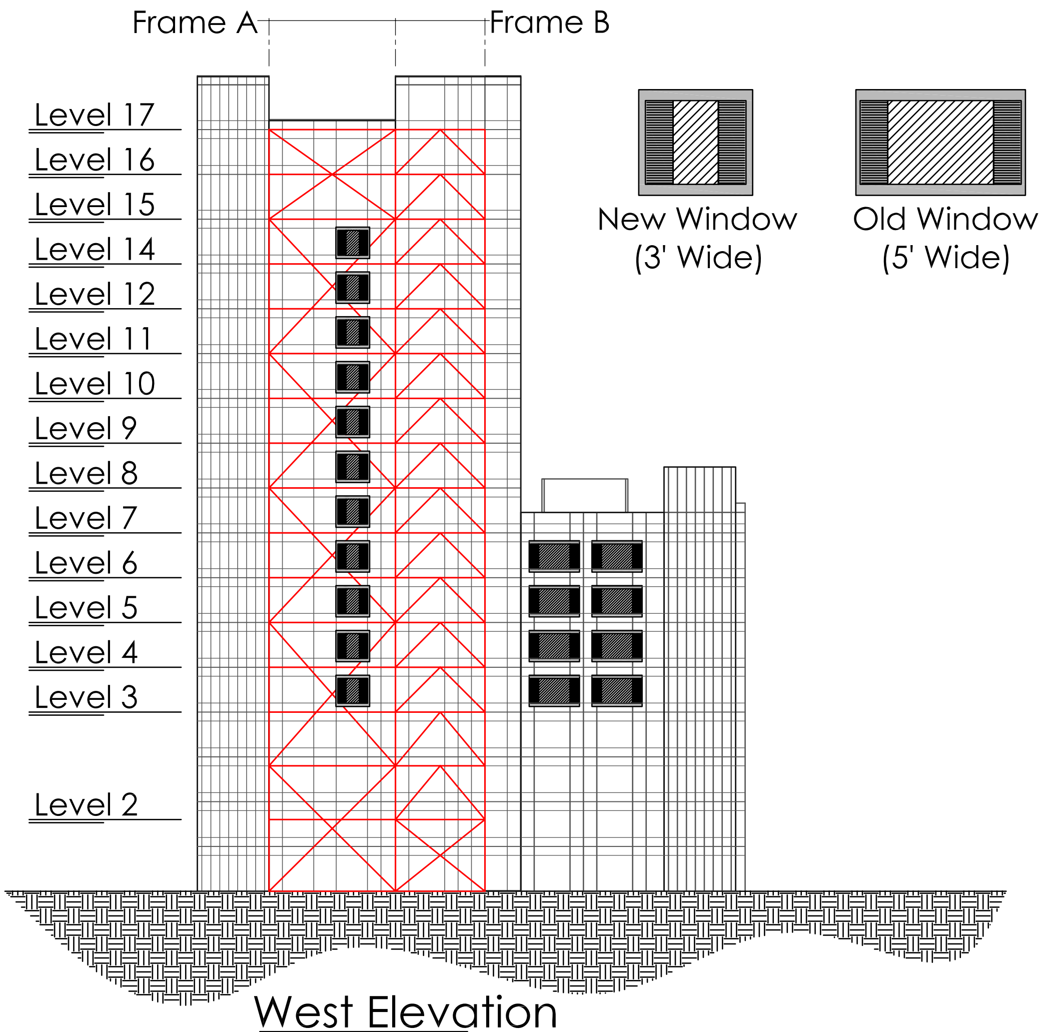


Figure 7 – Shown above is the elevation of the exterior face steel framing plan, including braced frames as highlighted. The bracing members require that the windows in the corridor space of the building be reduced to 3' from their original 5' width.

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.

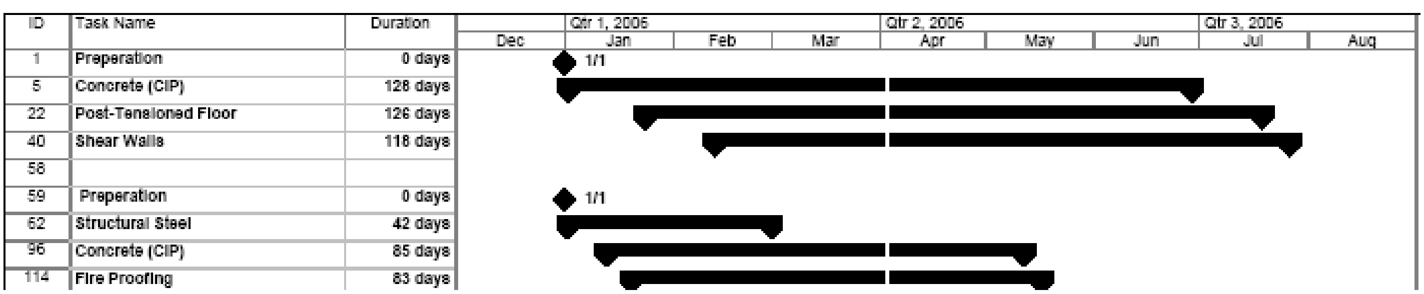


Figure 8 – Schedule Comparison

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.

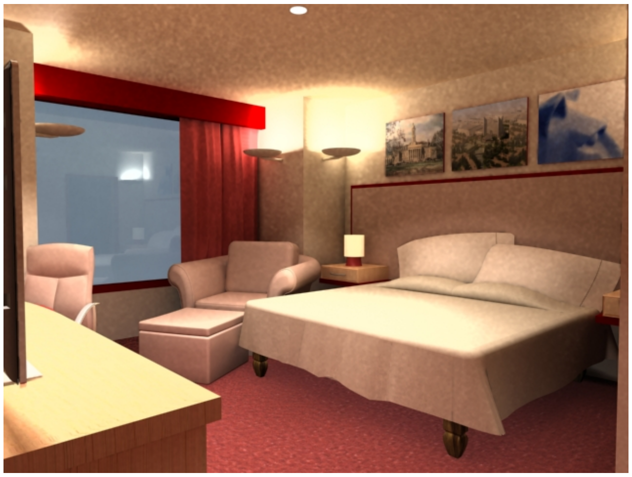
Type	#	Wattage/fixture	Ballast	Location
Erco	3	33W		Mirror (bath)
Erco	1	68W		Downlights
Erco	2	34W		Wall wash
Erco	4	34W		Single Down

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

I would like to begin by thanking everyone who helped with this project. Specifically, Chris Hewitt at the AISC who was instrumental in finding a suitable building, along with Todd Alwood. Josh Munson of Halvorson and Partners for supplying the structural documentation that accompanied the project. John Portman and Associates that made the architectural and MEP drawings available. And finally Brian Townsend and Ken Fritz at the village of Schaumburg for allowing me to use their building as my thesis project. The Architectural Engineering faculty have been the most instrumental in completing this project and were a pleasure to spend the last 5 years of my education learning under, specifically for his help in this project Kevin Parfitt deserves a big thank you for helping me and the other 90+ students in the program.

Personal

I still cannot believe that five years have past. There is an incredibly long list of people to thank for shaping me into the student and more importantly, the person I am today. First and foremost, an unconditional thank you to my parents who have always encouraged me to do my best in all facets of life. For my father's work ethic, and my mother's limitless ability to absorb knowledge, I am thankful that so much of what makes you both great people has had the chance to rub off on me. To my younger brother, who may be the smartest member of the family, thank you for making all my memories of childhood wonderful, and for always having the patients to deal with my questions (no matter how remedial you may find them).

I would also like to thank Jamee, who is the reason that I made it through college in one piece. Quite possibly the hardest working person I've met while at school, or anywhere for that fact. I love you, and I'm so excited about our future together. The last four years have been amazing and I cannot wait to see what Chicago has in store for us.

My Friends and extended family also must be included, the tightly knit group of friends from high school, to those I met while at Penn State, to all the family members that have guided me in the right direction (including the Netwalls). Without their support in the past there is no doubt I would not have the chance or ability to finish this project.

And finally to all of the friends I've made while in the Architectural Engineering program, from faculty to students. You all have made this year very memorable, and most importantly, made it bearable. Thesis lab was often more fun than it was work, and I can only hope that I meet peers in the professional world that are as easy going, fun, and intelligent as all of you.

Specifically, I would like to thank the follow students who answered countless questions over the last year:

Beth Hostutler
Chris Shelow$_s$
Mike Troxell
Justin Mulhollan
Justin Strauser
Steve Puchek
Ming "Norm" Tsui
Dave Melfi
Jeff Sutterlin
Chris McCune
Tim Moore
Greg Eckel

Bibliography *— Books, Articles, Codes, Documentation and Codes used during the completion of this project*

American Concrete Institute (2002)
Building Code Requirements for Structural Concrete (ACI 318-02), Farmington Hills, MI

American Institute of Steel Construction (AISC) (2001)
Manual of Steel Construction, 3rd Edition LRFD, Chicago, IL

American Society of Civil Engineers (2003)
Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), Reston, VA

International Code Council, Inc (IBC) (2002)
2003 International Building Code, Falls Church, VA

International Code Council, Inc (IBC) (2002)
2003 International Fire Code, Falls Church, VA

R.S. Means Company, Inc. (2005)
2006 RS Means Building Construction Cost Data, Kingston, MA

Appendices

Appendix A – Loads and Checks

Dead Loads (psf):

o Mechanical/Ceiling	10 psf
o Finishes	10 psf
o Carpet/Miscellaneous	5 psf
o 4”slab	150 psf
o 10”slab	375 psf
	Design Total = 175 psf or 400 psf

Live Loads (psf):

o Typical Floors (Hotels refer to residential)	40 psf
o Public rooms and Corridors	100 psf

Load Combinations (Controlling Case):
- o U=1.2D+1.6L .... (Gravity)
- o U=1.2D+1.6W+L..... (Overturning)
- o U=1.2D+1.6L+0.8W.......... (Wind and Gravity)
- o U=1.2D+E+L..................... (Seismic)

<u>*Beam-Column Analysis*</u>

Considering a column on the second level (due to its great height), that is part of both the east-west and north-south lateral system, it is necessary to ensure that a moment due to a load case that produced forces in each direction (Case 3 see ASCE 7-02 excerpt) is considered. In the figure below one can notice that (after the lateral forces are distributed according to relative stiffness) the torsional effects induce a moment in both the strong and weak axis of the column member which must be analyzed.

Following the Aminmansour method

Given:

50ksi Steel and current beam size of W14x211 ($\phi_c P_n = 1810k$)

24.33 feet from floor to floor

Assume K=1.0

Table 6-2 of AISC Manual of Steel Construction LRFD 3rd Edition yields figures to the right.

$b = 0.516\text{x}10^3$

$m = 0.632\text{x}10^3$

$n = 1.22\text{x}10^3$

$$P_u = 882.91k$$

$$M_{ux} = 11.69\ ft - k$$

$$M_{uy} = 4.7\ ft - k$$

$$M_u = B_1 M_{nt} + B_2 M_{lt} \quad (LRFD \quad Equation \quad C1-1) =$$

$$\frac{P_u}{\phi_c P_n} = \frac{882.91}{1810} = 0.488 > 0.2 \therefore LFRD \quad Equation \quad H1-1a$$

$$For\ \frac{P_u}{\phi P_m} \geq 0.2$$

$$\left(\frac{1}{\phi_c P_m}\right)P_u + \left(\frac{8}{9\phi_b M_{nx}}\right)M_{ux} + \left(\frac{8}{9\phi_b M_{ny}}\right)M_{uy} \leq 1.0 \cong bP_u + mM_{ux} + nM_{uy} \leq 1.0$$

$$bP_u + mM_{ux} + nM_{uy} = \left(10^{-3}\right)\left(\left(0.516\right)883 + \left(0.632\right)11.69 + \left(1.22\right)4.7 = 0.5\right) < 1 \therefore OK$$

This column appears to satisfy the interact-action equation therefore the W14X211 is suitable

Composite Slab and Beam Design

Following method as outlined in AISC Manual of Steel Construction 3rd Edition LRFD
Given:
Considering a typical bay as shown below
50ksi Steel
3/4" Shear Studs
3" deep deck
4" 4ksi Concrete Slab with 6.x6-W1.4xW1.4
Table 5-14 of AISC Manual of Steel Construction LRFD 3rd Edition.

$$DL = 1.93k/ft$$

$$LL = 0.44k/ft$$

$$TL = 1.2DL \times 1.6LL = 3.02k/ft$$

$$M_u = \frac{wl^2}{8} = \frac{3.02k/ft(27')^2}{8} = 275.2k-ft$$

$$b_{eff} = lesser\begin{cases} \frac{1}{4} span = 0.25(27) = 81'' \leftarrow \\ spacing = 11' = 132'' \end{cases}$$

$$assume \quad a = 2''$$

$$Y_2 = Y_{conn} - \frac{a}{2} = 7'' - 1'' = 6''$$

$$Y_1 = 0.0''$$

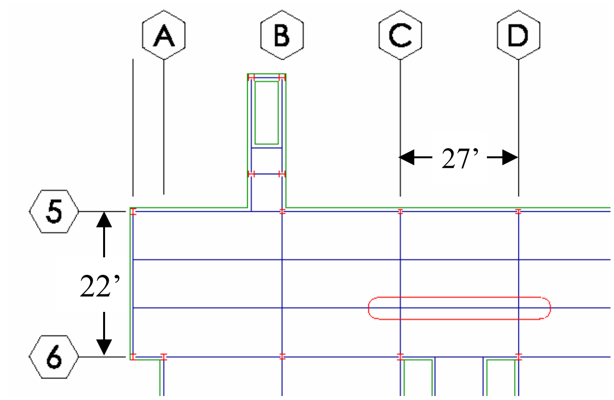
$$AISC \quad Manual \quad Table \quad 5-14:$$

$$possibletrials: W12x22, W14x22$$

$$try \quad W14x22 \quad (A_s = 6.29in^2) \quad \phi M_n = 307k-ft \quad \Sigma Q_n = 325k$$

$$a_{req} = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{\Sigma Q_n}{0.85 f'_c b} = 1.18 < 2 \therefore OK(Conservative)$$

$$Q_n = 26.1 \therefore \frac{\Sigma Q_n}{Q_n} 2 = \# studs = \frac{\Sigma Q_n}{Q_n} 2 = 25$$

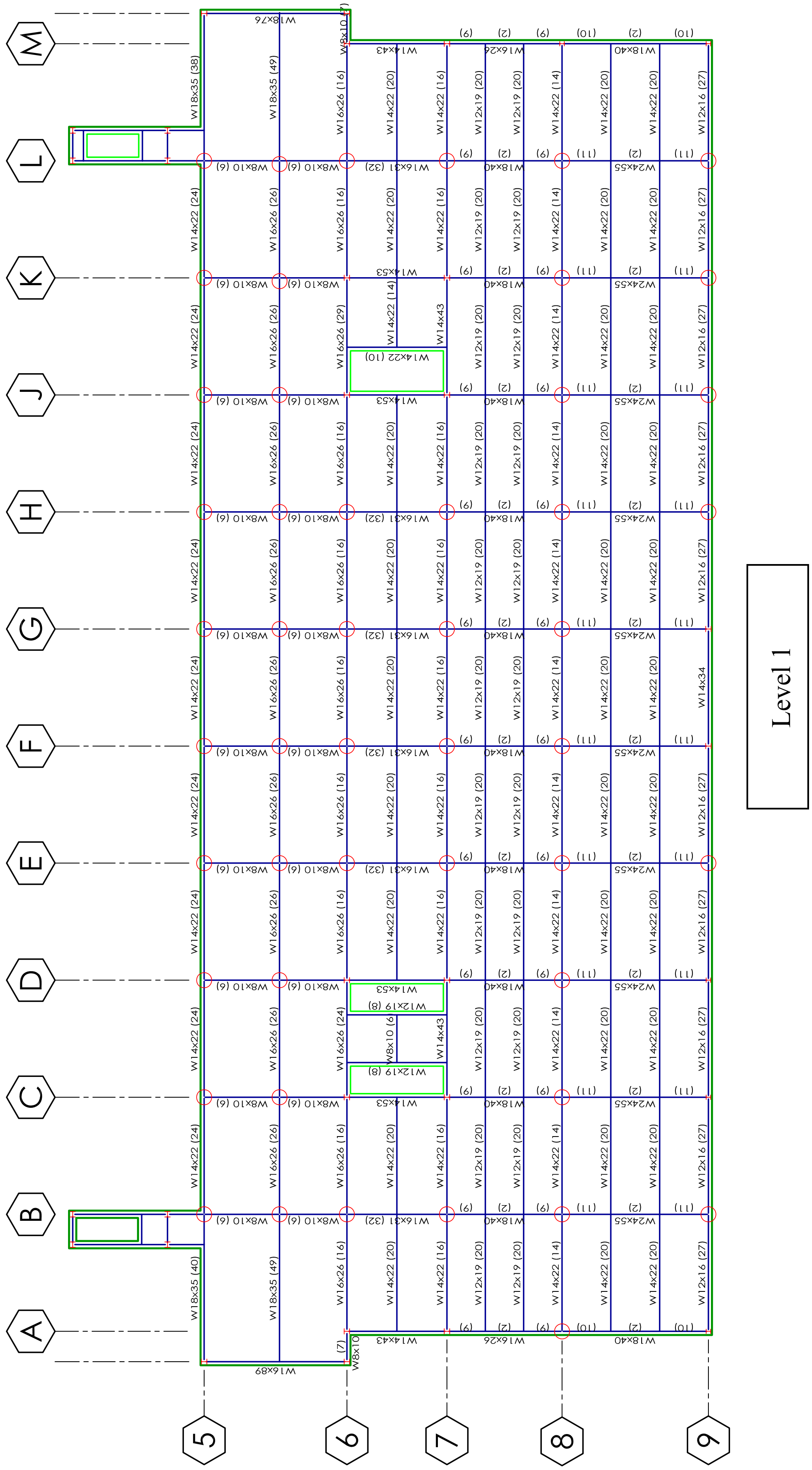


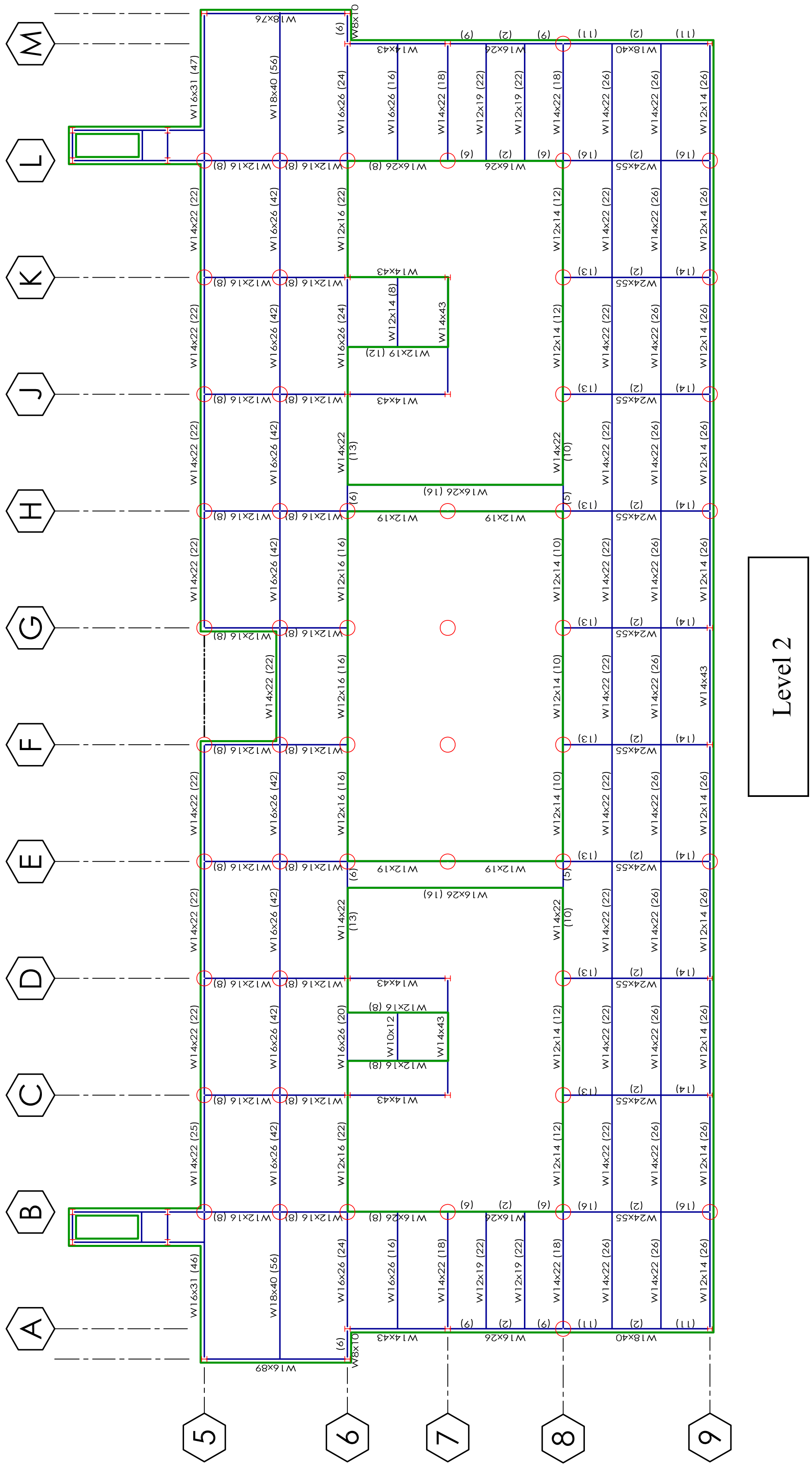
Typical Bay

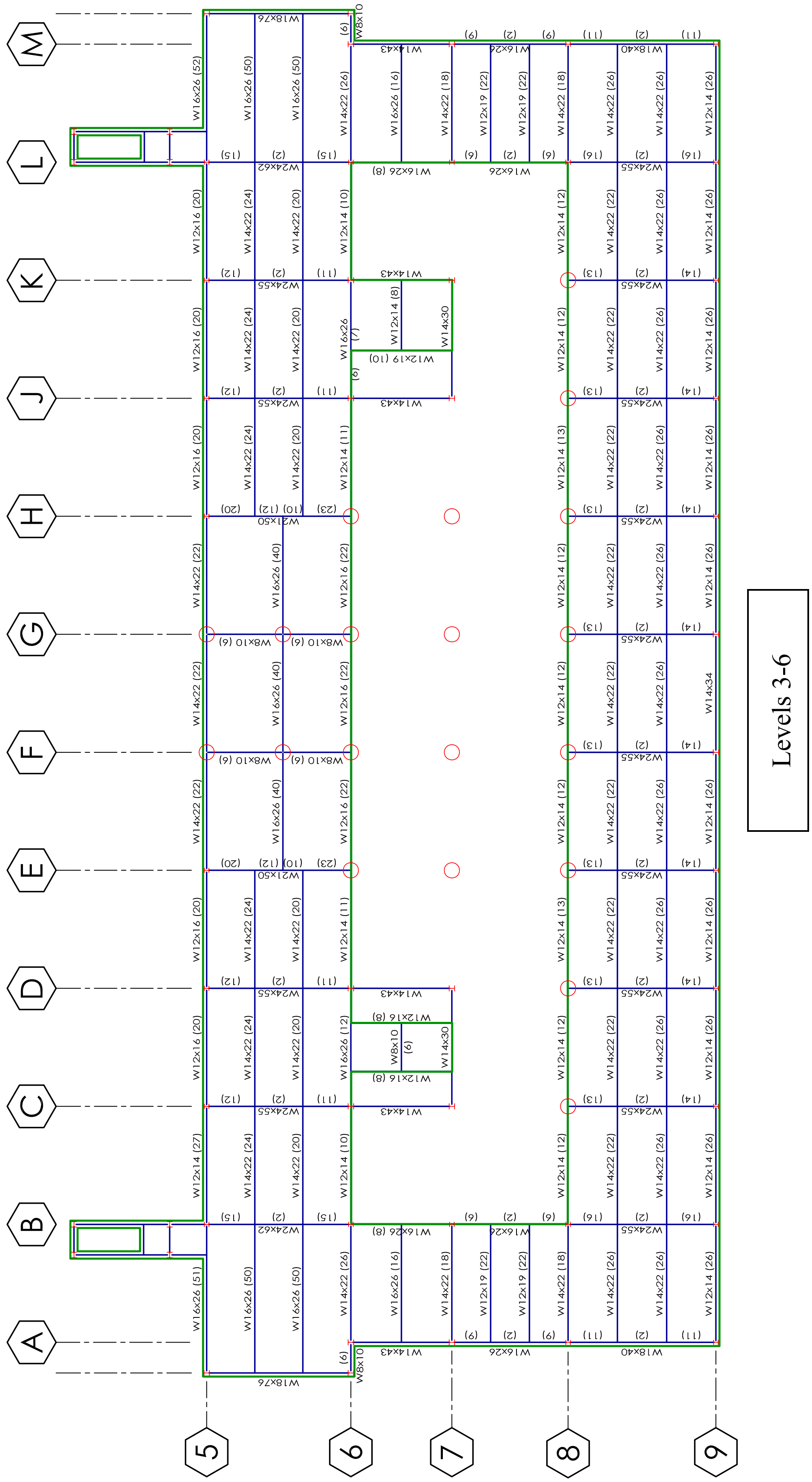
Ram calls for a W14x22 with 26 shear studs which is only 1 stud over my design above which could be due rounding differences

Appendix B – Structural Depth Information and Calculations

Typical floor framing plans for the proposed steel system can be found below.

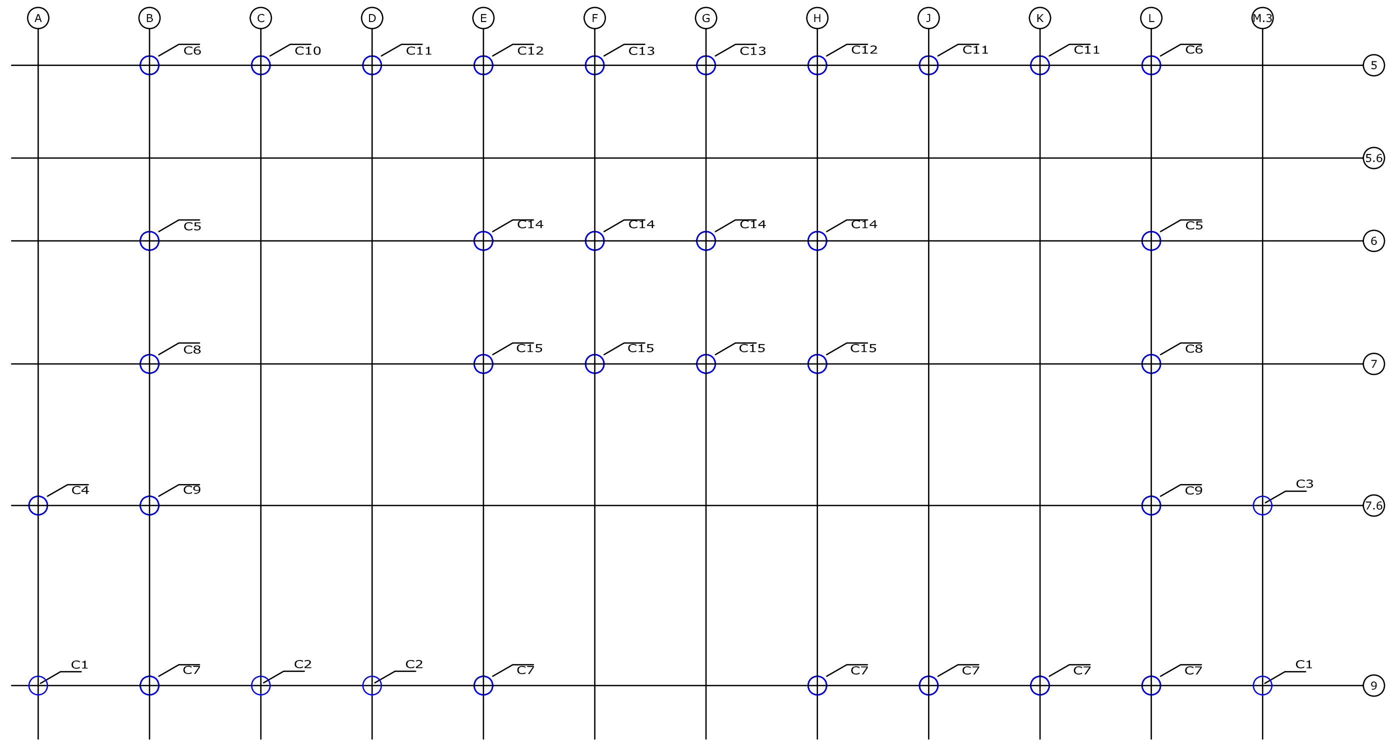



Level 2
A
B
C
D
E
F
G
H
J
K
L
M
5
6
7
8
9


Levels 3-6
M
L
K
J
H
G
F
E
D
C
B
A
5
6
7
8
9


Levels 8 and ↑

Column Schedule:



COLUMN LAYOUT

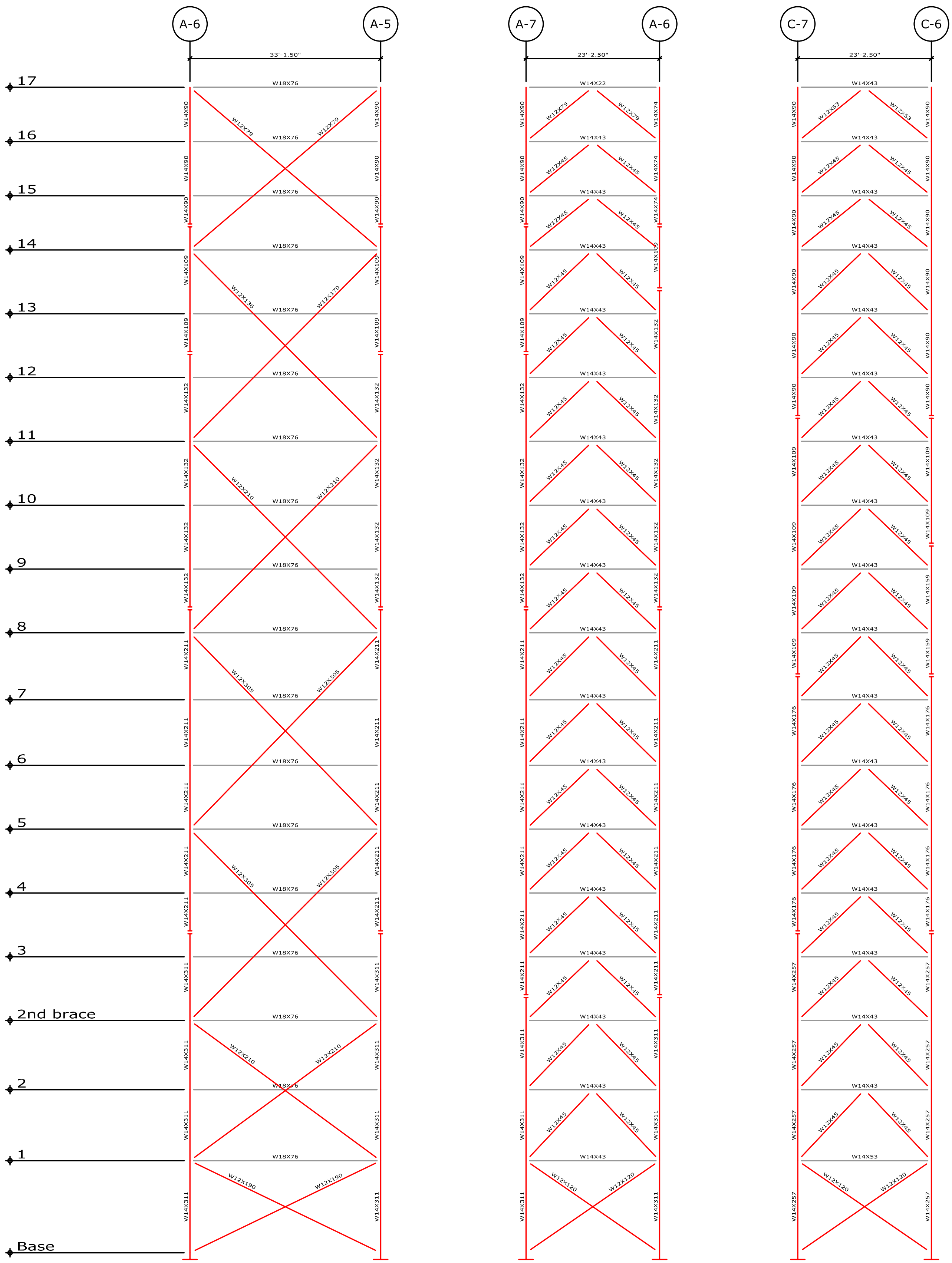
SCALE: NTS

Level	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13	C14	C15
17															
16					W14X43	W14X43		W14X43		W14X43	W14X43	W14X43	W14X43	W14X43	W10X33
15					W14X43	W14X43		W14X43		W14X43	W14X43	W14X43	W14X43	W14X43	W10X33
14					W14X61	W14X61		W14X43		W14X48	W14X48	W14X48	W14X48	W14X61	W10X39
13					W14X61	W14X61		W14X43		W14X48	W14X48	W14X48	W14X48	W14X61	W10X39
12					W14X90	W14X74		W14X53		W14X61	W14X61	W14X61	W14X61	W14X82	W10X49
11					W14X90	W14X74		W14X53		W14X61	W14X61	W14X61	W14X61	W14X82	W10X49
10					W14X109	W14X90		W14X68		W14X82	W14X82	W14X82	W14X82	W14X99	W10X68
9					W14X109	W14X90		W14X68		W14X82	W14X82	W14X82	W14X82	W14X99	W10X68
8					W14X132	W14X109		W14X90		W14X99	W14X90	W14X90	W14X90	W14X109	W10X68
7			W12X40	W12X40	W14X132	W14X109		W14X90	W14X43	W14X99	W14X90	W14X90	W14X90		
6			W12X40	W12X40	W14X145	W14X120		W14X90	W14X43	W14X109	W14X99	W14X109	W14X99		
5	W10X33	W12X40	W12X45	W12X45	W14X176	W14X145	W12X40	W14X109	W14X61	W14X120	W14X120	W14X120			
4	W10X33	W12X40	W12X45	W12X45	W14X176	W14X145	W12X40	W14X109	W14X61	W14X120	W14X120	W14X120			
3	W10X54	W12X72	W12X87	W12X87	W14X233	W14X211	W12X72	W14X159	W14X90	W14X176	W14X176	W14X176			
2nd Intermediate (2nd Inter.)															
2				W12X87	W14X233	W14X211	W12X72	W14X159	W14X90	W14X176	W14X176	W14X176			
1															
Base	W10X54	W12X72	W12X87												
BASEPLATE / COLUMN MARK	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11	C12	C13	C14	C15

MARK ORDER: BY COORDINATES

COLUMN SCHEDULE

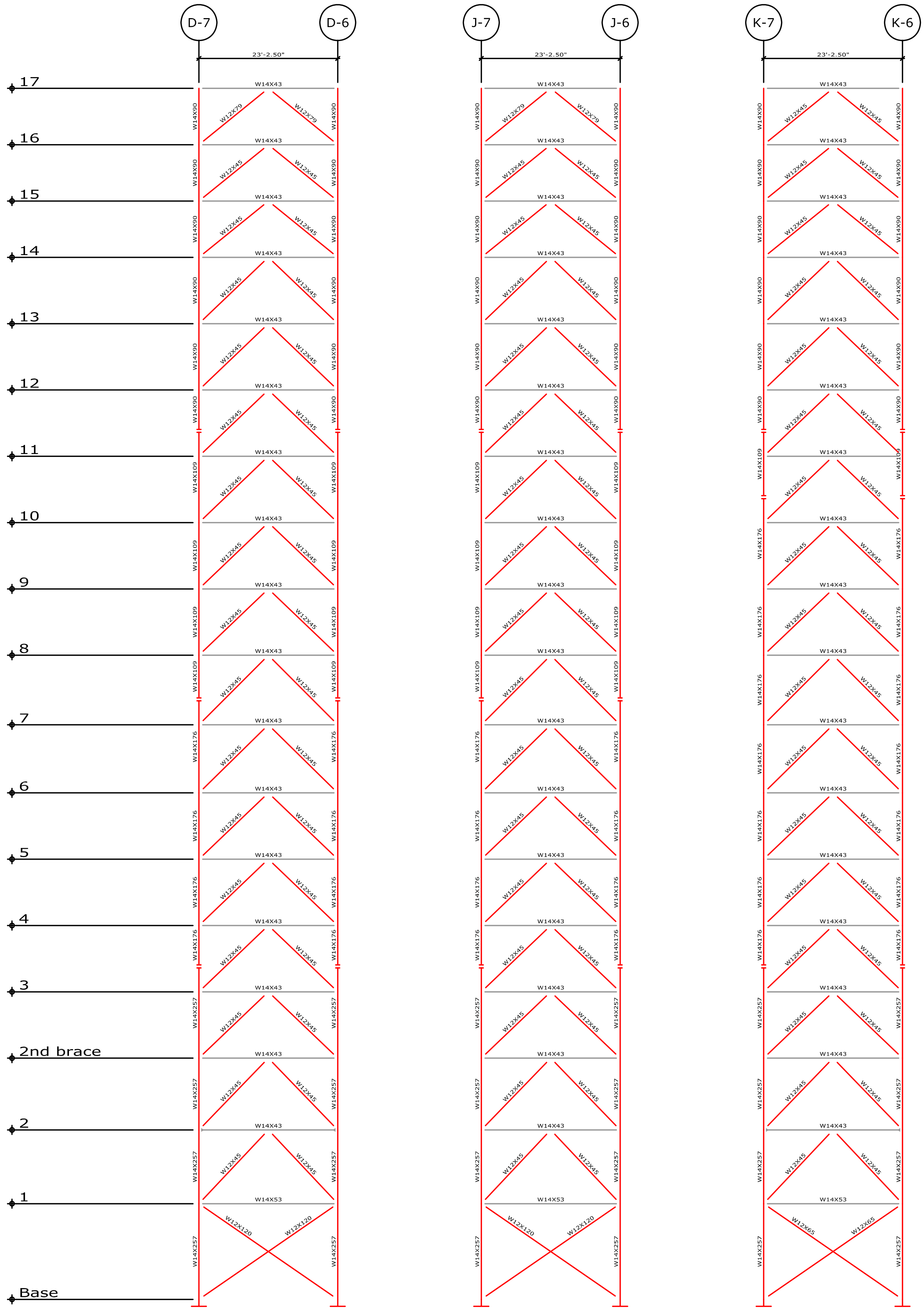
SCALE: NTS



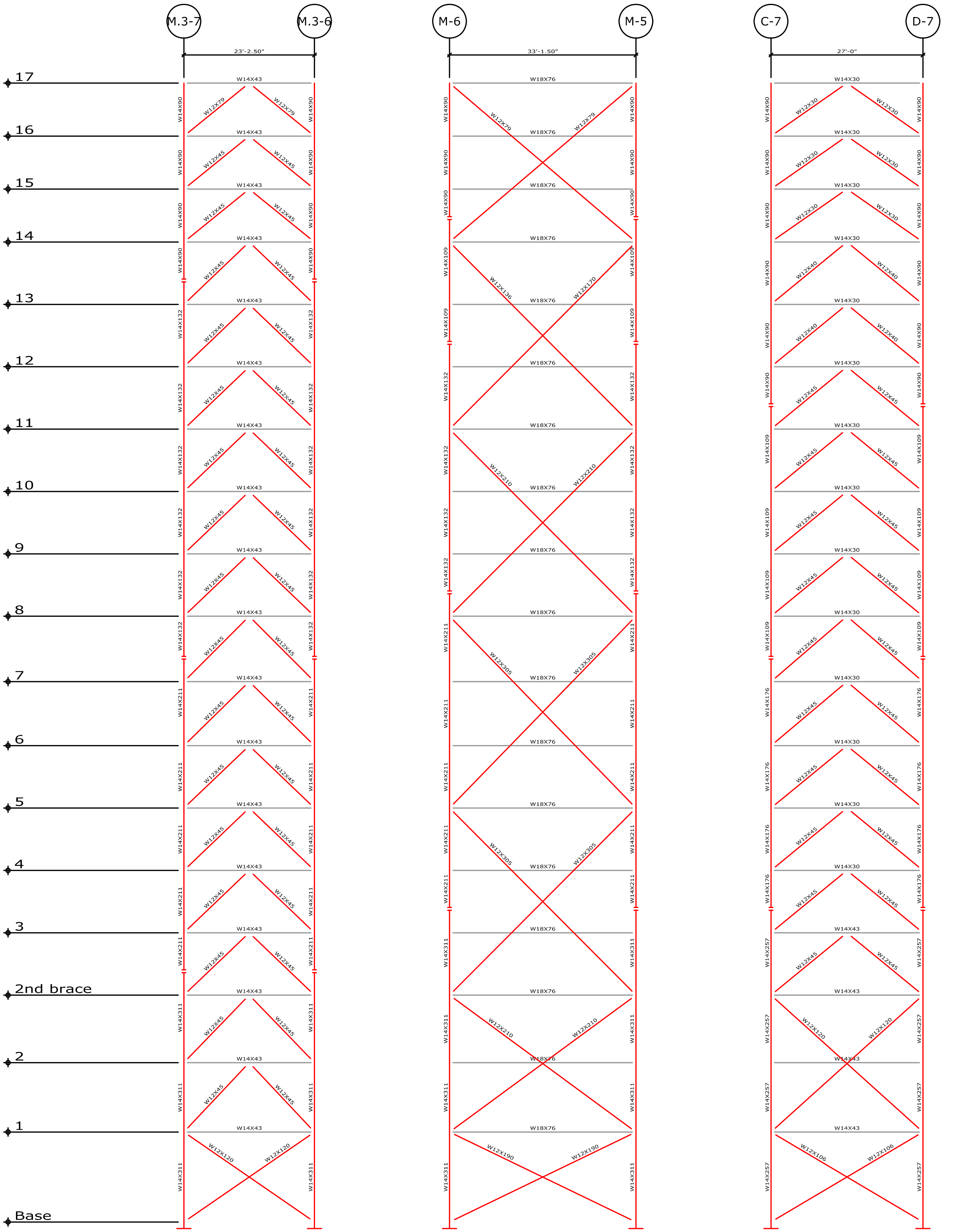
Frame A

Frame B

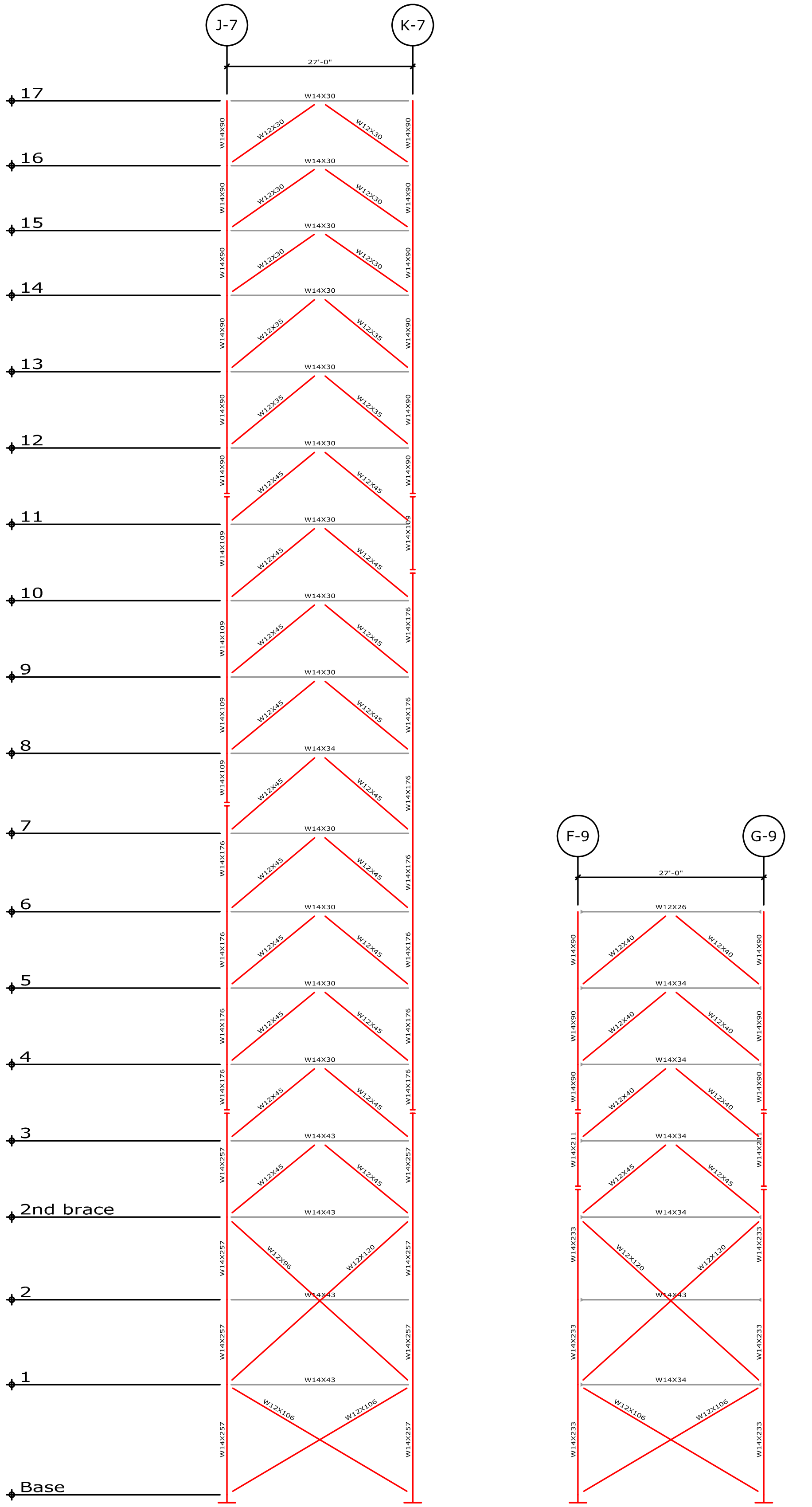
Frame C



Frame D Frame E Frame F



Frame G Frame H Frame I



Frame J

Frame K

Appendix C – *Construction Breadth Information and Calculations*

RS Means (2006) was used in order to obtain the following cost and scheduling data. Some assumptions were made in the justification of prices and time data, these are listed below:

•8 hour days •5 work days a week •Typical RS Means Crew Sizes
•Ram output /structural plans used for all takeoffs

Cost/Time Calculations:

Steel System Cost Calculations

Beams

Type	Amount			RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	#	Length (ft)	Weight (ton)	Section			$	$
Beams	1937	50471.23	657.761	05120-680	13.9	47.32	$ 2,384.00	$ 1,568,102.22
Studs	41265			05090-860	1,040.0	39.68	$ 1.37	$ 56,533.05

Columns

Type	Amount			RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	#	Length (ft)	Weight (ton)	Section			$	$
Columns	189	4092.9	424.1085	05120-680	13.9	30.51	$ 2,384.00	$ 1,011,074.66

Braced Frames

Type	Amount			RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	#	Length (ft)	Weight (ton)	Section			$	$
Framing	324	6421.2	282.55375	05120-680	13.9	98.39	$ 2,384.00	$ 673,608.14

Composite Slab-on-Metal Deck - ***Concrete Cost/Time Includes Materail and Placement**

Type	Amount			RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Floor	# Similar	Area	Section			$	$
4" Slab*	9-17	9	18338.4 sq.ft.	03310-220 and 240	140.0	14.55	$ 102.45	$ 521,880.30
4" Slab*	7-8	2	26315.2 sq.ft.	03310-220 and 240	140.0	4.64	$ 102.45	$ 166,419.27
4" Slab*	3-6	4	36457.5 sq.ft.	03310-220 and 240	140.0	12.86	$ 102.45	$ 461,119.86
4" Slab*	2	1	36014.2 sq.ft.	03310-220 and 240	140.0	3.18	$ 102.45	$ 113,878.23
4" Slab*	1	1	36457.5 sq.ft.	03310-220 and 240	140.0	3.21	$ 102.45	$ 115,279.97
Decking	9-17	9	18338.4 sq.ft.	05310-300	1,350.0	12.23	$ 2.88	$ 475,331.33
Decking	7-8	2	26315.2 sq.ft.	05310-300	1,350.0	3.90	$ 2.88	$ 151,575.55
Decking	3-6	4	36457.5 sq.ft.	05310-300	1,359.0	10.73	$ 2.88	$ 419,990.40
Decking	2	1	36014.2 sq.ft.	05310-300	1,350.0	2.67	$ 2.88	$ 103,720.90
Decking	1	1	36457.5 sq.ft.	05310-300	1,359.0	2.68	$ 2.88	$ 104,997.60

Slab Reinforcing

Type	Height	Amount	Area	RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Floor	#	C.S.F.	Section			$	$
6x6-W1.4xW1.4	9-17	9	183.384	03220-200	31.0	53.24	$ 36.70	$ 60,571.74
6x6-W1.4xW1.4	7-8	2	263.152	03220-200	31.0	16.98	$ 36.70	$ 19,315.36
6x6-W1.4xW1.4	3-6	4	364.575	03220-200	31.0	47.04	$ 36.70	$ 53,519.61
6x6-W1.4xW1.4	2	1	360.142	03220-200	31.0	11.62	$ 36.70	$ 13,217.21
6x6-W1.4xW1.4	1	1	364.575	03220-200	31.0	11.76	$ 36.70	$ 13,379.90

Fire Proofing

Type	Height	Area (ft^2)	RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Floor		Section			$	$
Decking	ALL	435,977.70	07812-600	1,250.0	38.75	$ 1.45	$ 632,167.67
Beams	ALL	34,704.42	07812-600	1,500.0	2.57	$ 1.13	$ 39,215.99
Columns	ALL	36,076.43	07812-600	1,100.0	3.64	$ 2.09	$ 75,399.74
Bracing	ALL	137,088.20	07812-600	1,100.0	13.85	$ 2.09	$ 286,514.34

Summary	Total Cost
	$ 7,136,813.04

Concrete System Cost Calculations

Beams - *includes reinforcing/concrete/placement						
Type	Amount	RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Volume (CY)	Section			$	$
6ksi	1618.49	03310-240	15.6	103.62	$ 935.50	$ 1,514,092.86
5ksi	1409.62	03310-240	15.6	90.24	$ 872.50	$ 1,229,891.15

Columns - *includes reinforcing/concrete/placement						
Type	Amount	RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Volume (CY)	Section			$	$
8ksi	2266.35	03310-240	27.1	83.75	$ 1,199.00	$ 2,717,348.51
6ksi	456.42	03310-240	12.6	36.31	$ 1,086.00	$ 495,671.85

Post Tensioned Floor System - *Concrete Cost/Time Includes Materail and Placement								
Type	Amount			RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Floor	# Similar	Area	Section			$	$
10" Slab*	9-17	9	18338.4 sq.ft.	03310-220 and 240	140.0	14.55	$ 100.45	$ 511,692.30
10" Slab*	7-8	2	26315.2 sq.ft.	03310-220 and 240	140.0	4.64	$ 100.45	$ 163,170.48
10" Slab*	3-6	4	36457.5 sq.ft.	03310-220 and 240	140.0	12.86	$ 100.45	$ 452,118.01
10" Slab*	2	1	36014.2 sq.ft.	03310-220 and 240	140.0	3.18	$ 100.45	$ 111,655.14
10" Slab*	1	1	36457.5 sq.ft.	03310-220 and 240	140.0	3.21	$ 100.45	$ 113,029.50
	Floor	# Similar	Weight	Section			$	$
Tendons	9-17	9	7.34 ton	03230-600	1,650.0	8.89	$ 2.50	$ 330,091.20
Tendons	7-8	2	10.53 ton	03230-600	1,650.0	12.76	$ 2.50	$ 105,260.80
Tendons	3-6	4	14.58 ton	03230-600	1,650.0	17.68	$ 2.50	$ 291,660.00
Tendons	2	1	14.41 ton	03230-600	1,650.0	17.46	$ 2.50	$ 72,028.40
Tendons	1	1	14.58 ton	03230-600	1,650.0	17.68	$ 2.50	$ 72,915.00

Shear Walls								
Type	Amount			RS Means	Daily Output	Work Days	Unit Cost	Total Cost
	Height	$Length_b$	Number	Section			$	$
Largest	24 '	119.73 '	2	03310-220 and 700	1,375.0	4.18	$ 102.45	$ 24,532.68
Average	10.3 '	119.73 '	13	03310-220 and 700	1,375.0	11.66	$ 102.45	$ 159,462.40
Smallest	8 '	119.73 '	3	03310-220 and 700	1,375.0	2.09	$ 1.99	$ 714.79
Wall Reinforcement	Ton			Section			$	$
#3-7	32.9			03210-600	3.0	10.97	$ 1,165.00	$ 38,328.50
#8-18	77.95			03210-600	4.0	19.49	$ 1,065.00	$ 83,016.75

Summary	**Total Cost**
	$ 8,486,680.32

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.

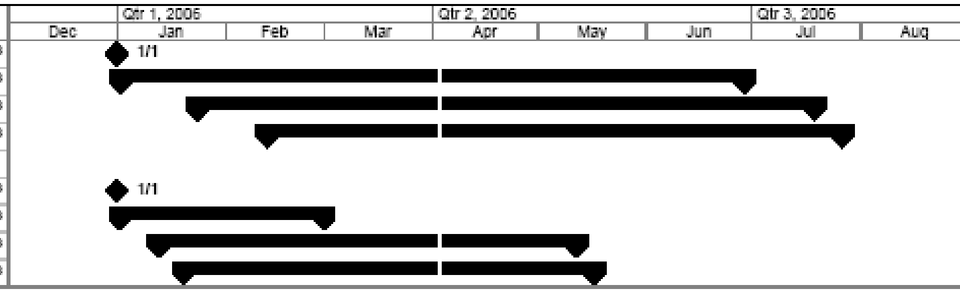
Total square footage
435977.7

Concrete System			
ID	Objective	Duration	Predessor
1	**Preperation**	**0 days**	
2	Procure Steel	0 days	
3	Procure Concrete	0 days	
4	Misc. Preperation (site work)	0 days	
5	**Concrete (CIP)**	**128 days**	
6	Columns Floors 1-3	8 days	
7	Beams Floors 1-3	8 days	6
8	Columns Floors 4-5	8 days	7
9	Beams Floors 4-5	8 days	8
10	Columns Floors 6-7	8 days	9
11	Beams Floors 6-7	8 days	10
12	Columns Floors 8-9	8 days	11
13	Beams Floors 8-9	8 days	12
14	Columns Floors 10-11	8 days	13
15	Beams Floors 10-11	8 days	14
16	Columns Floors 12-14	8 days	15
17	Beams Floors 12-14	8 days	16
18	Columns Floors 15-16	8 days	17
19	Beams Floors 15-16	8 days	18
20	Columns Floors M1-M2	8 days	19
21	Beams Floors M1-M2	8 days	20
22	**Post-Tensioned Floor**	**126 days**	
23	Floor 1	14 days	7
24	Floor 2	14 days	7
25	Floor 3	14 days	7
26	Floor 4	14 days	9
27	Floor 5	14 days	9
28	Floor 6	14 days	11
29	Floor 7	14 days	11
30	Floor 8	14 days	13
31	Floor 9	14 days	13
32	Floor 10	14 days	15
33	Floor 11	14 days	15
34	Floor 12	14 days	17
35	Floor 14	14 days	17
36	Floor 15	14 days	19
37	Floor 16	14 days	19
38	Mech 1	14 days	21
39	Mech 2	14 days	21
40	**Shear Walls**	**118 days**	
41	Floor 1	3 days	23
42	Floor 2	3 days	24,41
43	Floor 3	3 days	25,42
44	Floor 4	3 days	26,43
45	Floor 5	3 days	27,44
46	Floor 6	3 days	28,45
47	Floor 7	3 days	29,46
48	Floor 8	3 days	30,47
49	Floor 9	3 days	31,48
50	Floor 10	3 days	32,49
51	Floor 11	3 days	33,50
52	Floor 12	3 days	34,51
53	Floor 14	3 days	35,52
54	Floor 15	3 days	36,53
55	Floor 16	3 days	37,54
56	Mech 1	3 days	38,55
57	Mech 2	3 days	39,56
	Total Duration	**148 days**	

Steel System			
ID	Objective	Duration	Predessor
1	**Preperation**	**0 days**	
2	Concrete/Steel Procument	0 days	
3	Site Prep / Site Work	0 days	
4	**Structural Steel**	**3 days**	
5	Columns Floors 1-3	3 days	
6	Beams Floors 1-3	3 days	5
7	Columns Floors 4-5	3 days	6
8	Beams Floors 4-5	3 days	7
9	Columns Floors 6-7	2 days	8
10	Beams Floors 6-7	2 days	9
11	Columns Floors 8-9	2 days	10
12	Beams Floors 8-9	2 days	11
13	Columns Floors 10-11	2 days	12
14	Beams Floors 10-11	2 days	13
15	Columns Floors 12-14	2 days	14
16	Beams Floors 12-14	2 days	15
17	Columns Floors 15-16	2 days	16
18	Beams Floors 15-16	2 days	17
19	Columns Floors M1-M2	3 days	18
20	Beams Floors M1-M2	3 days	19
21	Metal Deck Floor 1	2 days	6
22	Metal Deck Floor 2	2 days	6,21
23	Metal Deck Floor 3	2 days	6,22
24	Metal Deck Floor 4	2 days	8,23
25	Metal Deck Floor 5	2 days	8,24
26	Metal Deck Floor 6	2 days	10,25
27	Metal Deck Floor 7	2 days	10,26
28	Metal Deck Floor 8	2 days	12,27
29	Metal Deck Floor 9	2 days	12,28
30	Metal Deck Floor 10	2 days	14,29
31	Metal Deck Floor 11	2 days	14,30
32	Metal Deck Floor 12	2 days	16,31
33	Metal Deck Floor 14	2 days	16,32
34	Metal Deck Floor 15	2 days	18,33
35	Metal Deck Floor 16	2 days	18,34
36	Metal Deck M1	2 days	20,35
37	Metal Deck M2	2 days	20,36
38	**Concrete (CIP)**	**85 days**	
39	Floor 1	5 days	7,21
40	Floor 2	5 days	7,22,39
41	Floor 3	5 days	7,23,40
42	Floor 4	5 days	9,24,41
43	Floor 5	5 days	9,25,42
44	Floor 6	5 days	11,26,43
45	Floor 7	5 days	11,27,44
46	Floor 8	5 days	13,28,45
47	Floor 9	5 days	13,29,46
48	Floor 10	5 days	15,30,47
49	Floor 11	5 days	15,31,48
50	Floor 12	5 days	17,32,49
51	Floor 14	5 days	17,33,50
52	Floor 15	5 days	19,34,51
53	Floor 16	5 days	19,35,52
54	Mech 1	5 days	20,36,53
55	Mech 2	5 days	20,37,54
56	**Fire Proofing**	**83 days**	
57	Floor 1	3 days	39
59	Floor 3	3 days	41,58
60	Floor 4	3 days	42,59
61	Floor 5	3 days	43,60
62	Floor 6	3 days	44,61
63	Floor 7	3 days	45,62
64	Floor 8	3 days	46,63
65	Floor 9	3 days	47,64
66	Floor 10	3 days	48,65
67	Floor 11	3 days	49,66
68	Floor 12	3 days	50,67
69	Floor 14	3 days	51,68
70	Floor 15	3 days	52,69
71	Floor 16	3 days	53,70
72	Mech 1	3 days	54,71
73	Mech 2	3 days	55,72
	Total Duration	**97 Days**	

Project Schedule Comparison	Days
Steel Duration	97
Concrete Duration	148

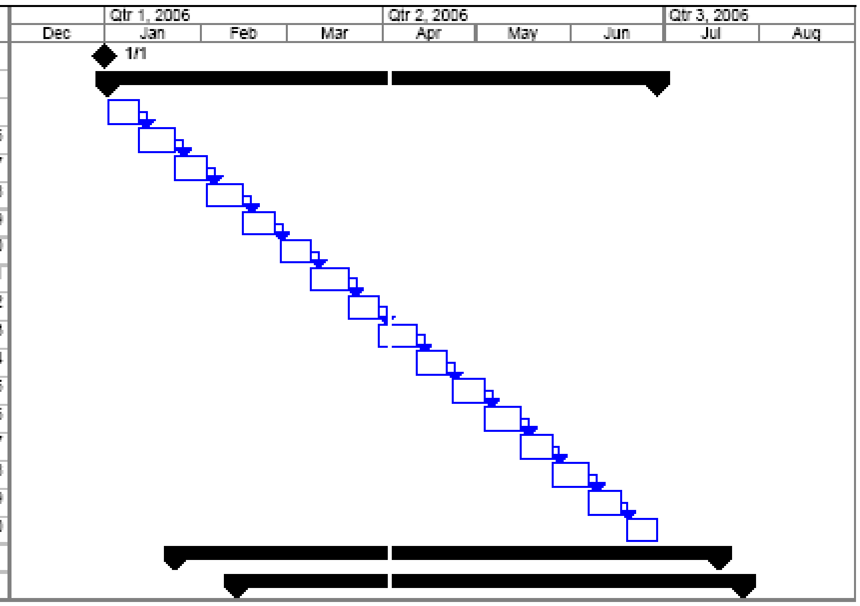
ID	Task Name	Duration
1	Preperation	0 days
5	Concrete (CIP)	128 days
22	Post-Tensioned Floor	126 days
40	Shear Walls	118 days
58		
59	Preperation	0 days
62	Structural Steel	42 days
96	Concrete (CIP)	85 days
114	Fire Proofing	83 days



Concrete System Schedule

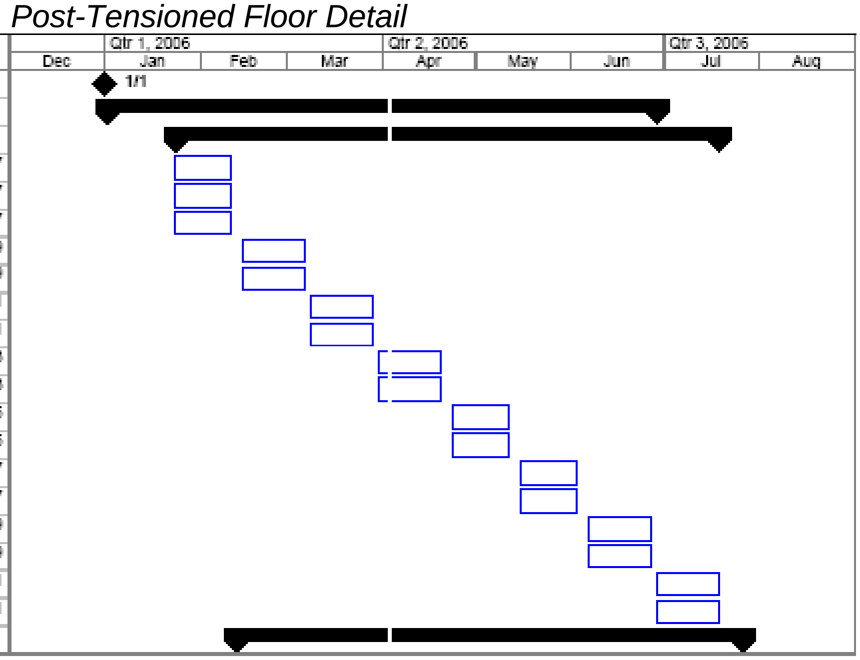
Cast-in-Place Detail

ID	Task Name	Duration	Predecessors
1	Preperation	0 days	
5	Concrete (CIP)	128 days	
6	Columns Floors 1-3	8 days	
7	Beams Floors 1-3	8 days	6
8	Columns Floors 4-5	8 days	7
9	Beams Floors 4-5	8 days	8
10	Columns Floors 6-7	8 days	9
11	Beams Floors 6-7	8 days	10
12	Columns Floors 8-9	8 days	11
13	Beams Floors 8-9	8 days	12
14	Columns Floors 10-11	8 days	13
15	Beams Floors 10-11	8 days	14
16	Columns Floors 12-14	8 days	15
17	Beams Floors 12-14	8 days	16
18	Columns Floors 15-16	8 days	17
19	Beams Floors 15-16	8 days	18
20	Columns Floors M1-M2	8 days	19
21	Beams Floors M1-M2	8 days	20
22	Post-Tensioned Floor	126 days	
40	Shear Walls	118 days	



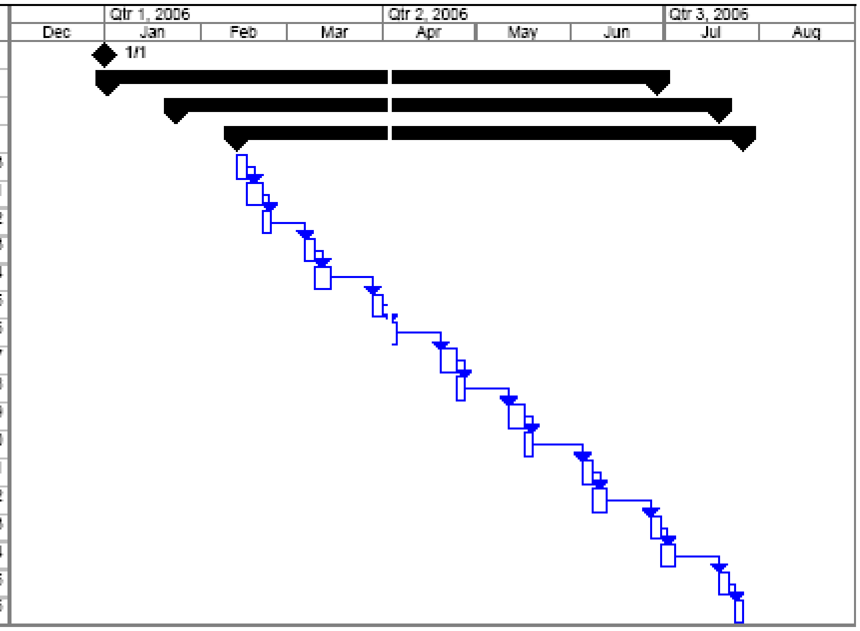
Post-Tensioned Floor Detail

ID	Task Name	Duration	Predecessors
1	Preperation	0 days	
5	Concrete (CIP)	128 days	
22	Post-Tensioned Floor	126 days	
23	Floor 1	14 days	7
24	Floor 2	14 days	7
25	Floor 3	14 days	7
26	Floor 4	14 days	9
27	Floor 5	14 days	9
28	Floor 6	14 days	11
29	Floor 7	14 days	11
30	Floor 8	14 days	13
31	Floor 9	14 days	13
32	Floor 10	14 days	15
33	Floor 11	14 days	15
34	Floor 12	14 days	17
35	Floor 14	14 days	17
36	Floor 15	14 days	19
37	Floor 16	14 days	19
38	Mech 1	14 days	21
39	Mech 2	14 days	21
40	Shear Walls	118 days	



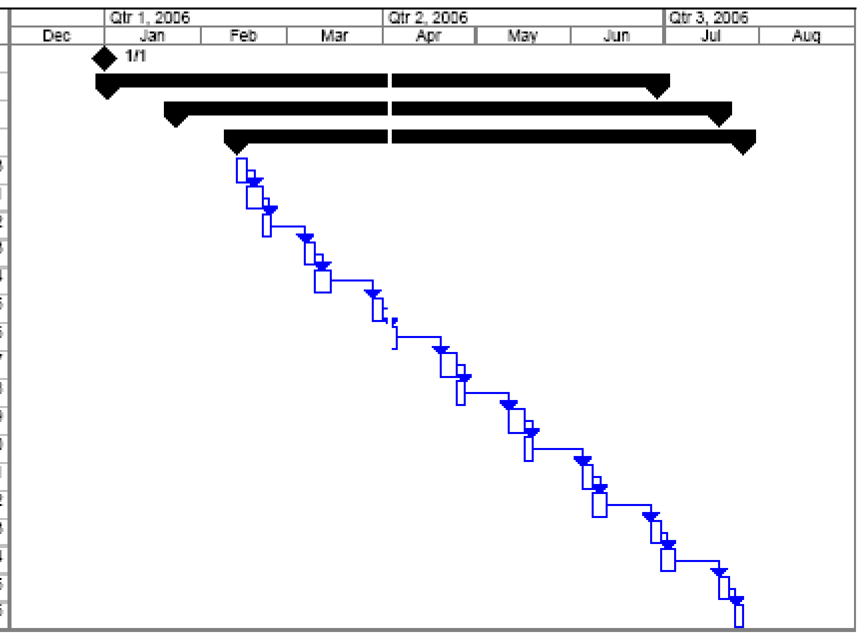
Shear Wall Detail

ID	Task Name	Duration	Predecessors
1	**Preparation**	**0 days**	
5	**Concrete (CIP)**	**128 days**	
22	**Post-Tensioned Floor**	**126 days**	
40	**Shear Walls**	**118 days**	
41	Floor 1	3 days	23
42	Floor 2	3 days	24,41
43	Floor 3	3 days	25,42
44	Floor 4	3 days	26,43
45	Floor 5	3 days	27,44
46	Floor 6	3 days	28,45
47	Floor 7	3 days	29,46
48	Floor 8	3 days	30,47
49	Floor 9	3 days	31,48
50	Floor 10	3 days	32,49
51	Floor 11	3 days	33,50
52	Floor 12	3 days	34,51
53	Floor 14	3 days	35,52
54	Floor 15	3 days	36,53
55	Floor 16	3 days	37,54
56	Mech 1	3 days	38,55
57	Mech 2	3 days	39,56



Post-Tensioned Floor Detail

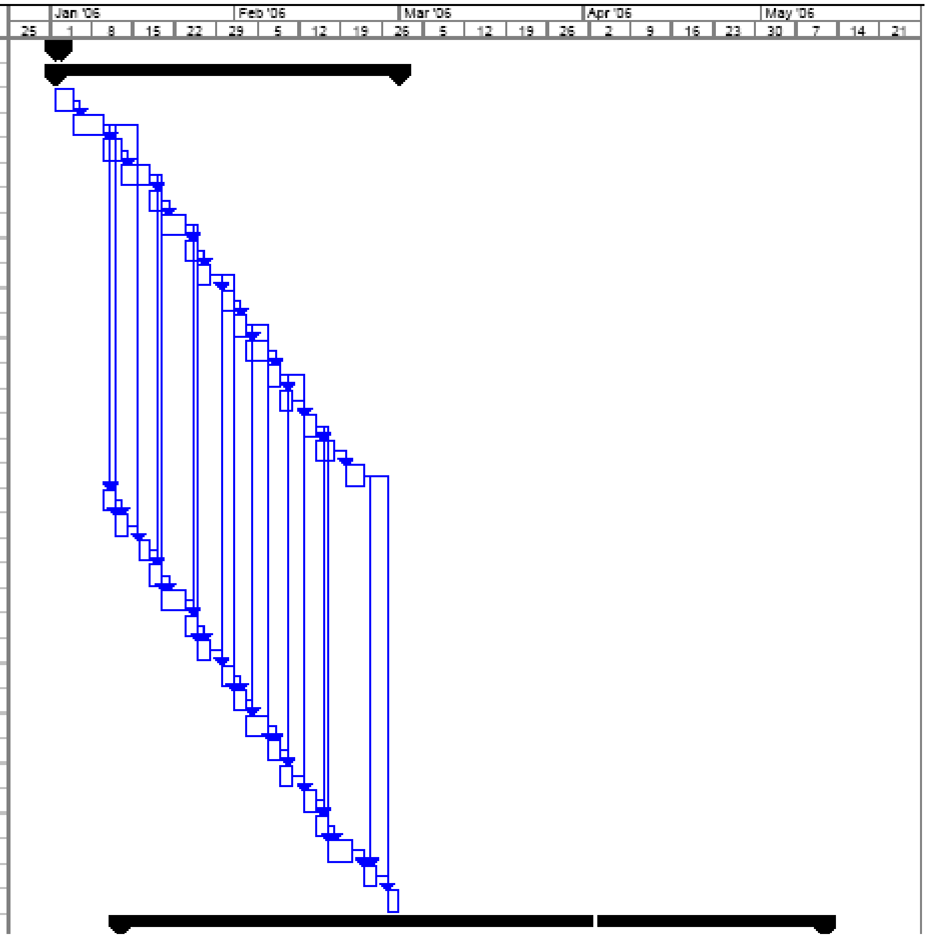
ID	Task Name	Duration	Predecessors
1	**Preparation**	**0 days**	
5	**Concrete (CIP)**	**128 days**	
22	**Post-Tensioned Floor**	**126 days**	
40	**Shear Walls**	**118 days**	
41	Floor 1	3 days	23
42	Floor 2	3 days	24,41
43	Floor 3	3 days	25,42
44	Floor 4	3 days	26,43
45	Floor 5	3 days	27,44
46	Floor 6	3 days	28,45
47	Floor 7	3 days	29,46
48	Floor 8	3 days	30,47
49	Floor 9	3 days	31,48
50	Floor 10	3 days	32,49
51	Floor 11	3 days	33,50
52	Floor 12	3 days	34,51
53	Floor 14	3 days	35,52
54	Floor 15	3 days	36,53
55	Floor 16	3 days	37,54
56	Mech 1	3 days	38,55
57	Mech 2	3 days	39,56



Steel System Schedule

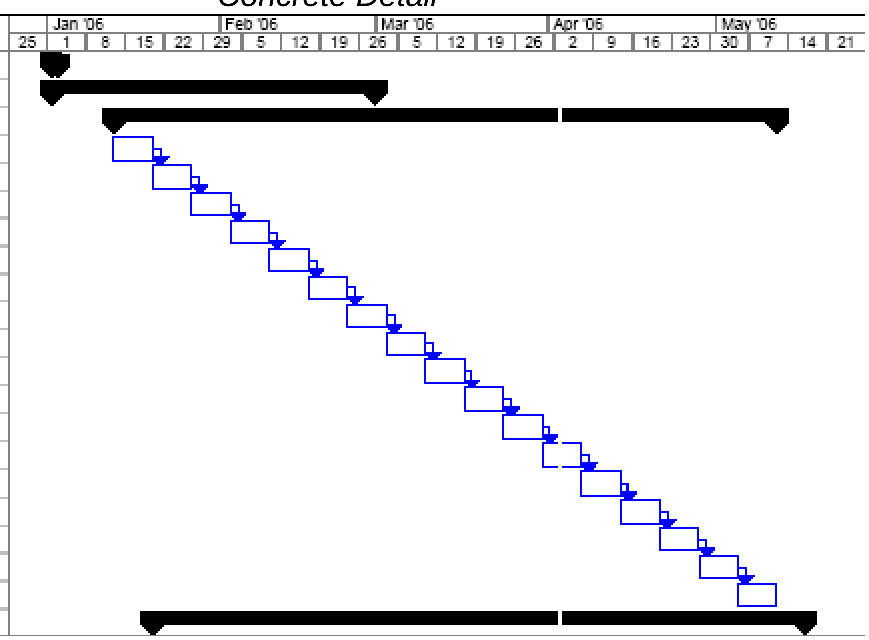
Steel Detail

ID	Task Name	Duration	Predecessors
1	**Preperation**	**1 day?**	
4	**Structural Steel**	**42 days**	
5	Columns Floors 1-3	3 days	
6	Beams Floors 1-3	3 days	5
7	Columns Floors 4-5	3 days	6
8	Beams Floors 4-5	3 days	7
9	Columns Floors 6-7	2 days	8
10	Beams Floors 6-7	2 days	9
11	Columns Floors 8-9	2 days	10
12	Beams Floors 8-9	2 days	11
13	Columns Floors 10-11	2 days	12
14	Beams Floors 10-11	2 days	13
15	Columns Floors 12-14	2 days	14
16	Beams Floors 12-14	2 days	15
17	Columns Floors 15-16	2 days	16
18	Beams Floors 15-16	2 days	17
19	Columns Floors M1-M2	3 days	18
20	Beams Floors M1-M2	3 days	19
21	Metal Deck Floor 1	2 days	6
22	Metal Deck Floor 2	2 days	6,21
23	Metal Deck Floor 3	2 days	6,22
24	Metal Deck Floor 4	2 days	8,23
25	Metal Deck Floor 5	2 days	8,24
26	Metal Deck Floor 6	2 days	10,25
27	Metal Deck Floor 7	2 days	10,26
28	Metal Deck Floor 8	2 days	12,27
29	Metal Deck Floor 9	2 days	12,28
30	Metal Deck Floor 10	2 days	14,29
31	Metal Deck Floor 11	2 days	14,30
32	Metal Deck Floor 12	2 days	16,31
33	Metal Deck Floor 14	2 days	16,32
34	Metal Deck Floor 15	2 days	18,33
35	Metal Deck Floor 16	2 days	18,34
36	Metal Deck M1	2 days	20,35
37	Metal Deck M2	2 days	20,36
38	**Concrete (CIP)**	**86 days**	



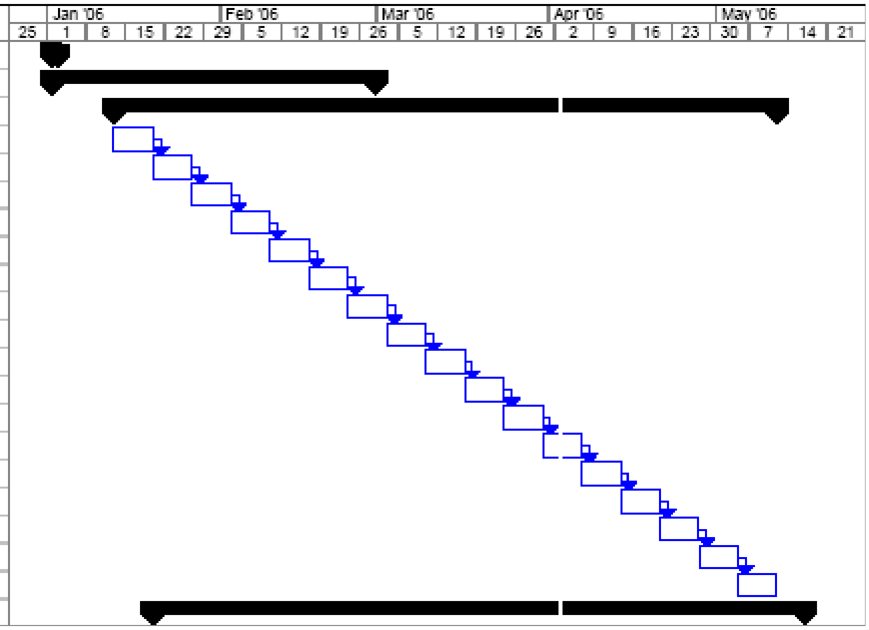
Concrete Detail

ID	Task Name	Duration	Predecessors
1	**Preperation**	**1 day?**	
4	**Structural Steel**	**42 days**	
38	**Concrete (CIP)**	**85 days**	
39	Floor 1	5 days	7,21
40	Floor 2	5 days	7,22,39
41	Floor 3	5 days	7,23,40
42	Floor 4	5 days	9,24,41
43	Floor 5	5 days	9,25,42
44	Floor 6	5 days	11,26,43
45	Floor 7	5 days	11,27,44
46	Floor 8	5 days	13,28,45
47	Floor 9	5 days	13,29,46
48	Floor 10	5 days	15,30,47
49	Floor 11	5 days	15,31,48
50	Floor 12	5 days	17,32,49
51	Floor 14	5 days	17,33,50
52	Floor 15	5 days	19,34,51
53	Floor 16	5 days	19,35,52
54	Mech 1	5 days	20,36,53
55	Mech 2	5 days	20,37,54
56	**Fire Proofing**	**83 days**	



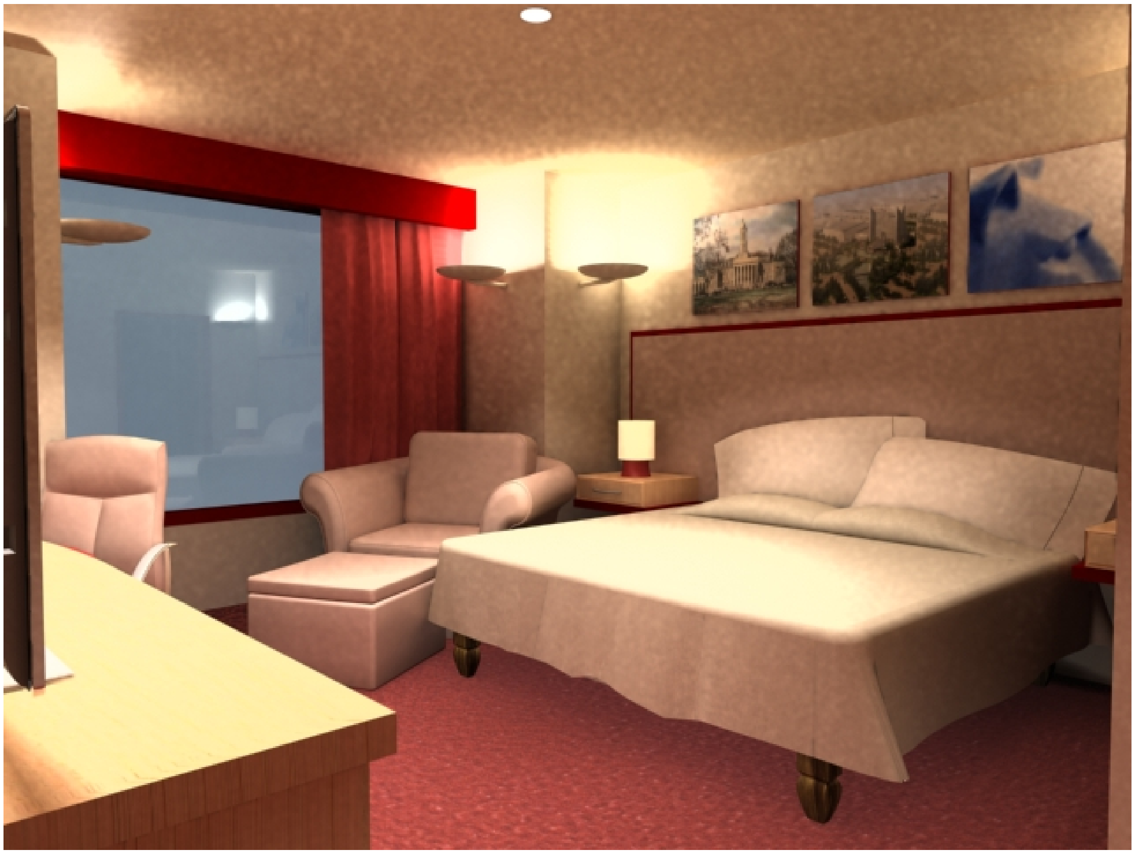
Cast in Place Concrete Detail

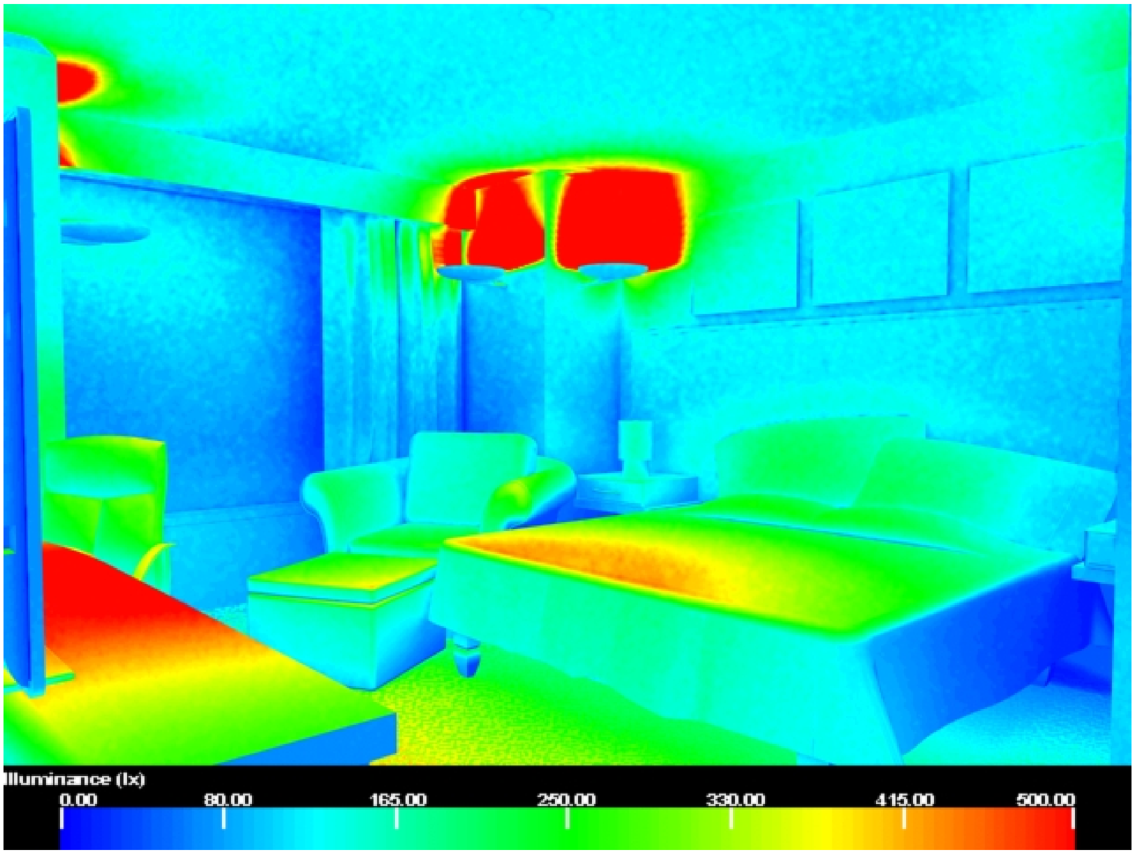
ID	Task Name	Duration	Predecessors
1	**Preperation**	**1 day?**	
4	**Structural Steel**	**42 days**	
38	**Concrete (CIP)**	**85 days**	
39	Floor 1	5 days	7,21
40	Floor 2	5 days	7,22,39
41	Floor 3	5 days	7,23,40
42	Floor 4	5 days	9,24,41
43	Floor 5	5 days	9,25,42
44	Floor 6	5 days	11,26,43
45	Floor 7	5 days	11,27,44
46	Floor 8	5 days	13,28,45
47	Floor 9	5 days	13,29,46
48	Floor 10	5 days	15,30,47
49	Floor 11	5 days	15,31,48
50	Floor 12	5 days	17,32,49
51	Floor 14	5 days	17,33,50
52	Floor 15	5 days	19,34,51
53	Floor 16	5 days	19,35,52
54	Mech 1	5 days	20,36,53
55	Mech 2	5 days	20,37,54
56	**Fire Proofing**	**83 days**	

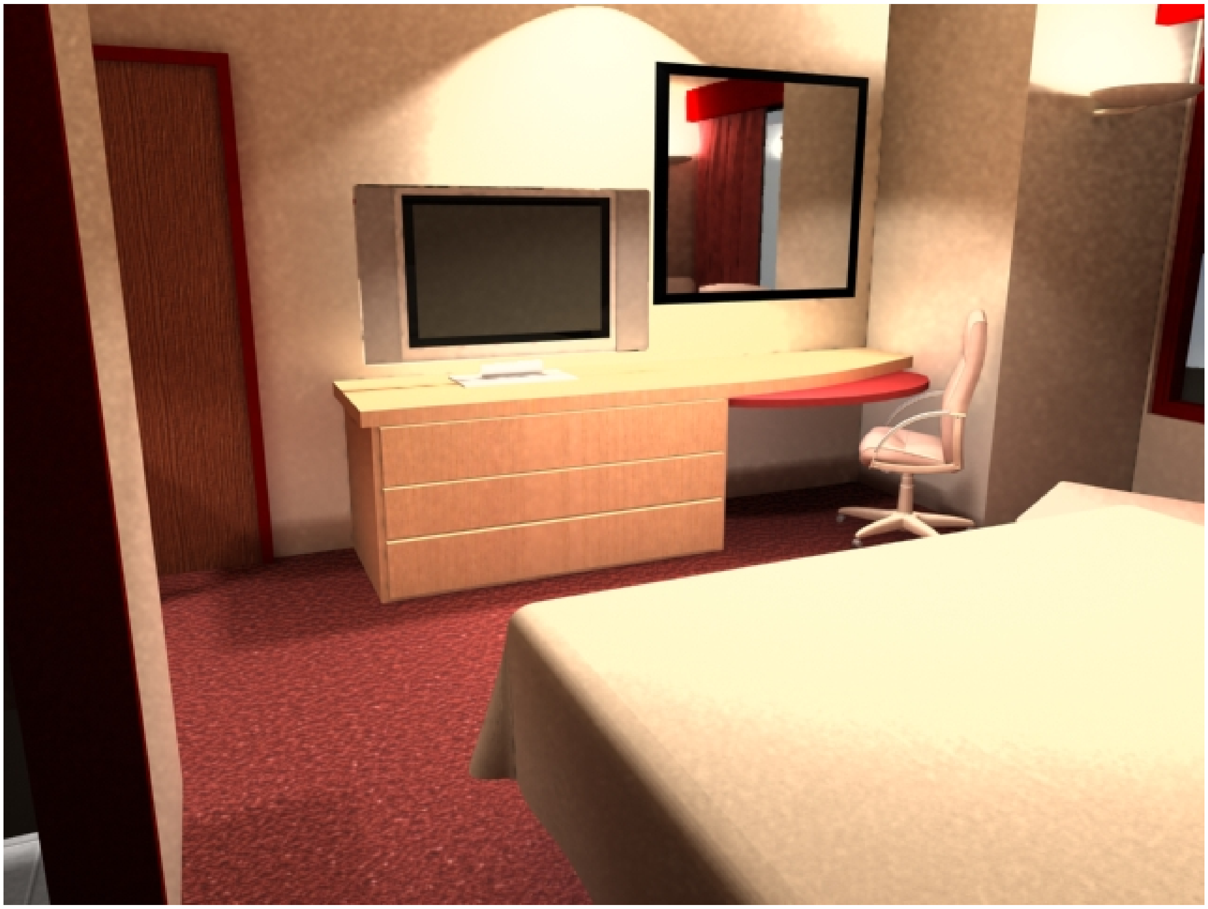


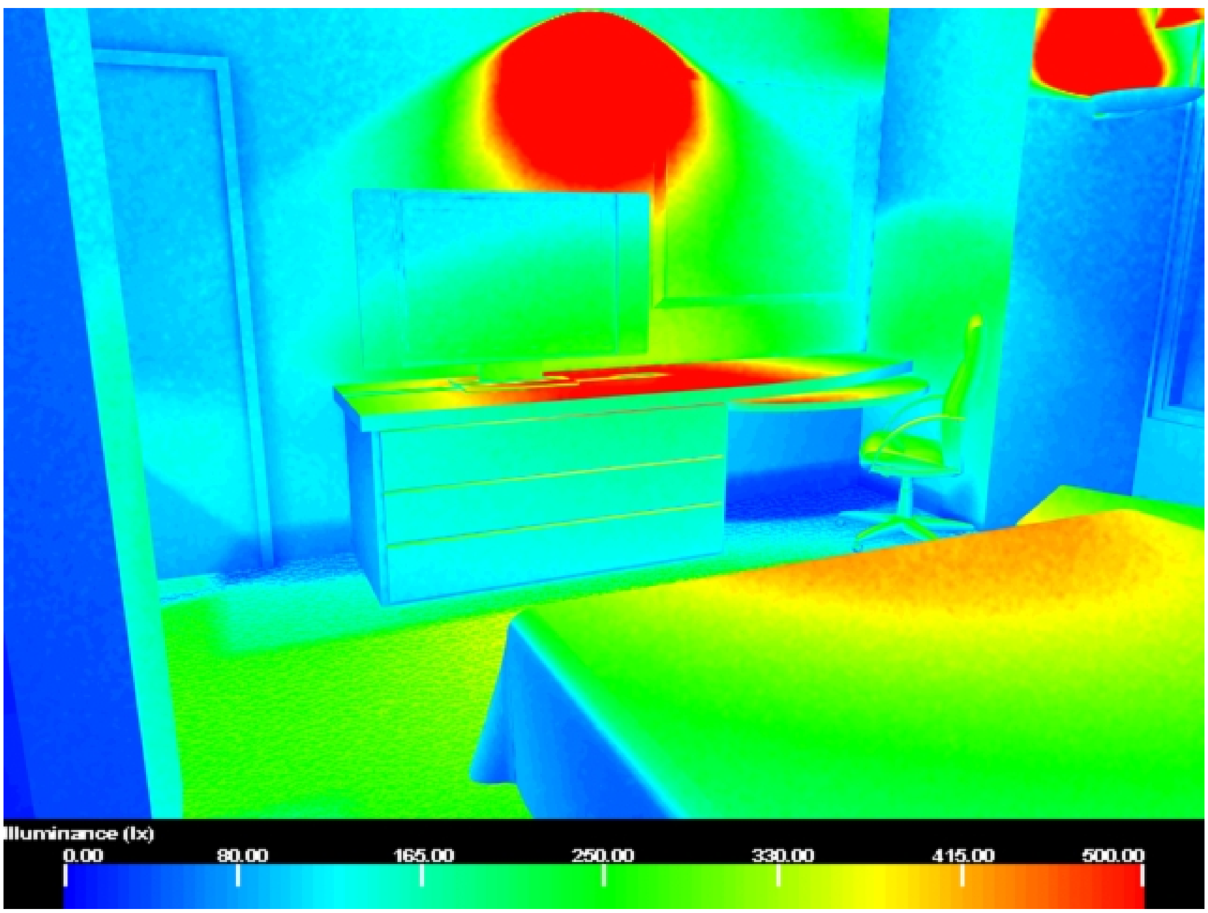
Appendix D –Lighting Breath Information and Calculations

Below are the rendered images, and their illuminance pseudo-color counterparts which represent the amount of light that reaches the surface of each object.







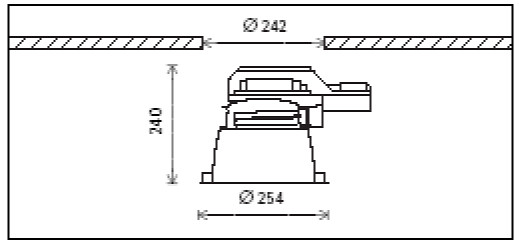

Illuminance (lx)
0.00
80.00
165.00
250.00
330.00
415.00
500.00

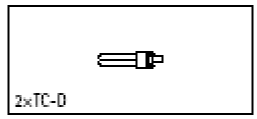
Downlight Fixture Cut-sheet

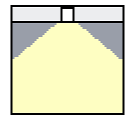
ERCO

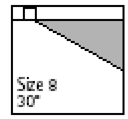
Lightcast Downlight

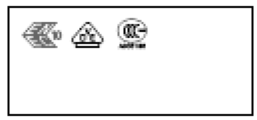
for TC-D lamps





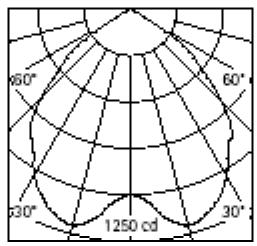






22214.000 Reflector silver
2×TC-D 26W G24d-3 1800lm

Product description
Housing: cast aluminium, designed as heat sink.
Mounting ring: cast aluminium, white (RAL9002) powder-coated.
Tools not required for mounting with 4-point support and screw fixing.
Junction box for through-wiring, 5-pole terminal block, integrated cable clamp.
2 sets of low-loss control gear.
Darklight reflector: aluminium, bright anodised. Cut-off angle 30°.
Weight 3.70kg



2×TC-D 26W G24d-3 1800lm

LOR 0.64
UGR 21.4
65° < 200 cd/m²

ERCO Leuchten GmbH
Postfach 24 60
58505 Lüdenscheid
Germany
Tel.:+49 2351 551-0
Fax:+49 2351 551-300
info@erco.com

Technical Region: 230V/50Hz
Edition: December 16, 2005
Please download the current version from
www.erco.com/22214.000

1/3

ERCO

Lightcast Downlight

Planning data

22214.000	TC-D 26W G24d-3 1800lm
Connected load	P: 62 W
Connected load per 100lx	P*: 2.7 W/m²
Number of luminaires per 100lx	n*: 4.3 1/100m²

22214.000	TC-D 26W G24d-3 1800lm			
Number of luminaires per 100m² for	100lx	200lx	300lx	500lx
	5	9	13	22

22214.000	TC-D 26W G24d-3 1800lm			
Module (m)	1.2x1.8	1.8x1.8	1.8x2.4	2.4x2.4
Illuminance E_n (lx)	1077	718	538	404

Cleaning (a)	1				2				3			
Ambient conditions	P	C	N	D	P	C	N	D	P	C	N	D
LMF	0.94	0.89	0.81	0.72	0.88	0.80	0.69	0.59	0.84	0.74	0.61	0.52
RSMF	0.99	0.98	0.96	0.95	0.97	0.96	0.95	0.94	0.97	0.96	0.95	0.94

Hours of operation (h)	1000	2000	4000	6000	8000	10000
LLMF	0.97	0.92	0.88	0.85	0.83	0.83
LSF	1	1	1	1	1	1

MF	LMFxRSMFxLLMFxLSF
MF	Maintainance Factor
LMF	Lumiaire Maintenance Factor
RSMF	Room Surface Maintenance Factor
LLMF	Lamp Lumens Maintenance Factor
LSF	Lamp Survival Factor
P	Room pure
C	Room clean
N	Room normal
D	Room dirty

Correction table

Ceiling		0.70	0.70	0.70	0.50	0
Wall		0.70	0.50	0.20	0.20	0
Floor		0.50	0.20	0.20	0.10	0
k	0.6	76	57	47	47	43
k	1.0	100	76	68	66	62
k	1.5	116	90	83	80	76
k	2.5	130	100	95	90	85
k	3.0	134	103	99	93	89

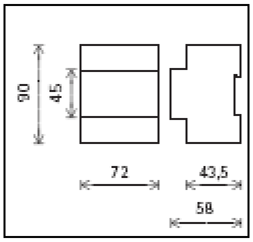
Lightcast Downlight
22214.000 2/3

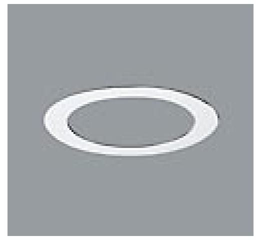
ERCO

Lightcast Downlight

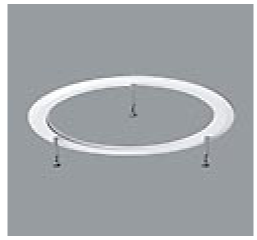
Accessories

83816.000
DALI switching actuator, double, 16A
Two voltage-free contacts for switching ohmic, inductive and capacitive loads max 16A.
DALI interface with two independent addresses.
Mounting on DIN rail.
Weight 0.21kg





83980.000
Cover ring
Metal, white. For covering the gap where ceiling cut-outs are too big. Inner and outer diameter to be specified when placing order.

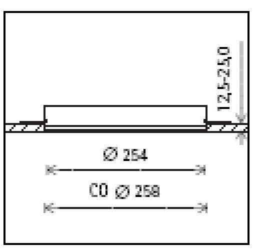


83973.000
Fixture
For decorative disc size 8.
Metal ring, white. 3 spacer sleeves, metal chrome-plated.

83955.000
Plaster ring
Metal, white.
Height 20mm.

83943.000
Decorative circular disc
Size 8
Plastic white, translucent, mirror-finish.
Only in conjunction with: 83973.000

83777.000
Mounting ring
Metal, white powder-coated. For flush-mounting installation in plasterboard ceilings.



82950.000
Mounting plate for panelled ceilings
Metal, white (RAL9002) powder-coated. Individual design of mounting plates according to ceiling type and luminaire. Quote ceiling type and dimensions.

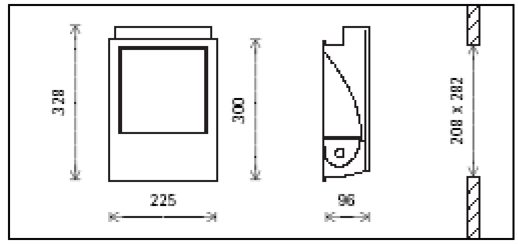
Lightcast Downlight
22214.000 3/3

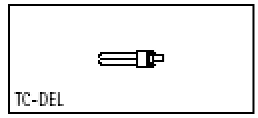
Wall Wash Fixture Cut

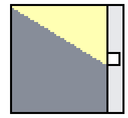
ERCO

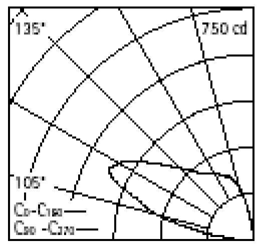
Atrium Uplight

for compact fluorescent lamps









TC-DEL 26W G24q-3 1750lm

33514.000 Reflector silver
TC-DEL 26W G24q-3 1750lm
ECG

Product description
Size 7
Housing for mounting in dry-wall partitions: metal, black powder-coated. Mounting bracket with screw fixing. Clamp extension 10-25mm.
Electronic control gear.
2 cable entries, through-wiring possible. 5-pole terminal block.
Front cover: cast aluminium, white (RAL9002) powder-coated.
Darklight reflector: aluminium, anodised. Horizontal diffuser.
Weight 2.70kg

Cleaning (a)	1				2				3			
Ambient conditions	P	C	N	D	P	C	N	D	P	C	N	D
LMF	0.91	0.86	0.81	0.74	0.86	0.77	0.66	0.57	0.80	0.70	0.55	0.45
RSMF	0.94	0.88	0.82	0.77	0.91	0.84	0.77	0.70	0.84	0.78	0.72	0.64

Hours of operation (h)	1000	2000	4000	6000	8000	10000
LLMF	0.97	0.92	0.88	0.85	0.83	0.83
LSF	1	1	1	1	1	1

MF LMFxRSMFxLLMFxLSF
MF Maintainance Factor
LMF Lumiaire Maintenance Factor
RSMF Room Surface Maintenance Factor
LLMF Lamp Lumens Maintenance Factor
LSF Lamp Survival Factor
P Room pure
C Room clean
N Room normal
D Room dirty

ERCO Leuchten GmbH
Postfach 24 60
58505 Lüdenscheid
Germany
Tel.:+49 2351 551-0
Fax:+49 2351 551-300
info@erco.com

Technical Region: 230V/50Hz
Edition: December 16, 2005
Please download the current version from
www.erco.com/33514.000

1/2

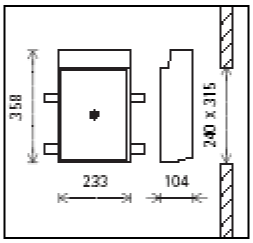
Sheet

ERCO

Atrium Uplight

Accessories

33518.000
Recessed housing
Flush-mounting in brickwork or in dry-wall partitions. Metal, white (RAL9010) powder-coated.
Mounting bracket: metal. Clamp extension 10-25mm.
4 cable entries.
Weight 1.50kg

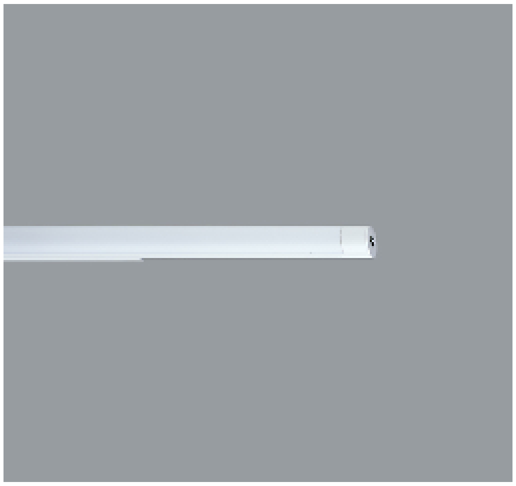


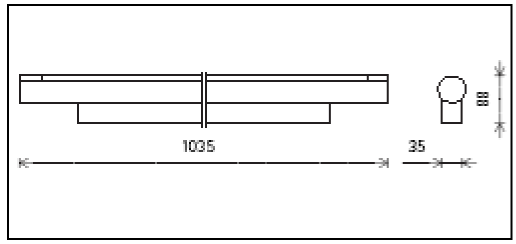
Mirror Fixture Cut-sheet

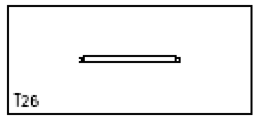
ERCO

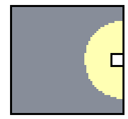
Mirror luminaire

with switch for fluorescent lamps









12182.000 White (RAL9002)
T26 30W G13 2400lm

Product description
Housing: aluminium.
End plates: plastic, white. Round rocker switch.
Wall fixture: aluminium, white (RAL9002) powder-coated, L 485mm.
Control gear, uncorrected.
Cover: acrylic, white.
Weight 2.30kg

ERCO Leuchten GmbH
Postfach 24 60
58505 Lüdenscheid
Germany
Tel.:+49 2351 551-0
Fax:+49 2351 551-300
info@erco.com

Technical Region: TechnischeUmgebung
Edition: December 16, 2005
Please download the current version from
www.erco.com/Artikelnummer

1/1

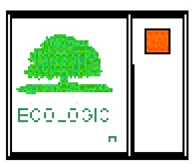
Ballast Cut-sheets

| Return to: DULUX D/E (double, 4-Pin) | | Print Page |

Product Number:	20722
Order Abbreviation:	CF26DD/E/830
General Description:	DULUX 26W double compact fluorescent lamp with 4-pin base, integral EOL, 3000K color temperature, 82 CRI, for use with electronic and dimming ballasts, ECOLOGIC

Product Information	
Abbrev. With Packaging Info.	CF26DDE830 105V 50/CS 1/SKU
Average Rated Life (hr)	12000
Base	G24Q-3
Bulb	T4
Color Rendering Index (CRI)	82
Color Temperature/CCT (K)	3000
Family Brand Name	Dulux® D/E
Industry Standards	IEC 60901- 2526
Initial Lumens at 25C	1710
Mean Lumens at 25C	1470
Maximum Overall Length - MOL (in)	6.5
Maximum Overall Length - MOL (mm)	166
NEMA Generic Designation (current)	CFQ26W/G24Q/830
Nominal Wattage (W)	26.00

Additional Product Information
Product Documents, Graphs, and Images
Compatible Ballast
Packaging Information



Footnotes

- Approximate initial lumens after 100 hours operation.
- The life ratings of fluorescent lamps are based on 3 hr. burning cycles under specified conditions and with ballast meeting ANSI specifications. If burning cycle is increased, there will be a corresponding increase in the average hours life.
- Rule of Thumb for Compact Fluorescent Lamps: Divide wattage of incandescent lamp by 4 to determine approximate wattage of compact fluorescent lamp that will provide similar light output.
- Minimum starting temperature is a function of the ballast; consult the ballast manufacturer.
- There is a NEMA supported, industry issue where T2, T4, and T5 fluorescent and compact fluorescent lamps operated on high frequency ballasts may experience an abnormal end-of-life phenomenon. This end-of-life phenomenon can resultin one or both of the following: 1. Bulb wall

ADVANCE

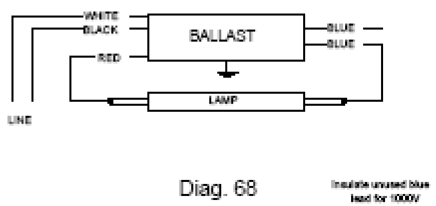
VOP-2P32-SC

Brand Name	OPTANIUM
Ballast Type	Electronic
Starting Method	Instant Start
Lamp Connection	Parallel
Input Voltage	277
Input Frequency	50/60 HZ
Status	Active

Electrical Specifications

Lamp Type	Num. of Lamps	Rated Lamp Watts	Min. Start Temp (°F/C)	Input Current (Amps)	Input Power (ANSI Watts)	Ballast Factor	MAX THD %	Power Factor	MAX Lamp Current Crest Factor	B.E.F.
F17T8	1	17	0/-18	0.08	19	1.02	15	0.95	1.7	5.37
F17T8	2	17	0/-18	0.11	31	0.90	15	0.97	1.7	2.90
F25T8	1	25	0/-18	0.10	27	1.02	15	0.97	1.7	3.78
F25T8	2	25	0/-18	0.16	43	0.88	10	0.99	1.7	2.05
F32T8	1	32	0/-18	0.13	35	1.01	15	0.98	1.7	2.89
F32T8	2	32	0/-18	0.20	55	0.88	10	0.99	1.7	1.60
* F32T8/ES (30W)	1	30	60/16	0.12	33	1.01	15	0.97	1.7	3.06
F32T8/ES (30W)	2	30	60/16	0.19	52	0.88	10	0.99	1.7	1.69
F40T8	1	40	32/00	0.15	41	1.01	15	0.99	1.7	2.46
F40T8	2	40	32/00	0.24	67	0.88	10	0.99	1.7	1.31

Wiring Diagram



Diag. 68

Insulate unused blue lead for 1000V

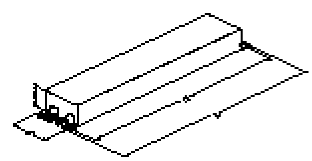
The wiring diagram that appears above is for the lamp type denoted by the asterisk (*)

Standard Lead Length (inches)

	in.	cm.
Black	25L	63.5
White	25L	63.5
Blue	31R	78.7
Red	37L	94
Yellow	0	0
Gray	0	0
Violet	0	0

	in.	cm.
Yellow/Blue	0	0
Blue/White	0	0
Brown	0	0
Orange	0	0
Orange/Black	0	0
Black/White	0	0
Red/White	0	0

Enclosure



Enclosure Dimensions

OverAll (L)	Width (W)	Height (H)	Mounting (M)
9.50 "	1.7 "	1.18 "	8.90 "
9 1/2	1 7/10	1 9/50	8 9/10
24.1 cm	4.3 cm	3 cm	22.6 cm

Revised 06/09/2003

Data is based upon tests performed by Advance Transformer in a controlled environment and representative of relative performance. Actual performance can vary depending on operating conditions. Specifications are subject to change without notice. All specifications are nominal unless otherwise noted.

ADVANCE TRANSFORMER CO.
O'HARE INTERNATIONAL CENTER · 10275 WEST HIGGINS ROAD · ROSEMONT, IL 60018
Customer Support/Technical Service: Phone: 800-372-3331 · Fax: 630-307-3071
Corporate Offices: Phone: 800-322-2086

ADVANCE

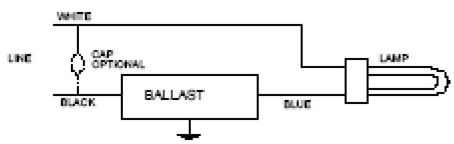
VLO-2S13-TP

Brand Name	COMPACT-OPEN
Ballast Type	Magnetic
Starting Method	Pre-Heat
Lamp Connection	Series
Input Voltage	277
Input Frequency	60 HZ
Status	Active

Electrical Specifications

Lamp Type	Num. of Lamps	Rated Lamp Watts	Min. Start Temp (°F/C)	Input Current (Amps)	Starting Current (Amps)	Open Circuit (Amps)	Input Power (Watts)	Ballast Factor	MAX THD %	Power Factor
CFQ13W/GX23	2	13	32/00	0.31	0.38	0.00	34	0.95	15	0.40
* CFQ26W/G24D	1	26	50/10	0.33	0.40	0.00	34	0.98	15	0.37
CFT13W/GX23	2	13	32/00	0.25	0.38	0.00	34	0.95	15	0.40

Wiring Diagram



Diag. 140

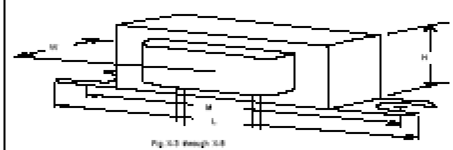
The wiring diagram that appears above is for the lamp type denoted by the asterisk (*)

Standard Lead Length (inches)

	in.	cm.
Black	14	35.6
White		0
Blue	7	17.8
Red		0
Yellow		0
Gray		0
Violet		0

	in.	cm.
Yellow/Blue		0
Blue/White		0
Brown		0
Orange		0
Orange/Black		0
Black/White		0
Red/White		0

Enclosure



Enclosure Dimensions

OverAll (L)	Width (std)/(TP)	Height (H)	Mounting (M)
4.0 "	1.5625 "/1.8125 "	2.25 "	3.5 "
4	1 9/16 / 1 13/16	2 1/4	3 1/2
10.2 cm	4 cm / 4.6 cm	5.7 cm	8.9 cm

Revised 11/02/1999

UL

Data is based upon tests performed by Advance Transformer in a controlled environment and representative of relative performance. Actual performance can vary depending on operating conditions. Specifications are subject to change without notice. All specifications are nominal unless otherwise noted.

ADVANCE TRANSFORMER CO.
O'HARE INTERNATIONAL CENTER · 10275 WEST HIGGINS ROAD · ROSEMONT, IL 60018
Customer Support/Technical Service: Phone: 800-372-3331 · Fax: 630-307-3071
Corporate Offices: Phone: 800-322-2086