[Helios Plaza]

Houston, Texas

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Structural Option
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[THESIS FINAL REPORT]

Helios Plaza

201 Helios Way

Houston, Texas

General Building Data

Size: 423,500 GSF

Height: 113'

Occupancy: Office, Conference Center

Structure

Framing System: 24 in^2 concrete columns and post-tensioned concrete girders with cylindrical steel columns and long span steel W-shapes.

Lateral System: Concrete moment frames in combination with rigid diaphragm floor system.

Floor System: Two systems typically used. Composite 20 gage decking with lightweight concrete topping and one-way pan joist systems.

Foundation: Spread concrete footings with 4000 psi strength.

Project Team

Owner: BP p.l.c.

Architect: Gensler

CM: Bovis Lend Lease Construction Ltd

Structural: Walter P. Moore & Associates, Inc.

MEP: I.A. Naman + Associates, Inc.

Vertical Transportation: Persohn/Hahn Associates, Inc.

Security: CPP and Associates

CHPP: Turbine Air Systems

Architecture

The design principle is based upon functional, pragmatic design. Utilizing a simple box shape, the building is built in a three stack design to accommodate for two-story trading floors. A true campus environment is achieved by incorporating large expanses of Katy Prairie land in addition to multiple International Cafes on every floor of the six-story complex.

MEP Systems

Mechanical: VAV systems with 555,500 CFM exchange rate. 5 MW natural gas fired combined heat and power system in combination with chillers.

Electrical: 3ϕ 208Y/120V service voltage. 2 UPS Systems with 3-500kVa modules.

Lighting: Aggressive lighting scheme with high efficiency direct/indirect fixtures. 82% of regularly occupied spaces day lit.

Kevin Zinsmeister

http://www.engr.psu.edu/ae/thesis/portfolios/2011/kzz5000/

Structural Option

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Acknowledgements

I would like to thank the following individuals for supporting my thesis project with their time and understanding.

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Dennis Wittry

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I would also like to thank my family and friends for the constant support and belief. I hope that I will always be able to repay your kindness with my own.

Executive Summary

The purpose of this report is to present the investigations conducted on Helios Plaza as part of its redesign. Helios Plaza is an office building that houses the IST and oil trading divisions of its owner BP. The plaza is located in Houston, Texas in an area zoned for office buildings and suburban housing. The overall building height is 113' with a typical floor-to-floor height of 15'.

With respect to the structural system of Helios Plaza, the gravity system mainly consists of a one-way concrete pan joists system supported on concrete columns, but certain areas are composite steel deck supported on long-span, castellated steel wide flanges. Lateral forces in the building are resisted by concrete moment frames and some steel moment frames composed of HSS beams welded to concrete filled steel pipe columns. The overall effect of this design results in a relatively high building self-weight, requiring the use of large spread footing foundations and seismic loads controlling design in one direction.

In an attempt to remedy the large building weight, a composite steel system was designed as an alternative to the existing system. Prior investigations had shown that a composite steel system was feasible in strength design and had potential to reduce the weight of the building. The redesign successfully reduced the weight of the building.

The entire structure was redesigned in RAM and ETABS and checked with hand calculations. Steel pipe braces were used as the lateral resisting system and were chosen for their aesthetic and strength properties. A typical brace was chosen to be representative of the brace to beam to column connection and was designed by hand.

Two depths are presented in this report that are related to the lateral braces in particular. Architectural considerations of the braces will be addressed and analyzed. The analysis shows why certain decisions were made in placing the braced frames in the building.

The second breadth presented deals with construction management principles. The cost and schedule of the redesign were compared with the original structure. The findings showed that the redesign was more expensive, but was able to be constructed much quicker.

As part of the MAE requirements, coursework from Computer Modeling of Building Structures was utilized in creating the computer models. Additionally, principles from Earthquake Resistant Design of Buildings were used to design the lateral bracing system and the braces' connections were designed using Design of Steel Connection course notes.

Introduction

Helios Plaza is a corporate campus located in Houston, Texas that is comprised of three main structures. The first structure, which is the focus of this report, is a six-story office building that houses the IST and oil trading divisions of BP, the building owner. In addition to the office building, there is a 1,909 car capacity parking deck adjacent to a five megawatt combined heat and power plant separate from the office building. Construction was completed in September 2009. The office building will be referred to as Helios Plaza throughout the rest of this document.

The six-story office building is 423,500 gross square feet with an overall building height of 113 feet. The typical floor-to-floor height is 15 feet with exception at the first floor, the lower roof level and the roof level. The first floor height is 21.5 feet, the lower roof level is 17 feet and the roof level is 14 feet higher than the lower roof. Figure 1 represents these dimensions below.

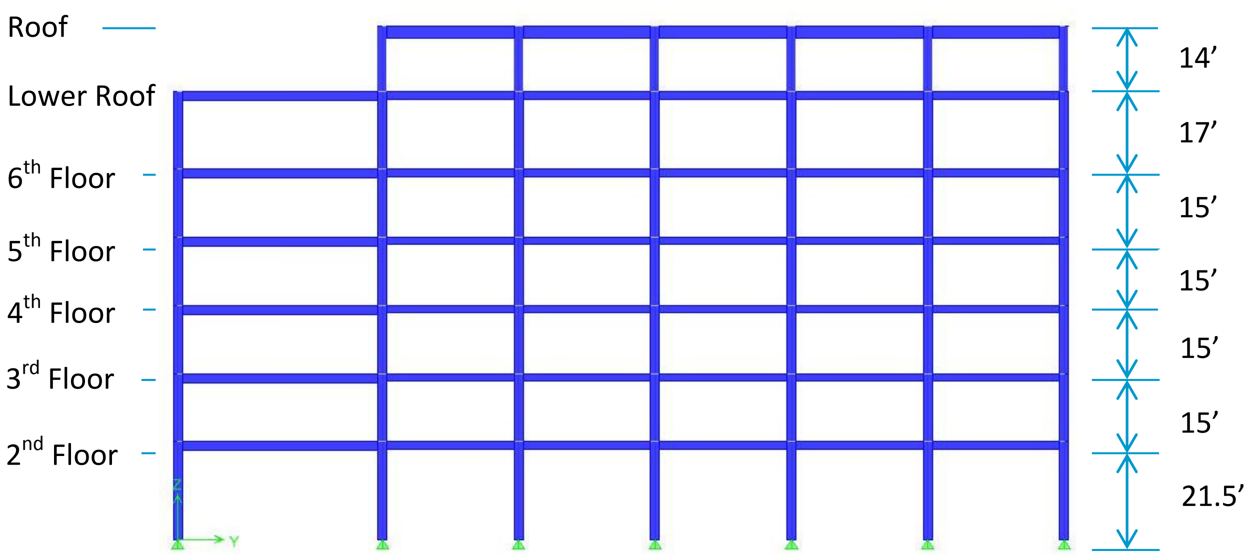


Figure 1: Building Frame Section

One of the more unique aspects of the office building is a result of the oil trading division wishes. The traders requested large, open areas to work in and these spaces are accommodated on the second, fourth and sixth floors. To make these areas more open, the floors above (i.e. the third floor, fifth floor, and lower roof level) are cut out over the trading floors to create double story spaces. To further the open feeling, the number of columns used is limited, which in turn creates long-span situations. Figures 2 and 3 on the next page illustrate simplified versions of the floor plans.

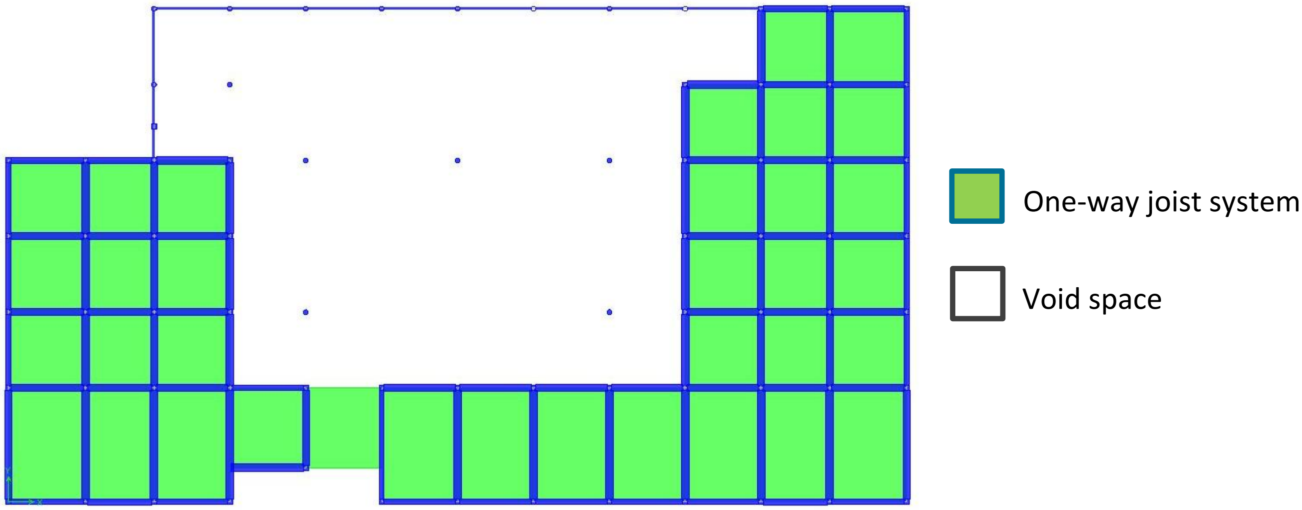


Figure 2: Cut-out Floor over Trading Floor

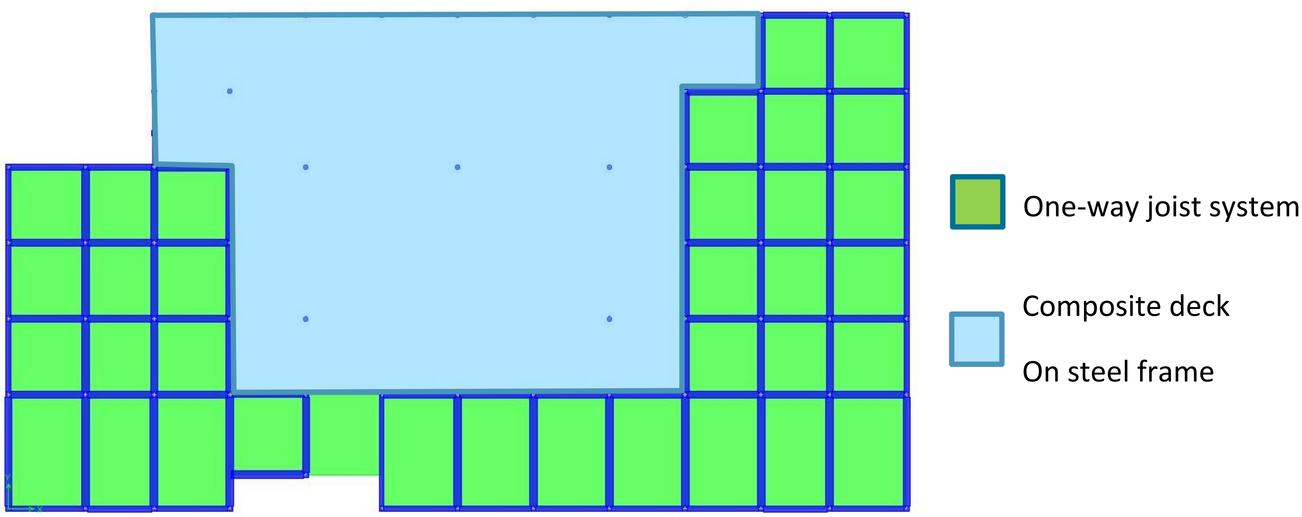


Figure 3: Composite Deck at Trading Floor

Existing Structural System Overview

For better understanding of the main redesign, some existing structural conditions will be addressed. The main structural system of Helios Plaza is framed in reinforced concrete and gravity loads are handled largely by square concrete columns, although concrete filled steel pipe columns are used for aesthetics in larger spaces. For shorter spans, averaging thirty feet, concrete girders in combination with pan beams are used. For longer spans of forty-five feet, post tensioned girders are employed. Finally, for spans of sixty feet, castellated wide flanges shapes are used to reduce the weight-span ratio while maintaining strength.

The floor is mainly a concrete one-way system that uses 66” span, 6” wide skip joists typically. In mechanical rooms, two-way slabs are used to distribute the larger live loads more evenly to the supporting members. Composite decking with lightweight concrete is used over the long span steel members in the trading rooms.

To resists lateral loads, the building relies on the typical framing members to perform as concrete moment frames. In the trading floor areas, steel moment frame comprised of 2’ diameter steel pipe columns are filled with 7000 psi concrete and 14” Ø HSS steel beams run the perimeter of the building to transfer lateral load.

Foundation

The site had to be extensively dewatered prior to the excavation for the project because of the porosity of the soil in Houston. Despite the initial site conditions, the bearing capacity of the soil was determined to be 6500 psf.

Spread concrete footings are placed at the base of all grade level columns. The typical depth of the footings is six feet below the member that they are supporting. Their sizes range from 4’ x 4’ x 15” to 17’ x 17’ x 57”.

Retaining walls are only used in the southeast corner of the building where there is a sub-grade basement with access to the adjacent parking structure via a tunnel. At level one, the floor is a slab on grade with thickness ranging from 5” to 12”. Grade beams are also implemented at level one sized at 42” x 30”.

One of the focal points of this thesis investigation regards the reduction of the foundations due to decreased building weight. This topic will be addressed later in the report.

Columns

Rectangular concrete columns are the predominant system used in Helios Plaza. For the most part these normal weight columns are 24” x 24” in size at all floors except level one where there is an increase in size to 30” x 30”. The concrete strength decreases as the levels increase from 6000 psi at the basement level and level one to 5000 psi at levels two and three to 4000 psi for levels four through six. The basement level only occurs in the southeast corner of the building to allow access from the underground tunnel to the rest of the building and accounts for only fifteen percent of the ground floor area. This space is spanned at level one by post-tensioned girders and one-way pan joists and can be seen in Figure 4.

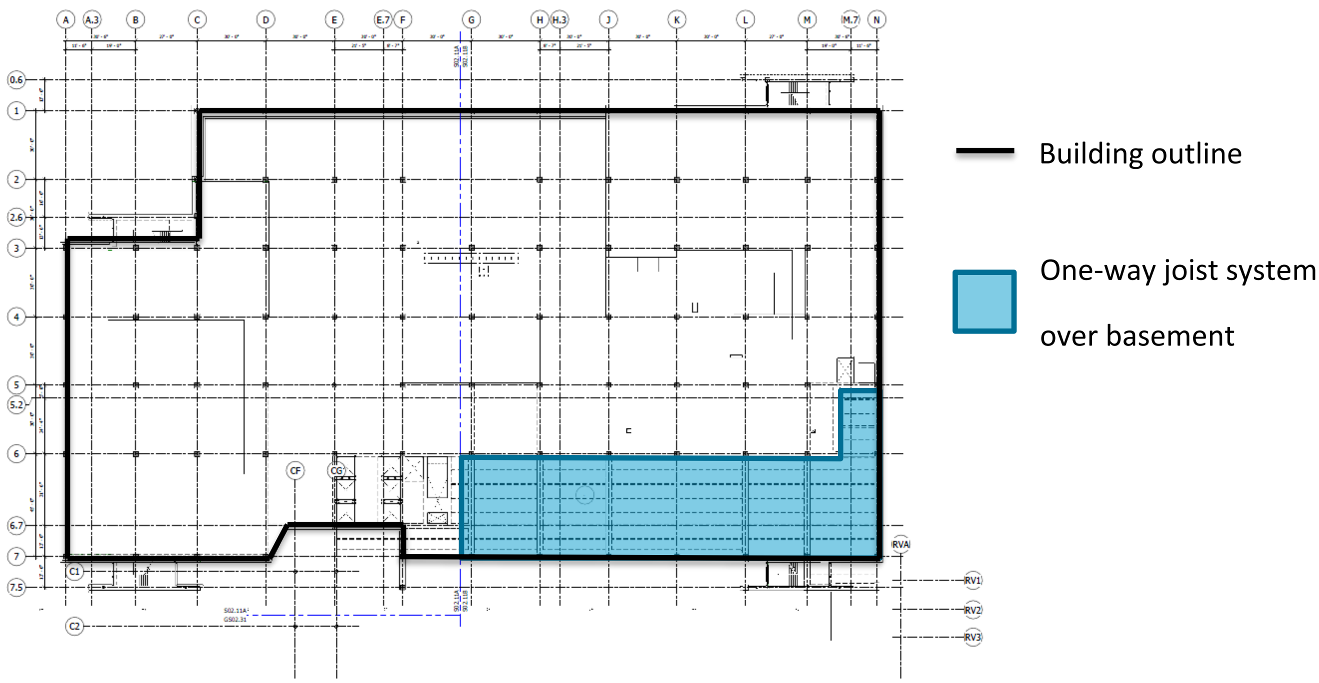


Figure 4: Basement Area

In addition to the rectangular concrete columns, concrete filled steel pipe columns are used in the double story trading spaces. These columns are 24Ø and are filled with 7000 psi strength concrete.

Floor Systems

As with the rest of the structural systems in Helios Plaza, the floor system is split into two main categories, one-way pan joists and composite deck. The one-way pan joist system is a 4" slab that rests on 16" deep pan typically. The one-way system frames into girders that range from 20" to 33" deep with a width ranging from 24" to 36". Girders also span in the same direction as the one-way joist system, but these members are there to create concrete moment frames to resist lateral loads.

Post-tensioned girders are used all along the south face of the building that span in the North-South direction. This is necessary to meet the strength requirements for the 45' distance that these members span. The tendons are typically bundled in groups of four and the minimum final post-tension force is 351 kips. Their locations can be seen in Figure 5 on the next page.

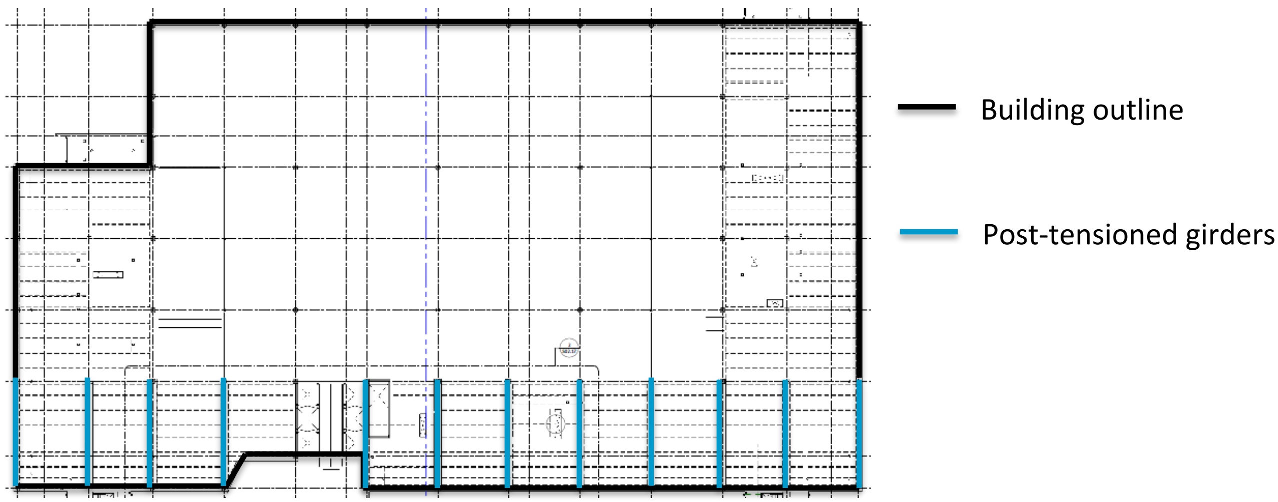


Figure 5: Post-Tensioned Girder Locations

Two-way slabs are implemented in areas where mechanical equipment is housed on every floor. The slabs are typically 10" thick, but in some cases they are 12" thick. Bathrooms usually share the same bays as the mechanical rooms because cutting holes in this system is efficiently achievable.

The second main floor system used in Helios Plaza is a composite deck on w-shapes. The change occurs because of the move to long span castellated beams to accommodate open, double story spaces for the trading floors. Spans of 60' dominate these spaces and the castellated beams vary between CB24x100 and CB30x44/62. In addition to the weight saving caused by punching out parts of the web, the beams are cambered 1.5" and 1.75" to meet deflection limits. The composite section used is typically 3 1/2" light weight concrete over 2" composite deck. Figure 6 below shows all three of the floor systems in adjacent bays of the building.

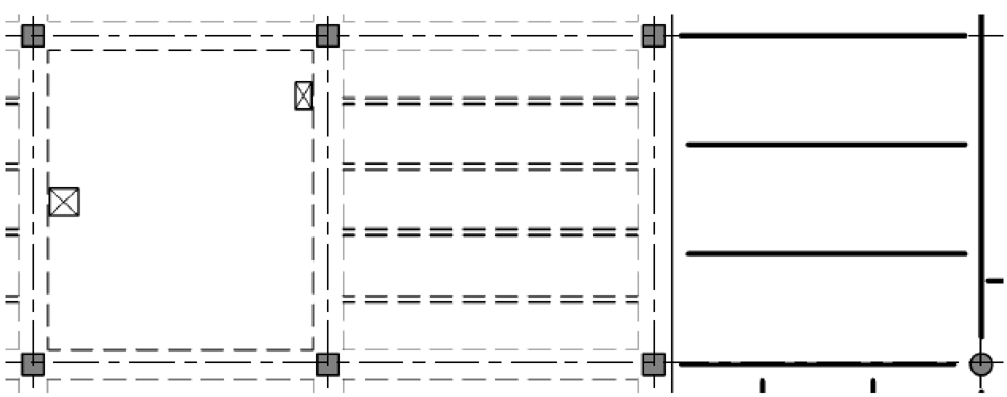


Figure 6: All Three Floor Systems in Adjacent Bays

Lateral Systems

Lateral forces are resisted in Helios Plaza by concrete moment frames. As mentioned before, girders run in the same direction as the one-way joist system to make up the frames in the East-West direction, while girders running in the North-South direction carry the pan joist loads in addition to transferring lateral load. When a double story occurs, several lateral resisting frames are interrupted and load transfers from the building's enclosure directly to moment frames are not possible. The force is instead transferred perpendicularly by horizontal circular HSS members to the one-way joists or to the floors above and below by the steel pipe columns. These beams are welded to the steel pipe columns and a detail can be seen below in Figure 7.

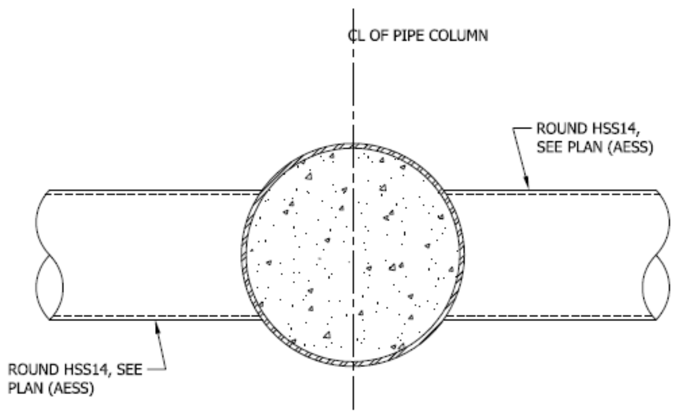


Figure 7: Round HSS Members Framing Into Each Other

Steel members that compose the floor system for the trading areas are not effective lateral members. They are not framed with moment connections and essentially only function to make a rigid diaphragm and to carry gravity loads. Overall, the building consists of twenty-two moment frames. Floor plans can be found in Appendix A.

Existing Lateral Load Conditions

Calculations in Technical Report I showed that the controlling load cases for the East-West and North-South directions were resultant of seismic loading and wind loading respectively. The relatively short width of the building compared to its length in combination with the building's large mass led to the seismic control in the East-West direction. Story force diagrams can be seen in Figures 8, 9 and 10.

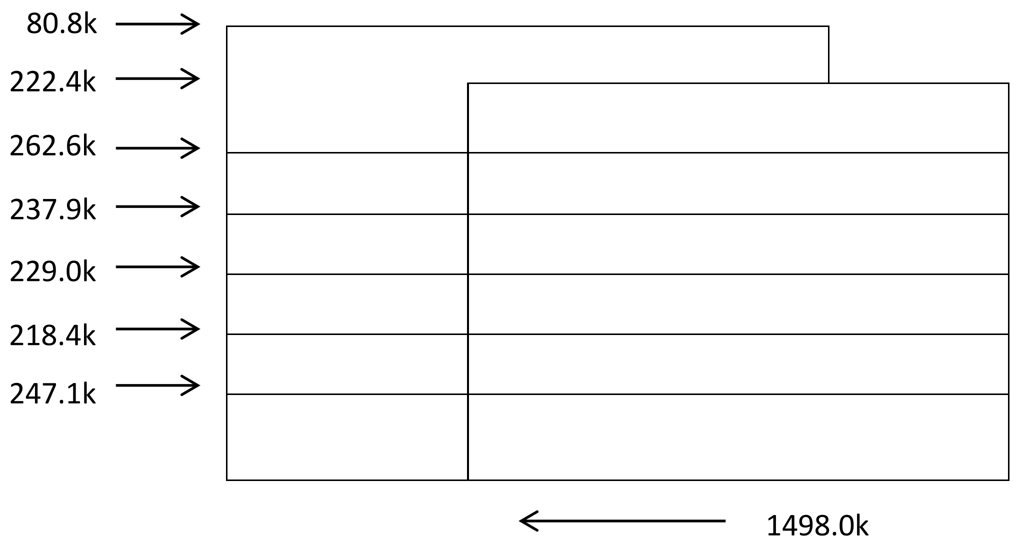


Figure 8: North-South Wind Story Forces

The figures were not drawn to scale, which makes the drastic difference between the story forces seem peculiar. In actuality, the North-South facades have a tributary width of 355' as compared to the East-West facades which have a tributary width of 195'. This ratio of approximately 1.8 accounts for the nearly doubled forces in the North-West direction for wind.

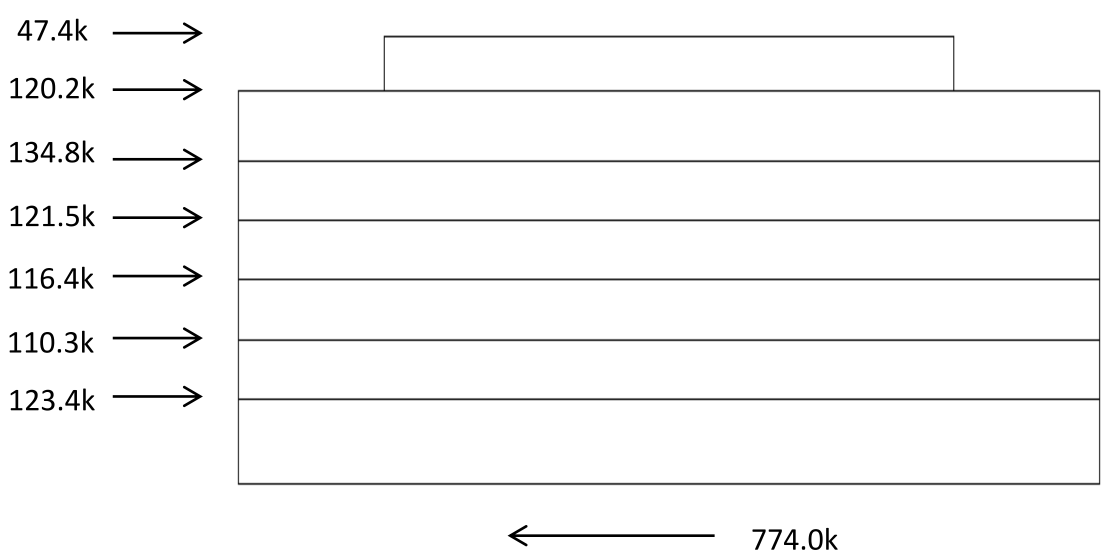


Figure 9: East-West Wind Story Forces

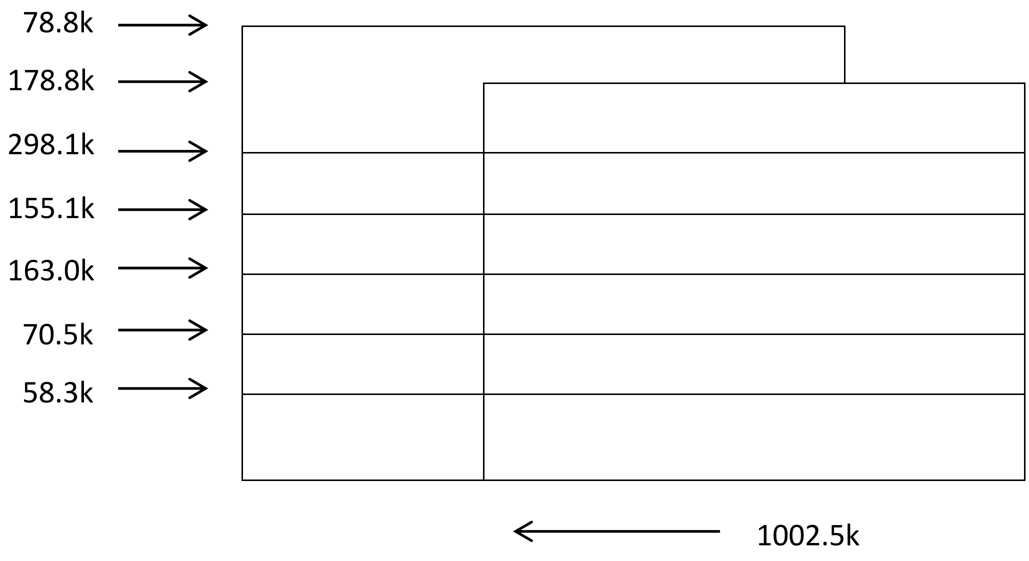


Figure 10: Seismic Force Diagram

Structural Redesign Philosophy

In an attempt to unify the structural system of Helios Plaza, the existing concrete framing system was redesigned in steel. Of the spaces in the building, only the trading floors were kept the same due to their unique design. For lateral resistance, a concentric diagonal braced frame system was picked for design due to potential efficiency and the generally higher cost of steel moment frames. With these decisions in mind, the design of the gravity system was approached first, but parameters for design needed to be defined first.

Codes and References

Helios Plaza was designed following all of the applicable guidelines for the state of Texas as well as the city of Houston. For the purpose of these thesis investigations, the latest design codes were utilized without specific regional additions.

Original Design Codes

- National Model Code:
  - 2003 International Building Code with City of Houston Amendments
- Design Codes:
  - Texas Architectural Barrier Act Standard
  - ANSI/AWS Structural Welding Code
- Structural Standards:
  - American Society of Civil Engineers, SEI/ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Code:
  - 2009 International Building Code
- Design Codes:
  - Steel Construction Manual 13th edition, AISC
  - ACI 318-05, Building Code Requirements for Structural Concrete
- Structural Standards:
  - American Society of Civil Engineers, SEI/ASCE 7-10, Minimum Design Loads for Buildings and Other Structures

Materials

In selecting the materials to design with, assumptions were made based upon both the existing structure and code requirements. Of particular importance was the material strength of the bracing members. For seismic design requirements, the steel pipe used in bracing needed to be ASTM A53 grade B steel. A summary of the rest of the design values can be seen in Table 1.

Concrete		f'c (psi)
Spread Footings		4000
Basement Walls		6000
Slabs	On-Grade	3500
	Metal Deck	3500
Reinforcement		**Fy (ksi)**
Rebar		60
Welded Wire	Smooth	65
Structural Steel		**Fy (ksi)**
Wide Flange Shapes		50
HSS		42
Edge Angles/Bent Plates		36
Plates		36
Pipe		35

Table 1: Redesign Material Strengths

Redesign Goals

Prior investigations in Technical Report II showed that there were many potential benefits in switching the structure from concrete to steel. This thesis attempted to maximize these benefits while limiting the negative effects of the steel frame. Amongst the detriments of the composite steel deck system was higher cost and a larger floor depth. Based upon these findings, the desired goals of the redesign investigation were as follows:

1) Reducing the overall building weight;
2) Eliminating the controlling seismic base shear in the East-West direction;
3) Minimizing floor plan impacts;
4) Creating aesthetically compatible braces;
5) Reducing the construction schedule; and
6) Offsetting the increased steel structure cost with foundation savings.

Parameters for the design of gravity members were determined at the beginning to select trial sizes. Prior investigations in Technical Report II showed that a Vulcraft 1.5VL17 composite deck with a 3.25" light weight concrete topping was adequate for the spans and fire-rating requirements. Not only was this deck more than sufficient to carry the loads placed upon it, but was also able to span between all of the framing members without the use of shoring for construction. The depth of the topping also allowed for a fire rating of two hours, which would benefit the cost of the building by eliminating the need for fireproofing on the underside of the metal deck. The composite deck constituted a majority of the dead load on the structure; however, additional superimposed dead loads and beam self-weight allocations were added for gravity design of beams and columns.

Structural Redesign

The main focus of this section of the report is on the redesign of the building from a predominantly concrete moment frame system to a steel braced frame system. Additionally, the structural depth involved designing the connection interface of the lateral braces, columns and beams.

Initial Design

To aid the design of the new system, two computer programs, RAM Structural System and ETABS, were utilized. To begin the design, typical framing members were designed by hand with the live load assumptions determined in Technical Report I and the dead load assumptions addressed above. Once these trial member sizes were determined, a model was built in RAM to perform initial member sizing for gravity loading to confirm the hand design for beams and columns. When the columns were laid out, the same centerline locations as the existing 24" square concrete columns used in Helios plaza were referenced. Hand calculations can be found in Appendix B.

In building the model, concepts learned in the Computer Modeling of Building Structures course were drawn on for accurate input. Column local axis orientations and beam member end releases were employed to ensure a proper output from the program's black box. One snafu in the design process that would need correction after several runs was the orientation of the steel decking. This initial error resulted in the program placing the entire load on the girders and columns and designing beams that were mainly W8x10s. Other input parameters that needed to be addressed were deflection and camber limitations. Without user guidance, the program would select the member size with least weight within a certain tolerance and camber the shape to meet standard deflection requirements. Once these program nuances were dealt with, the output was at a standard acceptable to the user. The completed model can be seen in Figure 11.

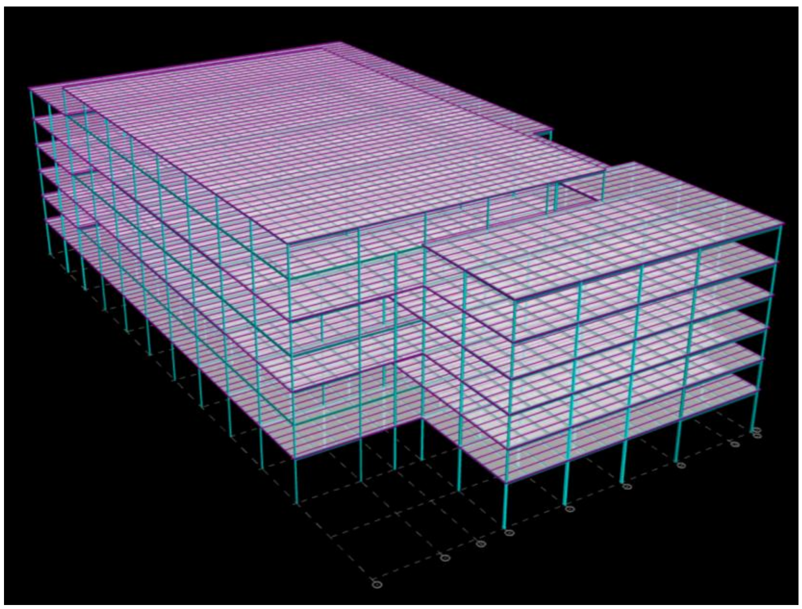


Figure 11: RAM Gravity Design Model

Minor size differences existed between the hand designed framing members and the RAM model output. The RAM output sizes were favored in several instances due to plastic neutral axis considerations (PNA) assumptions of the composite section. Hand designs relied on a PNA that was conservatively chosen at the lowest possible point, resulting in lower load resistance values. Additionally, the distance from the top of the steel beam to the concrete flange force was conservatively chosen in hand analysis. RAM was able to calculate the PNA location and flange force distance more accurately and usually resulted in one size smaller of a member.

Lateral Design

Moving on to lateral resistance of the structural system, initial sizes for the braces were determined utilizing the seismic provisions guide provided by the American Institute of Steel

Construction and supplemented by notes from the Earthquake Resistant Design of Buildings course. Steel pipe sections were chosen for aesthetic purposes and initial design forces were based on the wind forces in the North-South direction and the East-West since a building weight had yet to be established. The layout chosen was based upon exterior appearance and is addressed in the architectural breadth section of this report. In an attempt to limit torsional effects, the braces in each direction were made equal. This presented a problem in the North-South direction where there are only five bays. To account for this, the braces in the 45' bay were laid out as x-braces as can be seen in Figure 12.

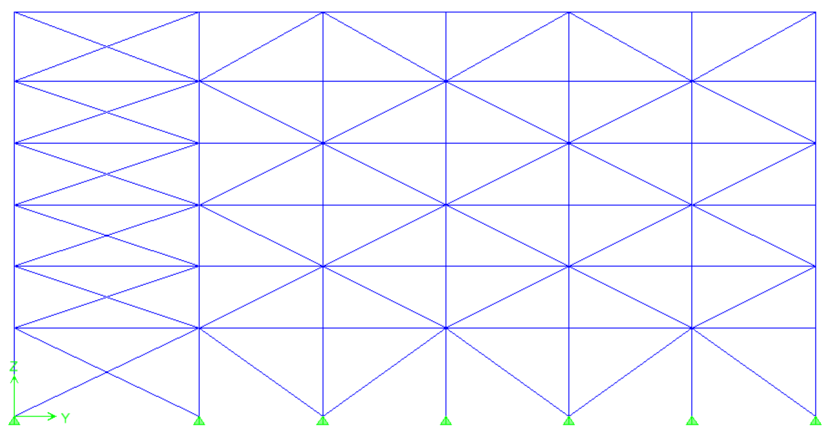


Figure 12: Brace Layout of East Facade

Brace sizes initially ranged from 6 Standard pipe to 12 x pipe based upon the preliminary brace locations. At this point in the design, the braces were only required to have a slenderness, $\frac{kL}{r}$, less than 200. The braces were only located on the perimeter of the building in an attempt to limit their effect on the floor plan of Helios Plaza. Their relative strengths were designed to distribute the lateral load between the frames as evenly as possible given the geometry.

Taking these brace designs forward, a computer model was built in the program ETABS to analyze the lateral brace design. Several assumptions were made when creating this model. First of all, both the beams and the braces were modeled with major axis moment released to simulate purely pinned conditions. In defining the material properties used in the building, the weights and masses were also removed and instead applied by the user to the diaphragm as unit dead load and unit mass respectively. Another assumption concerning the diaphragm was assuming that the composite steel deck and concrete were rigid, and thus formed a rigid diaphragm. The analysis showed that under wind loading, and all load combinations defined by ASCE 7-10 that included wind, the braces were sufficient in strength. A summary of the basic load combinations can be seen in Table 2.

Load Combination	Equation
1	1.4D
2	1.2D + 1.6L + 0.5(L_r or S or R)
3	1.2D + 1.6(L_r or S or R) + (L or 0.5W)
4	1.2D + 1.0W + L + 0.5(L_r or S or R)
5	1.2D + 1.0E + L + 0.2S
6	0.9D + 1.0W
7	0.9D + 1.0E

Table 2: ASCE7-10 2.3.2 Basic Load Combinations

With the members all accounted for, a preliminary building weight could be determined, and new seismic loads could be calculated. Besides changing the weight of the building, the Response Modification Coefficient, R, of Helios plaza was increased from R=3 for ordinary reinforced concrete moment frames to an R=3 ¼ for steel ordinary concentrically braced frames. This alteration in turn led to changes in the seismic response coefficient, C_S, as well as the approximate fundament period of the building, T_a. The combination of all of these changes resulted in much lower seismic design forces than originally encountered in previous investigations. A comparison of the final and original seismic design forces and weights can be seen in Table 3.

Seismic Forces						
	Original			Redesign		
Level	Weight (k)	F_x (k)	Shear (k)	Weight (k)	F_x (k)	Shear (k)
roof	1089	78.8	78.8	1329	88.2	88.2
lower roof	2961	178.8	257.6	1918	106.9	195.2
6	6332	298.1	555.7	4447	194.9	390.1
5	4304	155.1	710.7	2255	76.3	466.4
4	6332	163.0	873.7	4455	109.0	575.4
3	4304	70.5	944.2	2270	35.9	611.3
2	7146	58.3	1002.5	4116	33.2	644.5
Total	32468	1002.5	-	20790	644.5	-

Table 3: Building Weight and Seismic Force Comparison

These results, although successful in reducing the seismic base shear, did not reduce the loading enough to eliminate seismic forces as the controlling load case in the East-West direction. As a result, the braces needed to be redesigned to meet all of the criteria of steel ordinary concentrically braced frames. The main repercussion of this switch was ensuring that the braces met more stringent slenderness requirements. Now the seismic provisions stated that $\frac{kL}{r} \leq \frac{\overline{E}}{Fy}$, which in the case of ASTM A53 Grade B steel simplifies to $\frac{kL}{r} \leq 115$. With the unbraced lengths that these pipes needed to span regularly being on the larger side of 30', the minimum pipe size that could be used was 10 Standard. The new members were entered into the ETABS model and the seismic forces were placed on the building. Analysis showed that the braces all had adequate load resistance so other issues with the design could be addressed.

The addition of so much weight from the braces as well as the effect of lateral forces on the structure caused several columns to fail. The RAM output as well as hand calculations did not account for excessive lateral loads and the interaction equation for combined axial and bending in columns was exceeding 1.0 for many ground floor columns. To correct these failures, the axial forces in the columns was retrieved from the ETABS output and new members were sized based on unbraced length using Table 4-1 of the AISC Steel Construction Manual. After the members were upsized, analysis was run again and forces were checked again. This process was iterated until no more members failed under any load combination.

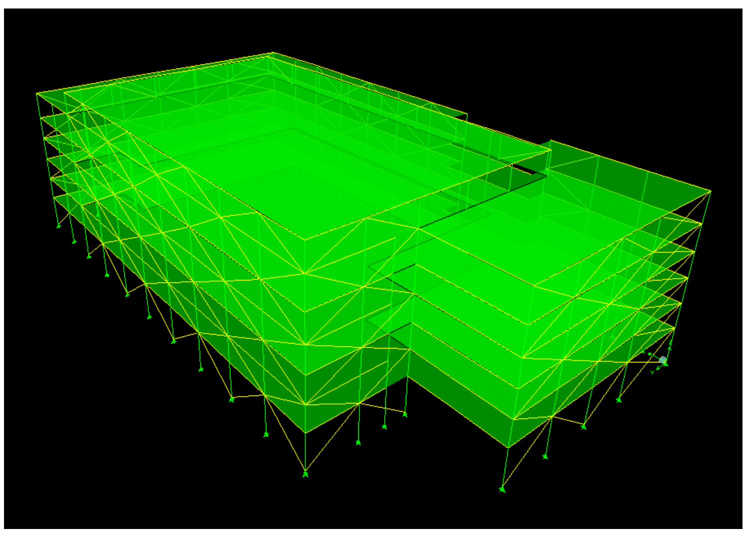


Figure 13: Preliminary Bracing Locations in ETABS I

The next check was for deflection limitations. Under wind loading, the deflection of the roof level was compared to $\frac{H}{400}$. The allowable limit for Helios Plaza was 3.39" and this was far exceeded by the building with a deflection of over 4.5" in the North-South direction. Brace sizes in the North-South direction were increased to 12 Standard pipe above the third floor and to 12 x pipe from the ground to the third floor. Heavier members (the 12x pipes) were used in the lower stories because these levels saw the most interstory drift. The effect of increasing the brace size was a decreased deflection, but the value still did not fall below the acceptable maximum.

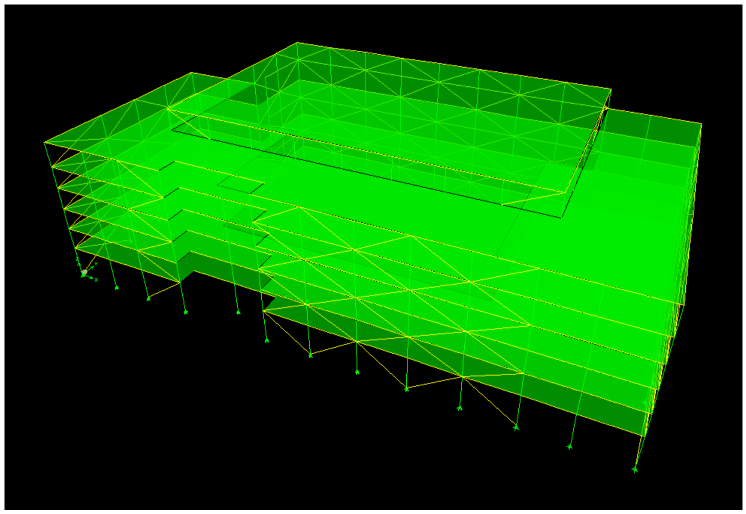


Figure 14: Preliminary Bracing Locations in ETABS II

At this point, 12 x pipe was the maximum strength pipe that still met slenderness requirements. This meant that either more frames were needed or a different section type needed to be picked as bracing members to lower the deflection of the building to an acceptable level. As mentioned before, the steel pipe sections were chosen for aesthetic purposes and were part of the architectural breadth of this thesis, so the option of more frames was chosen. Placement of the braces would ultimately affect the floor plan of the building so a maximum amount of stiffness per floor area was a priority. To achieve this, x-bracing was placed in two bays that had as minimal effect on the floor plan as possible. The braces would occur on every floor up until the sixth floor and would be 12 Standard pipe sections. The end result of the addition of these frames was a maximum deflection of 3.24".

Relative Frame Stiffness

With the frames finalized for strength and serviceability requirements, the relative stiffness of the frames was able to be determined. A 1000 kip load was assessed at the top of each frame and the deflections recorded for each frame. With this data, the relative stiffness of each frame could be compared for distribution of forces within the building.

The locations of each of the frames in plan can be seen in Figure 15 for the North-South direction and in Figure 16 for the East-West Direction. A summary of the relative stiffness of each frame can be seen in Table 4 and 5.

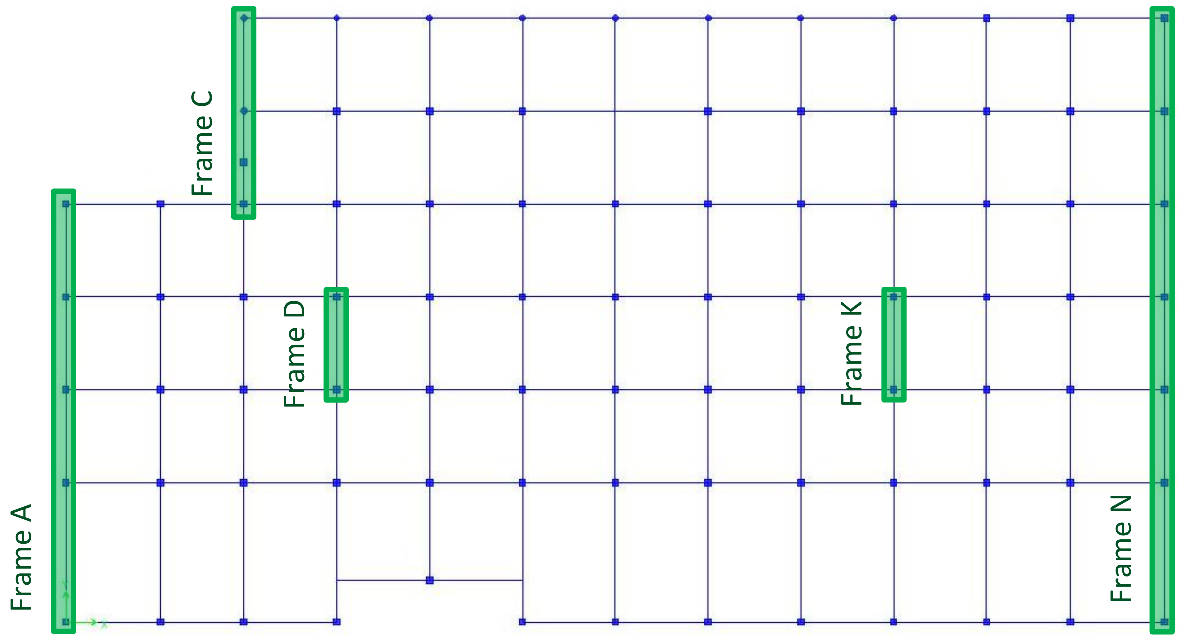


Figure 15: North- South Braced Frame Locations

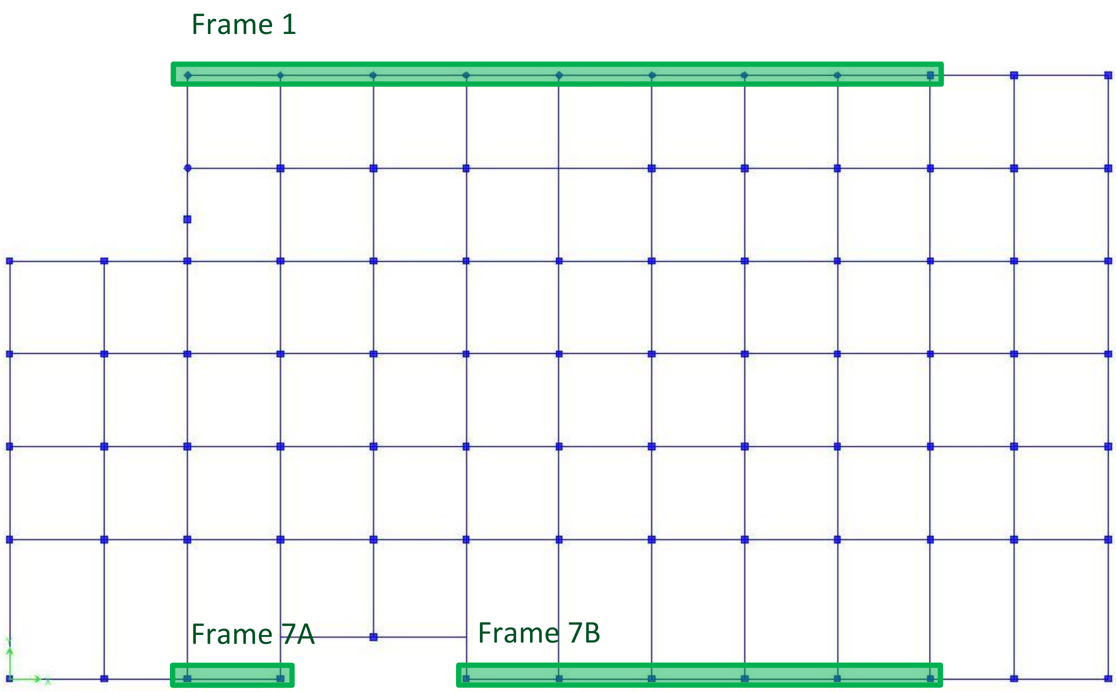


Figure 16: East-West Braced Frame Locations

1000 k Load In Y-Direction				
Frame	Δ	K (k/in)	$K_{relative}$	$K_{relative}$ (%)
A	9.650	103.6	0.2834	28.34
C	22.906	43.7	0.1194	11.94
D	31.919	31.3	0.0857	8.57
K	31.912	31.3	0.0857	8.57
N	6.423	155.7	0.4258	42.58
	Total	365.6	1	100

Table 4: Relative Stiffness in North- South Direction

1000 k Load In X-Direction				
Frame	Δ	K (k/in)	$K_{relative}$	$K_{relative}$ (%)
1	6.965	143.6	0.5934	59.34
7A	73.507	13.6	0.0562	5.62
7B	11.797	84.8	0.3504	35.04
	Total	241.9	1	100

Table 5: Relative Stiffness in East-West Direction

Despite the approximate ten percent imbalance in the relative stiffness in the x-direction the effects were not strong enough to cause serious torsional problems in the redesign of Helios Plaza. Analysis showed that the mode shapes for both mode one and two were x- and y-translation respectively. One effect that the stiffness imbalance did have was one column's local axis needed to be rotated. Once this rotation was applied to the member, the member was well under 1.0 for the combined axial and bending interaction equation.

Controlling Load Cases

Analysis of the redesigned system yielded the controlling loading cases as load combination 5 in the East-West direction and load combination 4 in the North-South direction. Load combination 5 means seismic controls in the x-direction and load combination 4 means that wind controls in the y-direction. As compared with Technical Report III, the redesign of the structure made no impact on the controlling load cases. This result was unfortunate since it negated one of the goals of the redesign, which was to eliminate the seismic control of the building in the East-West direction. The goal was technically met since the controlling base shear in the East-West direction was wind, but the design of the lateral system was still based upon the seismic forces.

Brace Connection Design

An investigation was performed to determine a potential connection method for the circular steel pipe braces to the wide flange beam column connection. The connection was designed to be as easily constructible as possible. Several types of steel connections were employed between the many elements involved in the interface. The following connections were designed:

1) Slotted steel pipe welded to gusset plate;
2) Gusset plate welded to beam flange;
3) Gusset plate welded to double angle;
4) Double angle bolted to column flange;
5) End plate welded to beam flanges and web; and
6) End plate bolted to column flange.

Refer to Appendix C for calculations. The end result can be seen in Figure 17.

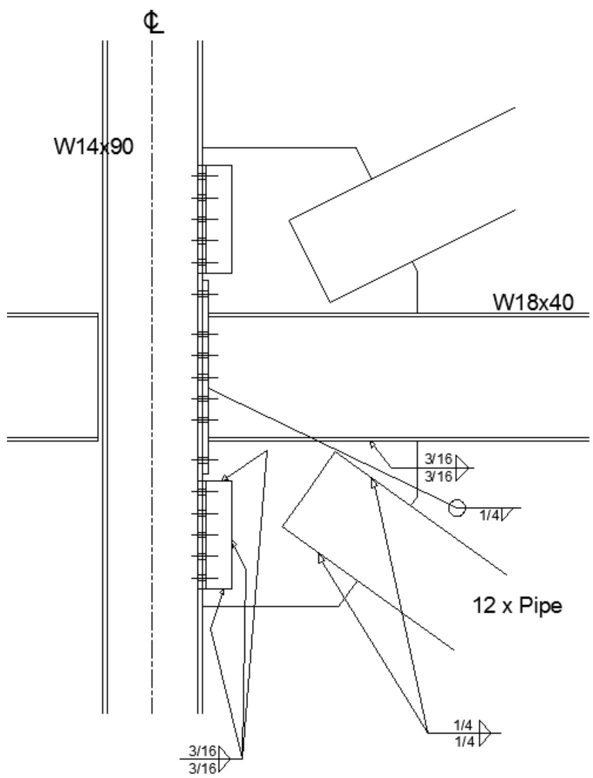


Figure 17: Steel Brace Connection

In designing the connection, several constructability issues became apparent. The low slope of the majority of the braces makes non-eccentric connections extremely unviable to erect. Calculations performed on the above connection showed that for the bottom brace to be non-eccentric by the uniform force method, the length of the weld for the beam to gusset plate connection needed to be 47.2”. This long of a connection could lead to major interruption of other trades plenum spaces and for braces that frame into the base of columns, interruptions in the floor plan.

The configuration in Figure 17 resulted in the least impact on the other spaces of Helios Plaza, but the line of action of the braces does not pass through the centroid of the beam to column connection. For this to occur in the top braces, the length of gusset plate would appear as in Figure 18. The length of the gusset plate in this instance is just short of 3'-4".

The best way for the connection to reduce the gusset plate size would be to have a β value of 7.04" for the bottom connection; however, this is not possible. The length of the double angle connection needs to be 15" to avoid tensile rupture of the bolts connecting it to the column flanges. The prying action of the double angle severely limits the ability to shorten this connection length.

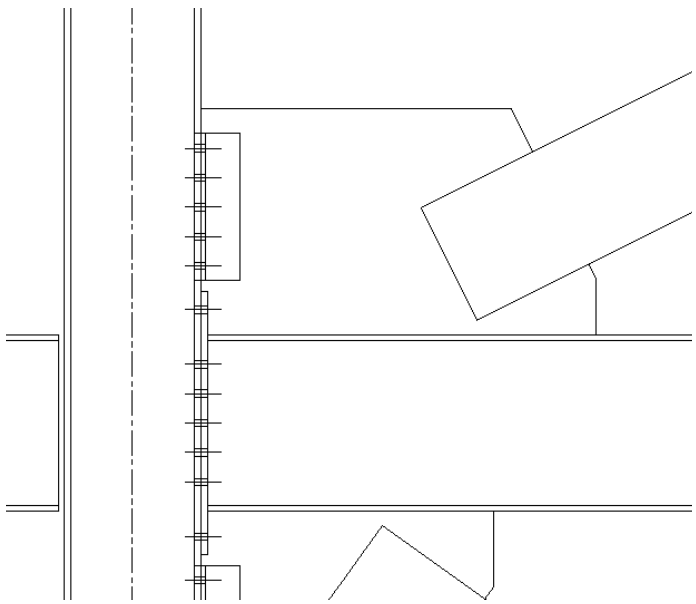


Figure 18: Upper Brace Configuration for Non-eccentric Connection

Based upon these findings, it would seem that switching the connections to welded moment connections could be more advantageous in both terms of constructability and cost. The connection that was designed has many components that have very specific tolerance that may be hard to meet in the field. While some parts can be attached in the shop for speed of erection, such as the double angles on the column and the gusset plates on the beams, this would seem to create a much more expensive connection due to components. One of the main reasons for avoiding moment connections is to limit the amount of welding that needs to be performed in the field, but this connection still requires field welding of the steel pipe to the gusset plate. This weld is much easier to complete due to the separation from nearby elements, but the time and labor involved is still substantial.

Architectural Breadth – Brace Selection and Layout

One of the main design decisions made in this thesis redesign was the bracing selection. To marry the architecture of the existing spaces, a circular section was chosen because one of the focal points of the building's design is the circular steel columns and beams located in the trading floors. Figure 19 is a picture of one of the trading floors. The large open spaces were a design goal of both the owner and the architect and the large columns that are sparingly used stand out as features in this space.

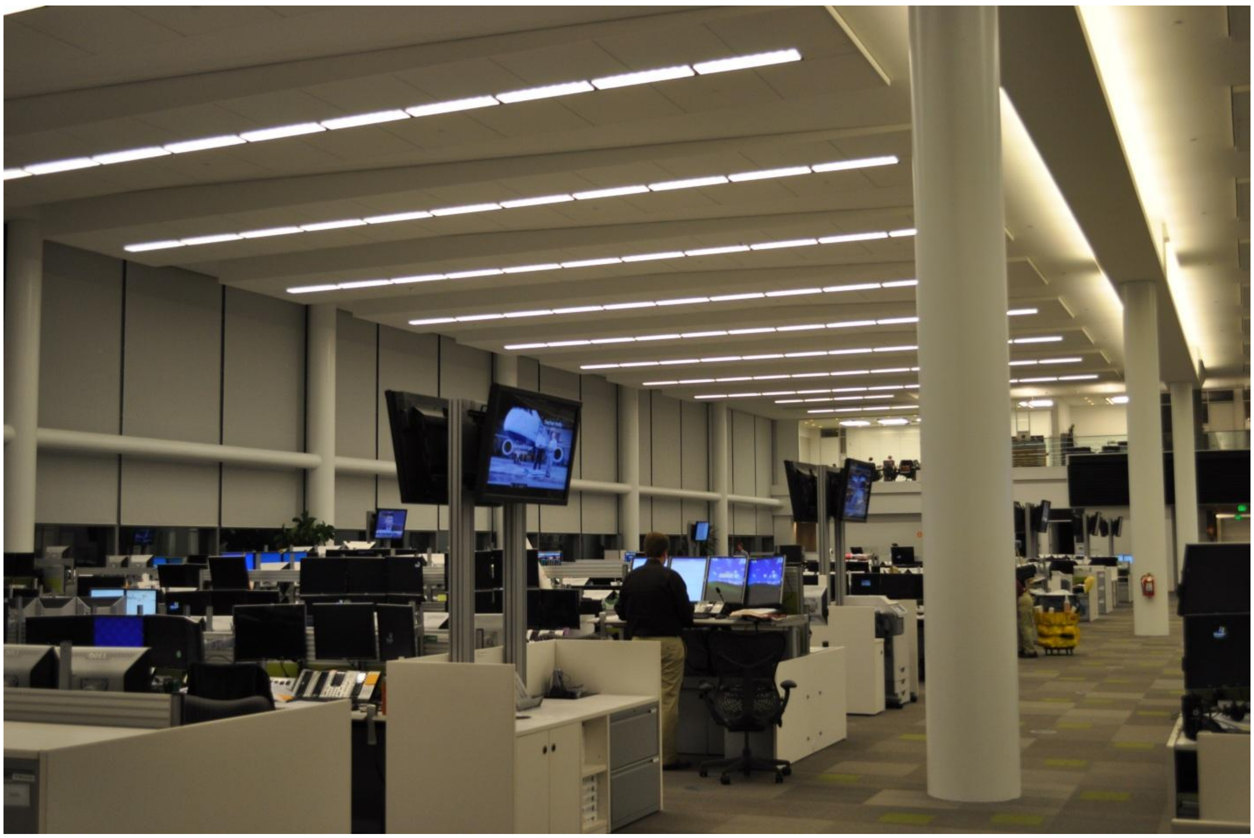


Figure 19: Helios Plaza Trading Floor

The visibility of the braces in these spaces warranted similar geometric properties, hence the design decision for steel pipe. Several configurations for the braces were explored for aesthetic purposes and a diamond pattern was chosen to create a simple repeating geometry in the space. Initial brace design can be seen in Figure 20.

To confirm the choice of the diamond pattern, a Revit model was created to explore the space. Once the existing conditions were created, the braces were added to the exterior wall line and a rendering was of the space was run. Figure 21 shows the outcome of the rendering. The connections in the rendering are clean moment connections, despite modeling them as pin connections and designing the connection as a pin connection as a structural depth.

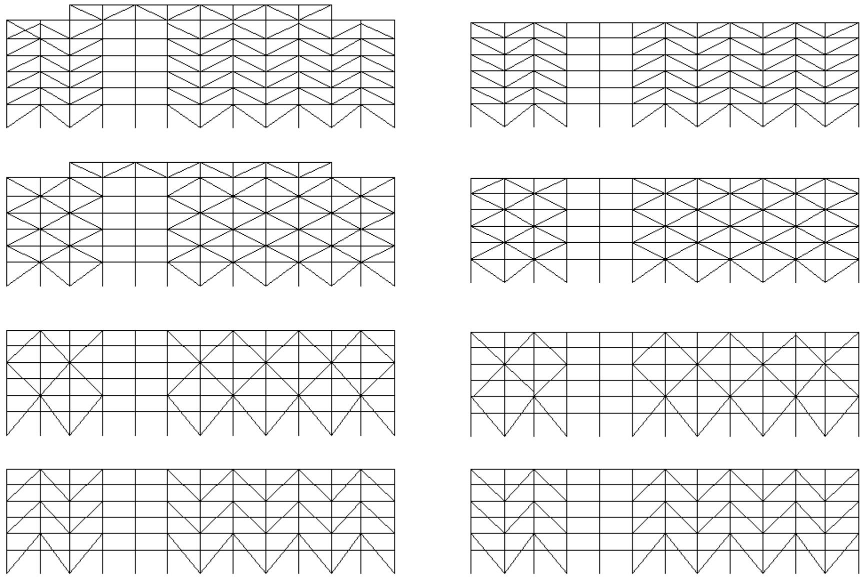


Figure 20: Initial Brace Configuration Considerations

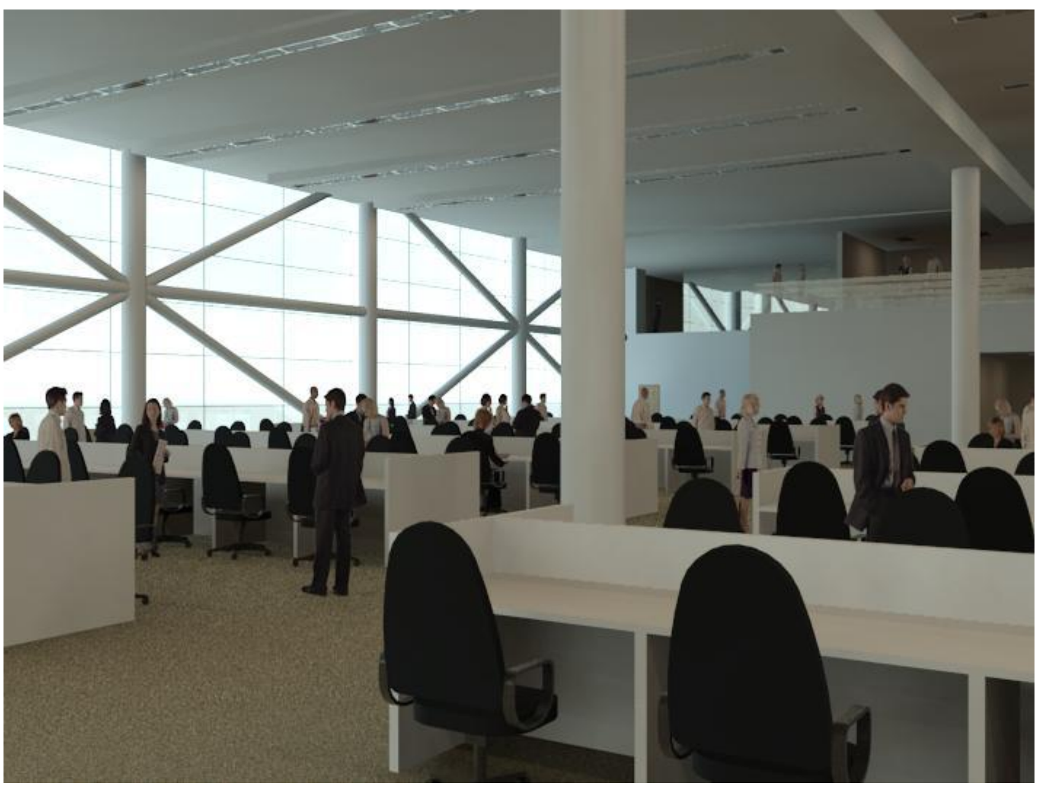


Figure 21: Interior Rendering of Trading Floor with Diagonal Braces

In an attempt to minimize the impact on the floor plan of the building, the locations of the braced frames were located originally on the exterior only. As discussed before, deflection criteria led placing braced frames on the interior of the building. To try and maintain the goal of minimizing floor plan impacts in this thesis, a location was picked that had limited traffic in the building. Due to the nature of x-braces, there would be no way for any people to pass through the chosen bays. The area least likely to be affected by the braces can be seen in Figure 22 called out in red.

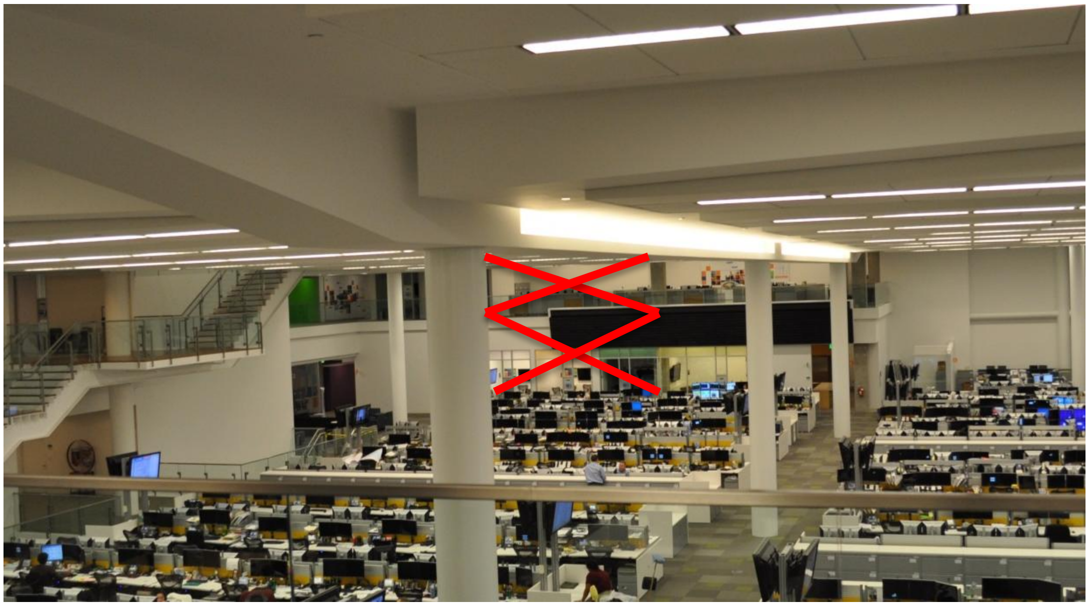


Figure 22: Area of Interest for Interior Braces

At the edge of the trading floor spaces, there is a perimeter walkway on the story above. Unfortunately, these braces will affect the floor plan of the trading floor spaces since they block the entrance into a conference room. There is potential for this room to have its entrance rerouted to the other side and to even keep the glass in place for a viewport into the trading floor. Because the trading floors only occur every other level, this limits the amount of floor plan that is hindered by the braces and could even be a feature of the space.

Construction Management Breadth – Cost and Schedule

A key component in verifying the redesign is whether or not it is an economically viable solution. Investigations in Technical Report II showed that there would be an increase in structure cost with the switch from one-way concrete pan joists to a composite steel deck system. These investigations were based off of RS Means assemblies costing information, which are very generalized and not particularly accurate when the assembly does not match the bay dimensions very well. To determine a more accurate cost difference, a detailed estimate was performed using RS Means CostWorks.

Foundation Reduction

Before the detailed estimate was performed, an investigation was performed on whether significant cost savings could be had from reducing the size of the foundations based upon the lighter weight of the redesign. The switch to steel resulted in a 39.5% reduction in the building weight, amounting to 11,678 kips.

The redesign of the foundation proceeded with determining which footings were most likely to have significant reduction potential. The footings investigated can be seen in Figure 23. Output from ETABS was drawn upon to determine the controlling load case for axial force in the columns above the footing of interest and analysis was performed in by hand to see if reductions could be made.

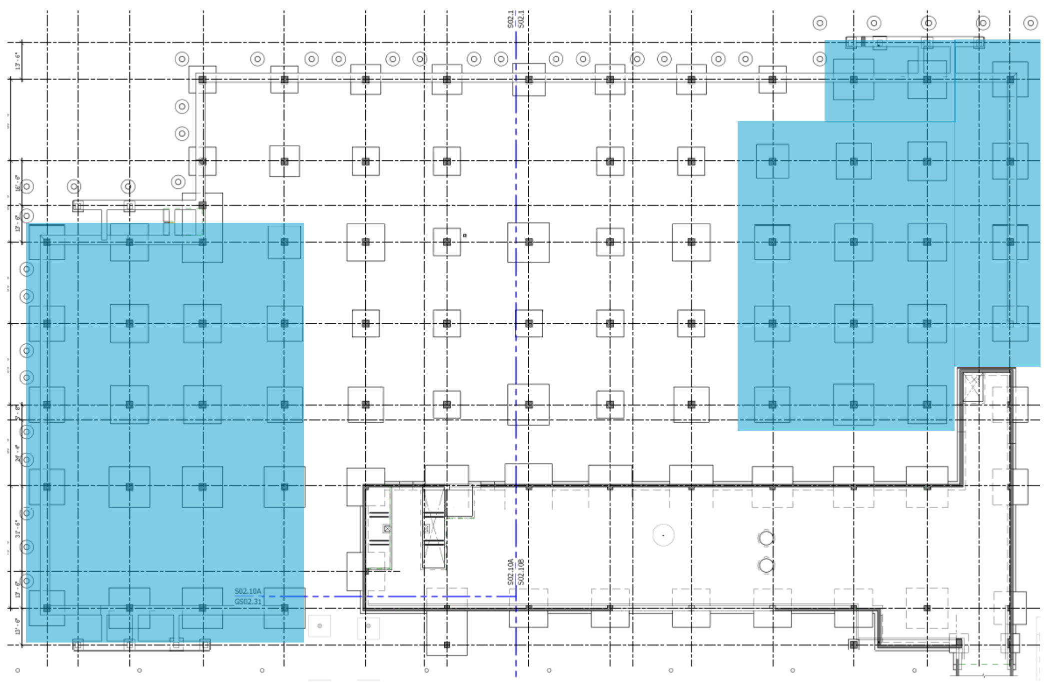


Figure 23: Foundations of Interest for Redesign

For design of the footings, an allowance of 500 psf was made for hydrostatic pressure, leaving the allowable bearing capacity of the soil at 6000 psf. Based upon the loads determined in ETABS, the require area of footing was calculated to keep the amount of force in the soil below 6000 psf. The area was then converted into square dimensions and rounded up to the nearest foot. These dimensions were then used to find the corresponding footing already designed for this project in the technical documents. The results of the foundation reductions can be seen in Table 6.

Foundation Savings			
	Concrete (CY)	Formwork (SFCA)	Cost
Original Design	2012	16131	$ 319,779.62
Redesign	1756	14355	$ 280,321.94
Savings	256	1776	$ 39,457.68

Table 6: Foundation Redesign Savings

Superstructure Cost Comparison

The savings for the foundation reduction were not significant enough to offset the switch to a composite concrete system. After the foundations were investigated, detailed estimates of the entire superstructure of both designs were prepared and the results can be seen in Table 7. Cost information was retrieved from RS Means for the majority of materials and processes, but some material information was not directly available. In these instances, costs and daily outputs were interpolated or extrapolated from similar materials to arrive at a reasonable value. For full cost analysis, refer to Appendix D. The cost difference between the two superstructures can be explained almost entirely by the applications of fireproofing; it alone accounted for $709,220 of the steel superstructure cost.

Superstructure Cost		
	Cost	Cost (O & P)
Original Design	$ 5,887,030.09	$ 7,254,951.27
Redesign	$ 6,866,659.78	$ 8,002,677.32
Savings	$ (979,629.70)	$ (747,726.05)

Table 7: Overall Superstructure Cost

Schedule Comparison

With the costing completed, the schedules for the two superstructures were compiled and compared. Several assumptions were made when determining the construction durations of certain tasks. In regards to the steel superstructure, four crews were used standardly to get building output. This assumption was related to the assumption that two cranes would be used to construct the superstructure. Once the erection times were compiled, they were sequenced in Microsoft Project. The steel superstructure schedule can be found in Appendix E.

The construction process for the existing concrete structure was much more involved than the redesign since the formwork process needed to be staggered to achieve remotely comparable construction times. Eight crews were used standardly for the erection of the concrete superstructure. Another assumption was that the building would be divided into three parts for the placing of formwork and concrete, with two of the sections being larger than the third. The proposed separations can be seen in Figure 24.

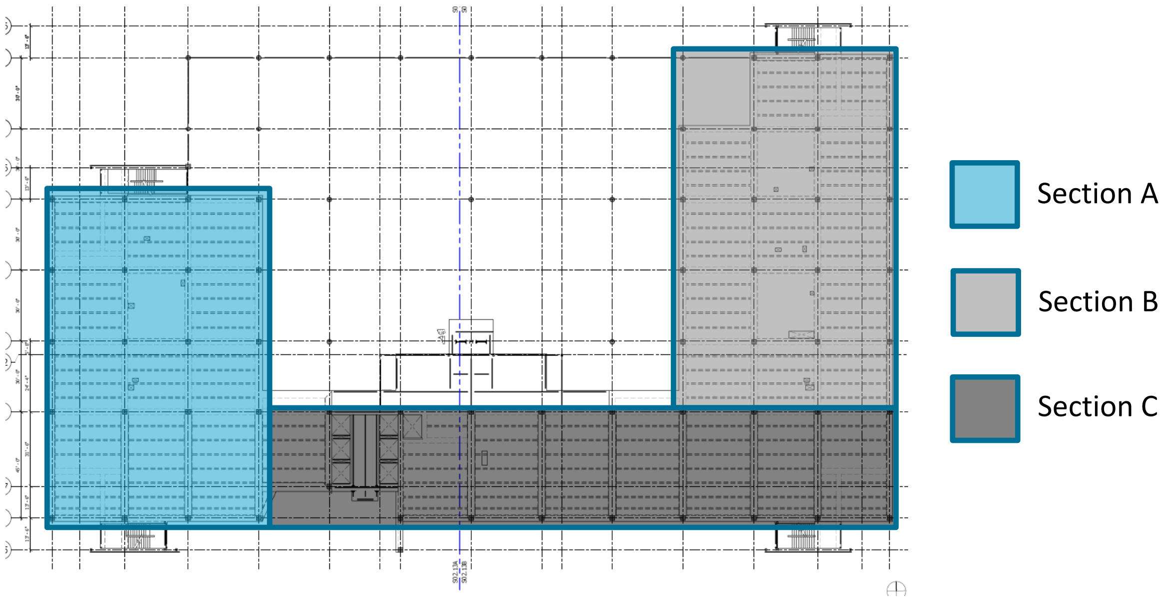


Figure 24: Proposed Concrete Pour Separations

A total superstructure build time of 143 days was achieved for the steel redesign as compared with the build time of 194 days for the concrete structure. This time saving is significant since the amount of labor used in the construction of the original concrete structure is double what is used in the steel construction.

MAE Considerations

Throughout the design process, specific tasks were completed with MAE coursework as the knowledge foundation. For accurate modeling of the structures in the various software programs, principles and guidelines from the Computer Modeling of Building Structures class were utilized. In regards to the design of the lateral bracing system, guidelines learned in the Earthquake Resistant Design of Buildings course were utilized for proper strength, local buckling, and slenderness requirements. Finally, the Design of Steel Connections course notes were crucial in the design of the brace to column to beam connection. All three of these courses were helpful aids in expanding practical Master's knowledge into this thesis report.

Conclusion

The redesign of Helios Plaza from a mainly concrete moment frame system to a concentrically braced steel frame system managed to achieve four of its initial six goals. The goals achieved were:

1) Reducing the overall building weight;
2) Minimizing floor plan impacts;
3) Creating aesthetically compatible braces; and
4) Reducing the construction schedule.

The two goals that were not met were eliminating the seismic control of forces in the East-West direction and offsetting the cost of the steel structure by reducing the amount of foundations needed. As part of the investigations, a typical steel connection involving all types of members in Helios Plaza was designed.

The design and analysis of the steel structure showed that despite large weight savings, the controlling load cases stayed the same in each direction. Deflection criteria were of particular importance in this design since the building had a relatively soft design.

Architectural concepts explore that had the connections been designated as moment connection and welded in place, the aesthetic of the trading floors would have been upheld. With the welded connections, the impact on the floor plan would have been minimal.

The benefits of switching to steel were decreased schedule time and a nearly comparable cost. If further analysis were to be carried out on labor costs as a function of building time, the gap between costs could potentially close substantially.

Issues with the design that became apparent during analysis were related to the brace orientations. The slope of the braces was shallow enough that the connections would be very large and would certainly affect the floor plan and construction process of the building. The apparent solution to this problem is to make all of the connections welded.

Overall, the design was effective at resisting all loads placed upon it and would be a viable alternative to the existing structure.

References

American Concrete Institute, 2008, Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, Farmington Hills, MI.

American Institute of Steel Construction, Inc., 2005, *Steel Construction Manual*, 13th Edition, Chicago, Illinois.

American Institute of Steel Construction, Inc., 2005, Seismic Provisions for Structural Steel Buildings, AISC 341-05, Chicago, IL.

American Society of Civil Engineers, 2010, *Minimum Design Loads for Building and Other Structures,* ASCE 7-10, Reston, VA

Geschwindner, Louis F., 2008, *Unified Design of Steel Structures,* John Wiley & Sons, Inc., Hoboken, NJ

RS Means, 2010, *Assemblies Cost Data,* 34th Edition, Reed Construction Data, Kinston, Ma.

Appendix

Appendix A: Existing Structural Floor Plans

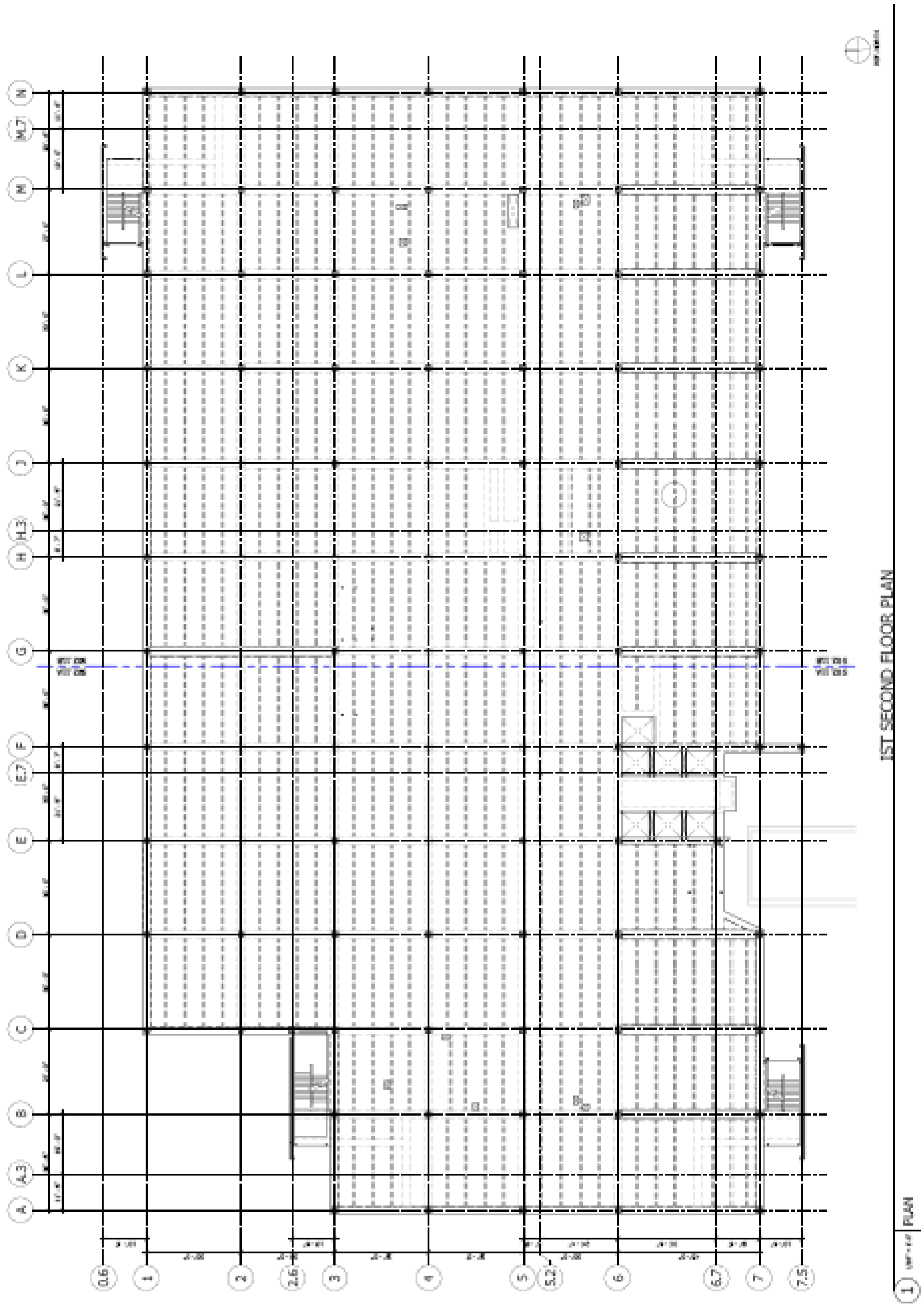


Figure 25: Original Second Floor Plan

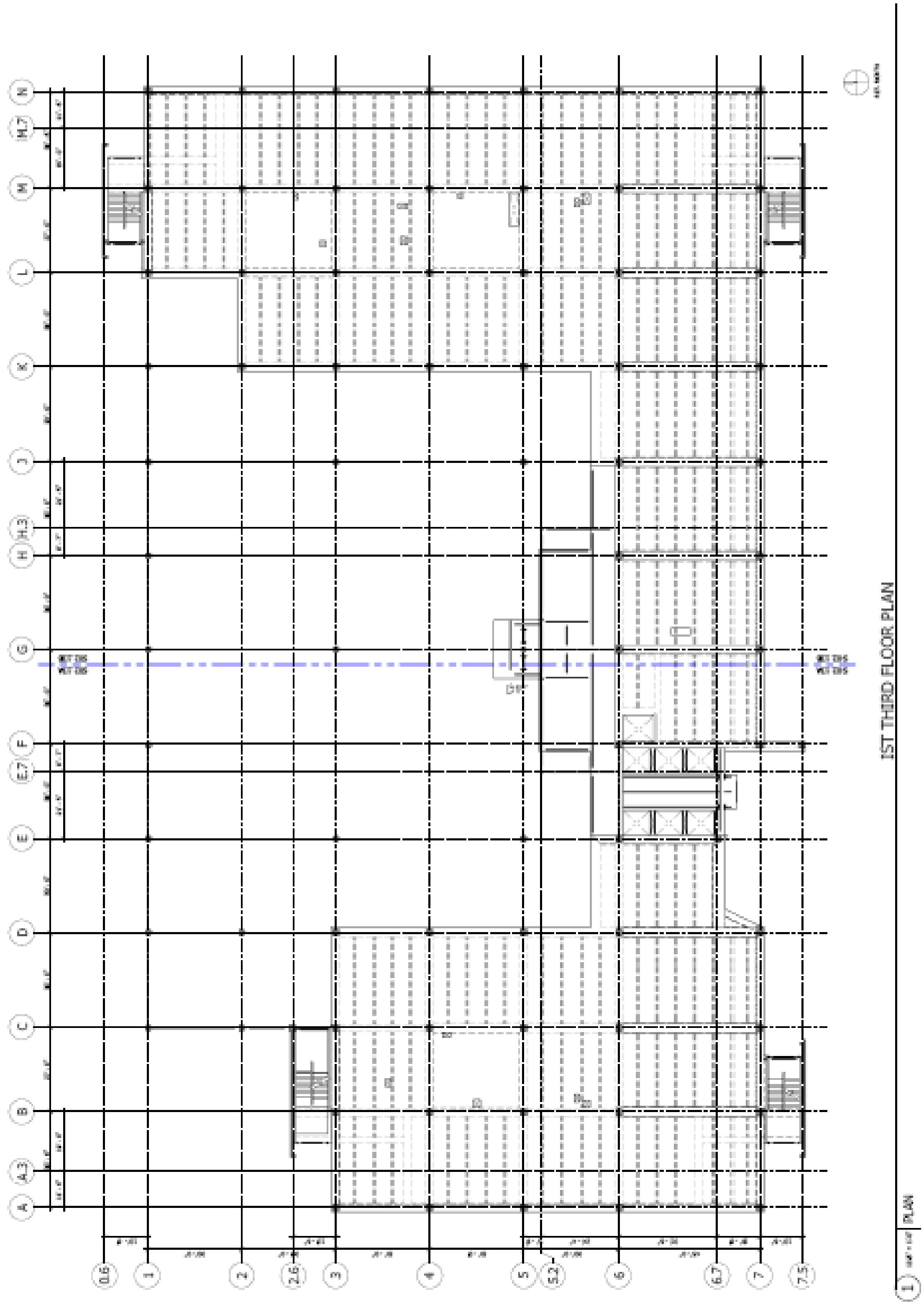


Figure 26: Original Third Floor Plan

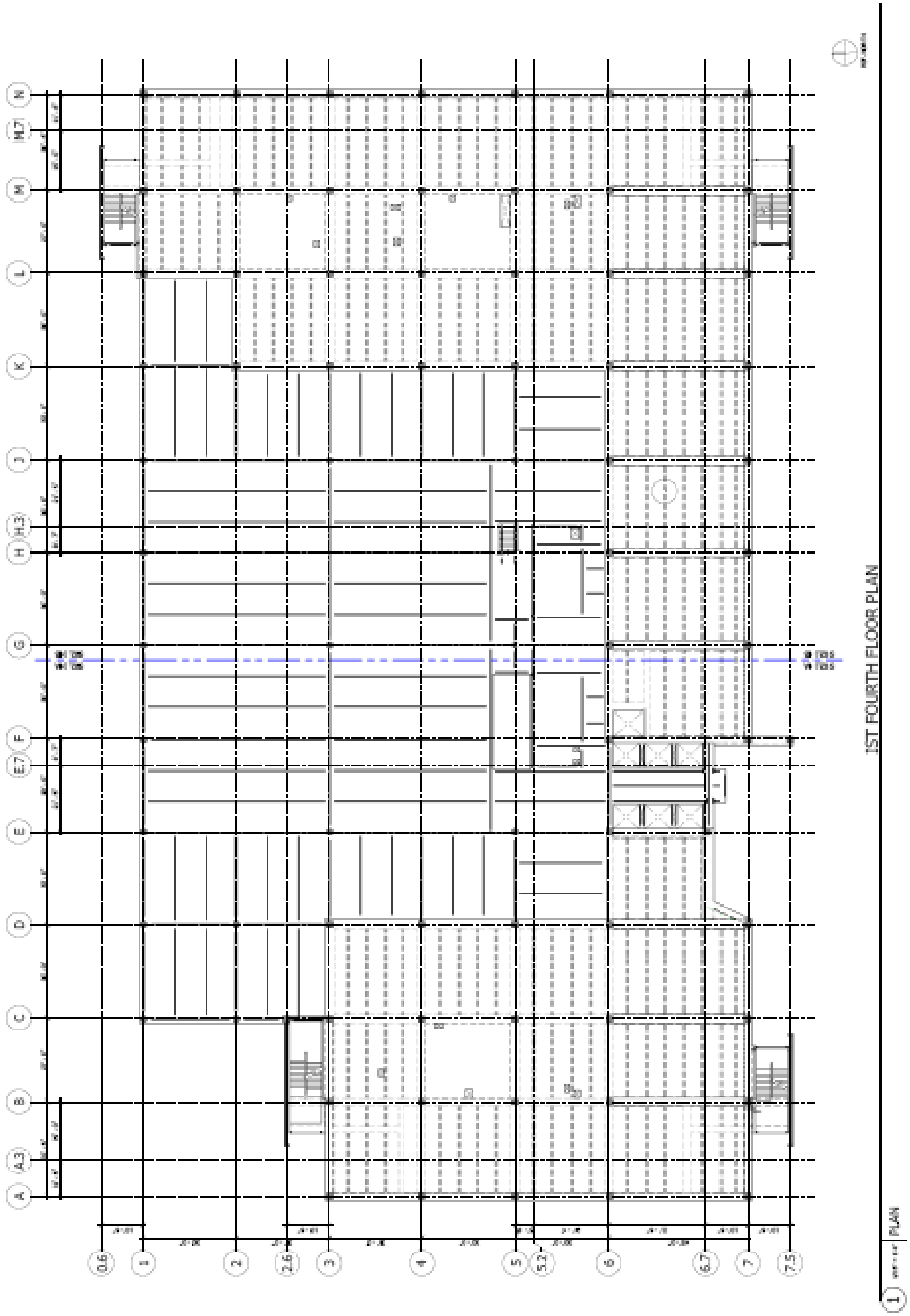
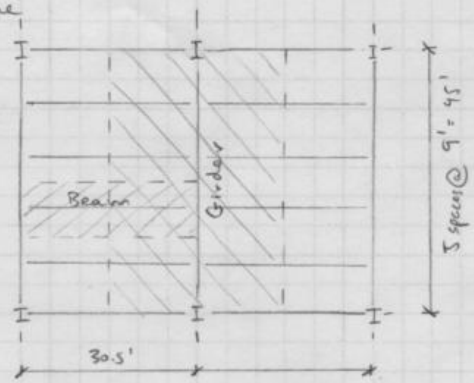


Figure 27: Original Fourth Floor Plan

Appendix B: Redesign Hand Calculations

Kevin Zinsmeister	Thesis Calcs	45' span sections	1/4

Deck Design: Assume



$K_{LL} A_T = 1(9)30.5 = 274.5\ ft^2 < 400\ ft^2$ ∴ live load reduction not allowed

$w_u = 15 + 80 + 37 = 132$ psf

∴ Pick 1.5 VL 17 w/ 3 1/4" LWC topping (4 3/4" thickness total)

Check Unshored Clear Span (7 spans): 10'-6" > 9'-0" ∴ okay

Check Load Capacity: 184 psf > 132 psf ∴ okay

Beam Design

$K_{LL} A_T = 2(9)30.5 = 549\ ft^2 > 400\ ft^2$ ∴ live load reduction allowed

$LL = 80\left(0.25 + \frac{15}{\sqrt{549}}\right) = 71.2$ psf

$w_u = 1.2(15+37+5) + 1.6(71.2) = 182.3$ psf $\Rightarrow w_u = 9(182.3) = 1.64$ klf

$M_u = \frac{wl^2}{8} = \frac{1.64(30.5)^2}{8} = 190.7\ ^{'k}$

Assume a Y2 value using a = 1.0 in, $Y2 = 3.75 - \frac{1.0}{2} = 3.25$ in ∴ round down to 3 in

From Table 3-19 pick W12x30 w/ $M_u = 220\ ^{'k}$ & $\Sigma Q_n = 110\ ^k$

$b_{eff} = \min \begin{cases} 2(span/8) = 30.5(12)/4 = 91.5" \Leftarrow \text{controls} \\ 2(s/2) = 9(12) = 108" \end{cases}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{110}{0.85(3.5)91.5} = 0.40" < 1.5"$ assumed "a" value ∴ okay

Kevin Zinsmeister | Thesis Calcs | 45' span sections | 2/4

Check Bare Steel Strength: Use DL & Construction live load

$w_u = 1.2(15+37+5) + 1.6(20) = 100.4$ psf $\Rightarrow w_u = 100.4(9) = 0.90$ klf $\Rightarrow M_u = \frac{0.9(30.5)^2}{8} = 104.7'^k$

W12x30 has $\phi M_n = 162'^k > 104.7'^k$ ∴ okay

Check $\Delta_{LL} = \frac{l}{360} = \frac{30.5(12)}{360} = 1.02''$

From Table 3-20, $I_{LB} = 386$ in^4

$$\Delta_{LL} = \frac{5 w_{LL} l^4}{384EI} = \frac{5(0.71)30.5^4(1728)}{384(29000)386} = 1.23'' > 1.02'' \therefore \text{not okay}$$

$$I_{req,LB} = \frac{5 w_{LL} l^4}{384 E \Delta_{LL}} = \frac{5(0.71)30.5^4(1728)}{384(29000)1.02} = 467 \text{ in}^4$$

Check $\Delta_{WC} = \frac{l}{240} = \frac{30.5(12)}{240} = 1.53''$

$$I_{req,BB} = \frac{5(0.57)30.5^4(1728)}{384(29000)1.53} = 250 \text{ in}^4$$

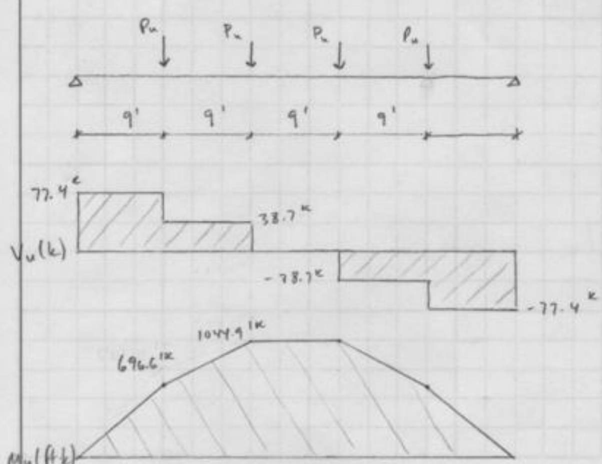
∴ Pick W16x26 w/ $I_{LB} = 482$ in^4 > 467 in^4 ∴ okay for Δ_{LL}
$I_{BB} = 301$ in^4 > 250 in^4 ∴ okay for Δ_{WC}
$\phi M_n = 230'^k > 190.7'^k$ ∴ okay for composite strength
$\phi M_n = 166'^k > 104.7'^k$ ∴ okay for bare beam strength

$\Sigma Q_n = 96.0^k$ ∴ # studs $= \frac{96}{14.6} = 6.6$ studs ∴ 14 studs for full beam span

<u>Girder Design</u>

$K_{LL}A_T = 80\left(0.25 + \frac{15}{\sqrt{2(28.75)30}}\right) = 48.9$ psf

$w_u = 1.2(57) + 1.6(48.9) = 146.6$ psf, $P_u = 146.6(9)28.75 + 26(28.75) = 38.7^k$



Kevin Zinsmeister	Thesis Calcs	45' Span Sections	3/4

From Table 3-19 pick W24×84 w/ $\phi M_n = 1110^{'k}$; $\Sigma Q_n = 309^k$

$$b' = \min \begin{cases} \text{span}/8 = 45(12)/8 = 67.5" \\ \ell_2/2 = 30.5(12)/2 = 183" \\ \ell_1/2 = 27(12)/2 = 162" \end{cases} \quad \therefore \ b_{eff} = 2(67.5) = 135"$$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{309}{0.85(3.5)135} = 0.77" < 1.5" \ \therefore \text{ choice is conservative}$$

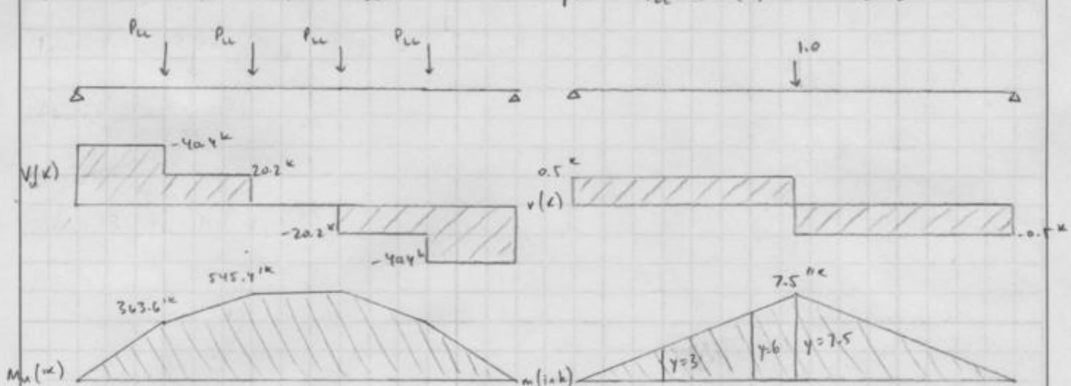
Check Bare Steel Strength: Use DL & Construction Live Load

$w_u = 1.2(57) + 1.6(20) = 100.4$ psf , $P_u = 100.4(9)\,28.75 = 26.0^k$

$M_u = 52(9) + 26(9) = 702^{'k}$ W24×84 has $\phi M_n = 804^{'k} > 702^{'k}$ ∴ okay

Check $\Delta_{LL} \le \frac{\ell}{360} = \frac{45(12)}{360} = 1.5"$

Virtual work for Δ_{max}: $w_{LL} = 1.6(48.9) = 78.2$ psf → $P_{LL} = 78.2(9)28.75 = 20.2^k$



$$\Delta_{max} = \left[\frac{\frac{1}{2}(3)9(363.6)9\left(\frac{2}{3}\right)}{EI} + \frac{\left[\frac{1}{2}(3)9 + 3(9)\right]\left[(545.4-363.6)\frac{1}{2}9 + 363.6(9)\right]}{EI} + \frac{\left[\frac{1}{2}(1.5)4.5 + 6(4.5)\right]545.4(4.5)}{EI}\right]2 \leftarrow \text{symmetry}$$

$$\Delta_{max} = \left[\frac{22088.7}{EI} + \frac{165665.25}{EI} + \frac{74549.3625}{EI}\right]2 = \frac{524606.625\ ft^2k^2(144)}{29000(3490)} = 0.75" < 1.5" \ \therefore \text{okay}$$

From Table 3-20, $I_{LB} = 3490\ in^4$

Check $\Delta_{WC} \le \frac{\ell}{240} = \frac{45(12)}{240} = 2.25"$, $P_{WC} = 57(9)28.75 = 14.8^k$

$$\Delta_{max} = \left[\frac{13.5(29.6)9\left(\frac{2}{3}\right)}{EI} + \frac{40.5\left[\left(\frac{1}{2}\right)133.2(9) + 266.4(9)\right]}{EI} + \frac{399.6(4.5)30.375}{EI}\right]2 = \left[\frac{16183.8}{EI} + \frac{121378.5}{EI} + \frac{54620.325}{EI}\right]2$$

$$\Delta_{max} = \frac{384365.25(144)}{29000(2370)} = 0.81" < 2.25" \ \therefore \text{okay}$$

Kevin Zinsmeister | Thesis Calcs | 45' span sections | 4/4

Check $\Delta_{TL} \leq 2.25"$

$$\Delta_{max} = \left[\frac{13.5\left(\frac{1}{3}\right)696.6(9)}{EI} + \frac{40.5\left[\frac{1}{2}(348.3)9 + 9(696.6)\right]}{EI} + \frac{30.375(1044.9)4.5}{EI} \right] 2$$

$$\Delta_{max} = \left[\frac{42318.45 + 317388.375 + 142824.7688}{EI} \right] 2 = \frac{1005063.138(144)}{29000(3490)} = 1.43" < 2.25" \therefore \text{okay}$$

Kevin Zinsmeister | Thesis Calcs | Column Gravity Design | 1/2

<u>DL</u> Roof: 30 psf   <u>LL</u>: office: 80 psf   wall: 2

Floors: 57 psf + 8 psf   Mechanical: 100 psf (unreduceable)

<u>Column A7 @ 5</u>

$A_T = \left(\frac{45}{2}\right)\left(\frac{30.5}{2}\right) = 343$ ф $LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(343)}}\right) = 80(0.655) = 52.4$ psf

$P_L = 52.4(343) + 100(343) = 52.3^k$ $P_D = 65(343)2 + 388\left(\frac{45}{2} + \frac{30.5}{2}\right) + 400\left(\frac{45}{2} + \frac{30.5}{2}\right) = 74.3^k$

$P_u = 1.2D + 1.6L = 1.2(74.3) + 1.6(52.3) = 173^k$ ∴ $\boxed{W10\times33}$ From Table 4-1

<u>Column B6 @ 5</u>

$A_T = \left(\frac{45+30}{2}\right)\left(\frac{30.5+27}{2}\right) = 1078$ ф $LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(1078)}}\right) = 80(0.478) = 38.3$ psf

$P_L = 38.3(1078) + 100(1078) = 149^k$ $P_D = 65(1078)2 = 140^k$

$P_u = 1.2D + 1.6L = 1.2(140) + 1.6(149) = 406^k$ $\boxed{W10\times49}$ From Table 4-1

<u>Column A5 @ 5</u>

$A_T = 30\left(\frac{30.5}{2}\right) = 458$ ф $LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(458)}}\right) = 80(0.601) = 48.1$ psf

$P_L = 48.1(458) + 100(458) = 46.3^k$ $P_D = 65(458)2 + 388(30) + 400(30) = 53.4^k$

$P_u = 1.2D + 1.6L = 1.2(53.4) + 1.6(46.3) = 138^k$ $\boxed{W10\times33}$ From Table 4-1

<u>Column A7 @ 3</u>

$LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(343)3}}\right) = 80(0.484) = 38.7$ psf

$P_L = 38.7(343)3 + 100(343) = 74.1^k$ $P_D = 65(343)4 + (388 + 400 + 2(375))(37.75) = 147^k$

$P_u = 1.2D + 1.6L = 1.2(147) + 1.6(74.1) = 295^k$ $\boxed{W10\times45}$ From Table 4-1

<u>Column B6 @ 3</u>

$LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(343)3}}\right) = 80(0.382) < 0.4$ ∴ $= 80(0.4) = 32$ psf

$P_L = 32(1078)3 + 100(1078) = 211^k$ $P_D = 65(1078)4 = 280^k$

$P_L = 1.2(280) + 1.6(211) = 674^k$ $\boxed{W12\times72}$ From Table 4-1

Kevin Zinsmeister | Thesis Calcs | Column Gravity Design | 2/2

Column A5 @ 3

$LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(458)3}}\right) = 80(0.452) = 36.2 \text{ psf}$

$P_L = 36.2(458)3 + 100(458) = 95.5^k$ $P_D = 65(458)4 = 119^k$

$P_u = 1.2(119) + 1.6(95.5) = 296^k$ $\boxed{W10\times45}$ From Table 4-1

Column A7 @ Grand

$LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(343)5}}\right) = 80(0.431) = 34.5 \text{ psf}$

$P_L = 34.5(343)5 + 100(343) = 93.5^k$ $P_D = 65(343)6 = 134^k$

$P_u = 1.2(134) + 1.6(93.5) = 310^k$ $\boxed{W10\times54}$ From Table 4-1

Column B6 @ Ground

$P_L = 32(1078)5 + 100(1078) = 280^k$ $P_D = 65(1078)6 = 420^k$

$P_u = 1.2(420) + 1.6(280) = 952^k$ $\boxed{W14\times109}$ From Table 4-1

Column A5 @ Ground

$LL_{red} = 80\left(0.25 + \frac{15}{\sqrt{4(458)5}}\right) = 80(0.407) = 32.5 \text{ psf}$

$P_L = 32.5(458)5 + 100(458) = 120^k$ $P_D = 65(458)6 = 179^k$

$P_u = 1.2(179) + 1.6(120) = 407^k$ $\boxed{W12\times65}$ From Table 4-1

Kevin Zinsmeister	Thesis Calcs	Brace Design	1/3

EW South Facade

1&2 $P_u = 1.2D + 1.0W + 0.5L = 1.2\left(\frac{30.5+27}{2}\right)\frac{45}{2}(0.057)2 + \frac{773.9}{2(5)} + \frac{650.5}{2(5)} + 0.5(646.875)(0.08)(2)$

$$P_u = 1.2(36.9)2 + 77.4 + 65.1 + 0.5(51.75)2 = 282.8^k$$

$T_u = 0.7D + 1.0W = 0.7(36.9)2 - (77.4 + 65.1) = -90.9^k$

$L_{unbraced} = 28.02\text{ ft} \sim 28\text{ ft}$

Try Pipe 8 x-Strong w/ $A_g = 11.9\text{ in}^2$ $t_{nom} = 0.5$ $t_{des} = 0.465$

$D = 8.63\text{ in}$, $r = 2.89\text{ in}$

Check local Buckling

$\lambda \le \lambda_{ps} \Rightarrow \lambda = 18.6 < 36.5 = \lambda_{ps}$ ∴ okay

$\lambda = \frac{D}{t} = \frac{8.63}{0.465} = 18.6$, $\lambda_{ps} = 0.044\frac{E}{F_y} = \frac{0.044(29000)}{35} = 36.5$

Check Slenderness

$\frac{KL}{r} \le 4.0\sqrt{\frac{E}{F_y}} \Rightarrow \frac{KL}{r} = 116.3 \nleq 115 = 4.0\sqrt{\frac{E}{F_y}}$ ∴ try Pipe 10 x-strong

$\frac{KL}{r} = \frac{1.0(28)12}{2.89} = 116.3$, $4.0\sqrt{\frac{E}{F_y}} = 4.0\sqrt{\frac{29000}{35}} = 115$

$A_g = 15.0\text{ in}^2$, $t_{nom} = 0.5\text{ in}$, $t_{design} = 0.465\text{ in}$, $D = 10.8\text{ in}$, $r = 3.64\text{ in}$

Check Local Buckling

$\lambda = D/t = 10.8/0.465 = 23.2 < 36.5 = \lambda_{ps}$ ∴ okay

Check Slenderness

$\frac{KL}{r} = \frac{1.0(28)12}{3.64} = 92.3 < 115 = 4.0\sqrt{\frac{E}{F_y}}$ ∴ okay

Compressive Strength

Check $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}}$, $4.71 > 4.0$ ∴ okay

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{92.3^2} = 33.6\text{ ksi}$

Kevin Zinsmeister | Thesis Calcs | Brace Design | 2/3

$F_{cr} = \left(0.658^{F_y/F_e}\right) F_y = \left(0.658^{35/33.6}\right) 35 = 22.6$ ksi

$\phi P_n = 0.9 F_{cr} A_g = 0.9(22.6)15.0 = 305.1^k > 282.8^k = P_u \therefore$ okay

Tensile Strength

$\phi P_n = 0.9 F_y A_g = 0.9(35)15 = 472.5^k > 90.9^k \therefore$ okay

3 & 4 $\quad P_u = 1.2(646.875)0.057(2) + 54.0 + 42.4 + 0.5(646.875)0.08/(2) = 236.6^k$

$T_u = 0.7(646.875)0.057(2) - (54.0 + 42.4) = -44.8^k$

$L_{unbraced} = 21.4$ ft

Try Pipe 8 x-strong w/ $A_g = 11.9\ in^2$, $t_{nom} = 0.5$ in, $t_{design} = 0.465$ in

$D = 8.63$ in, $r = 2.89$ in

Check Local Buckling

$\lambda = D/t = 8.63/0.465 = 18.6 < 36.5 = \lambda_{ps} \therefore$ okay

Check Slenderness

$\frac{KL}{r} = \frac{1.0(21.4)12}{2.89} = 88.9 < 115 = 4.0\sqrt{\frac{E}{F_y}} \therefore$ okay

Compressive Strength

$\frac{KL}{r} < 4.71\sqrt{\frac{E}{F_y}} \therefore F_{cr} = \left(0.658^{F_y/F_e}\right) F_y = \left(0.658^{35/36.2}\right) 35 = 23.35$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{88.9^2} = 36.2$ ksi

$\phi P_n = 0.9 F_{cr} A_g = 0.9(23.35)11.9 = 250.0^k > 236.6^k \therefore$ okay

Tensile Strength

$\phi P_n = 0.9 F_y A_g = 0.9(35)11.9 = 374.9^k > 44.8^k \therefore$ okay

5 & 6 $\quad P_u = 1.2(646.875)0.057(2) + 16.8 + 30.2 + 0.5[646.875(0.08+0.1)] = 193.7^k$

$T_u = 0.7(646.875)0.057(2) - (16.8 + 30.2) = 4.6^k$

$L_{unbraced} = 21.8$ ft

Try Pipe 6 x-strong w/ $A_g = 7.88\ in^2$, $t_{nom} = 0.432$ in, $t_{design} = 0.403$ in

$D = 6.63$ in, $r = 2.20$ in

Kevin Zinsmeister | Thesis Calcs | Brace Design | 3/3

Check Local Buckling

$\lambda = D/t = 6.63/0.403 = 16.5 < 36.5 = \lambda_{ps}$ ∴ okay

Check Slenderness

$\frac{KL}{r} = \frac{1.0(21.8)12}{2.2} = 118.9 \nless 115 = 4.0\sqrt{\frac{E}{F_y}}$ ∴ not okay

∴ Pick Pipe 8 std.

$A_g = 7.85\ in^2$, $t_{nom} = 0.322\ in$, $t_{des} = 0.300\ in$, $D = 8.63\ in$, $r = 2.95\ in$

Check Local Buckling

$\lambda = D/t = 8.63/0.3 = 28.8 < 36.5 = \lambda_{ps}$ ∴ okay

Check Slenderness

$\frac{KL}{r} = \frac{1.0(21.8)12}{2.95} = 88.7 < 115 = 4.0\sqrt{\frac{E}{F_y}}$ ∴ okay

Compressive Strength

$\frac{KL}{r} < 4.71\sqrt{\frac{E}{F_y}}$ ∴ $F_{cr} = \left(0.658^{F_y/F_e}\right)F_y = \left(0.658^{35/36.4}\right)35 = 23.4\ ksi$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{88.7^2} = 36.4$

$\phi P_n = 0.9 F_{cr} A_g = 0.9(23.4)7.85 = 165.3^k < 193.7^k$ ∴ Try Pipe 10 std.

$A_g = 11.1\ in^2$, $t_{nom} = 0.365\ in$, $t_{des} = 0.340\ in$, $D = 10.8$, $r = 3.68\ in$

Check Local Buckling

$\lambda = D/t = 10.8/0.34 = 31.8 < 36.5 = \lambda_{ps}$ ∴ okay

Check Slenderness

$KL/r = 1.0(21.8)12/3.68 = 71.1 < 115 = 4.0\sqrt{\frac{E}{F_y}}$ ∴ okay

Compressive Strength

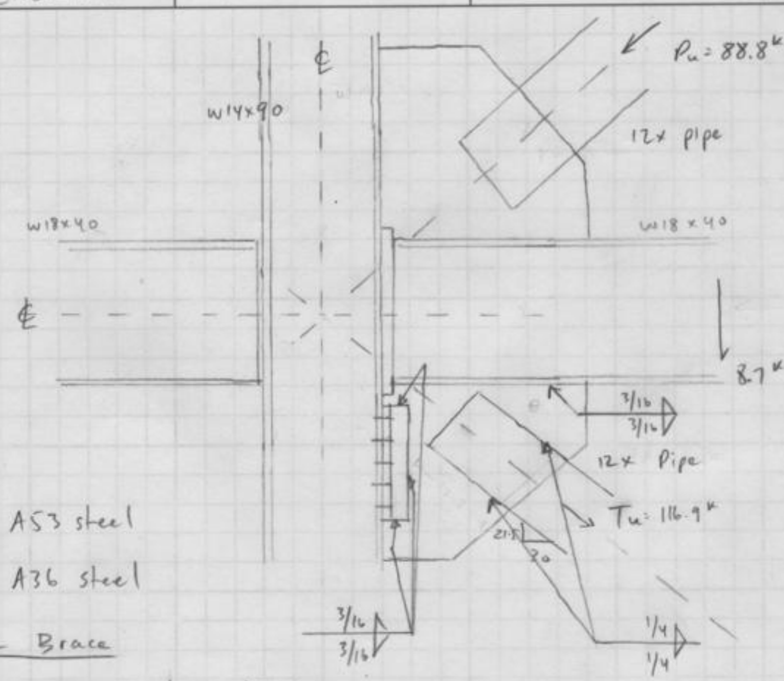
$KL/r < 4.71\sqrt{\frac{E}{F_y}}$ ∴ $F_{cr} = 0.658^{F_y/F_e} F_y = \left(0.658^{35/56.6}\right)35 = 27.0$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{71.1^2} = 56.6$

$\phi P_n = 0.9 F_{cr} A_g = 0.9(27.0)11.1 = 269.7^k > 193.$

Appendix C: Brace Connection Design

Kevin Zinsmeister | Thesis Calcs | Brace Connection Design | 1/6



Brace: A53 steel

Plate: A36 steel

Check Brace

Tension Yielding: $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(35)\,17.9 = 563.9^k > 116.9^k \therefore$ okay

Tension Rupture: $\phi R_n = \phi F_u A_e$

$A_e = U A_n$, where $U = 1 - \bar{x}/\ell$ since $D \le \ell \le 1.3D$ (From Table D3.1)

$\bar{x} = D/\pi = 12.8/\pi = 4.07$

$U = 1 - 4.07/12.8 = 0.68$

$A_e = 0.68(17.9) = 12.2\ in^2 \quad \therefore \phi R_n = 0.75(60)12.2 = 549^k > 116.9^k \therefore$ okay

Check Brace & Plate

Weld Rupture: $\phi R_n = 1.392(D)L_w$

Determine weld size, D: $t_{pipe} = 0.465 \ge 0.25$

$\therefore t_{w\,max} = t_p - 1/16\ in = 0.465 - 1/16 = 0.4025''$

$1.392D(12.8)\underbrace{2}_{\#\text{ of welds}} \ge 116.9^k \quad \therefore D \ge 3.3 \quad \therefore$ Pick $t_w = 4/16'' = 1/4''$

Kevin Zinsmeister | Thesis Calcs | Brace Connection Design | 2/6

Since $t_w = 1/4"$, $t_{gusset} \geq t_p + 1/16" = 5/16"$

Base Metal Strength: $\phi R_n = 0.75(0.6 F_u A_{nw})$

Plate: $\phi R_n = 0.75(0.6)58(5/16)12.8(\underset{\#\text{ of planes}}{2}) = 208.8^k > 116.9^k$ ∴ okay

Brace: $\phi R_n = 0.75(0.6)60(0.465)12.8(4) = 642.8 > 116.9^k$ ∴ okay

Plate Yielding: $\phi R_n = \phi F_y A_g$

$A_g = 5/16(12.8) = 4 \text{ in}^2$

$\phi R_n = 0.9(36)4 = 129.6^k > 116.9^k$ ∴ okay

Plate Rupture: $\phi R_n = \phi F_u A_e$

$A_e = A_g$

$\phi R_n = 0.75(58)4 = 174^k > 116.9^k$ ∴ okay

Plate Buckling:

$\frac{kl}{r} = \frac{0.5(12)}{5/16} = 19.2 < 25$ ∴ From Section J4.4 $P_n = F_y A_g$

$\phi P_n = \phi F_y A_g = 0.9(36)5/16(12.8) = 129.6 > 116.9^k$ ∴ okay

Gusset to Beam Connection

Weld Rupture: $\phi R_n = 1.392(D)L_w(1 + 0.5\sin^{1.5}\theta)$

$\phi R_n = 1.392(3)24(1 + 0.5\sin^{1.5}35.6)$

$= 1.392(3)24(1.22) = 122.5^k > 116.9^k$ ∴ okay

Base Metal Strength: $\phi R_n = 0.75(0.6 F_u A_{nw})$

Plate: $\phi R_n = 0.75(0.6)58(3/16)24 = 117.6^k > 116.9^k$ ∴ okay

Beam: $\phi R_n = 0.75(0.6)65(0.525)24 = 368.6^k > 116.9^k$ ∴ okay

Beam Web Yielding: $R_n = (5k + N)F_{yw}t_w$

$\phi R_n = 1.0(50)[5(0.927) + (5/16)]0.315 = 77.9^k > 68.1^k$ ∴ okay

68.1^k = brace vertical compressive force

Kevin Zinsmeister	Thesis Calcs	Brace Connection Design	3/6

Beam Web Crippling: $R_n = 0.8\, t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$

$\phi R_n = 0.75(0.8)\, 0.315^2 \left[1 + 3\left(\frac{5/16}{17.9}\right)\left(\frac{0.315}{0.525}\right)^{1.5}\right]\sqrt{\frac{29000(50)\, 0.525}{0.315}}$

$\phi R_n = 94.8^k > 68.1^k \therefore$ okay

Gusset to Column Connection

$T_u = 95^k$, $V_u = 68.1 + 8.7 = 76.8^k$

Pick $3/4''\ \phi$ A-325N bolts w/ $\phi r_n = 15.9^k$, $\phi r_{nt} = 29.8^k$

$\frac{76.8}{15.9} = 4.8$, $\frac{95}{29.8} = 3.2 \therefore$ Try 2 rows of 4 bolts

Check Prying: $F'_t = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v$

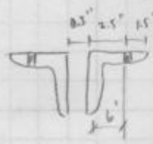
$F'_t = 1.3(90) - \frac{90}{0.75(48)}\left[\frac{76.8}{2(4)\, 0.442}\right] = 62.7^{ksi} < 90^{ksi}$

$\therefore \phi r_{nt} = 0.75(62.7)\, 0.442 = 20.79^k$, $r_{ut} = \frac{95}{8} = 11.88^k$

Try 2 - L4x4x1/2

p_{min} = | gage: $2.5(2) + 0.5 = 5.5$ | $3(4) - 3\ /4 = 3.75 \Leftarrow$ controls

$\phi M_{n1} = \frac{(0.9)\, 58\, (0.5)^2\, 3.75}{4} = 12.2$



$t = 1/2''$

$a = 1.5''$

$b = 2.5 - 0.25 = 2.25$, $b' = 2.25 - 3/8 = 1.88$

$r_{ut} b' = 11.9(1.88) = 22.4 > \phi M_{n1} = 12.2 \therefore$ prying will occur

$\phi M_{n2} = 12.2\left(1 - \frac{7/8}{2}\right) = 6.9$

$\frac{\phi M_{n1}}{b'} = \frac{12.2}{1.88} = 6.5 < r_{ut} = 11.9 > \frac{\phi M_{n1} + \phi M_{n2}}{b'} = 10.2 \therefore$ bolts will rupture

Add more bolts, Try 2 rows of 5 bolts

Kevin Zinsmeister	Thesis Calcs	Brace Connection Design	4/6

$F'_t = 1.3(90) - \frac{90}{0.75(48)}\left[\frac{76.3}{2(5)\,0.442}\right] = 73.6^{ksi} < 90^{ksi}$

$\phi r_{nt} = 0.75(73.6)0.442 = 24.4^k$, $r_{ut} = \frac{95}{10} = 9.5^k$

$p = 3(5)+3/5 = 3.6$, $\phi M_{n_1} = \frac{0.9(58)0.5^2(3.6)}{4} = 11.7$

$r_{ut}b' = 9.5(1.88) = 17.7 > \phi M_{n_1} = 11.7$ ∴ prying will occur

$\phi M_{n_2} = 11.7(1 - \frac{7/8}{2}) = 6.6$

$\frac{\phi M_{n_1}}{b'} = \frac{11.7}{1.88} = 6.2 < r_{ut} = 9.5^k < \frac{\phi M_1 + \phi M_2}{b'} = \frac{18.3}{1.88} = 9.7$

∴ flange yields at first hinge, but has sufficient strength to carry r_{ut}

Determine bolt strength:

$a = \min \begin{cases} a = 1.5 \leftarrow \text{controls} \\ 1.25b = 1.25(2.25) = 2.8125 \end{cases}$

$a' = 1.5 + 3/8 = 1.875$, $q_u = \frac{9.5(1.88) - 11.7}{1.88} = 3.3$

$r_{ub} = 9.5 + 3.3 = 12.8^k \leq \phi r_{nt} = 24.4^k$ ∴ okay

Check Angle

Shear Yielding: $\phi V_n = \phi(0.6)F_y L_p t_p$

$\phi V_n = 1.0(0.6)36(18)0.5\,\underbrace{(2)}_{\#\text{ of angles}} = 388.8^k > 68.1 + 7.6 = 75.7^k$ ∴ okay

Shear Rupture: $\phi V_n = \phi(0.6)F_u A_n$

$A_n = 18(0.5) - 5(3/4 + 1/8)0.5 = 6.8\ in^2$

$\phi V_n = 0.75(0.6)6.8(58) = 177.5^k > 75.7^k$ ∴ okay

Block Shear:

From Table 9.3 $\begin{cases} a = 46.2 \\ b = 219 \leftarrow \text{controls} \\ c = 250 \end{cases}$

∴ $\phi V_n = 2(46.2 + 219)0.5 = 265.2^k > 75.7^k$ ∴ okay

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Kevin Zinsmeister | Thesis Calcs | Brace Connection Design | 5/6

Check Bolts:

Bearing & Tearout:

Angle: $\phi r_n = 0.75(2.4) d_b F_u t = 0.75(2.4)\,3/4\,(58)\,0.5 = 39.2^k$ (B)

$\phi r_n = 0.75(1.2) L_c F_u t = 0.75(1.2)\left[1.5 - \frac{0.875}{2}\right] 58(0.5) = 27.7^k$ (To)

Column: $\phi r_n = 0.75(2.4)\,3/4\,(65)\,0.710 = 62.3^k$ (B)

$\phi r_n = 0.75(1.2)\left[1.5 - \frac{0.875}{2}\right] 65(0.710) = 44.1^k$ (To)

other: $\phi r_n = 0.75(1.2)\left[3 - \frac{0.75 + 1/16}{2}\right] 58(0.5) = 57.2^k$ (To)

$\therefore \phi r_n = 4(39.2) + 27.7 = 184.5^k > 75.7^k \therefore$ okay

Check Plate to Angle

Weld Rupture: $\phi R_n = 1.392(D)L_w$

$\phi R_n = 1.392(3)18(2) = 150.3^k > 116.9^k \therefore$ okay

Base Metal Strength: $\phi R_n = \phi(0.6) F_u A_{nw}$

Angle: $0.75(0.6)\,58\,(18)(0.5)2 = 469.8^k > 116.9^k \therefore$ okay

Plate: $0.75(0.6)\,58(18)\,5/16 = 146.8^k > 116.9^k \therefore$ okay

Plate Yielding: $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(36)\,5/16\,(18) = 182.3^k > 116.9^k \therefore$ okay

Plate Rupture: $\phi R_n = \phi F_u A_e$

$\phi R_n = 0.75(58)\,5/16\,(18) = 146.8^k > 116.9^k \therefore$ okay

Beam to Column Connection

$t_{w\,min} = 1/4''$

Weld Rupture: $\phi R_n = 1.392(D)L_w(1.5)$

$\phi R_n = 1.392(4)[2(6.02) + 15.5]\,1.5 = 230^k > 95^k \therefore$ okay
transverse weld

Base Metal Strength $\phi R_n = \phi(0.6)F_u A_{nw}$

Beam: $0.75(0.6)\,65\,[(12.04)\,0.525 + 15.5(0.315)] = 327.7^k > 95^k \therefore$ okay

Plate: $0.75(0.6)\,58(27.5)(1/4 + 1/16) = 224.3^k > 95^k \therefore$ okay

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Kevin Zinsmeister	Thesis Calcs	Brace Connection Design	6/6

Beam Tension Yielding: $\phi R_n = \phi F_y A_g$

$\phi R_n = 0.9(50)\,11.8 = 531^k > 95^k \therefore$ okay

Beam Tension Rupture: $\phi R_n = \phi F_u A_e$

$A_e = U A_g = 1.0 A_g \quad \therefore \quad \phi R_n = 0.75(65)\,11.8 = 575^k > 95^k \therefore$ okay

Check End Plate

$T_u = 95^k, \quad V_u = 76.8^k$

Pick 3/4" ϕ A325N bolts w/ $\phi r_n = 15.9^k$, $\phi_{rt} = 29.8^k$, 2 rows of 5 bolts

$F'_t = 1.3(90) - \frac{90}{36}\left[\frac{76.8}{10/(0.442)}\right] = 73.6^{ksi} < 90^{ksi}$

$\therefore \phi r_{nt} = 0.75(73.6)\,0.442 = 24.4^k, \quad r_{ut} = \frac{95}{10} = 9.5^k$

$p = (3(5)+3)/5 = 3.6, \quad \phi M_n = \frac{0.9(58)\,0.3125^2(3.6)}{4} = 4.6$

By inspection, increase plate thickness

Try 3/4" thick $\therefore \quad \phi M_n = \frac{0.9(58)\,3.6\,(.75)^2}{4} = 26.4^{"k}$

$b = 2.5 - 0.315/2 = 2.34 \quad b' = 2.34 - 3/8 = 1.97$

$r_{ut} b' = 1.97(9.5) = 18.7^{"k} < \phi M_n = 26.4^{"k} \therefore$ prying does not occur

Shear Yielding: $\phi V_n = \phi(0.6)\,F_y L_p t_p$

$\phi V_n = 1.0(0.6)\,36(18)\,0.75(2) = 583.2^k > 76.8^k$

Shear Rupture $= \phi V_n = \phi(0.6)\,F_u A_n$

$A_n = 18(0.75) - 5(3/4 + 1/8)\,0.75 = 10.2 \text{ in}^2$

$\phi V_n = 0.75(0.6)\,10.2\,(58) = 266.2^k > 76.8^k$

Block Shear: Not possible with this arrangement

Bolt strength:

From Table 7-5, Column Bearing $= 0.710(87.8) = 62.3^k$
Plate Bearing $= 0.75(78.3) = 58.7 \in$ controls
From Table 7-6 Plate Tearout $= 0.75(44) = 33^k$

$\phi R_n = 33 + 4(58.7) = 267.8^k > 76.8^k \therefore$ okay

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Kevin Zinsmeister | Thesis Calcs | Uniform Force Method | 1/1

$\beta = 12 + 1 = 13"$

$e_b = \frac{17.9}{2} = 8.95"$

$e_c = \frac{14}{2} = 7"$

$\tan\theta = \frac{30}{21.5} = 1.395$

$\alpha - \beta\tan\theta = e_b\tan\theta - e_c$

$\alpha = e_b\tan\theta - e_c + \beta\tan\theta = 8.95(1.395) + 13(1.395) - 7 = 23.6"$

Results in 45.7" weld ∴ Move bolted connection down

$\alpha_{min} = 0.75 + \frac{29.125}{2} = 15.3125$

$\beta\tan\theta = e_c - e_b\tan\theta + \alpha$

$\beta = \frac{e_c + \alpha - e_b\tan\theta}{\tan\theta} = \frac{7 + 15.3125 - 8.95(1.395)}{1.395} = 7.04"$

∴ Not possible ∴ Use $\alpha = 23.6"$, or have eccentric connection

Appendix D: Cost Analysis

Quantity	LineNumber	Description	Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Total	Total O&P	Cost	Cost O&P
231120	031113353600	C.I.P. concrete forms, elevated s	C2	475	0.101	S.F.	$ 2.95	$ 2.68	$ -	$ 5.63	$ 7.36	$ 1,301,205.60	$ 1,701,043.20
26850.8125	031113201600	C.I.P. concrete forms, beams an	C2	315	0.152	SFCA	$ 0.74	$ 4.05	$ -	$ 4.79	$ 7.01	$ 128,615.39	$ 188,224.20
93365	031113202600	C.I.P. concrete forms, beams an	C2	385	0.125	SFCA	$ 0.75	$ 3.32	$ -	$ 4.07	$ 5.88	$ 379,995.55	$ 548,986.20
59144	031113256650	C.I.P. concrete forms, column, s	C1	238	0.134	SFCA	$ 0.73	$ 3.48	$ -	$ 4.21	$ 6.15	$ 248,996.24	$ 363,735.60
5500	031113500050	C.I.P. concrete forms, grade bea	C2	580	0.083	SFCA	$ 1.20	$ 2.20	$ -	$ 3.40	$ 4.68	$ 18,700.00	$ 25,740.00
16131	031113455050	C.I.P. concrete forms, footing, s	C1	371	0.086	SFCA	$ 0.93	$ 2.23	$ -	$ 3.16	$ 4.44	$ 50,973.96	$ 71,621.64
3774.333333	033105350300	Structural concrete, ready mix,				C.Y.	$ 98.67	$ -	$ -	$ 98.67	$ 108.25	$ 372,413.47	$ 408,571.58
233.5135135	033105350400	Structural concrete, ready mix,				C.Y.	$ 106.34	$ -	$ -	$ 106.34	$ 116.88	$ 24,831.83	$ 27,293.06
199.8918919	033105350411	Structural concrete, ready mix,				C.Y.	$ 121.67	$ -	$ -	$ 121.67	$ 133.16	$ 24,320.85	$ 26,617.60
838.2202202	033105700800	Structural concrete, placing, col	C20	92	0.696	C.Y.	$ -	$ 17.42	$ 8.43	$ 25.85	$ 35.91	$ 21,667.99	$ 30,100.49
2011.861111	033105702600	Structural concrete, placing, spr	C6	120	0.4	C.Y.	$ -	$ 9.77	$ 0.38	$ 10.15	$ 15.09	$ 20,420.39	$ 30,358.98
3926.290574	033105700050	Structural concrete, placing, be	C20	60	1.067	C.Y.	$ -	$ 26.64	$ 12.92	$ 39.56	$ 54.89	$ 155,324.06	$ 215,514.09
3033.333333	033105701400	Structural concrete, placing, ele	C20	140	0.457	C.Y.	$ -	$ 11.47	$ 5.52	$ 16.99	$ 23.53	$ 51,536.33	$ 71,374.33
7768.923907	033116100820	Structural concrete, ready mix,				C.Y.	$ 135.08	$ -	$ -	$ 135.08	$ 148.49	$ 1,049,426.24	$ 1,153,607.51
1001.157407	033105704300	Structural concrete, placing, slal	C6	110	0.436	C.Y.	$ -	$ 10.65	$ 0.41	$ 11.06	$ 16.50	$ 11,072.80	$ 16,519.10
356.4814815	033105703200	Structural concrete, placing, gra	C6	150	0.32	C.Y.	$ -	$ 7.82	$ 0.31	$ 8.13	$ 12.22	$ 2,898.19	$ 4,356.20
90	051223750600	Structural steel member, 100-to	E2	600	0.093	L.F.	$ 15.59	$ 3.95	$ 2.66	$ 22.20	$ 26.80	$ 1,998.00	$ 2,412.00
235.6666667	051223751300	Structural steel member, 100-to	E2	880	0.064	L.F.	$ 28.35	$ 2.69	$ 1.81	$ 32.85	$ 38.05	$ 7,741.65	$ 8,967.12
195.6666667	051223751900	Structural steel member, 100-to	E2	990	0.057	L.F.	$ 33.60	$ 2.39	$ 1.62	$ 37.61	$ 43.10	$ 7,359.02	$ 8,433.23
75.16666667	051223752700	Structural steel member, 100-to	E2	1000	0.056	L.F.	$ 33.60	$ 2.37	$ 1.60	$ 37.57	$ 43.06	$ 2,824.01	$ 3,236.68
60	051223752900	Structural steel member, 100-to	E2	900	0.062	L.F.	$ 40.43	$ 2.63	$ 1.77	$ 44.83	$ 50.51	$ 2,689.80	$ 3,030.60
1440	by extrapolation		E5	960	0.083	L.F.	$ 26.80	$ 3.56	$ 1.77	$ 32.13	$ 35.67	$ 46,270.08	$ 51,359.79
2412	by extrapolation		E5	960	0.083	L.F.	$ 45.42	$ 3.56	$ 1.77	$ 50.75	$ 56.33	$ 122,409.00	$ 135,873.99
480	051223753500	Structural steel member, 100-to	E5	960	0.083	L.F.	$ 51.98	$ 3.56	$ 1.77	$ 57.31	$ 65.30	$ 27,508.80	$ 31,344.00
60	051223753700	Structural steel member, 100-to	E5	912	0.088	L.F.	$ 65.10	$ 3.75	$ 1.87	$ 70.72	$ 79.89	$ 4,243.20	$ 4,793.40
120	051223753900	Structural steel member, 100-to	E5	912	0.088	L.F.	$ 71.40	$ 3.75	$ 1.87	$ 77.02	$ 87.24	$ 9,242.40	$ 10,468.80
680	051223754100	Structural steel member, 100-to	E5	1064	0.075	L.F.	$ 57.23	$ 3.21	$ 1.61	$ 62.05	$ 70.30	$ 42,194.00	$ 47,804.00
120	051223754300	Structural steel member, 100-to	E5	1064	0.075	L.F.	$ 65.10	$ 3.21	$ 1.61	$ 69.92	$ 78.70	$ 8,390.40	$ 9,444.00
436	051223754900	Structural steel member, 100-to	E5	1110	0.072	L.F.	$ 71.40	$ 3.08	$ 1.54	$ 76.02	$ 85.76	$ 33,144.72	$ 37,391.36
360	051223755300	Structural steel member, 100-to	E5	1110	0.072	L.F.	$ 88.20	$ 3.08	$ 1.54	$ 92.82	$ 104.14	$ 33,415.20	$ 37,490.40
555.8333333	051223755700	Structural steel member, 100-to	E5	1080	0.074	L.F.	$ 109.20	$ 3.17	$ 1.58	$ 113.95	$ 126.89	$ 63,337.21	$ 70,529.69
1392.333333	051223755740	Structural steel member, 100-to	E5	1050	0.076	L.F.	$ 135.45	$ 3.26	$ 1.63	$ 140.34	$ 156.46	$ 195,400.06	$ 217,844.47
174.8333333	051223755780	Structural steel member, 100-to	E5	1050	0.076	L.F.	$ 190.05	$ 3.26	$ 1.63	$ 194.94	$ 216.31	$ 34,082.01	$ 37,818.20
240	by extrapolation		E5	1190	0.067	L.F.	$ 71.19	$ 2.88	$ 1.43	$ 75.50	$ 126.00	$ 18,120.00	$ 30,239.06
510	051223755800	Structural steel member, 100-to	E5	1190	0.067	L.F.	$ 109.20	$ 2.88	$ 1.43	$ 113.51	$ 126.24	$ 57,890.10	$ 64,382.40
360	051223755900	Structural steel member, 100-to	E5	1190	0.067	L.F.	$ 121.80	$ 2.88	$ 1.43	$ 126.11	$ 140.94	$ 45,399.60	$ 50,738.40
600	by extrapolation		E5	1200	0.067	L.F.	$ 89.78	$ 2.88	$ 1.42	$ 94.08	$ 104.42	$ 56,445.00	$ 62,653.95
212	by extrapolation		E5	1200	0.067	L.F.	$ 117.81	$ 2.85	$ 1.42	$ 122.08	$ 135.51	$ 25,880.96	$ 28,727.87
240	051223756100	Structural steel member, 100-to	E5	1200	0.067	L.F.	$ 129.15	$ 2.85	$ 1.42	$ 133.42	$ 148.23	$ 32,020.80	$ 35,575.20
120	051223756540	Structural steel member, 100-to	E5	1160	0.069	L.F.	$ 192.15	$ 2.96	$ 1.47	$ 196.58	$ 217.72	$ 23,589.60	$ 26,126.40
160.5	051223756560	Structural steel member, 100-to	E5	1120	0.071	L.F.	$ 224.70	$ 3.05	$ 1.53	$ 229.28	$ 254.75	$ 36,799.44	$ 40,887.38
360	051223757600	Structural steel member, 100-to	E5	1150	0.07	L.F.	$ 220.50	$ 2.97	$ 1.49	$ 224.96	$ 249.28	$ 80,985.60	$ 89,740.80
240	by extrapolation		E5	1050	0.077	L.F.	$ 292.95	$ 3.30	$ 1.65	$ 297.90	$ 330.67	$ 71,496.00	$ 79,360.56
240	by extrapolation		E5	1050	0.077	L.F.	$ 341.25	$ 3.30	$ 1.65	$ 346.20	$ 384.28	$ 83,088.00	$ 92,227.68
	051223600600	Pipe support framing, structural	E4	5400	0.006	Lb.	$ 1.51	$ 0.26	$ 0.02	$ 1.79	$ 2.14	$ 171,323.67	$ 204,822.72
36000	53123502700	Metal roof decking, steel, open	E4	4300	0.007	S.F.	$1.22	$0.33	$0.03	$1.58	$1.95	$56,880.00	$70,200.00
58272	053113505300	Metal floor decking, steel, non-	E4	3600	0.009	S.F.	$ 1.50	$ 0.40	$ 0.03	$ 1.93	$ 2.38	$ 112,464.96	$ 138,687.36
253.0461201	032110600150	Reinforcing Steel, in place, bean	4 Rodm	2.7	11.852	Ton	$ 891.90	$ 376.63	$ -	$ 1,268.53	$ 1,580.42	$ 320,996.59	$ 399,919.15
141.67695	032110600250	Reinforcing Steel, in place, colur	4 Rodm	2.3	13.913	Ton	$ 891.90	$ 442.13	$ -	$ 1,334.08	$ 1,685.22	$ 189,001.30	$ 238,756.83
												$ 5,887,030.09	$ 7,254,951.27

Figure 28: Existing Concrete Cost Analysis

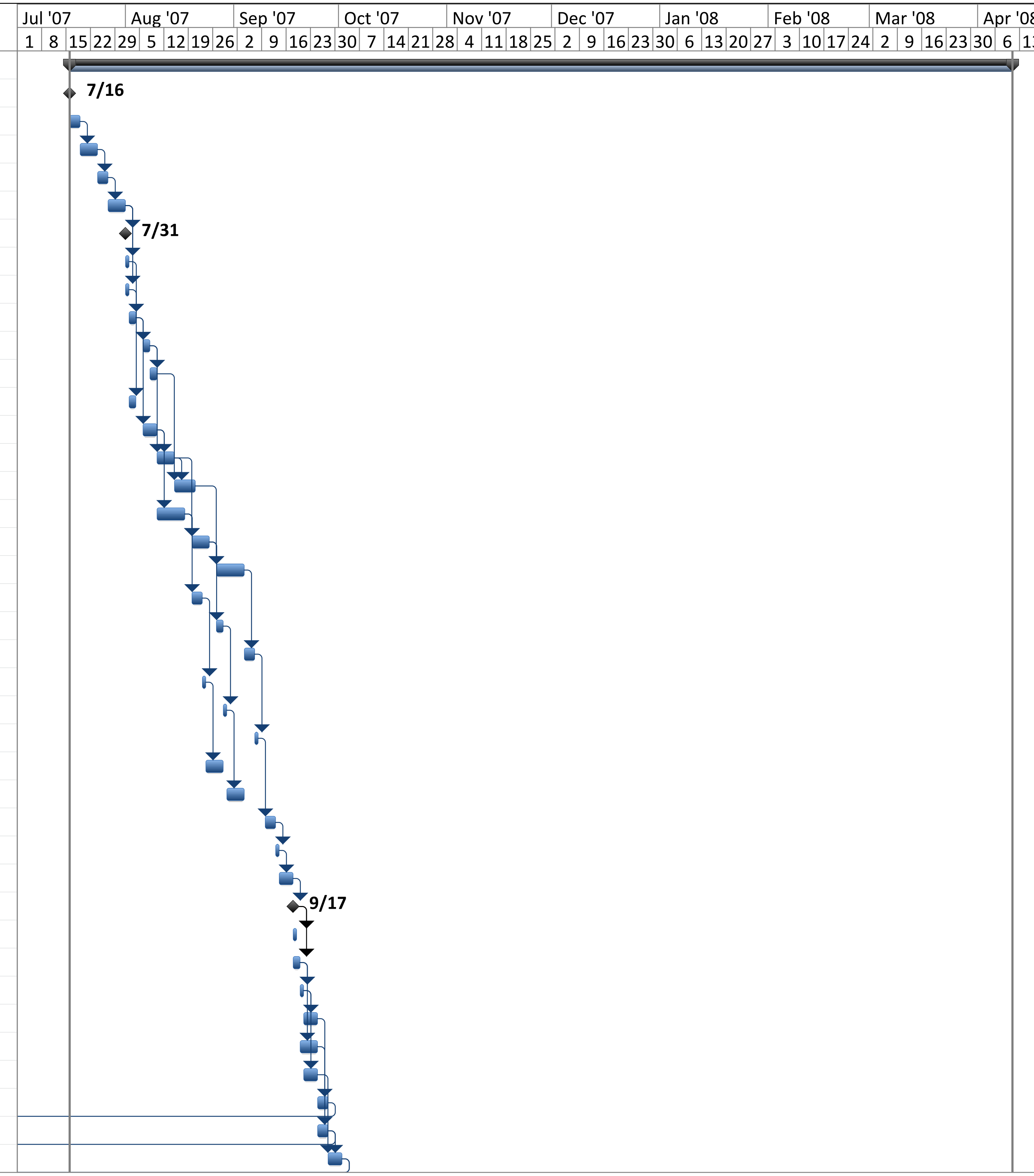
Quantity	Line Number	Description	Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Total	Total O&P	Cost	Cost O&P
13.5	051223750600	Structural steel m	E2	600	0.093	L.F.	$ 15.59	$ 3.95	$ 2.66	$ 22.20	$ 26.80	$ 299.70	$ 361.80
34.66667	051223750700	Structural steel m	E2	600	0.093	L.F.	$ 28.35	$ 3.95	$ 2.66	$ 34.96	$ 41.13	$ 1,211.95	$ 1,425.84
16.5	051223751100	Structural steel m	E2	880	0.064	L.F.	$ 20.79	$ 2.69	$ 1.81	$ 25.29	$ 29.65	$ 417.29	$ 489.23
3753.333			E2	990	0.057	L.F.	$ 33.60	$ 2.39	$ 1.62	$ 37.61	$ 43.10	$ 141,162.87	$ 161,768.67
94.5	051223751900	Structural steel m											
21550	051223752700	Structural steel m	E2	1000	0.056	L.F.	$ 33.60	$ 2.37	$ 1.60	$ 37.57	$ 43.06	$ 809,633.50	$ 927,943.00
608	051223752900	Structural steel m	E2	900	0.062	L.F.	$ 40.43	$ 2.63	$ 1.77	$ 44.83	$ 50.51	$ 27,256.64	$ 30,710.08
1440	051223753300	Structural steel m	E5	960	0.083	L.F.	$ 45.68	$ 3.56	$ 1.77	$ 51.01	$ 57.95	$ 73,454.40	$ 83,448.00
2884.167													
2227.5	051223753500	Structural steel m	E5	960	0.083	L.F.	$ 51.98	$ 3.56	$ 1.77	$ 57.31	$ 65.30	$ 127,658.03	$ 145,455.75
120	051223753700	Structural steel m	E5	912	0.088	L.F.	$ 65.10	$ 3.75	$ 1.87	$ 70.72	$ 79.89	$ 8,486.40	$ 9,586.80
4073.667	051223754100	Structural steel m	E5	1064	0.075	L.F.	$ 57.23	$ 3.21	$ 1.61	$ 62.05	$ 70.30	$ 252,771.02	$ 286,378.77
1080	051223754300	Structural steel m	E5	1064	0.075	L.F.	$ 65.10	$ 3.21	$ 1.61	$ 69.92	$ 78.70	$ 75,513.60	$ 84,996.00
1796.667	051223754900	Structural steel m	E5	1110	0.072	L.F.	$ 71.40	$ 3.08	$ 1.54	$ 76.02	$ 85.76	$ 136,582.60	$ 154,082.13
225	051223755100	Structural steel m	E5	1110	0.072	L.F.	$ 80.33	$ 3.08	$ 1.54	$ 84.95	$ 95.74	$ 19,113.75	$ 21,541.50
1290	051223755300	Structural steel m	E5	1110	0.072	L.F.	$ 88.20	$ 3.08	$ 1.54	$ 92.82	$ 104.14	$ 119,737.80	$ 134,340.60
1125	051223755500	Structural steel m	E5	1110	0.072	L.F.	$ 98.70	$ 3.08	$ 1.54	$ 103.32	$ 115.16	$ 116,235.00	$ 129,555.00
1320	051223755740	Structural steel m	E5	1050	0.076	L.F.	$ 135.45	$ 3.26	$ 1.63	$ 140.34	$ 156.46	$ 185,248.80	$ 206,527.20
2940	051223755800	Structural steel m	E5	1190	0.067	L.F.	$ 109.20	$ 2.88	$ 1.43	$ 113.51	$ 126.24	$ 333,719.40	$ 371,145.60
510													
420	051223755900	Structural steel m	E5	1190	0.067	L.F.	$ 121.80	$ 2.88	$ 1.43	$ 126.11	$ 140.94	$ 52,966.20	$ 59,194.80
555.6667	051223756100	Structural steel m	E5	1200	0.067	L.F.	$ 129.15	$ 2.85	$ 1.42	$ 133.42	$ 148.23	$ 74,137.05	$ 82,366.47
240													
221.6667	051223756560	Structural steel m	E5	1120	0.071	L.F.	$ 224.70	$ 3.05	$ 1.53	$ 229.28	$ 254.75	$ 50,823.75	$ 56,469.58
240	by extrapolation		E5	1000	0.071	L.F.	$ 226.20	$ 3.04	$ 1.63	$ 230.87	$ 256.27	$ 55,408.80	$ 61,503.77
120	by extrapolation		E5	1000	0.071	L.F.	$ 245.00	$ 3.10	$ 1.63	$ 249.73	$ 277.20	$ 29,967.60	$ 33,264.04
240	by extrapolation		E5	1000	0.071	L.F.	$ 256.30	$ 3.10	$ 1.59	$ 260.99	$ 289.70	$ 62,637.60	$ 69,527.74
240	by extrapolation		E5	1000	0.073	L.F.	$ 278.50	$ 3.16	$ 1.59	$ 283.25	$ 314.41	$ 67,980.00	$ 75,457.80
419589.2	051223600600	Pipe support fram	E4	5400	0.006	Lb.	$ 1.51	$ 0.26	$ 0.02	$ 1.79	$ 2.14	$ 751,064.67	$ 897,920.89
158	051223750740	Structural steel m	E2	550	0.102	L.F.	$ 43.05	$ 4.30	$ 2.91	$ 50.26	$ 57.72	$ 7,941.08	$ 9,119.76
62	051223750900	Structural steel m	E2	550	0.102	L.F.	$ 63.53	$ 4.30	$ 2.91	$ 70.74	$ 80.30	$ 4,385.88	$ 4,978.60
493.5	by extrapolation		E2	450	0.075	L.F.	$ 52.01	$ 3.16	$ 2.91	$ 58.08	$ 65.63	$ 28,662.48	$ 32,388.60
258	by extrapolation		E2	750	0.075	L.F.	$ 58.41	$ 3.16	$ 2.91	$ 64.48	$ 72.86	$ 16,635.84	$ 18,798.50
380	051223751560	Structural steel m	E2	750	0.075	L.F.	$ 65.10	$ 3.16	$ 2.91	$ 70.39	$ 79.11	$ 26,748.20	$ 30,061.80
949	by interpolation		E2	750	0.075	L.F.	$ 69.04	$ 3.16	$ 2.91	$ 75.11	$ 84.12	$ 71,277.02	$ 79,830.26
300	051223751580	Structural steel m	E2	750	0.075	L.F.	$ 75.60	$ 3.16	$ 2.91	$ 80.89	$ 90.66	$ 24,267.00	$ 27,198.00
300	by interpolation		E2	720	0.072	L.F.	$ 84.53	$ 3.43	$ 2.70	$ 90.66	$ 101.53	$ 27,196.50	$ 30,460.08
120	051223751700	Structural steel m	E2	640	0.088	L.F.	$ 93.45	$ 3.70	$ 2.49	$ 99.64	$ 111.90	$ 11,956.80	$ 13,428.00
180	by interpolation		E2	640	0.088	L.F.	$ 102.76	$ 3.70	$ 2.49	$ 108.95	$ 122.02	$ 19,611.00	$ 21,964.32
180	051223751740	Structural steel m	E2	640	0.088	L.F.	$ 113.40	$ 3.70	$ 2.49	$ 119.59	$ 132.90	$ 21,526.20	$ 23,922.00
209	by extrapolation		E2	760	0.084	L.F.	$ 78.82	$ 3.12	$ 2.10	$ 84.04	$ 94.12	$ 17,564.36	$ 19,672.08
1045.5	051223752380	Structural steel m	E2	740	0.076	L.F.	$ 116.55	$ 3.20	$ 2.16	$ 121.91	$ 136.93	$ 127,456.91	$ 143,160.32
285.5	by interpolation		E2	720	0.076	L.F.	$ 128.52	$ 3.23	$ 2.18	$ 133.93	$ 148.66	$ 38,235.59	$ 42,441.50
227.5	by interpolation		E2	720	0.078	L.F.	$ 141.82	$ 3.26	$ 2.20	$ 147.28	$ 163.69	$ 33,505.06	$ 37,239.10
36.5	051223752500	Structural steel m	E2	720	0.078	L.F.	$ 156.45	$ 3.29	$ 2.22	$ 161.96	$ 179.18	$ 5,911.54	$ 6,540.07
51.5	by extrapolation		E2	720	0.078	L.F.	$ 172.41	$ 3.29	$ 2.22	$ 177.92	$ 197.49	$ 9,162.88	$ 10,170.80
146	by extrapolation		E2	720	0.078	L.F.	$ 189.70	$ 3.29	$ 2.22	$ 195.21	$ 216.68	$ 28,500.66	$ 31,635.73
103	by extrapolation		E2	720	0.078	L.F.	$ 208.32	$ 3.29	$ 2.22	$ 213.83	$ 237.35	$ 22,024.49	$ 24,447.18
36.5	by extrapolation		E2	720	0.078	L.F.	$ 234.92	$ 3.29	$ 2.22	$ 240.43	$ 266.88	$ 8,775.70	$ 9,741.02
21.5	by extrapolation		E2	720	0.078	L.F.	$ 253.54	$ 3.29	$ 2.22	$ 259.05	$ 287.55	$ 5,569.58	$ 6,182.23
73	by extrapolation		E2	720	0.078	L.F.	$ 306.74	$ 3.29	$ 2.22	$ 312.25	$ 346.60	$ 22,794.25	$ 25,301.62
1526.5	051223600600	Pipe support fram	E4	5400	0.006	Lb.	$ 1.51	$ 0.26	$ 0.02	$ 1.79	$ 2.14	$ 171,323.67	$ 204,822.72
49379.83	078116100400	Sprayed cementit	G2	1500	0.016	S.F.	$ 0.53	$ 0.38	$ 0.08	$ 0.99	$ 1.24	$ 195,544.14	$ 244,923.97
7587	078116100700	Sprayed cementit	G2	1100	0.022	S.F.	$ 0.59	$ 0.52	$ 0.11	$ 1.22	$ 1.56	$ 444,294.72	$ 568,114.56
197519.3	078116100900	Sprayed cementit	G2	5000	0.005	S.F.	$ 0.07	$ 0.11	$ 0.03	$ 0.21	$ 0.28	$ 69,381.90	$ 92,509.20
364176	by extrapolation	Weld stud, 3/4" d	E10	1000	16	Ea.	$ 0.83	$ 0.72	$ 0.35	$ 1.90	$ 2.66	$ 50,855.40	$ 71,197.56
286894.5	by interpolation	Metal floor deck	E4	3575	0.009	S.F.	$ 2.18	$ 0.40	$ 0.03	$ 2.61	$ 3.10	$ 748,794.65	$ 889,372.95
3541.907	033116100820	Structural concre	o			C.Y.	$ 135.08	-	-	$ 135.08	$ 148.49	$ 478,440.85	$ 525,937.83
36000	53123502700	Metal roof deckin	E4	4300	0.007	S.F.	$1.22	$0.33	$0.03	$1.58	$1.95	$56,880.00	$70,200.00
5500	031113500050	C.I.P. concrete fo	C2	580	0.083	SFCA	$ 1.20	$ 2.20	$ -	$ 3.40	$ 4.68	$ 18,700.00	$ 25,740.00
3213.706	033105350300	Structural concret				C.Y.	$ 98.67	$ -	$ -	$ 98.67	$ 108.25	$ 317,098.40	$ 347,883.71
1101.16	033105704300	Structural concre	C6	110	0.436	C.Y.	$ -	$ 10.65	$ 0.41	$ 11.06	$ 16.50	$ 12,178.83	$ 18,169.14
1756.046	033105702800	Structural concret	C8	120	0.4	C.Y.	$ -	$ 9.77	$ 0.38	$ 10.15	$ 15.09	$ 17,823.87	$ 26,498.74
14355	031113455050	C.I.P. concrete fo	C1	371	0.086	SFCA	$ 0.93	$ 2.23	$ -	$ 3.16	$ 4.44	$ 45,361.80	$ 63,736.20
3238.29	032205500100	Welded wire fabri	2 Rodm	35	0.457	C.S.F.	$ 12.39	$ 14.41	$ -	$ 26.80	$ 36.88	$ 86,786.17	$ 119,428.14
												$ 6,856,659.78	$ 8,002,677.52

Figure 29: Redesign Cost Analysis

Appendix E: Superstructure Schedules

Original Schedule and Tasks Follow

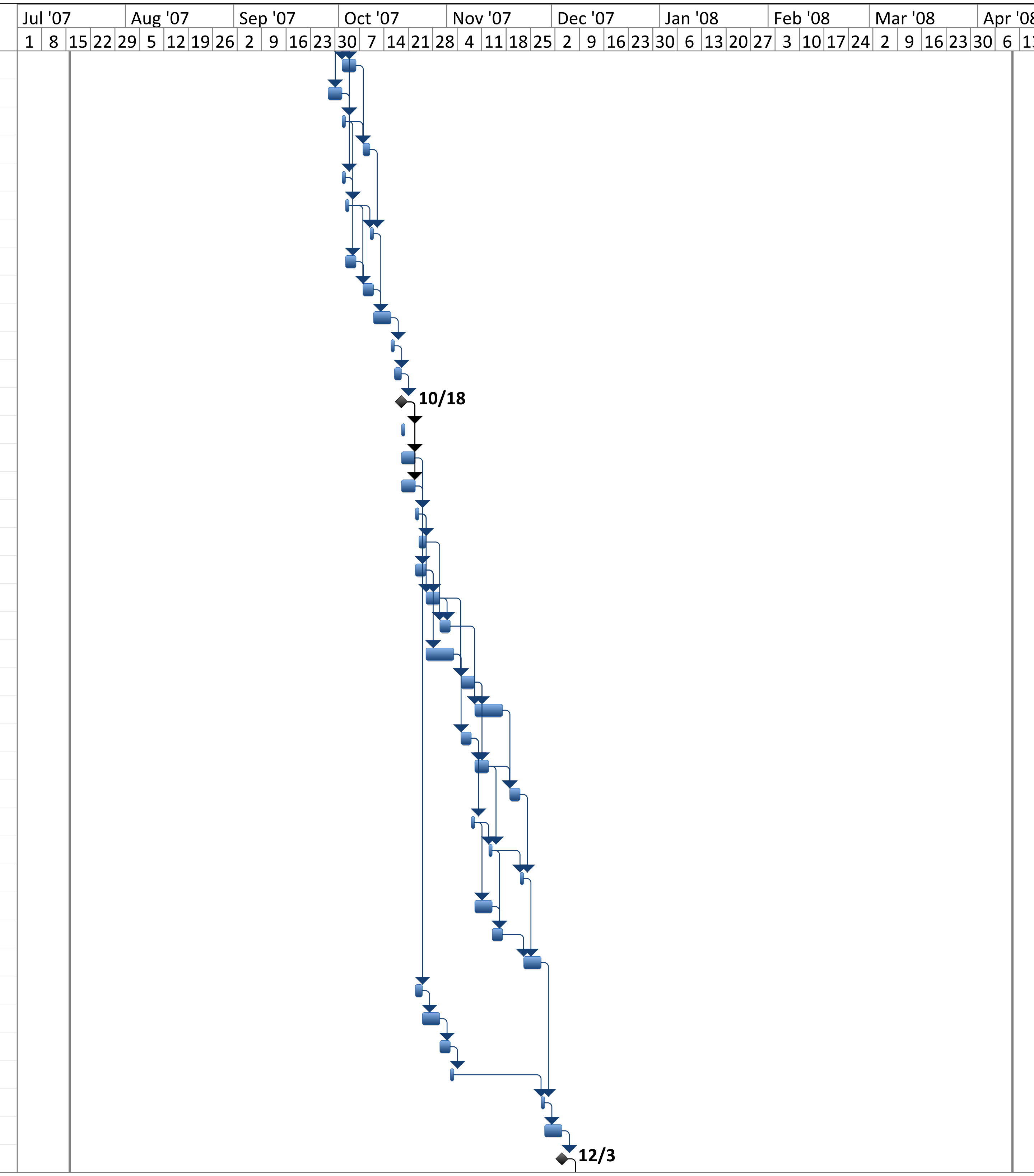
ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
1			**Superstructure**	**194 days**	**Mon 7/16/07**	**Thu 4/10/08**	
2			**Slab On Grade**	0 days	Mon 7/16/07	Mon 7/16/07	
3			Formwork	3 days	Mon 7/16/07	Wed 7/18/07	
4			Place Rebar	3 days	Thu 7/19/07	Mon 7/23/07	3
5			Place Concrete	3 days	Tue 7/24/07	Thu 7/26/07	4
6			Finish	3 days	Fri 7/27/07	Tue 7/31/07	5
7			**2nd Floor**	0 days	Tue 7/31/07	Tue 7/31/07	6
8			Shakeout	1 day	Wed 8/1/07	Wed 8/1/07	6
9			Place Baseplates and Grout	1 day	Wed 8/1/07	Wed 8/1/07	6
10			Frame Concrete Columns A (Ground to 2nd)	2 days	Thu 8/2/07	Fri 8/3/07	9
11			Frame Concrete Columns B (Ground to 2nd)	2 days	Mon 8/6/07	Tue 8/7/07	10
12			Frame Concrete Columns C (Ground to 2nd)	2 days	Wed 8/8/07	Thu 8/9/07	11
13			Erect Steel Columns (Ground to 3rd)	2 days	Thu 8/2/07	Fri 8/3/07	8
14			Frame Concrete Beams A	4 days	Mon 8/6/07	Thu 8/9/07	10
15			Frame Concrete Beams B	3 days	Fri 8/10/07	Tue 8/14/07	14,11
16			Frame Concrete Beams C	4 days	Wed 8/15/07	Mon 8/20/07	15,12
17			Frame Slab A	6 days	Fri 8/10/07	Fri 8/17/07	14
18			Frame Slab B	5 days	Mon 8/20/07	Fri 8/24/07	17,15
19			Frame Slab C	6 days	Mon 8/27/07	Mon 9/3/07	18,16
20			Set Reinforcement A	3 days	Mon 8/20/07	Wed 8/22/07	17
21			Set Reinforcement B	2 days	Mon 8/27/07	Tue 8/28/07	18
22			Set Reinforcement C	3 days	Tue 9/4/07	Thu 9/6/07	19
23			Place Concrete A	1 day	Thu 8/23/07	Thu 8/23/07	20
24			Place Concrete B	1 day	Wed 8/29/07	Wed 8/29/07	21
25			Place Concrete C	1 day	Fri 9/7/07	Fri 9/7/07	22
26			Finish Floor A	3 days	Fri 8/24/07	Tue 8/28/07	23
27			Finish Floor B	3 days	Thu 8/30/07	Mon 9/3/07	24
28			Finish Floor C	3 days	Mon 9/10/07	Wed 9/12/07	25
29			Erect Steel Beams (2nd)	1 day	Thu 9/13/07	Thu 9/13/07	28
30			Bolts/Welds	2 days	Fri 9/14/07	Mon 9/17/07	29
31			**3rd Floor**	0 days	Mon 9/17/07	Mon 9/17/07	30
32			Shakeout	1 day	Tue 9/18/07	Tue 9/18/07	31
33			Frame Concrete Columns A (2nd to 3rd)	2 days	Tue 9/18/07	Wed 9/19/07	31
34			Frame Concrete Columns B (2nd to 3rd)	1 day	Thu 9/20/07	Thu 9/20/07	33
35			Frame Concrete Columns C (2nd to 3rd)	2 days	Fri 9/21/07	Mon 9/24/07	34
36			Frame Concrete Beams A	3 days	Thu 9/20/07	Mon 9/24/07	33
37			Frame Concrete Beams B	2 days	Fri 9/21/07	Mon 9/24/07	34
38			Frame Concrete Beams C	3 days	Tue 9/25/07	Thu 9/27/07	35
39			Frame Slab A	3 days	Tue 9/25/07	Thu 9/27/07	36
40			Frame Slab B	2 days	Fri 9/28/07	Mon 10/1/07	39,37



Project: original schedule.mpp
Date: Thu 4/28/11

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

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
41		Frame Slab C	4 days	Tue 10/2/07	Fri 10/5/07	38,40
42		Set Reinforcement A	2 days	Fri 9/28/07	Mon 10/1/07	39
43		Set Reinforcement B	1 day	Tue 10/2/07	Tue 10/2/07	40,42
44		Set Reinforcement C	2 days	Mon 10/8/07	Tue 10/9/07	41,43
45		Place Concrete A	1 day	Tue 10/2/07	Tue 10/2/07	42
46		Place Concrete B	1 day	Wed 10/3/07	Wed 10/3/07	43,45
47		Place Concrete C	1 day	Wed 10/10/07	Wed 10/10/07	44,46
48		Finish Floor A	3 days	Wed 10/3/07	Fri 10/5/07	45
49		Finish Floor B	3 days	Mon 10/8/07	Wed 10/10/07	46,48
50		Finish Floor C	3 days	Thu 10/11/07	Mon 10/15/07	47,49
51		Erect Steel Beams (3rd)	1 day	Tue 10/16/07	Tue 10/16/07	50
52		Bolts/Welds	2 days	Wed 10/17/07	Thu 10/18/07	51
53		**4th Floor**	0 days	Thu 10/18/07	Thu 10/18/07	52
54		Shakeout	1 day	Fri 10/19/07	Fri 10/19/07	53
55		Erect Steel Columns (3rd to 5th)	2 days	Fri 10/19/07	Mon 10/22/07	53
56		Frame Concrete Columns A (3rd to 4th)	2 days	Fri 10/19/07	Mon 10/22/07	53
57		Frame Concrete Columns B (3rd to 4th)	1 day	Tue 10/23/07	Tue 10/23/07	56
58		Frame Concrete Columns C (3rd to 4th)	2 days	Wed 10/24/07	Thu 10/25/07	57
59		Frame Concrete Beams A	3 days	Tue 10/23/07	Thu 10/25/07	56
60		Frame Concrete Beams B	2 days	Fri 10/26/07	Mon 10/29/07	59,57
61		Frame Concrete Beams C	3 days	Tue 10/30/07	Thu 11/1/07	60,58
62		Frame Slab A	6 days	Fri 10/26/07	Fri 11/2/07	59
63		Frame Slab B	4 days	Mon 11/5/07	Thu 11/8/07	62,60
64		Frame Slab C	6 days	Fri 11/9/07	Fri 11/16/07	63,61
65		Set Reinforcement A	3 days	Mon 11/5/07	Wed 11/7/07	62
66		Set Reinforcement B	2 days	Fri 11/9/07	Mon 11/12/07	65,63
67		Set Reinforcement C	3 days	Mon 11/19/07	Wed 11/21/07	66,64
68		Place Concrete A	1 day	Thu 11/8/07	Thu 11/8/07	65
69		Place Concrete B	1 day	Tue 11/13/07	Tue 11/13/07	68,66
70		Place Concrete C	1 day	Thu 11/22/07	Thu 11/22/07	69,67
71		Finish Floor A	3 days	Fri 11/9/07	Tue 11/13/07	68
72		Finish Floor B	3 days	Wed 11/14/07	Fri 11/16/07	71,69
73		Finish Floor C	3 days	Fri 11/23/07	Tue 11/27/07	72,70
74		Erect Steel Beams (4th)	2 days	Tue 10/23/07	Wed 10/24/07	55
75		Bolts/Welds	3 days	Thu 10/25/07	Mon 10/29/07	74
76		Place Metal Deck	3 days	Tue 10/30/07	Thu 11/1/07	75
77		Deck Edge Forms	1 day	Fri 11/2/07	Fri 11/2/07	76
78		Place Deck Concrete	1 day	Wed 11/28/07	Wed 11/28/07	77,73
79		Finish Deck Concrete	3 days	Thu 11/29/07	Mon 12/3/07	78
80		**5th Floor**	0 days	Mon 12/3/07	Mon 12/3/07	79



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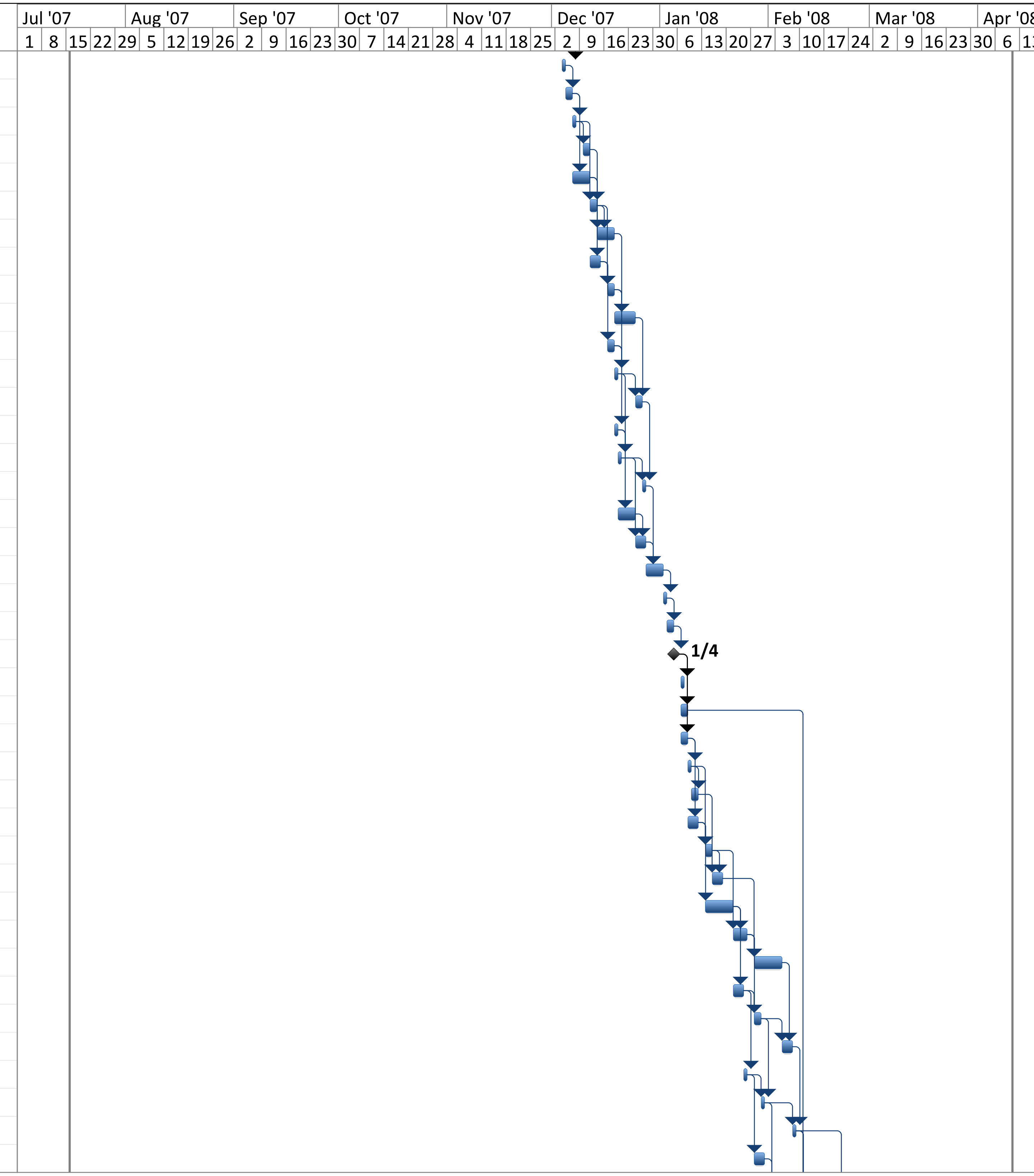
Task
Split
Milestone
Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
81			Shakeout	1 day	Tue 12/4/07	Tue 12/4/07	80
82			Frame Concrete Columns A (4th to 5th)	2 days	Wed 12/5/07	Thu 12/6/07	81
83			Frame Concrete Columns B (4th to 5th)	1 day	Fri 12/7/07	Fri 12/7/07	82
84			Frame Concrete Columns C (4th to 5th)	2 days	Mon 12/10/07	Tue 12/11/07	83
85			Frame Concrete Beams A	3 days	Fri 12/7/07	Tue 12/11/07	82
86			Frame Concrete Beams B	2 days	Wed 12/12/07	Thu 12/13/07	85,83
87			Frame Concrete Beams C	3 days	Fri 12/14/07	Tue 12/18/07	86,84
88			Frame Slab A	3 days	Wed 12/12/07	Fri 12/14/07	85
89			Frame Slab B	2 days	Mon 12/17/07	Tue 12/18/07	88,86
90			Frame Slab C	4 days	Wed 12/19/07	Mon 12/24/07	89,87
91			Set Reinforcement A	2 days	Mon 12/17/07	Tue 12/18/07	88
92			Set Reinforcement B	1 day	Wed 12/19/07	Wed 12/19/07	91,89
93			Set Reinforcement C	2 days	Tue 12/25/07	Wed 12/26/07	92,90
94			Place Concrete A	1 day	Wed 12/19/07	Wed 12/19/07	91
95			Place Concrete B	1 day	Thu 12/20/07	Thu 12/20/07	94,92
96			Place Concrete C	1 day	Thu 12/27/07	Thu 12/27/07	95,93
97			Finish Floor A	3 days	Thu 12/20/07	Mon 12/24/07	94
98			Finish Floor B	3 days	Tue 12/25/07	Thu 12/27/07	97,95
99			Finish Floor C	3 days	Fri 12/28/07	Tue 1/1/08	98,96
100			Erect Steel Beams (5th)	1 day	Wed 1/2/08	Wed 1/2/08	99
101			Bolts/Welds	2 days	Thu 1/3/08	Fri 1/4/08	100
102			**6th Floor**	0 days	Fri 1/4/08	Fri 1/4/08	101
103			Shakeout	1 day	Mon 1/7/08	Mon 1/7/08	102
104			Erect Steel Columns (5th to Lower Roof)	2 days	Mon 1/7/08	Tue 1/8/08	102
105			Frame Concrete Columns A (5th to 6th)	2 days	Mon 1/7/08	Tue 1/8/08	102
106			Frame Concrete Columns B (5th to 6th)	1 day	Wed 1/9/08	Wed 1/9/08	105
107			Frame Concrete Columns C (5th to 6th)	2 days	Thu 1/10/08	Fri 1/11/08	106
108			Frame Concrete Beams A	3 days	Wed 1/9/08	Fri 1/11/08	105
109			Frame Concrete Beams B	2 days	Mon 1/14/08	Tue 1/15/08	108,106
110			Frame Concrete Beams C	3 days	Wed 1/16/08	Fri 1/18/08	109,107
111			Frame Slab A	6 days	Mon 1/14/08	Mon 1/21/08	108
112			Frame Slab B	4 days	Tue 1/22/08	Fri 1/25/08	111,109
113			Frame Slab C	6 days	Mon 1/28/08	Mon 2/4/08	112,110
114			Set Reinforcement A	3 days	Tue 1/22/08	Thu 1/24/08	111
115			Set Reinforcement B	2 days	Mon 1/28/08	Tue 1/29/08	114,112
116			Set Reinforcement C	3 days	Tue 2/5/08	Thu 2/7/08	115,113
117			Place Concrete A	1 day	Fri 1/25/08	Fri 1/25/08	114
118			Place Concrete B	1 day	Wed 1/30/08	Wed 1/30/08	117,115
119			Place Concrete C	1 day	Fri 2/8/08	Fri 2/8/08	118,116
120			Finish Floor A	3 days	Mon 1/28/08	Wed 1/30/08	117



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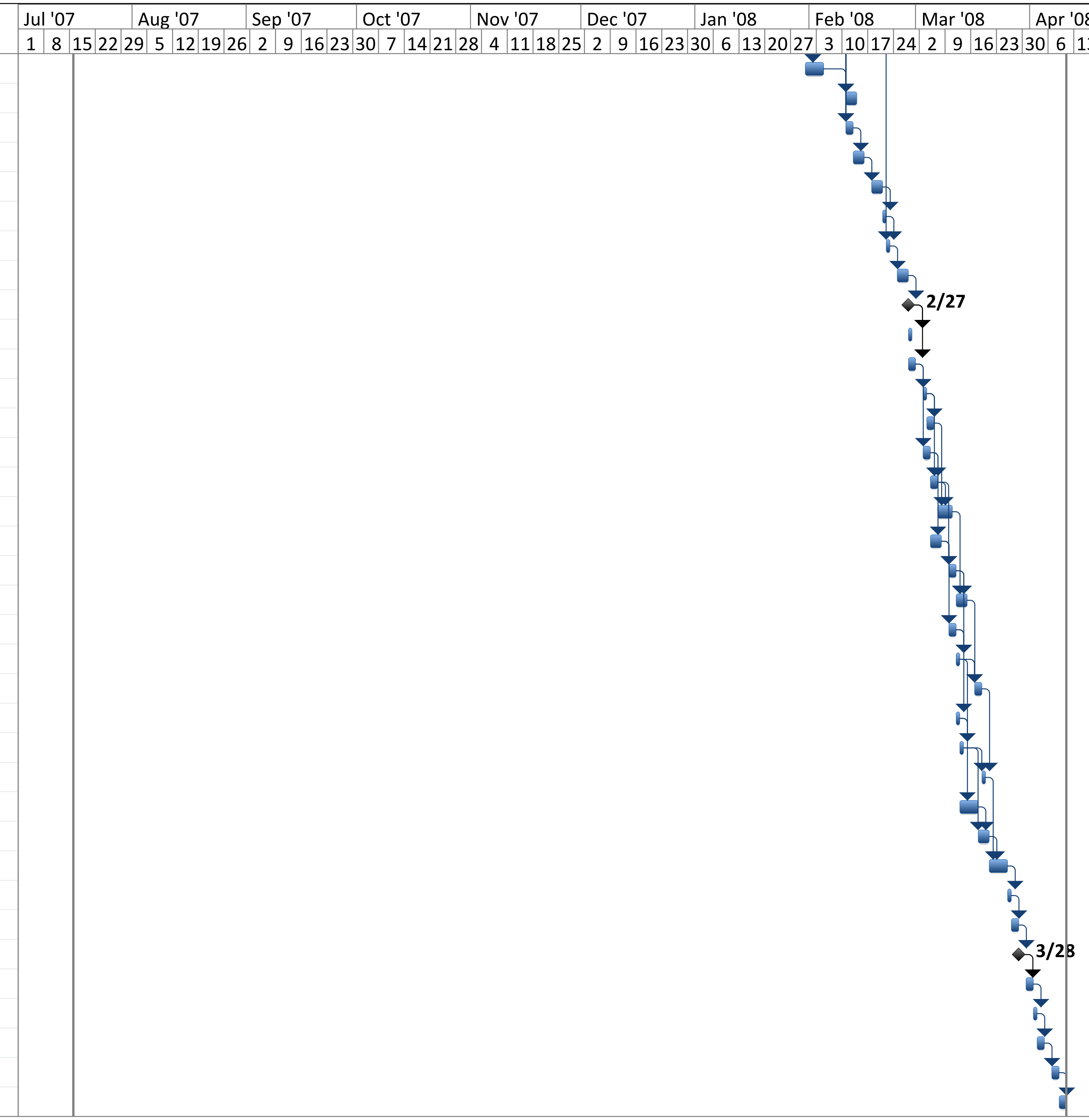
Task
Split
Milestone
Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
121			Finish Floor B	3 days	Thu 1/31/08	Mon 2/4/08	120,118
122			Finish Floor C	3 days	Mon 2/11/08	Wed 2/13/08	121,119
123			Erect Steel Beams (6th)	2 days	Mon 2/11/08	Tue 2/12/08	104,119
124			Bolts/Welds	3 days	Wed 2/13/08	Fri 2/15/08	123
125			Place Metal Deck	3 days	Mon 2/18/08	Wed 2/20/08	124
126			Deck Edge Forms	1 day	Thu 2/21/08	Thu 2/21/08	125
127			Place Deck Concrete	1 day	Fri 2/22/08	Fri 2/22/08	126,119
128			Finish Deck Concrete	3 days	Mon 2/25/08	Wed 2/27/08	127
129			**Lower Roof**	0 days	Wed 2/27/08	Wed 2/27/08	128
130			Shakeout	1 day	Thu 2/28/08	Thu 2/28/08	129
131			Frame Concrete Columns A (6th to Lower Roof)	2 days	Thu 2/28/08	Fri 2/29/08	129
132			Frame Concrete Columns B (6th to Lower Roof)	1 day	Mon 3/3/08	Mon 3/3/08	131
133			Frame Concrete Columns C (6th to Lower Roof)	2 days	Tue 3/4/08	Wed 3/5/08	132
134			Frame Concrete Beams A	2 days	Mon 3/3/08	Tue 3/4/08	131
135			Frame Concrete Beams B	2 days	Wed 3/5/08	Thu 3/6/08	134,132
136			Frame Concrete Beams C	2 days	Fri 3/7/08	Mon 3/10/08	135,133
137			Frame Slab A	3 days	Wed 3/5/08	Fri 3/7/08	134
138			Frame Slab B	2 days	Mon 3/10/08	Tue 3/11/08	137,135
139			Frame Slab C	3 days	Wed 3/12/08	Fri 3/14/08	138,136
140			Set Reinforcement A	2 days	Mon 3/10/08	Tue 3/11/08	137
141			Set Reinforcement B	1 day	Wed 3/12/08	Wed 3/12/08	140,138
142			Set Reinforcement C	2 days	Mon 3/17/08	Tue 3/18/08	141,139
143			Place Concrete A	1 day	Wed 3/12/08	Wed 3/12/08	140
144			Place Concrete B	1 day	Thu 3/13/08	Thu 3/13/08	143,141
145			Place Concrete C	1 day	Wed 3/19/08	Wed 3/19/08	144,142
146			Finish Floor A	3 days	Thu 3/13/08	Mon 3/17/08	143
147			Finish Floor B	3 days	Tue 3/18/08	Thu 3/20/08	146,144
148			Finish Floor C	3 days	Fri 3/21/08	Tue 3/25/08	147,145
149			Erect Steel Beams (Lower Roof)	1 day	Wed 3/26/08	Wed 3/26/08	148
150			Bolts/Welds	2 days	Thu 3/27/08	Fri 3/28/08	149
151			**Roof**	0 days	Fri 3/28/08	Fri 3/28/08	150
152			Erect Steel Columns (Lower Roof to Roof)	2 days	Mon 3/31/08	Tue 4/1/08	151
153			Erect Steel Beams	1 day	Wed 4/2/08	Wed 4/2/08	152
154			Place Metal Decking	2 days	Thu 4/3/08	Fri 4/4/08	153
155			Bolts/Welds	2 days	Mon 4/7/08	Tue 4/8/08	154
156			Place Shear Studs	2 days	Wed 4/9/08	Thu 4/10/08	155



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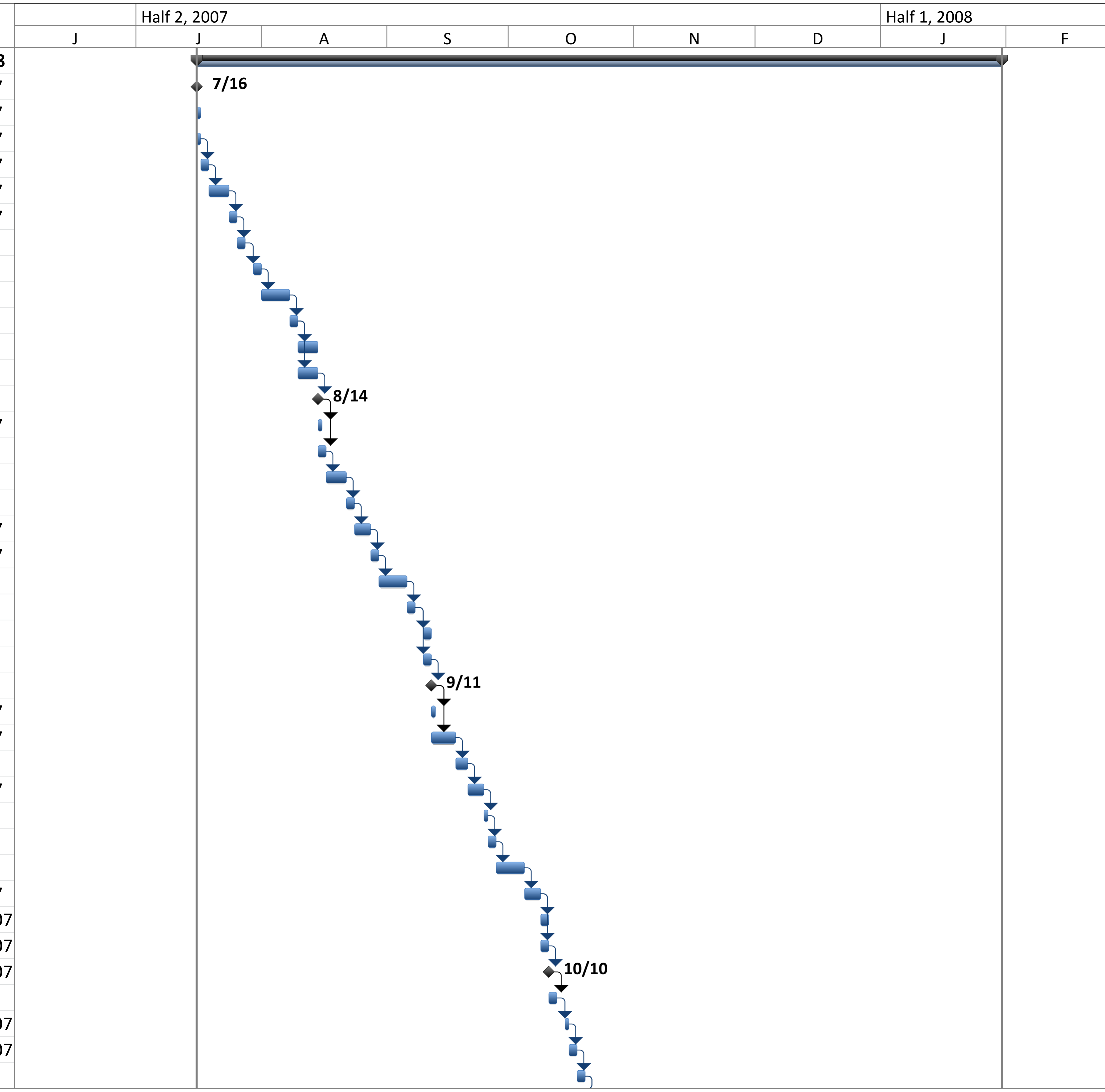
Task		Project Summary		Inactive Milestone		Manual Summary Rollup		Deadline	
Split		External Tasks		Inactive Summary		Manual Summary		Progress	
Milestone		External Milestone		Manual Task		Start-only			
Summary		Inactive Task		Duration-only		Finish-only			

Redesign Schedule and Tasks Follow

ID	Task Mode	Task Name	Duration	Start	Finish
1		**Superstructure**	**143 days**	**Mon 7/16/07**	**Wed 1/30/08**
2		**Steel Structure (Ground Floor to 3rd Floor)**	0 days	Mon 7/16/07	Mon 7/16/07
3		Place Baseplates and Grout	1 day	Mon 7/16/07	Mon 7/16/07
4		Shakeout	1 day	Mon 7/16/07	Mon 7/16/07
5		Erect Columns (Ground Floor to 3rd Floor	2 days	Tue 7/17/07	Wed 7/18/07
6		Erect Beams (2nd Floor)	3 days	Thu 7/19/07	Mon 7/23/07
7		Erect Braces (2nd Floor)	2 days	Tue 7/24/07	Wed 7/25/07
8		Erect Beams (3rd Floor)	2 days	Thu 7/26/07	Fri 7/27/07
9		Erect Braces (3rd Floor)	2 days	Mon 7/30/07	Tue 7/31/07
10		Place Metal Decking (2nd Floor)	5 days	Wed 8/1/07	Tue 8/7/07
11		Place Metal Decking (3rd Floor)	2 days	Wed 8/8/07	Thu 8/9/07
12		Place Shear Studs	3 days	Fri 8/10/07	Tue 8/14/07
13		Bolts/Welds	3 days	Fri 8/10/07	Tue 8/14/07
14		**Steel Structure (3rd Floor to 5th Floor)**	0 days	Tue 8/14/07	Tue 8/14/07
15		Shakeout	1 day	Wed 8/15/07	Wed 8/15/07
16		Erect Columns (3rd Floor to 5th Floor)	2 days	Wed 8/15/07	Thu 8/16/07
17		Erect Beams (4th Floor)	3 days	Fri 8/17/07	Tue 8/21/07
18		Erect Braces (4th Floor)	2 days	Wed 8/22/07	Thu 8/23/07
19		Erect Beams (5th Floor)	2 days	Fri 8/24/07	Mon 8/27/07
20		Erect Braces (5th Floor)	2 days	Tue 8/28/07	Wed 8/29/07
21		Place Metal Decking (4th Floor)	5 days	Thu 8/30/07	Wed 9/5/07
22		Place Metal Decking (5th Floor)	2 days	Thu 9/6/07	Fri 9/7/07
23		Place Shear Studs	2 days	Mon 9/10/07	Tue 9/11/07
24		Bolts/Welds	2 days	Mon 9/10/07	Tue 9/11/07
25		**Steel Structure (5th Floor to Lower Roof)**	0 days	Tue 9/11/07	Tue 9/11/07
26		Shakeout	1 day	Wed 9/12/07	Wed 9/12/07
27		Erect Columns (5th Floor to Lower Roof)	4 days	Wed 9/12/07	Mon 9/17/07
28		Erect Beams (6th Floor)	3 days	Tue 9/18/07	Thu 9/20/07
29		Erect Braces (6th Floor)	2 days	Fri 9/21/07	Mon 9/24/07
30		Erect Beams (Lower Roof)	1 day	Tue 9/25/07	Tue 9/25/07
31		Erect Braces (Lower Roof)	2 days	Wed 9/26/07	Thu 9/27/07
32		Place Metal Decking (6th Floor)	5 days	Fri 9/28/07	Thu 10/4/07
33		Place Metal Decking (Lower Roof)	2 days	Fri 10/5/07	Mon 10/8/07
34		Place Shear Studs	2 days	Tue 10/9/07	Wed 10/10/07
35		Bolts/Welds	2 days	Tue 10/9/07	Wed 10/10/07
36		**Steel Structure (Lower Roof to Roof)**	0 days	Wed 10/10/07	Wed 10/10/07
37		Erect Columns (Lower Roof to Roof)	2 days	Thu 10/11/07	Fri 10/12/07
38		Erect Beams (Roof)	1 day	Mon 10/15/07	Mon 10/15/07
39		Erect Braces (Roof)	2 days	Tue 10/16/07	Wed 10/17/07
40		Place Metal Decking (Roof)	2 days	Thu 10/18/07	Fri 10/19/07



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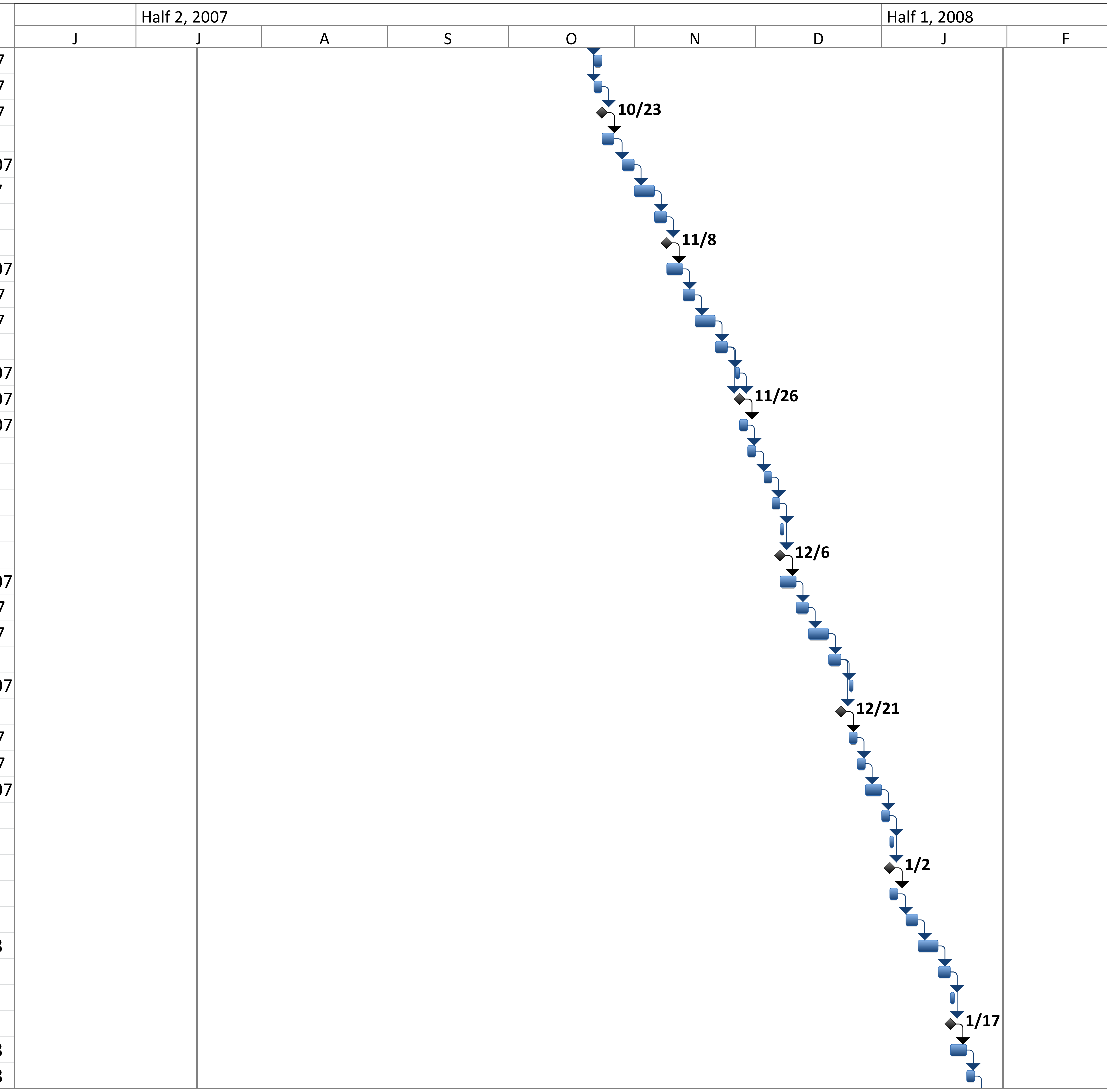
Task		Project Summary		Inactive Milestone		Manual Summary Rollup		Deadline	
Split		External Tasks		Inactive Summary		Manual Summary		Progress	
Milestone		External Milestone		Manual Task		Start-only			
Summary		Inactive Task		Duration-only		Finish-only			

ID		Task Mode	Task Name	Duration	Start	Finish
41			Place Shear Studs	2 days	Mon 10/22/07	Tue 10/23/07
42			Bolts/Welds	2 days	Mon 10/22/07	Tue 10/23/07
43			**Slab On Grade**	0 days	Tue 10/23/07	Tue 10/23/07
44			Formwork	3 days	Wed 10/24/07	Fri 10/26/07
45			Place Rebar	3 days	Mon 10/29/07	Wed 10/31/07
46			Place Concrete	3 days	Thu 11/1/07	Mon 11/5/07
47			Finish	3 days	Tue 11/6/07	Thu 11/8/07
48			**2nd Floor**	0 days	Thu 11/8/07	Thu 11/8/07
49			Formwork	2 days	Fri 11/9/07	Mon 11/12/07
50			Place WWF	3 days	Tue 11/13/07	Thu 11/15/07
51			Place Concrete	3 days	Fri 11/16/07	Tue 11/20/07
52			Finish	3 days	Wed 11/21/07	Fri 11/23/07
53			Spray Fireproofing	1 day?	Mon 11/26/07	Mon 11/26/07
54			**3rd Floor**	0 days	Mon 11/26/07	Mon 11/26/07
55			Formwork	2 days	Tue 11/27/07	Wed 11/28/07
56			Place WWF	2 days	Thu 11/29/07	Fri 11/30/07
57			Place Concrete	2 days	Mon 12/3/07	Tue 12/4/07
58			Finish	2 days	Wed 12/5/07	Thu 12/6/07
59			Spray Fireproofing	1 day	Fri 12/7/07	Fri 12/7/07
60			**4th Floor**	0 days	Thu 12/6/07	Thu 12/6/07
61			Formwork	2 days	Fri 12/7/07	Mon 12/10/07
62			Place WWF	3 days	Tue 12/11/07	Thu 12/13/07
63			Place Concrete	3 days	Fri 12/14/07	Tue 12/18/07
64			Finish	3 days	Wed 12/19/07	Fri 12/21/07
65			Spray Fireproofing	1 day	Mon 12/24/07	Mon 12/24/07
66			**5th Floor**	0 days	Fri 12/21/07	Fri 12/21/07
67			Formwork	2 days	Mon 12/24/07	Tue 12/25/07
68			Place WWF	2 days	Wed 12/26/07	Thu 12/27/07
69			Place Concrete	2 days	Fri 12/28/07	Mon 12/31/07
70			Finish	2 days	Tue 1/1/08	Wed 1/2/08
71			Spray Fireproofing	1 day	Thu 1/3/08	Thu 1/3/08
72			**6th Floor**	0 days	Wed 1/2/08	Wed 1/2/08
73			Formwork	2 days	Thu 1/3/08	Fri 1/4/08
74			Place WWF	3 days	Mon 1/7/08	Wed 1/9/08
75			Place Concrete	3 days	Thu 1/10/08	Mon 1/14/08
76			Finish	3 days	Tue 1/15/08	Thu 1/17/08
77			Spray Fireproofing	1 day	Fri 1/18/08	Fri 1/18/08
78			**Lower Roof**	0 days	Thu 1/17/08	Thu 1/17/08
79			Formwork	2 days	Fri 1/18/08	Mon 1/21/08
80			Place WWF	2 days	Tue 1/22/08	Wed 1/23/08



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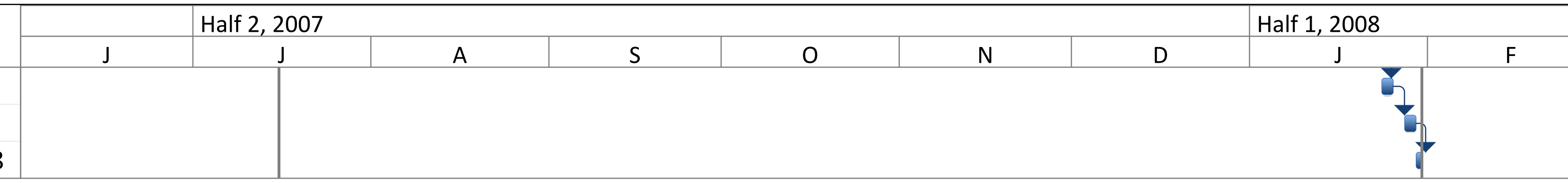
Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

ID		Task Mode	Task Name	Duration	Start	Finish
81			Place Concrete	2 days	Thu 1/24/08	Fri 1/25/08
82			Finish	2 days	Mon 1/28/08	Tue 1/29/08
83			Spray Fireproofing	1 day	Wed 1/30/08	Wed 1/30/08



Project: redesign schedule.mpp
Date: Thu 4/28/11

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