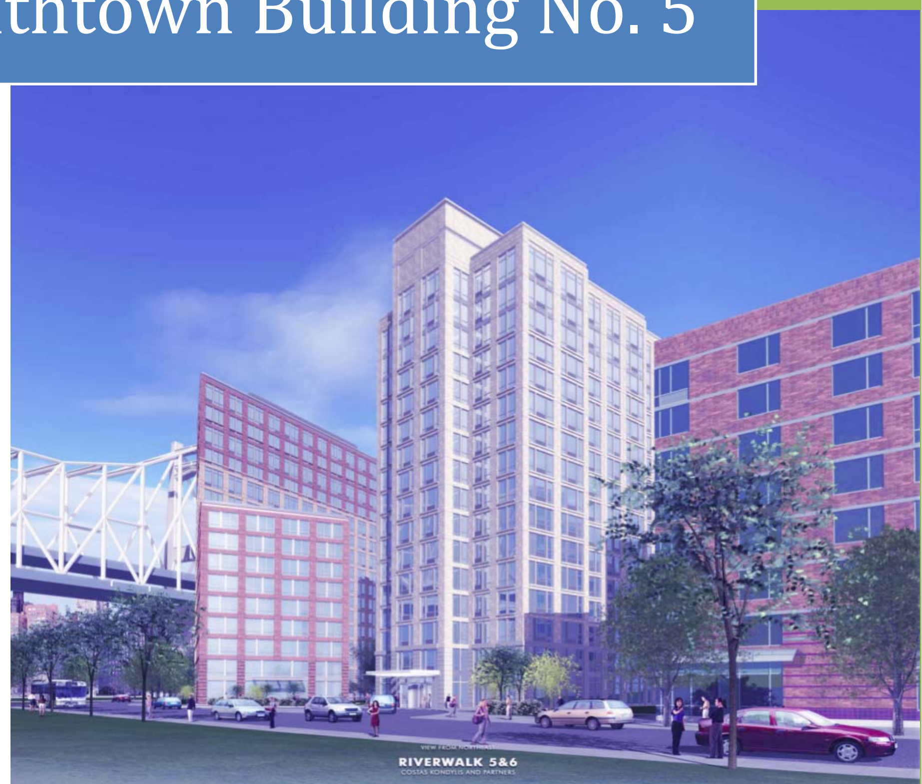
Spring 2008

Senior Thesis Final Report

Roosevelt Island Southtown Building No. 5



Steven Stein

Structural Option

April 9, 2008

Faculty Consultant:

Andres Lepage

ROOSEVELT ISLAND SOUTHTOWN BUILDING NO.5

NEW YORK, NY

PROJECT TEAM

Owner | The Hudson Companies
The Related Companies

Architect | Costas Kondylis and Partners

Structural Engineer | DeSimone Consulting Engineers

MEP Engineer | T/S Associates Mechanical Consultants

Geotechnical Engineer | Soil Mechanics Drilling Corp.

Interior Design | Kondylis Design

General Contractor/CM | Monadnock Construction, Inc.

ARCHITECTURE

Luxury apartment building located on the center of Roosevelt Island

Offers 123 apartments and beautiful views of Manhattan's skyline

Facade predominately consists of brck cavity walls with a white granite base

STRUCTURAL

Mainly cast-in-place reinforced concrete structure

12" Foundation wall around perimeter

Rectangular footings at base of each column with a large format foundation at base of shear wall

Rectangular concrete shear walls around elevator core act as primary lateral load resistance

Floor system consists of 8" two-way reinforced concrete flat plate floor slabs

BUILDING STATISTICS

Size| Approximately 130,000 sq. ft.

Stories | 16 stories

Cost | Approximately $52,000,000

Construction Dates | June 2007-October 2008

Delivery Method | Guaranteed Maximum Price

MECHANICAL

2 - 250 hp natural gas fired steam boilers with instantaneous domestic hot water coils rated at 10,461 MBTU

12- ½ ton capacity gas fired package heating and cooling rooftop HVAC unit services the corridors

2 Ceiling mounted package AC system with a hot water duct steam –reheat coil services the lobby

Each residential room (living room and bedrooms) have a P-TAC unit – ranging from 12,100 – 16,030 BTUH cooling capacity and 26,300 BTUH heating capacity

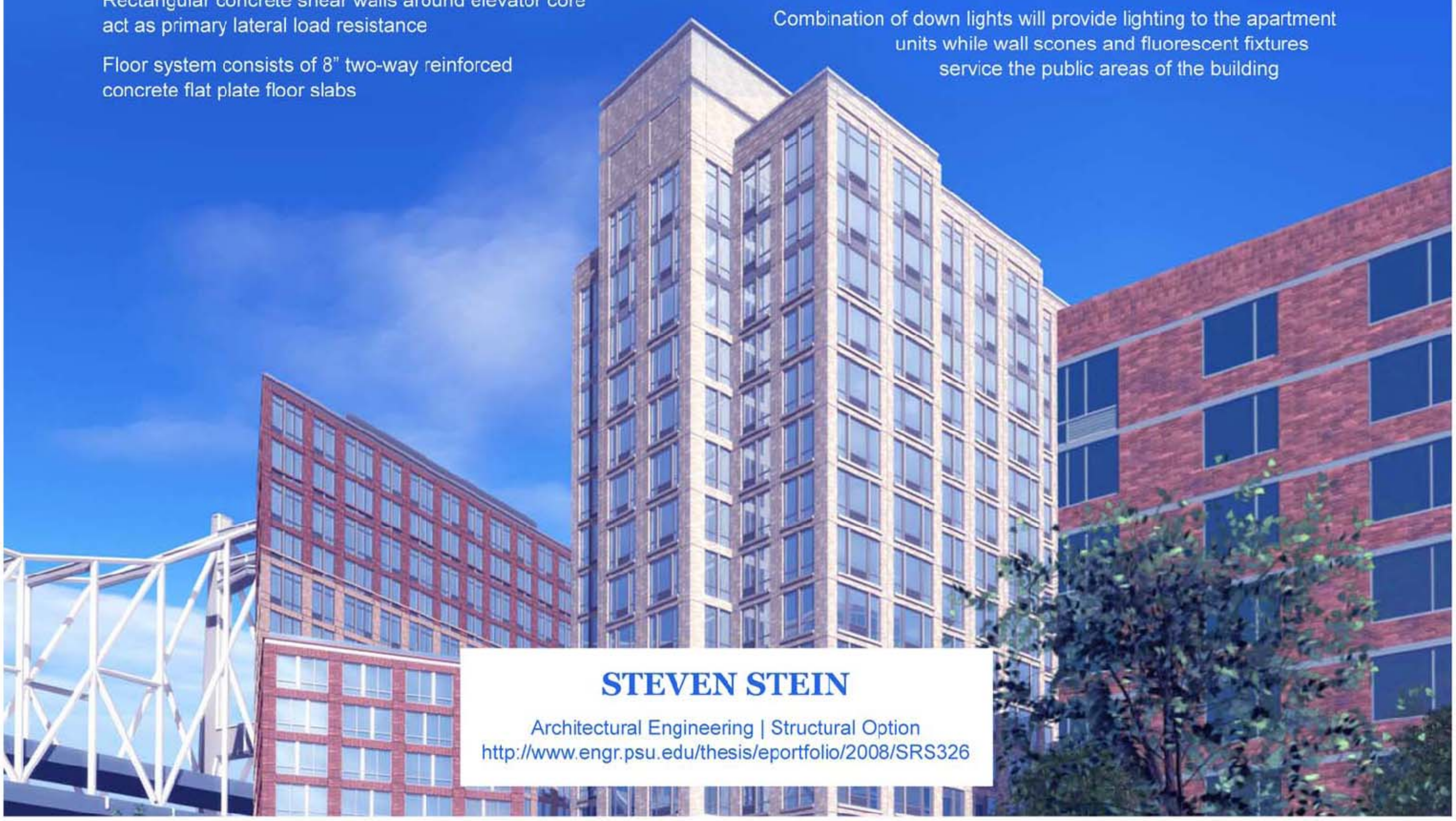
LIGHTING|ELECTRICAL

208/120V 3 phase, 4 wire main feeder

4000 Amp main electrical switchboard located in cellar

All apartments will have 125 ampere, 208 volt, and single-phase panels

Combination of down lights will provide lighting to the apartment units while wall scones and fluorescent fixtures service the public areas of the building



STEVEN STEIN

Architectural Engineering | Structural Option
http://www.engr.psu.edu/thesis/eportfolio/2008/SRS326

TABLE OF CONTENTS

Executive Summary

Southtown Building No. 5 is a luxury residential building located in the center of Roosevelt Island in Manhattan's East River. It houses 123 condominiums in 16 floors with an underground cellar which houses storage units as well as mechanical and electrical space. The building is the fifth out of nine apartment buildings in a development that is planning to revitalize the once industrial Roosevelt Island into a place in which people will live, work, and play. The apartment building also houses a full service lobby with a concierge, mail room, health club, multi-purpose room, children's play area, party room, and rooftop terraces.

With the building being located in New York City, height restrictions play a significant role in building construction. Having a height limitation of 187 feet from the datum, floor-to-floor heights as well as floor thicknesses are a very important factor. With this in mind, the structural engineer utilized a reinforced concrete flat plate floor slab with a thickness of 8" for typical floors. This type of system allowed for a fairly wide open layout for the architect's one, two, and three bedroom condos.

This report focuses on an in-depth study of engineering for an alternative structural steel system to the replace the existing cast-in-place concrete system. To keep floor thickness as close to 8" as possible, a Girder-Slab system was utilized for typical floors. This system consists of prefabricated steel beams, known as D-beams, which support prestressed hollow core floor planks. Additionally, the ground floor was designed as a composite steel and concrete deck system to allow for heavier loads. Additionally, the lateral system was redesigned from the original cast-in-place concrete shear walls to concentric braced frames to keep continuity of the steel system.

In addition to the depth study of an alternative steel system, two breadth studies were completed. The first breadth study analyzed construction management topics associated with the redesign of the structural system. This study compared existing costs and schedules associated with the concrete system to the new cost and construction schedule of the steel system. The second breadth study involved extensive research into achieving a LEED certified building. In addition to twenty-six points that were analyzed and applied to Southtown Building No. 5, a building energy analysis was performed to compare the existing PTAC air-conditioning units to an alternative air source heat pump system.

Introduction

Building Information and Architecture

Roosevelt Island Southtown Building No. 5 is located in the center of Roosevelt Island, just off of the coast of Manhattan’s eastern shoreline in the East River. The building is located just north of the Queensboro Bridge between Southtown Building No. 4 and Southtown Building No. 6.

Figure 2: Overhead view of Manhattan. Southtown Building No. 5 (yellow solid) is located on Roosevelt Island which is boxed in red.

Southtown Building No. 5 is the 5th building in a large scale new development. The building will house 123 luxury condominiums consisting of 1, 2 and 3 bedrooms. It will also house a full service health club, a children's day care center and additional occupant storage. The overall building is estimated to be a total of 130,000 sq. ft.

The building is 16 stories above ground with a one story cellar below grade. The building, therefore, is a total of 17 stories with an elevated mechanical, electrical, and plumbing room as well as an elevated elevator mechanical room.

Southtown Building No. 5 primarily utilizes light colored brick cavity walls for its façade. Large floor to ceiling windows make up a large percentage of the building's frontage. However, the base of the building has 4' of granite clad precast panels, followed by a two stories of a composite stone cavity wall. An artist's rendering of the building can be seen below in Figure 3.

Figure 3: Artist's rendering of Southtown Building No. 5 at dusk.

Mechanical

The heat source for the building's residential units consists of two (2) 250-hp natural gas fired steam boilers with instantaneous domestic hot water coils rated at 10,461 MBTU. A 12- ½ ton capacity gas fired package heating and cooling rooftop HVAC unit services the corridors. Two (2) ceiling mounted package AC system with hot water duct steam and reheat coils service the lobby. Each residential room (living room and bedrooms) have an incremental unit (P-TAC unit) – ranging from 12,100 – 16,030 BTUH cooling capacity and 26,300 BTUH heating capacity.

Electrical/Lighting

Power for this building is supplied via a 208/120V, three phase, four wire feeder to a 4000 amp main electrical switchboard. All apartments have 125 amp, 208 volt, and single-phase panels. A combination of down lights will provide lighting to the apartment units while wall scones and fluorescent fixtures service the public areas of the building.

Existing Structural Systems

Foundation

Three types of foundation systems are used for Southtown Building No. 5. Individual footings are used for interior columns of the building. These footings range from 3’-0” x 3’-0” to 4’-6” x 4’-6”. A mat footing is used at the base of the lateral force resisting shear walls. The mat is typically 42” thick with some step downs required for the elevator, boiler, and sump pits. Finally, a 12” thick foundation wall is used around the perimeter of the cellar. This system incorporates exterior concrete piers into the wall with footings at the base.

Columns

The columns in this New York building are typically rectangular reinforced concrete with varying sizes and reinforcement. The most common column size is 14” x 24” with 8 #6 bars as structural reinforcement. Column loads vary greatly within the building, especially as the elevation rises. The largest loads at the foundation level is 1056 kips of dead load and 139 kips of live load.

Floor System

The existing floor system for Roosevelt Island Southtown Building No. 5 is typically an 8" two-way normal weight reinforced concrete flat plate. Due to the building's primary use as a residential apartment building, there is no typical bay size to allow for different apartment layouts. The largest bay size, however, is 24'-0" x 26'-0". At the cellar floor, a 6" concrete slab is used with W2.0 x W2.0 welded wire fabric. At the first floor, a 9" concrete slab is used to accommodate for higher occupancy loads. Typical reinforcement for the 8" floor system is #4 @ 14" bottom steel and #4 @ 14" top steel. Middle strip reinforcement is used in the floor slab in some areas of higher stress. Additionally, several extra bars are required at the faces of the columns to resist the higher moments. The columns in the residential high rise are typically reinforced concrete. The most standard size for these is 14" x 24" with 8 #6 bars as structural reinforcement.

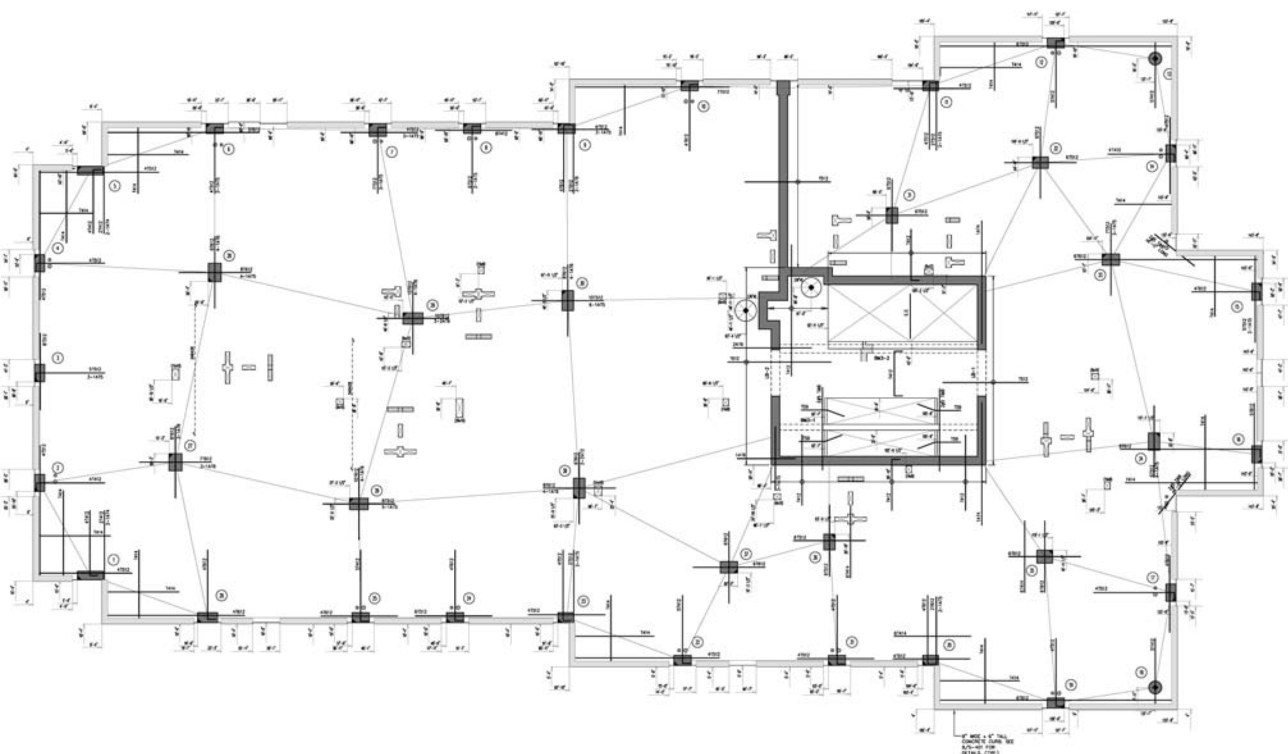


Figure 4: Typical structural floor plan of Southtown Building No. 5

Lateral System

Reinforced concrete shear walls make up the lateral force resisting system of the building. The elevator and stairwell core in the center of the building have been assigned as the location of these shear walls. The shear walls rise from the cellar level of the building all the way to the elevator mechanical room. A 12" typical shear wall section consists of #4 @ 12" horizontal reinforcement and #5 @ 12" vertical reinforcement. Openings in the shear walls require link beams in order to transfer high shear forces from one side of the opening to the other. The concrete used in the shear walls vary with the height of the building from 7ksi in the cellar to 5ksi at the roof.

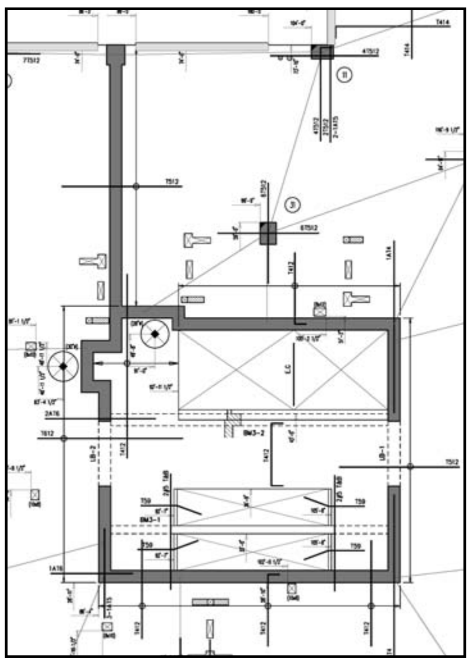


Figure 5: Typical reinforced concrete shear walls located around the building's elevator and stairwell cores

GRAVITY AND LATERAL LOADS

Dead Loads

1. Construction Dead Load
    a. Cellar Floor: 75 psf
    b. 1st Floor: 113 psf
    c. 2nd -16th Floor: 100 psf
    d. Main Roof: 113 psf
    e. Mechanical Room Floor: 100 psf
    f. E.M.R Floor: 100 psf
    g. E.M.R Roof: 100 psf
2. Superimposed Dead Load
    a. Cellar Floor: 25 psf
    b. 1st Floor: 30 psf
    c. 2nd -16th Floor: 20 psf
    d. Main Roof: 50psf
    e. Mechanical Room Floor: 25 psf
    f. E.M.R Floor: 25 psf
    g. E.M.R Roof: 25 psf

Live Loads

1. Cellar Floor:
    a. Equipment Rooms: 100 psf
    b. Offices: 50 psf
2. 1st Floor:
    a. Public Area: 100 psf
    b. Residential: 40psf
3. 2nd – 16th Floor: 40 psf
4. Main Roof: 100 psf – Public Area, Mechanical, Storage
5. Mechanical Room Floor: 100 psf
6. E.M.R. Floor: 100 psf – Mechanical + Machine Weight
7. E.M.R. Roof: 30 psf

Depth – Structural Redesign

Proposal

Due to the location of the project and the needs of the client, the most efficient structural system was originally designed. This became apparent in the previous Technical Assignment #2 where the existing floor system was compared to four other proposed systems. The intention of this proposal will be to redesign and reevaluate the current cast-in-place reinforced concrete system to a Girder-Slab system using asymmetrical steel beams and precast concrete planks. The serviceability and strength of the new system will be checked using codes and loads from the following but not limited codes: IBC, ASCE7-05, and AISC

Problem Solution

Floor System

The proposed floor structure to be analyzed and implemented is the Girder-Slab system. It is a new and innovative way to build that combines the benefits of steel and concrete into one monolithic floor system. The system is comprised of an interior girder known as an open-web dissymmetric beam, or D-Beam, which supports precast, prestressed hollow-core slabs on its bottom flange. The D-Beams also have openings in some of the web to allow for grouting of the hollow core planks. Upon grouting, the system develops composite action and is able to resist lateral movement between the planks and beams.

After the initial calculations performed in Technical Assignment #2, DB8x42 beams were chosen with an 8” x 4’-0” hollow core plank. The construction and economic costs associated with this system will be analyzed and reviewed. The Girder-Slab may prove to have a shorter erection time but a longer lead time than cast-in-place concrete. The erection time will allow for a quicker construction time but may be outweighed by the initial costs of the system. These topics as well as others will be reviewed and compared in the following pages.

Lateral System

In order to allow for the fastest erection time, a lateral resisting system consisting of diagonal braced frames will be investigated. Braced frames will be located around the elevator core and central stairwell. The braced frames will have to be able to resist the seismic and wind loads set by previous Technical Assignments. This lateral system must be able to resist any torsion effects created by these loads.

Structural Gravity System

Girder-Slab Floor System

Girder-Slab is a steel and precast plank flooring system developed by Girder-Slab Technologies LLC. This is the first hybrid floor system that fully integrates steel and precast planks to create a monolithic slab assembly. Specifically targeting mid to high-rise residential construction, The system is comprised of an interior girder known as an open-web dissymmetric beam, or D-Beam, which supports precast, prestressed hollow-core slabs on its bottom flange. The D-Beams are cut from a parent wide flange section which produces two D-beams which can be seen in Figure 6. Typically, there are two basic D-Beam sections that will work with the use of 8" pre-cast slabs, DB-8 and DB-9. Beams are corrugated when cut in half which allows for grout to flow through the web and the hollow core plank openings. Upon grouting, the system develops composite action, allowing it to support residential live loads. The transformed Girder-Slab section is able to withhold over twice the moment capacity of a sole D-Beam. The Girder- Slab system will also reduce

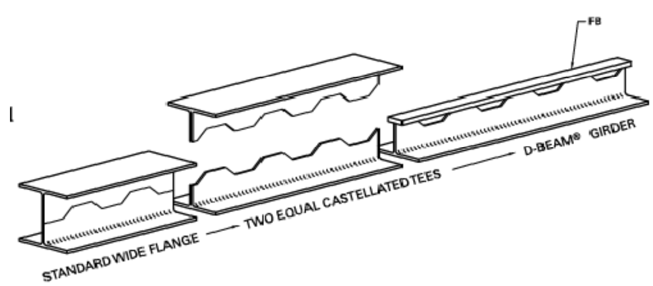


Figure 6: Two D-Beam Girders cut from a parent Wide-Flange

Figure 7: Composite transformed D-Beam Section

labor costs and improve construction operations. Girder-slab system and D-Beam girders are only distributed and assembled in New Jersey by Girder-Slab Technologies LLC.

In order to implement the Girder-Slab flooring system, the layout of the columns in the building needed to be changed. To allow for the floor planks to be aligned properly, an orthoganol grid was created. The Girder-Slab system was used for typical floors 2-16 during the structural redesign. In order to match the existing thickness of the flat plate floor slab (8”), DB-8’s were chosen with an eight inch hollow-core floor plank. In order to level the floor from differential deck cambers, a ¾” concrete topping will be used. The typical Girder-Slab layout can be seen in Figure 8.

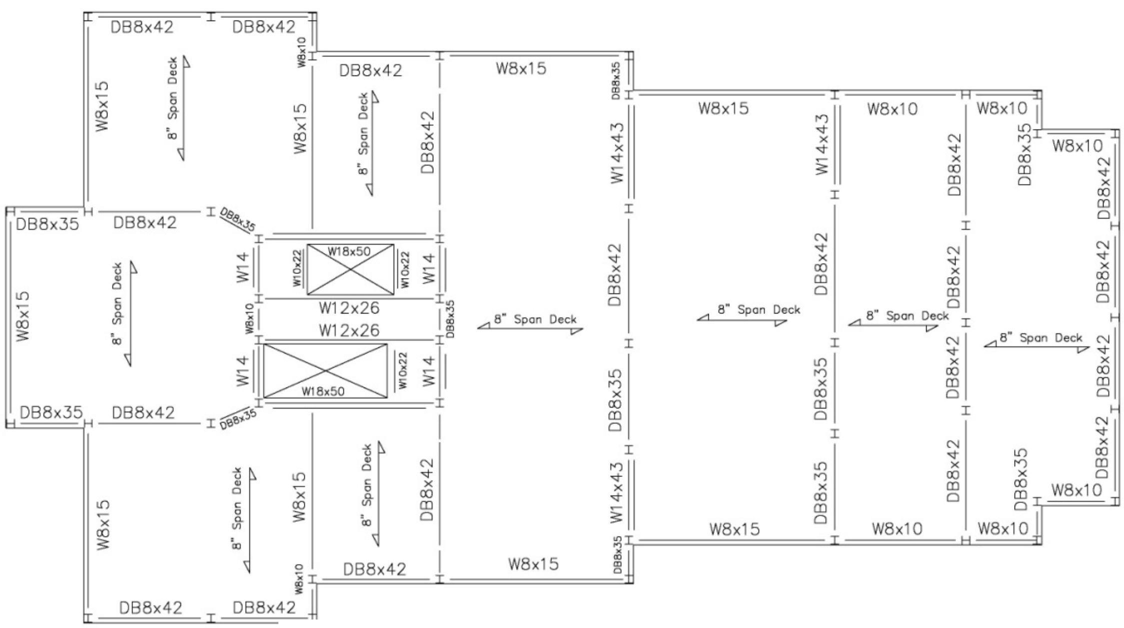


Figure 8: Typical floor plan utilizing Girder-Slab.

Typically, eight inch pre-cast planks will span the long direction of the bays, while DB8x42 will span the short directions. In some cases, DB8x35 will be used when acceptible. Along the plank edges, Wide-flange members will be used to span this long direction. J952 8” x 4’ Span Deck planks with 6-1/2” Ø strands will be used in the Girder-Slab system.

On the roof level, larger D-Beams were necessary to account for the increased live load. At this level, DB9's replace the DB8's in order to achieve the same spans while still maintaining the Girder-Slab system. Addtionally, a rubberized roof ver plank will be used to replace the hollow core concrete planks to withstand any severe weathering that may occur on the roof.

The Girder-Slab system was designed in accordance with the design specifications presented in the Girder-Slab Design Guide. This design guide uses Allowable Stress Design (ASD) calculations in accordance with with American Institute of Steel Construction (AISC).

When determining the types of D-Beams to use, the system must be checked twice, once for pre-composite action, and one for full composite. Pre-composite action occurs prior to grouting the system and before any curation takes place. The only load used in this design check is the weight of the pre-cast hollow core planks. Moment and deflection calculations are performed to make sure they meet the allowable criterion for the steel section. After the grout is injected and has cured, the transformed section is analyzed with the addition of the dead load of the plank, any superimposed dead load, and the live load per the occupancy (in accordance with ASCE7-05). At this point, the required section modulus is calculated and compared to the given transformed sections of the composite steel. Equation 1 exhibits how to calculate the required section modulus.

Equation 1:

Table 7: Existing Concrete System	
Floor Slabs	$2,036,000
Columns	$929,000
Shear Walls	$240,000
Foundation Walls	$76,000
Spread Footings	$10,900
Mat Foundation	$56,000
Total:	$3,347,900

Where: M_{TL} is the bending moment due to total loading

Fy is the yield strength of the steel

Deflection of the composite section is also checked against the allowable deflection criteria. In this case, the deflections were compared against the industry standard of L/360. Superimposed compressive stress on the concrete is checked against the allowable compressive stress. Next, the bottom flange tensile stress is checked for the total load and compared to the allowable yield stress of the steel section.

Equation 2: $$f_b = \frac{M_{DL}}{S_b} + \frac{M_{SUP}}{S_{b(Transformed)}} \leq 0.9F_y$$

Where: Sb is the section modulus of the D-beam before composite action

Sb(Transformed) is the section modulus of the transformed section

Finally, the last strength check is calculating the allowable shear stess of the D-Beam against the total loading.

Equation 3: $$f_v = \frac{R}{t_w b} \leq 0.4F_y$$

Where: R is the support reaction.

Figures 9 and 10 show some Girder-Slab details associated with the floor system.

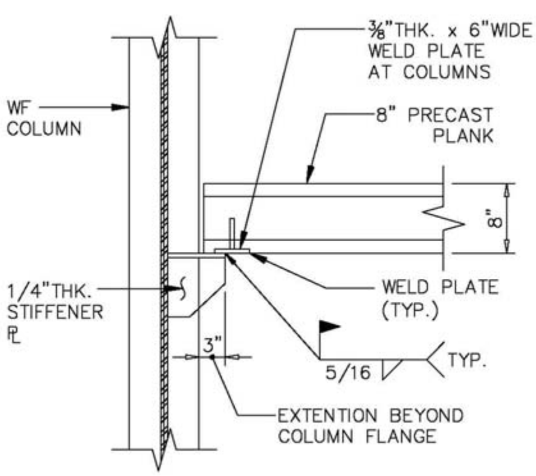


Figure 9: Typical precast plank details at columns

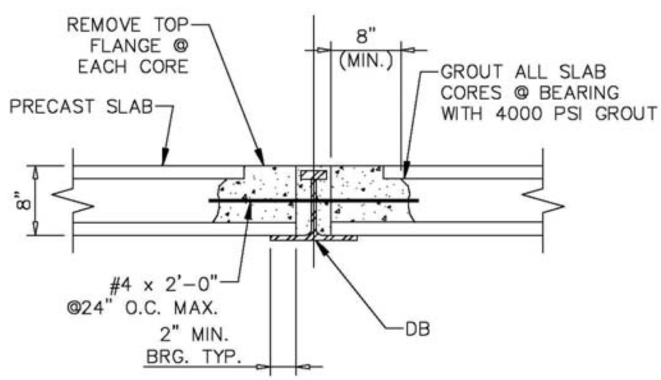


Figure 10: Typical detail of Girder-Slab system at D-Beam section

Girder-Slab Tree Columns and Connections

In order to use DB8 girders and hollow-core floor planks, wide-flange columns were designed for the gravity system of Southtown Building No. 5 as seen in Figure 4. Column sizes were determined using RAM Structural System and checked using hand calculations. To make the construction process more economical, the columns only change in size every 5 stories.

However, in order for some of the DB8x42 beams to span up to 19’, a “tree” column was utilized in some areas. A tree column is a WT section that is welded to a wide-flange column with a bevel weld and a fillet on both sides. This connection allows for a larger span of the D-beam without sacrificing any structural integrity. When selecting the WT shape to use, the same depth must be achieved as that of the D-Beam. In this particular case, a DB8 was used and therefore a WT8 section was selected. A typical connection was designed and a WT8x22.5 section was selected. Since the WT shape must be welded to the column, there will be a negative moment caused by this fixed connection of 44.5 ft-kips. Additionally, the WT will receive a shear force from the D-Beam of 19.8 K through a single plate connection with two bolts in each member. After sizing the WT shape, a 9” x 6” x ¾” plate with 7/16” A325N bolts will be able to resist the shear. Calculations for member sizing and connection may be found in the Appendix.

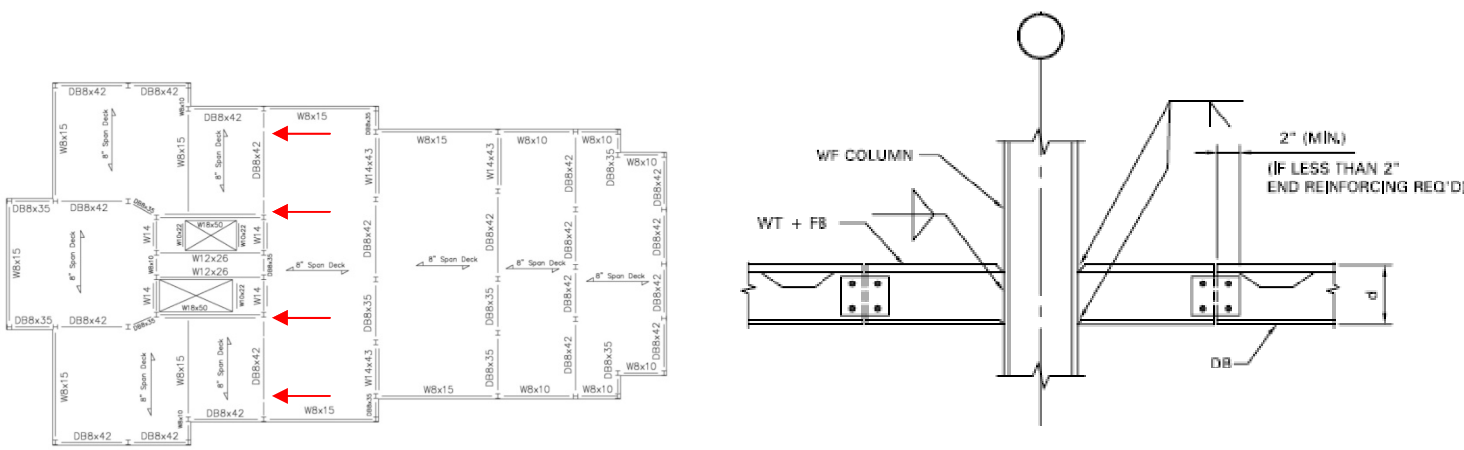


Figure 11: At left, red arrows indicate tree column locations located on a typical floor plan. At right, typical tree column connection utilizing WT shapes and single plate connection

Since tree column connections are very costly, they are only to be used in areas where the D-Beam spans over 16 feet. In such areas with shorter spans, an unstiffened seat connection is utilized. A typical connection for a DX8x42 spanning into the flange of a W12x96 was designed using Table 8-4 of AISC Steel Manual, 13^{th} Edition. Once picking a steel angle size, limit states for web crippling, web yielding, seat angle flexure, angle shear yielding, weld rupture were checked. After the angle size met all requirements, a 3/8" fillet weld on both sides of the angle was determined adequate. A stabalizing angle was used at the top of the D-Beam in order to resist flexure and rotation of the D-Beam. A detail can be seen below in Figure 12.

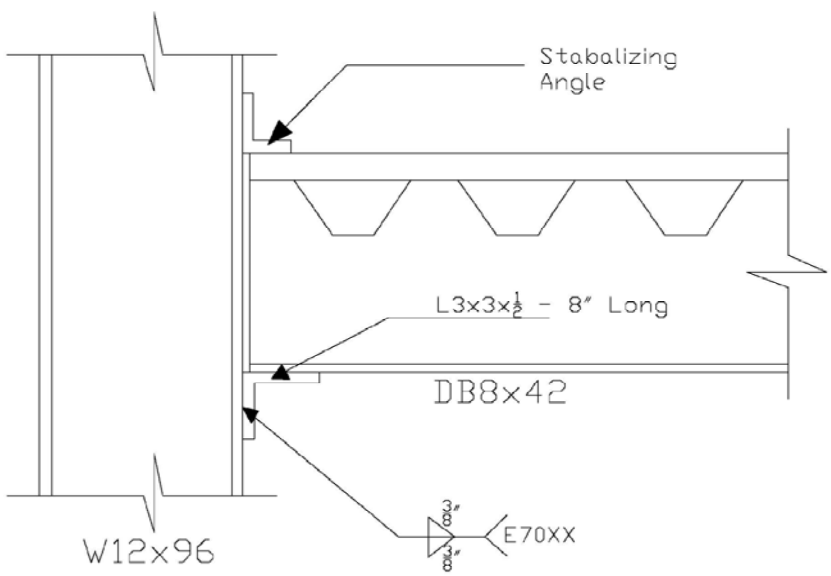


Figure 12: Connection detail using unstiffened seat connection at D-Beam – Column Flange location

Composite Beam Floor System

The first floor level of Southtown Building No. 5 was designed using a composite beam and concrete slab system. This system was chosen due to the increased live load per occupancy. Floor thickness was not critical in this floor because of the cellar bellow. Since the cellar is going to be an unoccupied space, the added thickness of the wide-flange girders will not add to the overall height of the building but detract from the story height of the cellar. With the first floor consisting of mostly public space, including the lobby, day care center, storage, fitness center, and conference room, the live load was increased from 40 psf of a typical residential floor to 100 psf. Using the column grid that was created by the Girder-Slab system above, the composite floor system has multiple bay sizes. The most common bay size, however, is a 27'-0" x 26'-0". Composite concrete and metal deck span perpendicular to beams spanning the 26'-0" distance and spaced 6'-9" o.c. Sixteen feet intermediate beams will then frame into the girders spanning 27'-0" which will, in turn, frame into the web of the wide-flange column.

The metal decking that was chosen was a 20 gauge USD 2" Lok-Floor deck with 4" concrete slab above for a total of six inches. The concrete compressive strength used was 3000 psi. The decking chosen was rated to span 7.85 feet without the use of shoring. The loading capacity of this deck is rated at 400 psf. Metal studs used were 4" x ¾" diameter, Grade 60.

The composite beams and girders were designed using American Institute of Steel Construction (AISC) Manual 13th Edition, Allowable Strength Design (ASD). The load combination used for this particular calculation was D+L. When designing a 27'-0" x 26'-0" bay by hand, a W12x19 with 14- ¾" diameter studs was found to be efficient as an intermediate beam. The maximum moment at midspan for this 26'-0" span was found to be 43.6 ft-kips.

The controlling factor for these intermediate beams was the deflection limitation. A moment of inertia that was required to limit the deflection to L/360 was figured to be 40.75 in^4. This moment of inertia was determined for a construction loading. The construction loading includes self-weight of the structure (i.e. deck, concrete, studs, and beams), as well as workers and equipment.

The deflections of the beams were also checked against live loads and total loads after the concrete cures and the system become fully composite. Given the industry standard of L/360 = 0.87". All beam sizes were well within this deflection criterion.

Once the intermediate beams were sized, a typical girder spanning 27'-0" was designed using a similar process. A maximum moment was found to be 121 ft-kips. By setting the deflection equation of a simple supported beam with 3 equal concentrated loads equal to the deflection limit of L/240. By manipulating this equation, a moment of inertia was found to be I_x= 448 in^4. This equation can be seen below.

Equation 4: $$f_b = \frac{M_{DL}}{S_b} + \frac{M_{SUP}}{S_{b(Transformed)}} \leq 0.9F_y$$

RAM results produced typical sizes for intermediate beams of W12x14 (18 studs) in this particular bay. This girder was checked against the necessary Ix value needed to limit the deflection. The girder in this bay spanning the 27'-0" length was sized W16x31 by RAM which was also checked against limiting deflection.

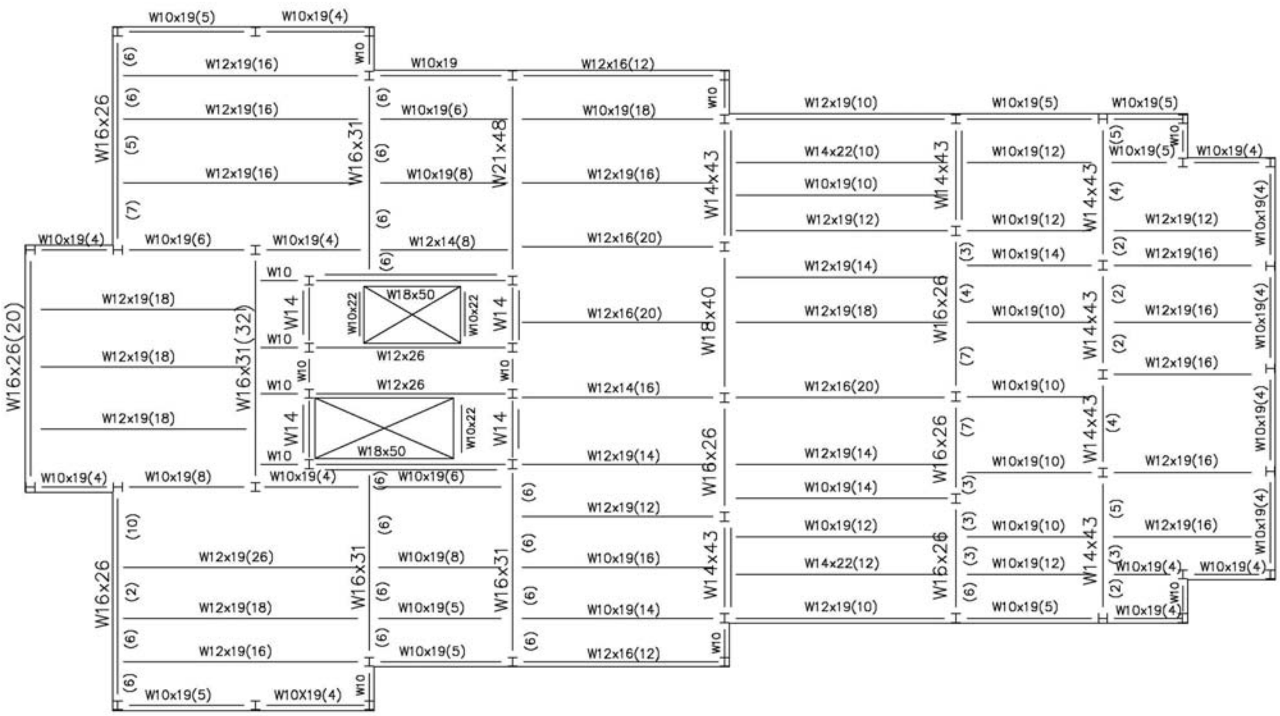


Figure 13: First floor structural plan utilizing steel beams and girders with composite steel deck

Composite Floor Connections

Three shear connections for the composite floor system were designed. All beams and girders that were modeled in RAM were assigned as pin-pin connections. In a typical bay, the three connections that were designed were 1) girder-web to intermediate beam-web, 2) girder-web to column-web and 3) beam-web to column-flange. The three connection types can be seen below in Figure 14.

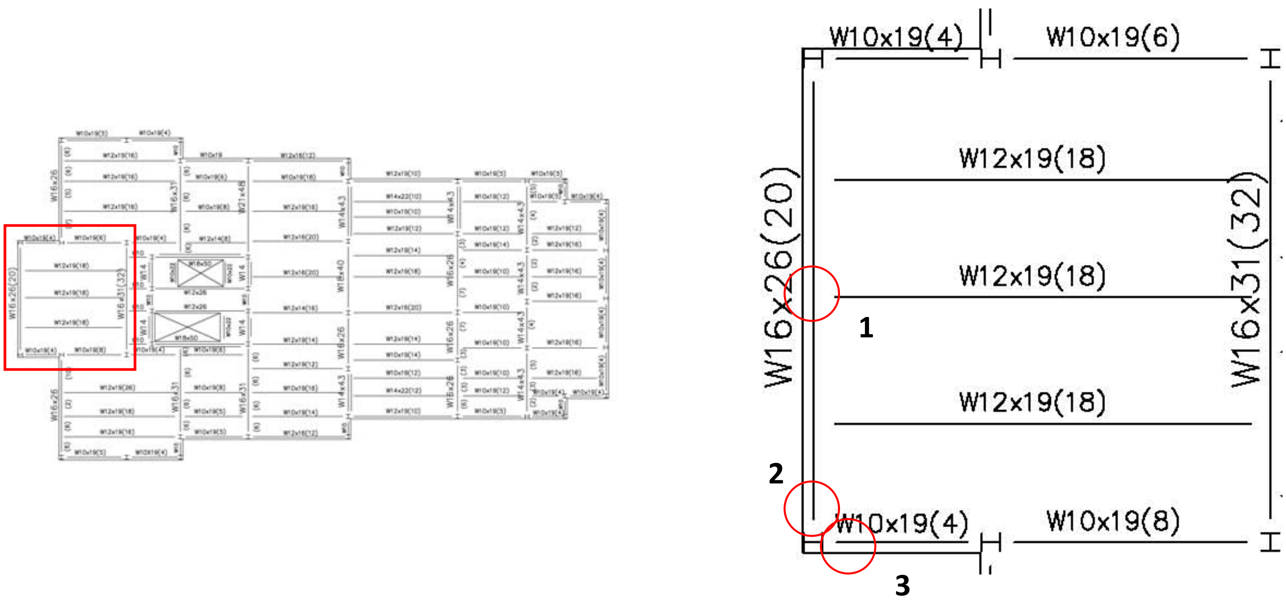


Figure 14: Typical composite steel bay with connection design locations. Typical bay located in red box on left.

Eccentric weld tables in Chapter 8 and design aids in chapter 10 of AISC Steel Manual, 13th Edition were used in the design of all connections. All calculations can be seen in the Appendix.

For connection 1, a 6"x5"x1/4" shear tab was used with 2-3/4" A325 Type-N Bolts. The beam will be coped at the top to allow for a level floor surface and ease of construction. A 3/16" E70XX fillet weld will be used to connect the shear tab to the web of the girder. This connection can be seen below in Figure 15.

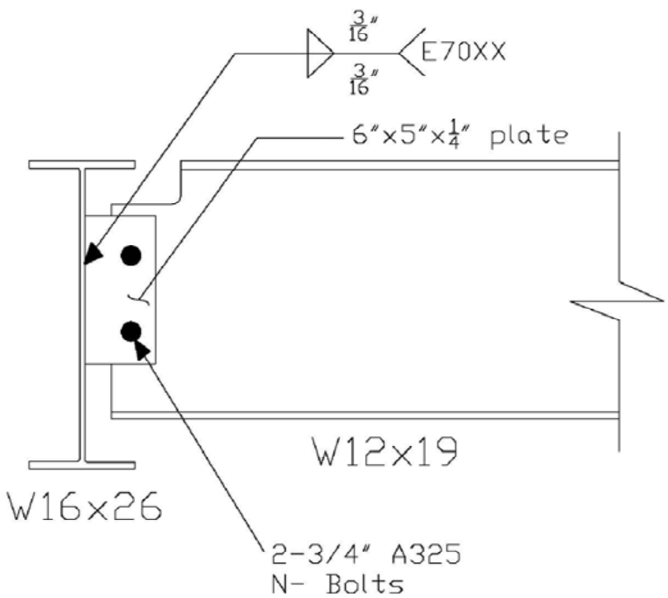


Figure 15: Shear tab connection at beam web
– girder web location

For connection 2, a bolted/welded single angle connection was used. An 11-inch L4"x4"x3/8" single angle was utilized with 4-3/4" A325 Type-N Bolts. A 3/16" E70XX fillet weld will be used to connect the angle to the column web. A 3/8 inch weld return is employed at the top of the single angle. This connection can be seen below in Figure 16.

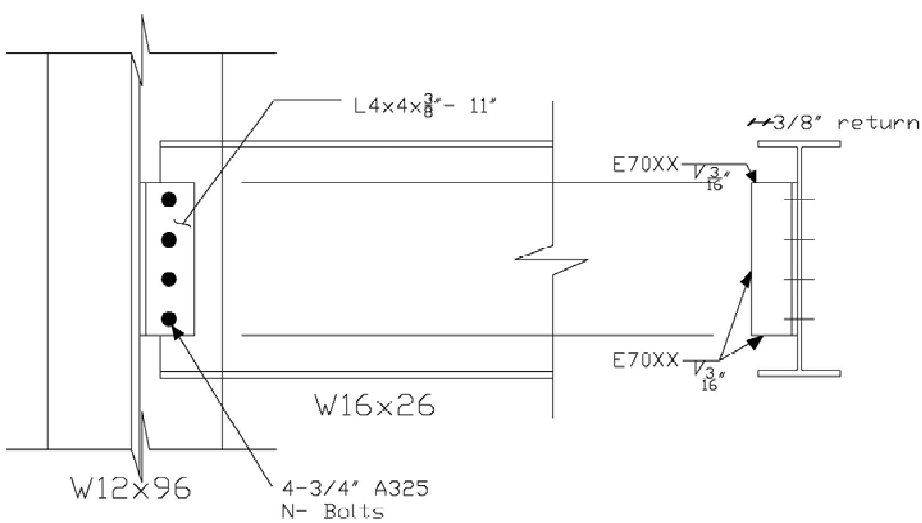


Figure 16: Bolted/welded single angle
connection at girder web- column web location

Finally, for connection 3, bolted/welded double angle connection is used. A 6 inch L3”x3”x1/4” double angle was used with 2-3/4” A325 Type-N Bolts. The beam will be coped at the bottom for constructability. A 3/18” E70XX fillet weld will be used to connect the double angles to the flange of the column. This connection can be seen below in Figure 17.

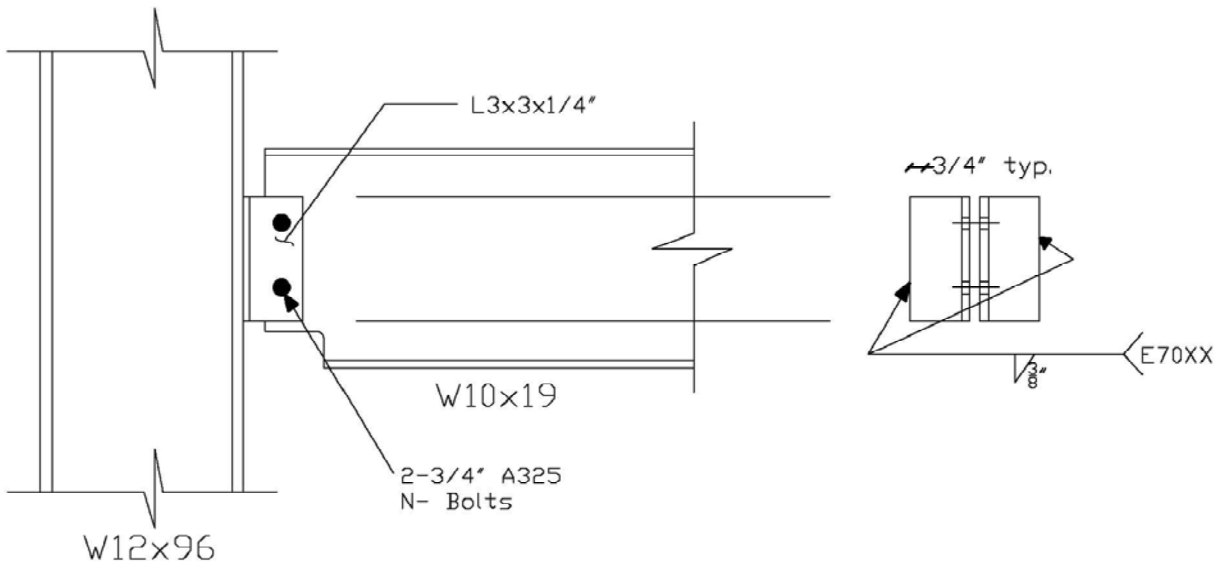


Figure 17: Bolted/welded double angle connection at beam web-column flange location

Gravity Columns

Gravity columns in Southtown Building No. 5 were designed by RAM Structural System and cross checked with hand calculations. All calculations were done using Allowable Strength Design in accordance with the AISC Steel Manual, 13th Edition.

Columns were designed to be spliced at every 4 floors. This was purposely designed this way to allow for the fastest erection time possible. In a four-floor tier, the raising gang will erect the first two levels of framing and the decking crew will pour the topping material and level out the 2nd floor. After this, the raising gang will continue with the 3rd and 4th floors as the decking crew continues with the 1st floor. Following completion of the 1st floor, the decking crew continues with the 4th floor as the raising gang continues to the next tier. Finally, the decking crew finishes by decking the 3rd floor and the process repeats.

Columns were checked at three splice points, 4th, 8th and 12th floors. Most bays in the typical floors did not require tree columns for the Girder-Slab system to work properly but a few columns would require this connection type. For these particular columns, a combined loading of axial and bending occurs at the column. As previously mentioned, the designed tree column would be subjected to a 50 ft-kip moment. For these few columns, interaction equation H1-1a governed.

Equation 5: H1-1a: $\frac{\mathrm{Pr}}{Pc}+\frac{8}{9}\left(\frac{Mr}{Mc}\right)\leq 1$

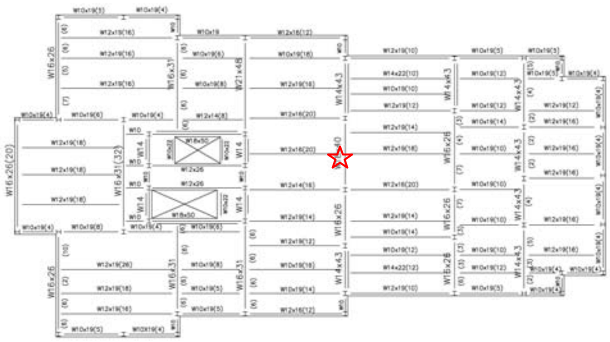
Where P_r is the axial load on column

M_r is the bending moment on column

P_c is the axial strength of column

M_c is the bending strength of column

For all other columns that did not utilize a tree column connection, the design was based purely on axial loading. This total axial force was determined via column load take down which can be seen in the appendix. The axial force in a typical interior column was determined and then cross checked with the RAM results. Less than a 5% difference in error was found. For this same column, an elevation of the column line can be seen in Figure 18.



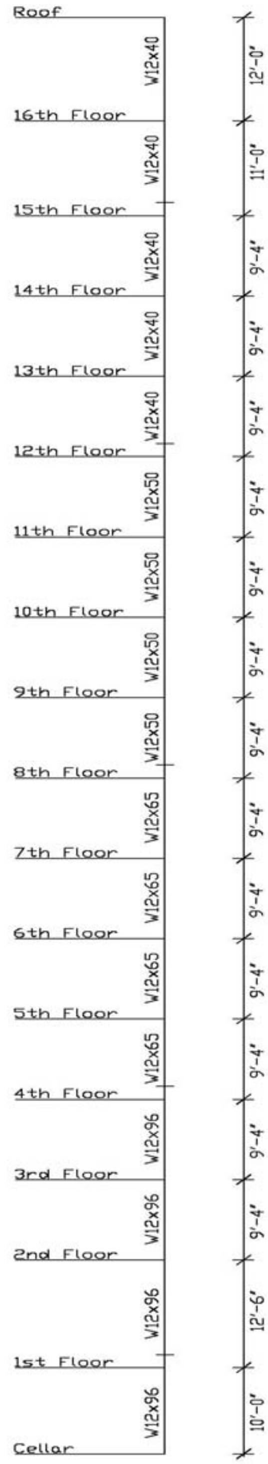


Figure 18: Gravity column line located at red star on floor plan above

Footing Redesign

With the proposal of switching from a concrete structural system to a steel system, comes the added benefit of reduced gravity loads. The original foundation wall is intended to stay in place to resist cladding and exterior column loads. Additionally, the mat foundation under the original shear walls can remain the same in depth and steel reinforcement but will be reduced in length since there is no long wall along the top in the North-South direction. This reduction in concrete mat will not add to the reduction in overall concrete since there will be newly placed braced frames in the North-South direction as described later in the Lateral Force Resisting System.

When redesigning typical interior footings, it was expected that the overall size of the footing would be reduced. However, when designing an interior footing, the size of the footings increased. This can be attributed to the larger spans and the reduced number of interior footings. With the use of steel beams instead of a flat plate floor system, the number of interior gravity columns was reduced from 13 on a typical concrete floor to 8 on a redesigned typical steel floor. Therefore, although individual footings will be increased in size, the overall amount of footings is reduced.

At the base, a ground column axial force of 617 kips must be transferred to the ground. As specified in the geotechnical report, an allowable bearing capacity of 12,000 psf can be used for foundations. At this interior footing location, a W12x96 was used. A 26” x 26” x 3”base plate was designed in according with AISC Steel Manual, 13th Edition. The base plate would be welded to the column and 4 anchor bolts would transfer the axial forces into the concrete pier. At this point, the concrete pier would then transfer the axial force to the footing.

The footing was designed by hand and cross-referenced with the Concrete Reinforcing Steel Institute (CRSI) Handbook, 2002. The design produced a 6’-0” square footing, 26 inches deep with (9) #6 bars in each direction compared to the existing footing, a 4’-6” square footing, 30 inches deep with (8) #8 bars in each direction. The new design requires 2.9 cubic yards per footing while the old design requires only 1.875 cubic yards. However, taking into account that there are 5 more interior footings in the original design, there is less overall concrete used in the proposed foundation design.

Structural Lateral System

Wind Design

Wind Loads were computed using Chapter 6 of ASCE7-05. Basic wind speeds for New York City were taken as 110 mph. The building exposure category was chosen to be C. The general parameters of the wind calculations can be seen in Table 1 below.

Table 1: General parameters	
Classification Category:	II
Basic Wind Speed, V:	110 mph
Importance factor, I:	1
Mean recurrence interval:	50 year
MRI factor:	1
Exposure Category:	C
a:	9.5
zg:	900
Topographic factor, Kzt:	1
Wind directionality factor, Kd:	0.85
Gust Factor, G (x-dir wind):	1.01
Gust Factor, G (y-dir wind):	0.969
Internal pressure coefficient, +GCpi:	0.18
Internal pressure coefficient, -GCpi:	-0.18
Windward pressure coefficient, Cp:	0.8
Side pressure coefficient, Cp:	-0.7

When sizing the lateral system for Southown Building No. 5, hand calculated wind forces were inputted into a RAM frame model. The applied story forces were then analyzed and found to be within 5% of the hand calculated story forces. Table 2 below shows a comparison between hand calculated story forces and RAM output story forces.

Table 2: Wind Applied Story Forces (kips)					
Level	Height (ft)	Manual	RAM Ouput	Manual	RAM Output
		N-S	N-S	E-W	E-W
Roof	157	39.9	43.11	20.3	21.92
16	145	75.7	79.26	38.4	40.2
15	134	68.4	72.24	34.6	36.54
14	124	61.3	65.32	31.0	33.03
13	115	57.5	61.61	29.0	31.08
12	106	60.1	64.24	30.3	32.39
11	96	59.3	62.6	29.9	31.57
10	87	55.5	59.26	27.9	29.79
9	78	57.8	61.28	29.0	30.75
8	68	56.9	59.9	28.4	29.9
7	59	53.0	55.19	26.4	27.49
6	50	54.8	55.53	27.2	27.56
5	40	53.4	54.87	26.4	27.12
4	31	49.2	49.73	24.2	24.44
3	22	50.0	49.84	24.4	24.29
2	12	55.6	50.01	27.0	24.16
	Total	908.2	943.99	454.4	472.23

Allowing RAM to calculate the different load cases given in Chapter 6 of ASCE7-05, the controlling load case was determined to be LC 1. Hand calculated wind forces, shears, and overturning moment is all shown below in Table 3.

Table 3: Wind Calculations						
Level	Force (k)		Shear (k)		Overturning Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	39.9	20.3	39.9	20.3	6261	3179
16	75.7	38.4	115.5	58.6	16753	8499
15	68.4	34.6	183.9	93.2	24646	12494
14	61.3	31.0	245.2	124.2	30403	15402
13	57.5	29.0	302.7	153.2	34807	17622
12	60.1	30.3	362.7	183.5	38450	19453
11	59.3	29.9	422.1	213.4	40517	20483
10	55.5	27.9	477.6	241.3	41549	20989
9	57.8	29.0	535.4	270.2	41761	21079
8	56.9	28.4	592.3	298.7	40274	20309
7	53.0	26.4	645.2	325.1	38069	19179
6	54.8	27.2	700.1	352.3	35003	17616
5	53.4	26.4	753.5	378.8	30140	15150
4	49.2	24.2	802.7	403.0	24882	12492
3	50.0	24.4	852.6	427.4	18758	9403
2	55.6	27.0	908.2	454.4	10898	5453
Total					473173	238804

Seismic Design

Seismic loads applied to the building were computed in accordance with chapters 11, 12 and 19 of ASCE7-05. Roosevelt Island Southtown Building No. 5 has a site class of C and a seismic design category of B, thus allowing, by code, the use of the Equivalent Lateral Force Method. The Seismic Design Criteria can be seen below in Table 4.

Table 4: Seismic Design Criteria	
Ss	0.36
S1	0.07
Site Class	C
Fa	1.52
Fv	2.4
SMS	0.544
SM1	0.168
SDS	0.363
SD1	0.112
Ct	0.02
hn(ft)	187.25
x	0.75
Ta	1.02
TL	6
k	1.255
Occ. Category	II
Importance factor (I)	1
Seismic Design Cat.	B

As for wind, the seismic parameters were inputted into RAM and the Equivalent Lateral Forces were then calculated for the building stories. A comparison of these forces can be seen in Table 5.

Table 5: Equivalent Lateral Forces				
Level	Height (ft.)	Weight (k)	Manual Force (k)	RAM Output Force (k)
Main Roof	157	2006	81.8	84.2
16	145	1221	45.0	47.6
15	134	1221	40.8	43.5
14	124	1221	37.2	40.0
13	115	1221	33.8	36.6
12	106	1221	30.4	33.0
11	96	1221	27.0	28.9
10	87	1221	23.8	26.0
9	78	1221	20.6	22.4
8	68	1221	17.6	18.9
7	59	1221	14.6	15.3
6	50	1221	11.8	11.5
5	40	1221	9.0	9.0
4	31	1221	6.5	5.9
3	22	1221	4.1	2.8
2	12	1221	2.0	2.3
		Total	406.0	427.9

It can be seen that the Total base shear for Seismic Design is 470 kips. Although the overall weight of the building is reduced with the use of a structural steel and hollow core plank system, the design parameters change. Instead of using an R value of 4 for concrete shear walls, 3.25 is used for concentric braced frames. This lower R value, in addition to a lower period increases the seismic response coefficient, Cs. When the base shear is determined, the higher Cs value is multiplied by the overall weight of the building and, thus creating a higher base shear. These formulas can be seen below.

Equation 6: Minimum of $C_s = \dfrac{S_{DS}}{\left(R/I\right)}$, $C_s = \dfrac{S_{D1}}{T\left(R/I\right)}$ for T ≤ T_L

Where: S_{DS} = the seismic design spectral response acceleration parameter in the short period range as determined from Section 11.4.4 of ASCE7-05

R = response modification factor in Table 12.2-1 of ASCE7-05

I = the occupancy importance factor determined in accordance with Section 11.5.1 of ASCE7-05

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 of ASCE7-05

T = the fundamental period of the structure determined in Section 12.8.2 of ASCE7-05

Equation 7: $T_a = C_t h_n^{\,x}$

Where: h_n is the height in ft. above the base to the heighest level of the structure

C_t and $_x$ are determined from Table 12.8-2 of ASCE7-05

As shown in the above tables, the wind forces and base shear are much more dominant in the design of the building's lateral system. The base shear generated by the wind loads in the East-West direction amount to 454 kips and 908 kips in the North-South direction. Therefore, the building's braced frame lateral system was designed based on wind criteria and calculations.

Braced Frame Design

In order to keep consistent with the change of the gravity system from reinforced concrete to a Girder-Slab and composite steel system, the existing shear wall lateral system was replaced with a braced frame steel system. In the existing system, the shear walls are typically surrounding the elevator core in the building as seen in Figure 19.

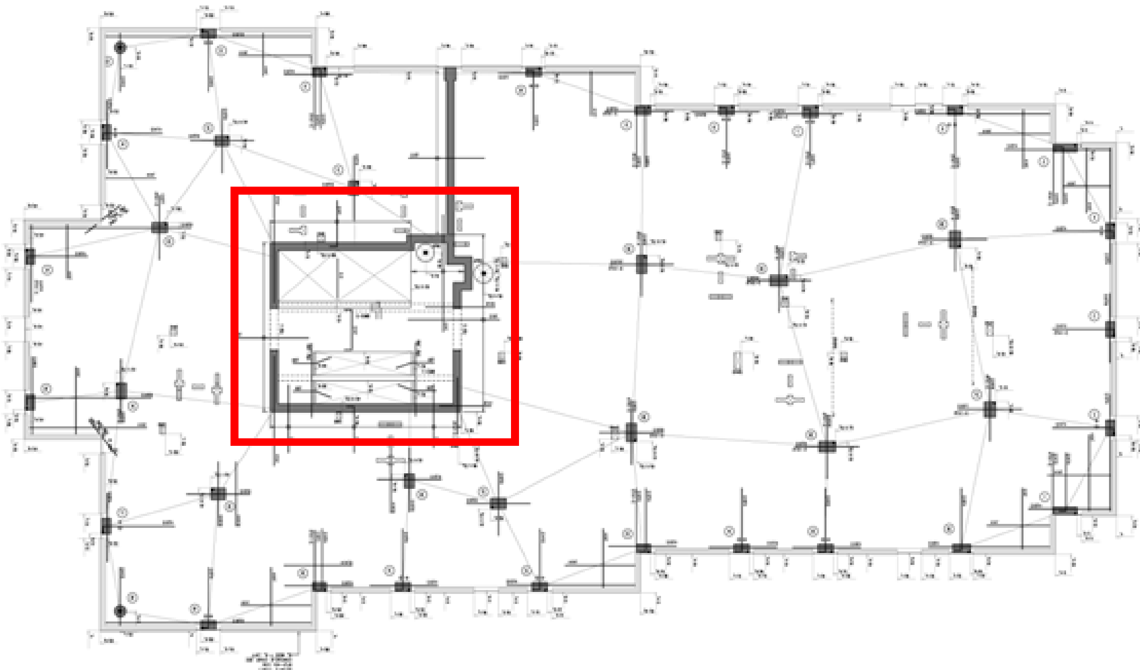


Figure 19: Typical existing structural floor plan with the elevator and stairwell core boxed in red

By replacing these walls with braced frames, the system needed to have more frames throughout the building in order to account for the long length of the North-South facing wall. This posed to be a problem with the existing architecture of the building. As shown in Figure 20, a layout of the braces can be seen.

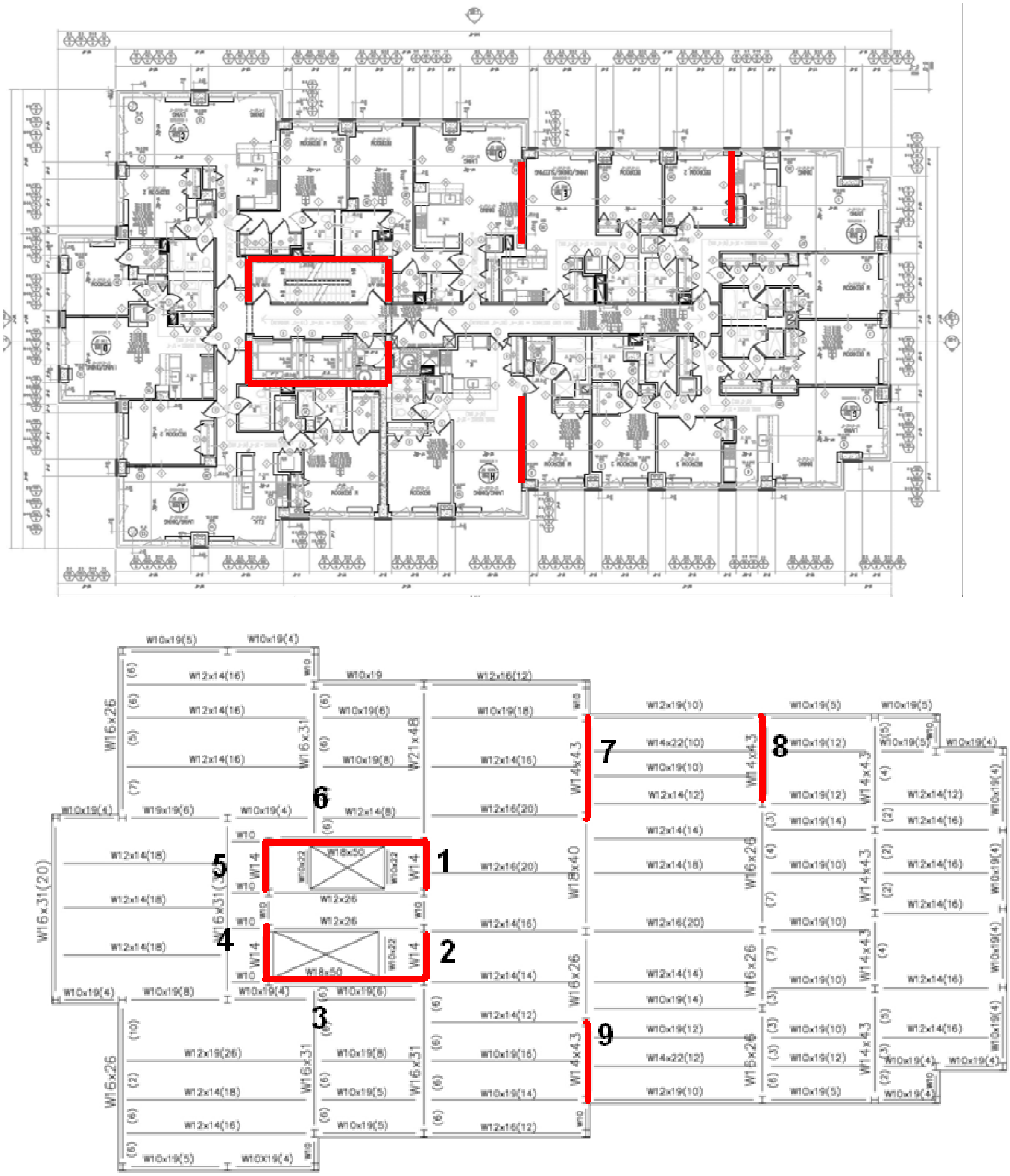


Figure 20: Above, the architectural floor plan can be seen with existing room layouts and partition walls. Below, a typical structural floor plan utilizing Girder-Slab floor system. Braced frame lateral systems are highlighted in red. Numbers correspond to frame number.

These braces utilize two different types of braced frames: the chevron brace and the cross brace. Frames 3 and 6 utilize the chevron brace while all of the other are fully crossed. Elevations for these braces can be seen in Figure 21.

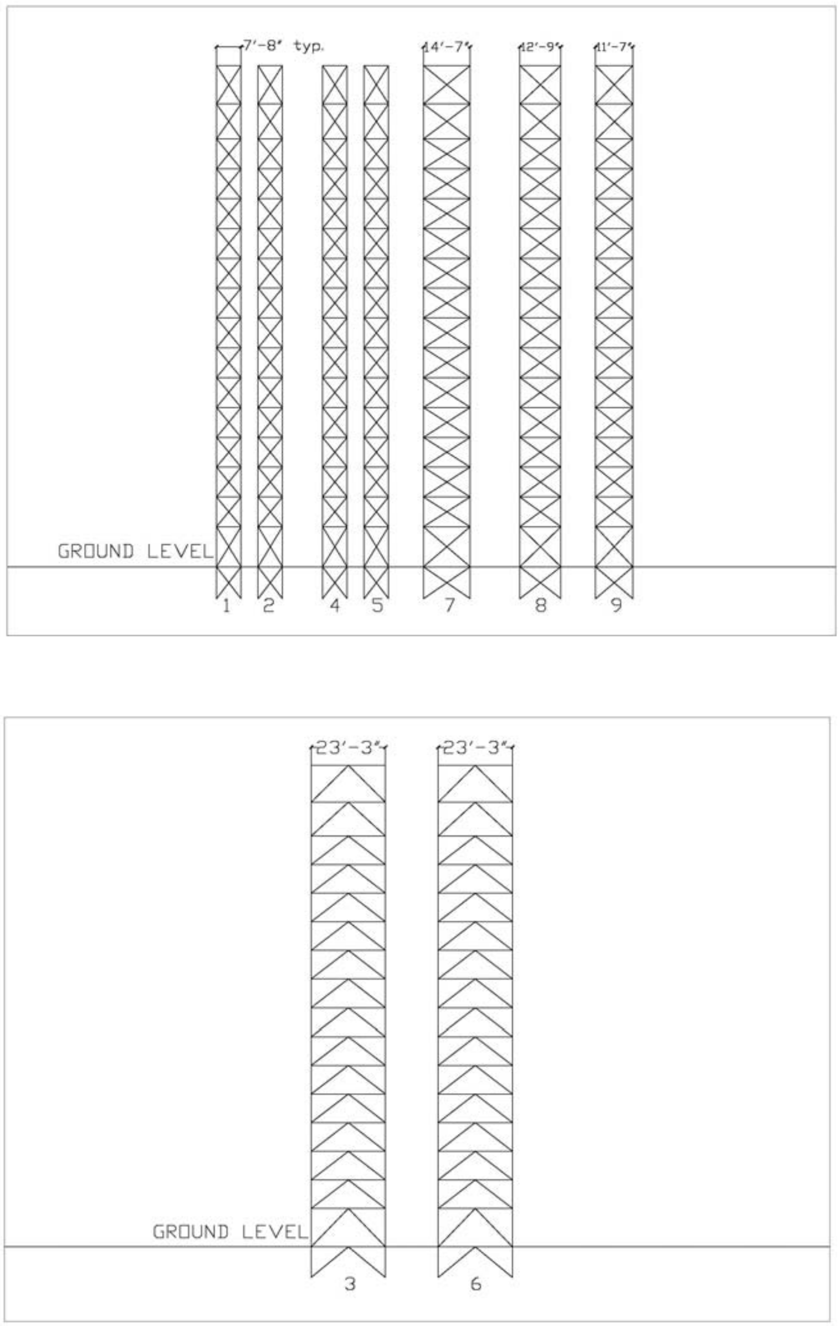


Figure 21: Braced frame lateral system. Frames 1, 2, 4, 5, 7, 8 and 9 utilize diagonal cross bracing while Frames 3 and 6 utilize Chevron braces.

The brace frames were designed using ASD load combination taken from ASCE7-05. The frames were assigned as gravity columns first and then assigned to lateral columns in RAM frame to determine initial member sizes. Once the story forces were applied to the building, the design of the frames became an iterative process. The overall displacement and torsion of the building was determined using RAM frame and it became apparent that the torsion was the controlling design factor. Using the industry standard of L/400 for the overall building displacement, 4.71 inches was the maximum displacement at the main roof. For Southtown Building No. 5, the maximum displacement was 3.53 inches due to wind in the N-S direction which occurred at the Main Roof level. Column sizes were increased to make sure that the displacement was within limits. A comparative analysis of story displacements and allowable displacements can be seen below in Table 6.

Table 6: Story Displacements

Level	Height (ft)	L/400 (in)	Wind Max Displ. (in)
Roof	157	4.71	3.53
16	145	4.35	3.27
15	134	4.02	3.03
14	124	3.72	2.8
13	115	3.45	2.57
12	106	3.18	2.34
11	96	2.88	2.1
10	87	2.61	1.86
9	78	2.34	1.63
8	68	2.04	1.39
7	59	1.77	1.16
6	50	1.5	0.94
5	40	1.2	0.74
4	31	0.93	0.54
3	22	0.66	0.35
2	12	0.36	0.18
1	0	0	0

When the total building displacement was within limits, members were checked using RAM steel check and ASD load combinations from ASCE7-05. The controlling load combination for members varied throughout the frames. Members were sized accordingly in order to meet all necessary code requirements. Story displacement by seismic loading was also within acceptable code limitations. The maximum story displacement was found to be 3.1” at the main roof.

Maximum story drift for seismic loading was found to be 0.257”. By multiplying this value by a Cd factor of 3.25, the code drift value was found to be 0.835”. From Chapter 12 of ASCE7-05, the allowable story drift $\Delta = 0.020h_{sx}$ for occupancy category II and braced frame lateral system. Given that the building height is 157 feet, the maximum allowable story drift by code is 3.14” which is much larger than 0.835”. Additionally, no torsional irregularity is considered for Southtown Building No. 5 since it is in the seismic design category B.

Overturning moment of the lateral system was checked for punching shear for the columns through the mat slab foundation. These calculations require the mat to be 36” thick. Since the existing mat foundation is 42” thick it will be able to resist the punching shear forces from the steel frame.

All member sizes and calculations can be found in the Appendix. Member sizes for braced frame 6 and 7 and 9 can be seen below in Figures 22 and 23, respectively.

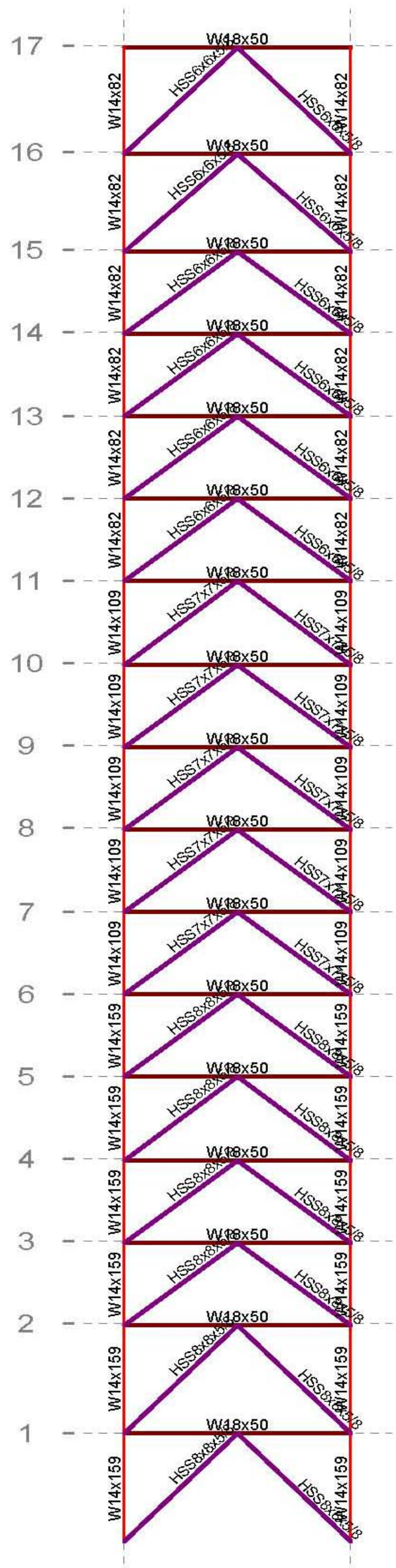


Figure 22: Frame 6 member sizes

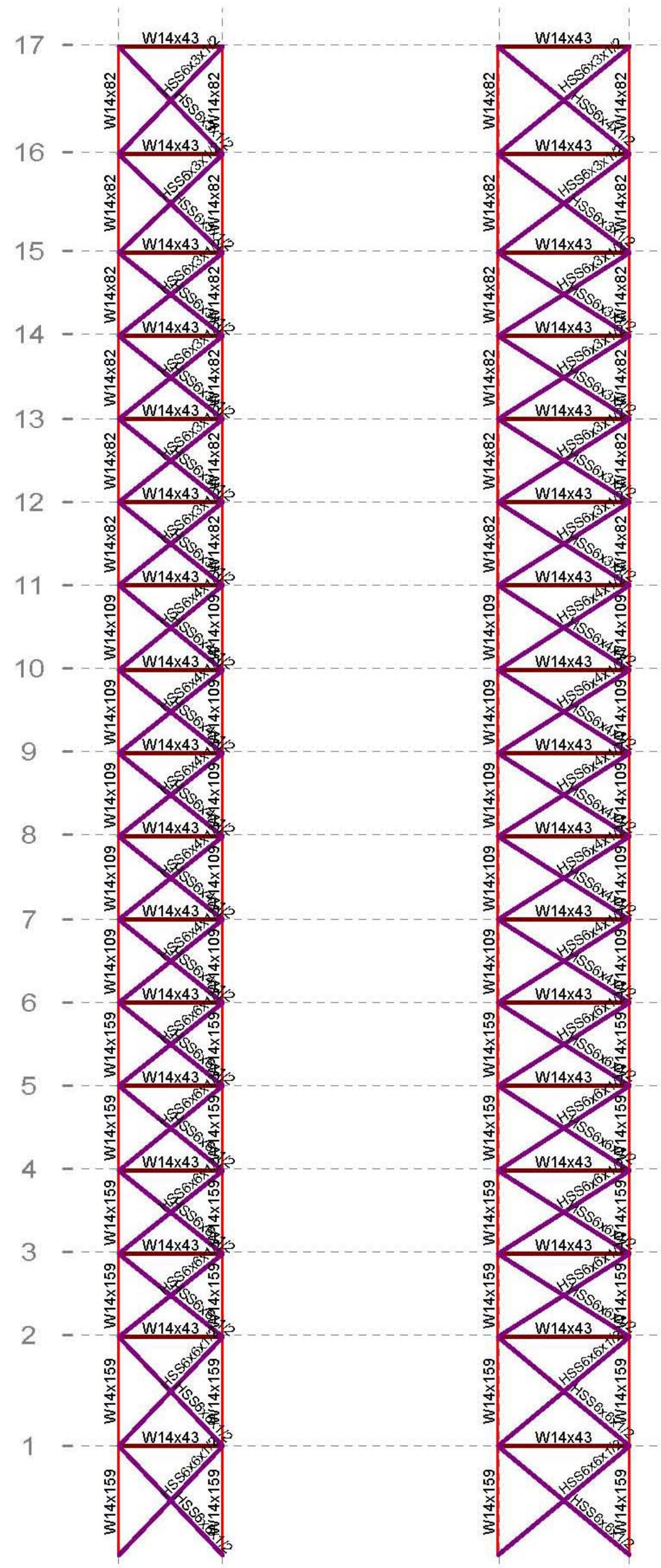


Figure 23: Frame 7 (left) and Frame 9 (right) member sizes

RAM Structural System Building Model

Southtown Building No. 5 was modeled using Bentley System RAM Structural System. To model the Girder-Slab system as accurately as possible, floors 2-Roof were modeled using a one-way deck. The deck was assigned the same weight as the 8" hollow-core floor planks and the ¾" topping material as specified before. Girder-Slab members were not designed using RAM but the model had to represent the typical floors as closely as possible to determine forces acting on adjacent columns and frames. The first floor was modeled using a composite steel deck and concrete system. Surface loads were applied to the floor diaphragms to accurately simulate applied forces on the floor slab. A 3D image of the RAM building model can be seen below in Figure 24.

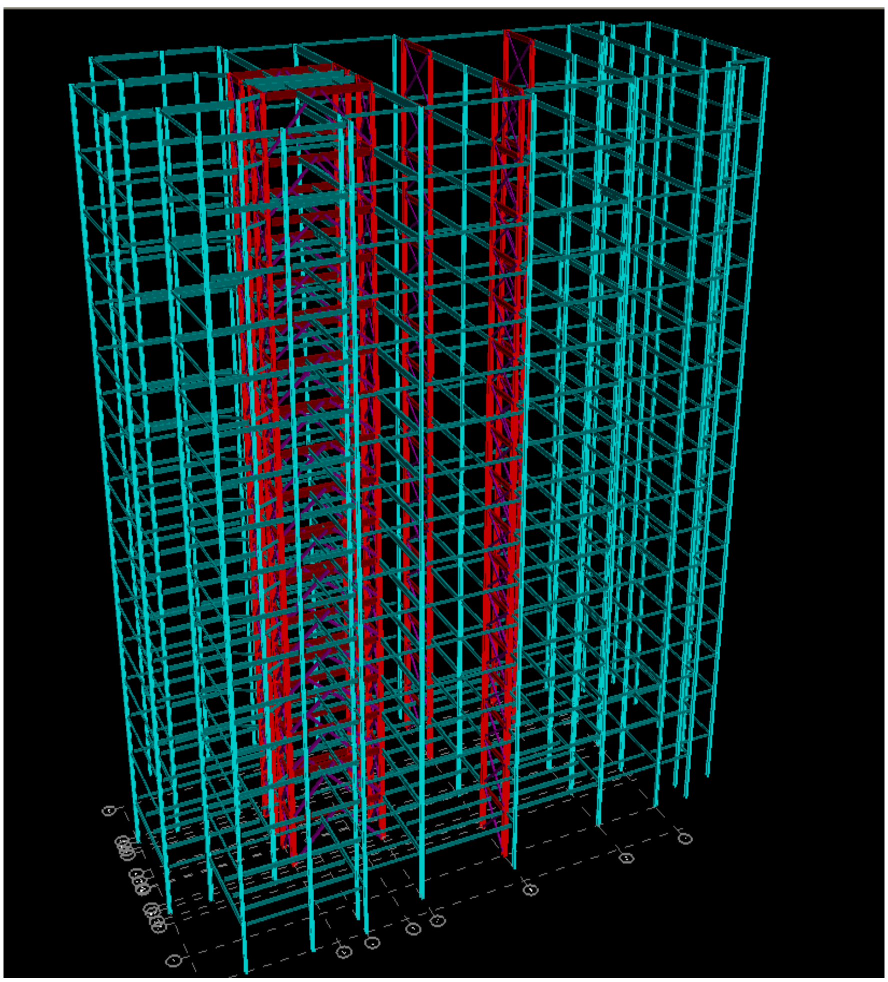


Figure 24: 3D RAM Building Model

Breadth Topics

Cost and Schedule Breadth

When altering the existing structure of Southtown Building No. 5 from a cast-in-place concrete system to a Girder-Slab system, there are many cost and schedule changes that are associated with the change. One main goal of this project was to compare the material, labor, and erection costs of the existing structure to those of the redesigned structure.

In order to achieve an accurate price of the existing superstructure, many factors were taken into account. The cost of the original structure was calculated using R.S. Means 2007 Construction Cost Data. The 2007 Cost Data was chosen to account for the June, 2007 start date. A factor of 1.31 was multiplied to the estimate for a location of New York City, NY.

The cost estimate of the existing system came to $3.35 million. This total was derived by contacting distributors and contractors in the New York area. Those numbers were cross referenced with R.S. Means to get the final total. The takeoff consisted of the existing floor slabs, Cast-in-place columns, shear walls, foundation walls, spread footings and the mat foundation under the core of shear walls. A summary of the costs per item can be found below in Table 7.

Table 7: Existing Concrete System	
Floor Slabs	$2,036,000
Columns	$929,000
Shear Walls	$240,000
Foundation Walls	$76,000
Spread Footings	$10,900
Mat Foundation	$56,000
Total:	$3,347,900

The cost estimate for the proposed Girder-Slab system was also derived by contacting local contractors and distributors as well as using R.S. Means. The Girder-Slab system was more expensive than the existing CIP structure and totaled $4.3 million. In this takeoff, the elements that were considered were the braced frame lateral system (consisting of columns, beams, and diagonal members), the composite floor system on the first floor (consisting of beams, metal deck, shear studs, and CIP concrete), the Girder-Slab system on floors 2-Roof (consisting of D-Beams, hollow core planks, and specialized planks for the roof), steel columns, spandrel beams, spread footings, fireproofing, and erection costs. A breakdown of the costs associated with each item can be seen in Table 8.

Table 8: Proposed Girder Slab System	
Braced Frame Lateral System	893,300
Composite Floor 1	140,000
Girder-Slab Floors 2-16	1,928,000
Columns	500,000
Erection Costs	354,600
Spandrel Beams	108,000
Fireproofing	259,000
Foundation Walls	75,500
Spread Footings	10,900
Mat Foundation	56,100
Total:	$4,325,400

Labor, equipment, erection times, profit and overhead were taken into account for both structural systems. A full graphic comparison can be seen in the Appendix.

Although the Girder-Slab system costs roughly $1 million more than the CIP system, the Girder-Slab system will be able to be erected approximately 2 months quicker than the existing system. Using Microsoft Project, a Gantt bar schedule was created which can be seen in the Appendix.

Using a start date of June 25, 2007 provided by the contractor, the Cast-in-place system will top out on December 11, 2007. This is approximately five months of construction for the superstructure. A majority of the time spent on the CIP system revolves around pouring the foundation. The footings, mat foundations, foundation walls take about two and a half months to complete. The cellar and floors 1-3 take about a week each to form and pour (including columns, shear walls, and slab) while floor 4-Roof take only 3 days each. This is unusually fast for cast-in-place concrete but in NYC a 3 day form, pour, strip period is very achievable.

With the same start date of June 25, 2007, the Girder-Slab system is able to top out on October 12, 2007. By eliminating a mat foundation and foundation walls, the erection time is significantly reduced. Also, according to a Girder-Slab contractor, up to 8000 square feet can be erected per day. Since the typical floor is just over 8000 square feet, one floor can be erected every 2 days. This number may be reduced depending on the speed and expertise of the erection contractor.

The steel system allowed for a decrease in erection time by 42 working days and over two months total. This can give way to opening the building sooner and enable the owner of the building to generate revenue earlier. Since each unit in the building is separately owned by each tenant, it is hard to estimate the amount of revenue the owner would generate with the time savings. With condominiums ranging in price from $575,000 for a one bedroom/one bathroom to $1,295,000 for a three bedroom /three bathroom, the amount of interest that the owner would generate from an earlier opening would contribute to the added costs of a steel system. Additionally, less time spent in construction means less money to pay back in construction loans. These loans are usually interest-only payments during construction and become due upon completion so the quicker the construction, the less money the owner has to dish out in interest rates. This faster erection process can not only save time during the construction but money for the owner, which is always a bonus.

LEED Design and Sustainability Breadth

The environmental impact of building construction and operation is very significant in today's world. According to the U.S. Green Building Council (USGBC), buildings annually consume more than 30% of the total energy and more than 60% of the electricity in the United States. To reduce such a large carbon footprint, the world is shifting to Green and sustainable building design.

Green building practices can substantially reduce or eliminate negative environmental impacts and improve existing unsustainable design, construction, and operational practices. Additionally, green design measures also reduce operating costs, enhance building marketability, increase worker productivity, and reduce potential liability resulting from indoor air quality problems.

Since Southtown Building No. 5 is one of nine buildings going up on Roosevelt Island, the environmental impact of these building is very significant. By designing the buildings with sustainability in mind, the island would be more environmentally friendly and reduce the amount of pollution that the island emits, especially into the already polluted East River. In order to achieve a more eco-friendly building, the USGBC's green-building rating system, Leadership in Energy and Environmental Design (LEED) will be applied to the Building No. 5. Twenty-six points will be evaluated in order to achieve a certified LEED rating.

Sustainable Sites

Southtown Building No. 5 is located in a prime area to achieve many sustainable site credits. It is located only 30 feet from the East River shoreline and utilizes many key sustainable features. The building has a rather small footprint compared to the amount of vegetative land surrounding the building. Beautiful landscaping compliments the building and reduces the surrounding hardscape.

As originally intended, Southtown Building No. 5 is to be designed with a vegetative roof. Of approximately 6000 square feet of exposed roof space, 1000 square feet of green roof was already in the existing plan. In order to achieve a LEED point for a vegetative roof, an additional 2000 square feet was added to the design as seen in Figure 24.

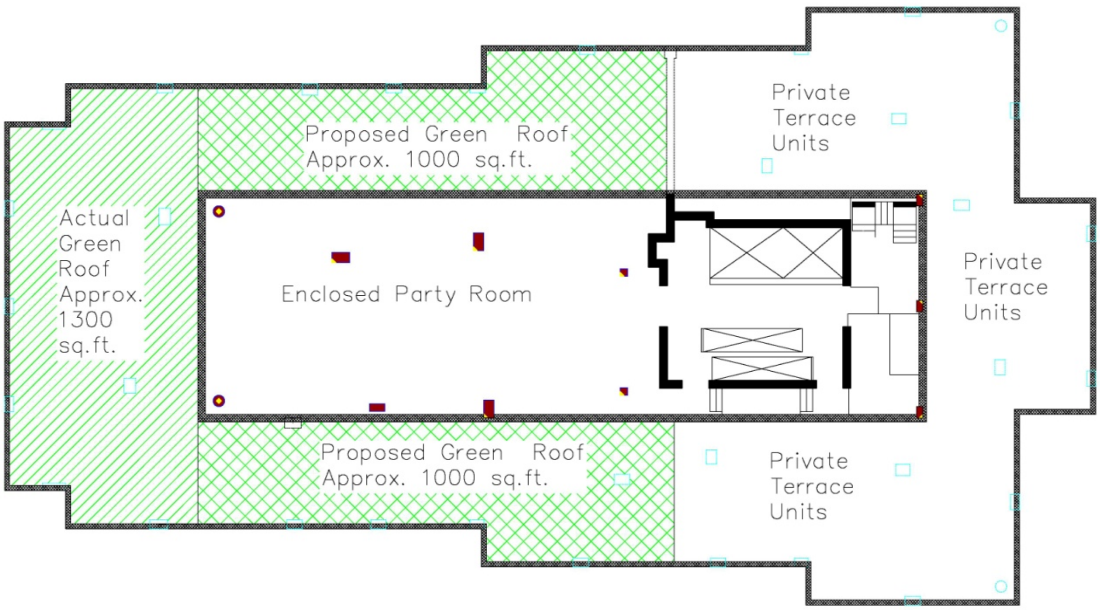


Figure 24: Roof plan for Southtown Building No. 5. Actual green roof and proposed green roof areas can be seen in the different green hatching.

Finally, the intended hardscape of Roosevelt Island is fairly minimal. With the addition of these nine new buildings, many walkways and streets must be added. To fully reduce the amount of reflective material, pavers and other permeable walkways were chosen instead of concrete sidewalks.

SS-1: “Previously undeveloped land that is within 50 feet of a body of water, defined as seas, lakes, rivers, streams and tributaries which support or could support fish, recreation or industrial use.”

SS-4.1: “Locate project within ½ mile of an existing commuter rail, light rail or subway station.”

*Located just a block away from Manhattan's F, and NRW lines.

SS-4.2: "For residential buildings, provide covered storage for securing bicycles for 15% or more of the building occupants in lieu of changing/shower facilities."

SS-4.4: "Provide no new parking."

SS-5.1: "On greenfield sites, limit all site disturbance to 40 ft. beyond the building perimeter; 10 ft. beyond surface walkways, patios, surface parking and utilities less than 12 inches in diameter; 15 ft. beyond primary roadway curbs and main utility branch trenches; and 25 ft. beyond constructed areas with permeable surfaces that require additional staging areas in order to limit compaction in the constructed area.

SS-5.2: "Where a zoning ordinance exists, but there is no requirement for open space, provide vegetated open space equal to 20% of the project's site area."

SS-6.1: "Implement a stormwater management plan that prevents the post-development peak discharge rate and quantity from exceeding the pre-development peak discharge rate and quantity for the one- and two-year, 24-hour design storms."

SS-7.1: "50% of site hardscape to be open grid pavement system."

SS-7.2: "Install a vegetated roof at least 50% of the roof area."

Water Efficiency

A case study of a 27-story green residential high rise in New York City was studied to compare similarities for the Water Efficiency credits. The Solaire building was able to earn all five WE credits. A wastewater treatment system treats 100% of the wastewater from the building. Water recaptured by the system is used to supply the building's toilets, and 5000 gallons per day are provided to the adjacent public park. A stormwater storage tank which harvests rainwater is used for all irrigation needs. 50% less potable water is needed from the municipal water supply than would be used in a conventional apartment building and no potable water is used outdoors. Additionally, low-flow appliances and fixtures were used, and the public restroom facilities use waterless urinals, contributing to a water use reduction of 88% within the building.

By implementing the same type of criteria for this residential high-rise and its surrounding buildings, I believe that the same credits could be obtained.

WE-1.1: "Reduce potable water consumption for irrigation by 50% from a calculated mid-summer baseline case."

WE-1.2: "Use only captured rainwater, recycled wastewater, recycled graywater, or water treated and conveyed by a public agency for non-potable uses for irrigation."

WE-2: "Reduce potable water use for the building sewage conveyance by 50% through the use of water-conserving fixtures or non-potable water."

WE-3.1: "Employ strategies that in aggregate use 20% less water than the water use baseline calculated for the building after meeting the Energy Policy Act of 1992 fixture performance requirements."

WE-3.2: Same as above with 30" less water than the water use…

Energy and Atmosphere

Using Energy-10, an energy building model was created to compare the existing HVAC system to a proposed system. Currently, a Packaged Terminal Air Conditioner (PTAC) & Electric Heat system is used in every room in each apartment. These systems are mounted beneath windows and combine heating and cooling. They use refrigeration components and forced ventilation to cool and reverse cycle refrigeration as the prime heating source. As these units are generally more expensive than window air conditioners and base board electric heat, they are often less environmentally friendly with the large amount of refrigerant used.

As a proposed method of heating and cooling, an air source heat pump was compared to the existing PTAC units. Additionally, photo-voltaic cells were added to the roof to help conserve the overall energy consumption of the building. A single floor was modeled in Energy-

10 and the two systems were compared. The total energy used in the existing system was 960000 kBtu versus 810000 kBtu of the proposed system. This equates to an 18% reduction per floor. Additionally, the emissions were reviewed for the two systems. Carbon dioxide emissions were reduced from 345000 lbs to 300000 lbs per floor and Nitrous Oxide was reduced from 1100 lbs to under 900 lbs.

As you can see, the proposed system would be environmentally beneficial to use. It would require a heat pump to be placed in every apartment in order for the building to measure individual energy costs per unit. However, with the amount of benefits gained by this new system, the installation costs would be offset by the energy savings. The following credits would be obtained from the new proposed system.

EA-1: "Demonstrate a percentage improvement in the proposed performance rating compared to the baseline building performance rating."

*Since the renovations are all proposed to a new building design, 18% improvement would allocate 3 points to the LEED rating.

EA-2: "Use pm-site renewable energy systems to offset building energy cost."

*Photovoltaic cells implemented on the roof would allow for a 3% reduction in heating costs. This equates to 1 point.

EA-6: " Provide at least 35% of the building's electricity from renewable sources by engaging in at least a two-year energy contract."

Materials and Resources

Building material choices are important in sustainable design due to the large amount of time it takes to extract, process, and transport them. The steps required to create the building materials may pollute air and water as well as deplete the natural resources used to make them. When selecting materials for a project, it is important to evaluate new and different sources.

Southtown Building No. 5 is one of nine buildings going up in its development. By using material left over from previous building construction, total material costs will be lowered and less new material will need to be processed. Also, with the proposal of developing the building in steel, recycled-content materials, such as steel, reuse waste products that would otherwise be deposited in landfills. Finally, the use of local materials, such as steel manufactured in Pennsylvania, support local economy and reduce transportation. These options would allow the building to earn the Materials and Resources credits listed below.

MR-2.1: "Recycle and/or salvage at least 50% of non-hazardous construction and demolition."

MR-3.1,2:" Reuse building material and products in order to reduce demand for vigin materials and to reduce waste."

*With the consistency between the nine proposed buildings, the construction material should be recycled and used for the next constructed building.

MR-4.1: "Use materials with recycled content such that the sum of post-consumer recycled content plus one-half of the pre-consumer content constitutes at least 10% of the total value of the materials in the project."

MR-5.1: "Use building materials or products that have been extracted, harvested or recovered, as well as manufactured, within 500 miles of the project site for a minimum of 10% of total materials value."

Indoor Environmental Quality

Using higher ratios of filtered outside air, increasing ventilation rates, managing moisture, and controlling the level of contaminants in the cleaning substances used can provide optimal air quality for building inhabitants. Additionally, occupant well-being can be improved by providing occupants with the ability to control their personal thermal environment.

The owner, building design team, contractors, subcontractors and suppliers must all play an integral role in order to achieve such improved indoor air quality and occupant satisfaction. The following Indoor Environmental Quality credits could be attained.

EQ-3.2: "Conduct baseline IAQ testing, after construction ends and prior to occupancy."

EQ-4.3: "All carpet installed in the building interior shall meet the testing and product requirements of the Carpet and Rug Institute's Green Label Plus program.

EQ-6.1: "Provide individual lighting controls for 90% of the building occupants enable adjustments to suit individual task needs and preferences AND provide lighting system controllability for all shared multi-occupant spaces to enable lighting adjustments that meets group needs and preferences.

Conclusions

In today's world, it is very important to study the impact of a building, from construction to maintenance to building management. As shown, twenty-six LEED points have been outlined and credited to achieve a LEED certified building. It would take a lot of effort from all parties involved in the development, design, and construction of the building but the environmental impact would greatly outweigh any additional costs.

If Southtown Building No. 5 was to achieve a LEED rating, it could possibly change the way owners and developers look to construct later buildings on Roosevelt Island; a once natural habitat to many plants and animals. As a result of this breadth topic, I believe that LEED building is a step in the right direction in order to preserve our Earth and its species.

Conclusion and Recommendation

Since New York City has very strict building height restrictions, the aim of construction is to maximize floor area without exceeding height limitations. In particular, condominium construction requires maximizing floor space in order for building owners to achieve the most money per square foot. In order to achieve these limiting factors, the floor thickness must be kept as thin as possible. As constructed, the cast-in-place concrete floors are 8” thick compared to the 8-3/4” thick floors of the proposed Girder-Slab system. This ¾” reduction in floor-to-floor height will not be a noticeable difference in appearance and the building will still be able to house the same number of stories in the governing height restriction.

There are many advantages associated with both the cast-in-place concrete and the Girder-Slab structural systems. But, in the end, I believe that the existing concrete system will prove to be the best choice for this project. Even though the steel system would be able to be erected faster, New York City’s concrete workers are very efficient and quick to erect concrete slabs, columns, and shear walls. The use of multiple unions for the construction of the Girder-Slab system could prove to be more strenuous than efficient. The job site could become cluttered and the erection time may be slowed with the use of multiple crews (i.e. concrete crew, erection crew, steel bolting crew, etc.) working at once.

The Girder-Slab system has proved that it would be structurally sufficient to withstand all applied loads and forces seen in a typical residential high-rise building. It has been shown that a braced frame lateral system would work well in such a building and resist all lateral forces. The Girder-Slab system has proven to be a sufficient method of construction, however, the conventionality of a cast-in-place concrete system seems to be a better fit in New York City with its employment of labor unions.

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References

Reference Standards

AISC Steel Construction Manual, 13th Edition

ASCE 7-05

ACI 318-05

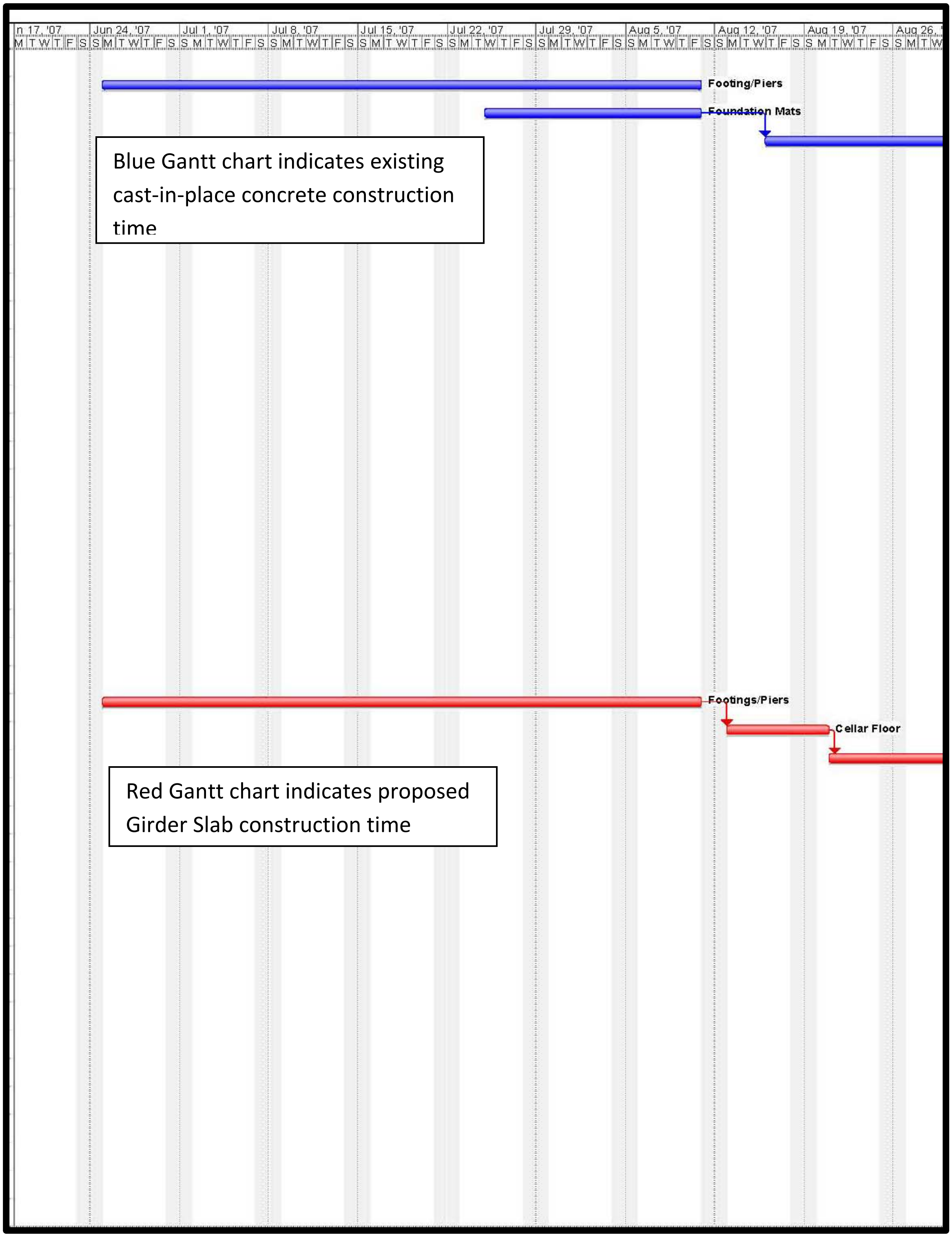
RS Means 2007 Construction Cost Data

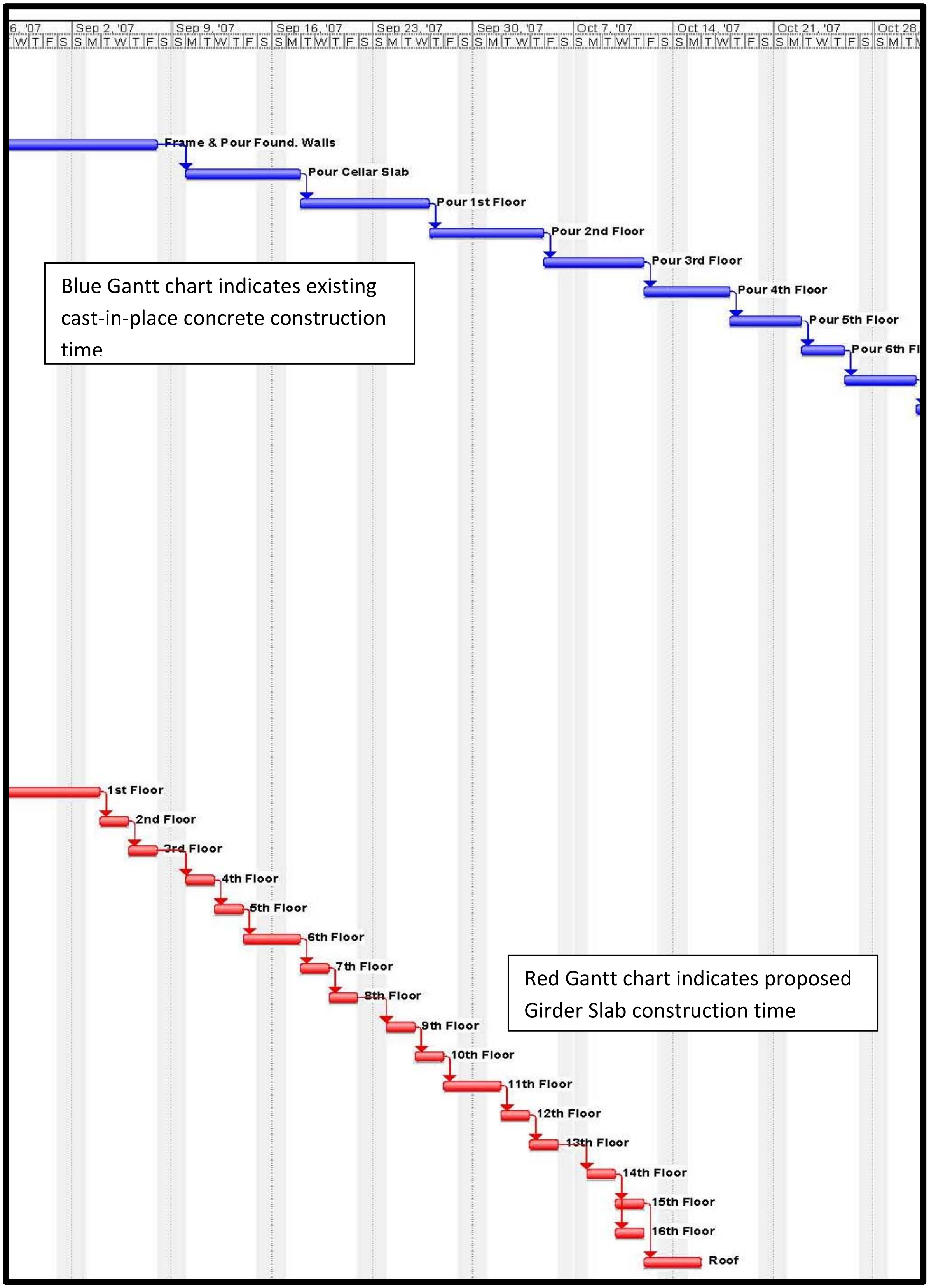
LEED-NC Version 2.2 Reference Guide

Girder-Slab Design Guide Version 1.4

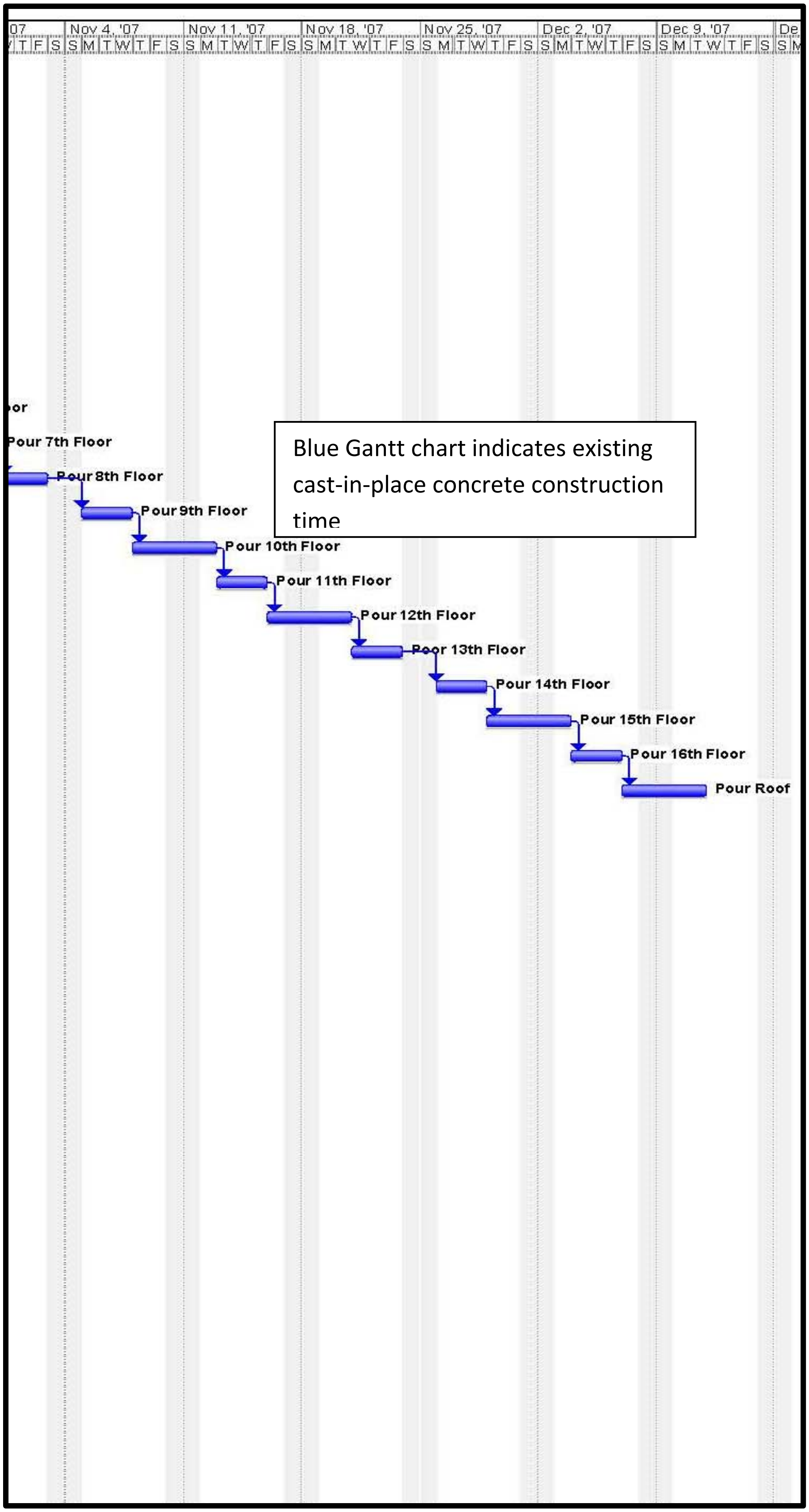
APPENDIX

Construction Schedule





Sep 2, '07
Sep 9, '07
Sep 16, '07
Sep 23, '07
Sep 30, '07
Oct 7, '07
Oct 14, '07
Oct 21, '07
Frame & Pour Found. Walls
Pour Cellar Slab
Pour 1st Floor
Pour 2nd Floor
Pour 3rd Floor
Pour 4th Floor
Pour 5th Floor
Blue Gantt chart indicates existing cast-in-place concrete construction time
1st Floor
2nd Floor
3rd Floor
4th Floor
5th Floor
6th Floor
7th Floor
8th Floor
9th Floor
10th Floor
11th Floor
12th Floor
13th Floor
14th Floor
15th Floor
16th Floor
Roof
Red Gantt chart indicates proposed Girder Slab construction time


Nov 4, '07
Nov 11, '07
Nov 18, '07
Nov 25, '07
Dec 2, '07
Dec 9, '07
Pour 7th Floor
Pour 8th Floor
Pour 9th Floor
Pour 10th Floor
Pour 11th Floor
Pour 12th Floor
Poor 13th Floor
Pour 14th Floor
Pour 15th Floor
Pour 16th Floor
Pour Roof
Blue Gantt chart indicates existing cast-in-place concrete construction time

Cost Calculations

Structural Steel System

Structural Steel Construction Cost Calculations				
Lateral Frame Takeoff	Weight (lbs)	Weight (tons)		
Columns				
W14x82	68965	34.5		
W14x109	71114	35.6		
W14x159	137531	68.8		
Beams				
W14x43	49453	24.7		
W18x50	39541	19.8		
Braces				
HSS6x6x1/2	37932	19.0		
HSS6x6x5/8	15270	7.6		
HSS6x4x1/2	25723	12.9		
HSS6x3x1/2	27637	13.8		
HSS7x7x1/2	2245	1.1		
HSS7x7x5/8	14202	7.1		
HSS8x8x5/8	20849	10.4	Price per Ton:	$3,500
Total	441497	255.2	Cost:	$893,308.50

Beam Takeoff				
1st Floor	28399	14.2		
Gravity Column Takeoff				
All Floors	285572	142.8		
			Price per Ton:	$3,500
Total	313971	157.0	Cost:	$549,498.25

Girder Slab Takeoff				
Floors 2-Roof				
DB8x35	54587.2	27.3		
DB8x42	206144.2	103.1	Price per Ton:	$2,600
Total	260731.4	130.4	Cost:	$338,951

Spandrel Beam Takeoff				
Floors 2-Roof				
W8x10	12280	6.1		
W8x15	54180	27.1		
W10x22	8089.6	4.0		
W12x26	19344	9.7	Price per Ton:	$2,300
Total	93893.6	46.9	Cost:	$107,978

Labor & Equipment per Ton:		Tons		
	$2,000	177.3	Cost:	$354,600

Girder-Slab/Steel Material Costs						
		Metal Deck	Concrete	Rubber Roofing Plank	Pre-Cast Plank	Fireproofing
Floor Area	Area(ft^2)	$3.30/s.f.	$6.80/s.f.	$12.5/s.f.	$11.00/s.f.	$1.70/s.f.
Roof	8952.2			$111,902		$15,219
Floors 2-16	134283.5				$1,477,119	$228,282
1st Floor	8952.2	$29,542	$60,875			$15,219
Total		$29,542	$60,875	$111,902	$1,477,119	$258,720

Proposed Girder Slab System Cost Totals	
Braced Frame Lateral System	893,300
Composite Floor 1	140,000
Girder-Slab Floors 2-16	1,928,000
Columns	500,000
Erection Costs	354,600
Spandrel Beams	108,000
Fireproofing	259,000
Foundation Walls	75,500
Spread Footings	10,900
Mat Foundation	56,100
Total:	$4,325,400

Cast-in-place concrete System

Cast-in-place Concrete System Construction Cost Calcs			
Floor Slabs			
Floor	Volume (ft^3)	C.Y.	Flat Plate $530 per C.Y.
Roof	7460	276.3	$146,439
Floors 2-16	89522	3315.6	$1,757,268
1st Floor	6714	249	$131,970
		Total	$2,035,677

Columns			
Floor	Volume (ft^3)	C.Y.	Columns $1475 per C.Y.
16	1142.5	42.3	$62,393
15	1068.5	39.6	$58,410
Floors 2-14	11752	435.3	$642,067
1st Floor	1221	45.2	$66,375
Cellar	1822.3	67.5	$99,562
		Total	$928,807

Shear Walls			
Floor	Volume (ft^3)	C.Y.	Walls $365 per C.Y.
16	1289	47.7	$17,425
15	1188	44	$16,060
Floors 2-14	13099.3	485.2	$177,083
1st Floor	1212.5	45	$16,391
Cellar	937.5	34.7	$12,674
		Total	$239,633

Foundation Wall			
	Perimeter (ft)	Volume (ft^3)	$365 per C.Y.
Cellar	447	5587.5	$75,534
		Total	$75,534

				Volume (ft^3)	C.Y.	Quantity	Total C.Y.	Footings
Footings								$300per C.Y.
Type	Width	Length	Depth	18	0.67	26	17.42	$5,226
F3	3'-0"	3'-0"	24"	24.5	0.9	3	2.7	$810
F3.5	3'-6"	3'-6"	24"	40	1.5	4	6	$1,800
F4	4'-0"	4'-0"	30"	33.75	1.25	2	2.5	$750
F4.3	4'-6"	3'-0"	30"	50.63	1.9	4	7.6	$2,280
F4.5	4'-6"	4'-6"	30"			Total	36.22	$10,866

Mat Foundation			
Area (ft^2)	Volume (ft^3)	C.Y.	$370 per C.Y.
1170	4095	151.67	$56,118
		Total	$56,118

Existing Concrete System Total Costs	
Floor Slabs	$2,036,000
Columns	$929,000
Shear Walls	$240,000
Foundation Walls	$76,000
Spread Footings	$10,900
Mat Foundation	$56,000
Total:	$3,347,900

Seismic Calculations

Seismic Design Criteria	
Ss	0.36
S1	0.07
Site Class	C
Fa	1.52
Fv	2.4
SMS	0.544
SM1	0.168
SDS	0.363
SD1	0.112
Ct	0.02
hn(ft)	187.25
x	0.75
Ta	1.02
TL	6
k	1.255
Occ. Category	II
Importance factor (I)	1
Seismic Design Cat.	B

Hand Generated Seismic Calculations							
	w_x	h_x	$w_x(h_x)$^k	C_{vx}	F_x	V_x	M_x
Level	(kips)	(ft.)			(kips)	(kips)	(ft-kips)
Main Roof	2006	157	1139528	0.201	81.8	0.0	12805
16	1221	145	627555	0.111	45.0	81.8	6511
15	1221	134	568226	0.100	40.8	126.8	5447
14	1221	124	518869	0.092	37.2	167.6	4627
13	1221	115	470397	0.083	33.8	204.8	3879
12	1221	106	422971	0.075	30.4	238.6	3205
11	1221	96	376603	0.067	27.0	268.9	2601
10	1221	87	331368	0.059	23.8	296.0	2067
9	1221	78	287311	0.051	20.6	319.8	1600
8	1221	68	244633	0.043	17.6	340.4	1198
7	1221	59	203421	0.036	14.6	357.9	860
6	1221	50	163804	0.029	11.8	372.5	583
5	1221	40	126095	0.022	9.0	384.3	364
4	1221	31	90566	0.016	6.5	393.3	201
3	1221	22	57670	0.010	4.1	399.8	89
2	1221	12	28335	0.005	2.0	404.0	25
Totals	20321		5657351	1.000	406.0	406.0	46063

Equivalent Lateral Forces				
Level	Height	Weight	Manual	RAM Output
	(ft.)	(k)	Force (k)	Force (k)
Main Roof	157	2006	81.8	84.2
16	145	1221	45.0	47.6
15	134	1221	40.8	43.5
14	124	1221	37.2	40.0
13	115	1221	33.8	36.6
12	106	1221	30.4	33.0
11	96	1221	27.0	28.9
10	87	1221	23.8	26.0
9	78	1221	20.6	22.4
8	68	1221	17.6	18.9
7	59	1221	14.6	15.3
6	50	1221	11.8	11.5
5	40	1221	9.0	9.0
4	31	1221	6.5	5.9
3	22	1221	4.1	2.8
2	12	1221	2.0	2.3
		Total	406.0	427.9

Wind Calculations

General parameters	
Classification Category:	II
Basic Wind Speed, V:	110 mph
Importance factor, I:	1
Mean recurrence interval:	50 year
MRI factor:	1
Exposure Category:	C
a:	9.5
zg:	900
Topographic factor, Kzt:	1
Wind directionality factor, Kd:	0.85
Gust Factor, G (x-dir wind):	1.01
Gust Factor, G (y-dir wind):	0.969
Internal pressure coefficient, +GCpi:	0.18
Internal pressure coefficient, -GCpi:	-0.18
Windward pressure coefficient, Cp:	0.8
Side pressure coefficient, Cp:	-0.7

Building Wind Calculations							
Level	Story Height (ft)	hx (ft)	Length E-W (ft)	Length N-S (ft)	z (ft)	Kz	qz (psf)
Roof	12	157	144	80	157	1.4	31.9
16	11	145	144	80	145	1.4	31.4
15	9.33	134	144	80	134	1.3	30.8
14	9.33	124	144	80	124	1.3	30.3
13	9.33	115	144	80	115	1.3	29.9
12	9.33	106	144	80	106	1.3	29.3
11	9.33	96	144	80	96	1.3	28.7
10	9.33	87	144	80	87	1.2	28.2
9	9.33	78	144	80	78	1.2	27.5
8	9.33	68	144	80	68	1.2	26.7
7	9.33	59	144	80	59	1.1	25.9
6	9.33	50	144	80	50	1.1	25.1
5	9.33	40	144	80	40	1.0	23.9
4	9.33	31	144	80	31	1.0	22.7
3	9.33	22	144	80	22	0.9	21.1
2	12.5	12	144	80	12	0.8	19.4
1	0	0	144	80	0	0.8	19.4

E-W DIRECTION WIND														
z (ft)	L/B	Leeward C_p	External wall pressure			w/ pos. internal pressure			w/ neg. internal pressure			Total pressure (psf)	Story Force (k)	Moment contribution (k-ft)
			p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)			
157	1.80	-0.340	29.6	-12.6	-25.9	23.0	-19.2	-32.5	36.2	-6.0	-19.3	42.2	20	3,179
145	1.80	-0.340	29.1	-12.6	-25.9	22.5	-19.2	-32.5	35.7	-6.0	-19.3	41.7	38	5,563
134	1.80	-0.340	28.6	-12.6	-25.9	22.0	-19.2	-32.5	35.2	-6.0	-19.3	41.2	35	4,640
124	1.80	-0.340	28.2	-12.6	-25.9	21.6	-19.2	-32.5	34.8	-6.0	-19.3	40.8	31	3,841
115	1.80	-0.340	27.7	-12.6	-25.9	21.1	-19.2	-32.5	34.3	-6.0	-19.3	40.3	29	3,338
106	1.80	-0.340	27.3	-12.6	-25.9	20.7	-19.2	-32.5	33.9	-6.0	-19.3	39.8	30	3,210
96	1.80	-0.340	26.7	-12.6	-25.9	20.1	-19.2	-32.5	33.3	-6.0	-19.3	39.3	30	2,866
87	1.80	-0.340	26.1	-12.6	-25.9	19.6	-19.2	-32.5	32.7	-6.0	-19.3	38.7	28	2,426
78	1.80	-0.340	25.6	-12.6	-25.9	19.0	-19.2	-32.5	32.1	-6.0	-19.3	38.1	29	2,261
68	1.80	-0.340	24.8	-12.6	-25.9	18.2	-19.2	-32.5	31.4	-6.0	-19.3	37.4	28	1,933
59	1.80	-0.340	24.1	-12.6	-25.9	17.5	-19.2	-32.5	30.7	-6.0	-19.3	36.7	26	1,558
50	1.80	-0.340	23.3	-12.6	-25.9	16.7	-19.2	-32.5	29.9	-6.0	-19.3	35.9	27	1,362
40	1.80	-0.340	22.2	-12.6	-25.9	15.6	-19.2	-32.5	28.8	-6.0	-19.3	34.8	26	1,057
31	1.80	-0.340	21.0	-12.6	-25.9	14.4	-19.2	-32.5	27.6	-6.0	-19.3	33.6	24	751
22	1.80	-0.340	19.6	-12.6	-25.9	13.0	-19.2	-32.5	26.2	-6.0	-19.3	32.2	24	538
12	1.80	-0.340	18.1	-12.6	-25.9	11.5	-19.2	-32.5	24.7	-6.0	-19.3	30.6	27	324
0	1.80	-0.340	18.1	-12.6	-25.9	11.5	-19.2	-32.5	24.7	-6.0	-19.3	30.6	15	0

N-S DIRECTION WIND														
z (ft)	L/B	Leeward C_p	External wall pressure			w/ pos. internal pressure			w/ neg. internal pressure			Total pressure (psf)	Story Force (k)	Moment contribution (k-ft)
			p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)			
157	0.56	-0.500	28.4	-17.8	-24.9	21.8	-24.3	-31.5	35.0	-11.2	-18.3	46.2	40	6,261
145	0.56	-0.500	27.9	-17.8	-24.9	21.3	-24.3	-31.5	34.5	-11.2	-18.3	45.7	76	10,970
134	0.56	-0.500	27.5	-17.8	-24.9	20.9	-24.3	-31.5	34.1	-11.2	-18.3	45.2	68	9,163
124	0.56	-0.500	27.0	-17.8	-24.9	20.4	-24.3	-31.5	33.6	-11.2	-18.3	44.8	61	7,597
115	0.56	-0.500	26.6	-17.8	-24.9	20.0	-24.3	-31.5	33.2	-11.2	-18.3	44.4	57	6,611
106	0.56	-0.500	26.2	-17.8	-24.9	19.6	-24.3	-31.5	32.7	-11.2	-18.3	43.9	60	6,366
96	0.56	-0.500	25.6	-17.8	-24.9	19.0	-24.3	-31.5	32.2	-11.2	-18.3	43.4	59	5,695
87	0.56	-0.500	25.1	-17.8	-24.9	18.5	-24.3	-31.5	31.7	-11.2	-18.3	42.8	56	4,830
78	0.56	-0.500	24.5	-17.8	-24.9	17.9	-24.3	-31.5	31.1	-11.2	-18.3	42.3	58	4,510
68	0.56	-0.500	23.8	-17.8	-24.9	17.2	-24.3	-31.5	30.4	-11.2	-18.3	41.6	57	3,867
59	0.56	-0.500	23.1	-17.8	-24.9	16.5	-24.3	-31.5	29.7	-11.2	-18.3	40.9	53	3,125
50	0.56	-0.500	22.3	-17.8	-24.9	15.7	-24.3	-31.5	28.9	-11.2	-18.3	40.1	55	2,741
40	0.56	-0.500	21.3	-17.8	-24.9	14.7	-24.3	-31.5	27.9	-11.2	-18.3	39.1	53	2,137
31	0.56	-0.500	20.2	-17.8	-24.9	13.6	-24.3	-31.5	26.8	-11.2	-18.3	37.9	49	1,524
22	0.56	-0.500	18.8	-17.8	-24.9	12.2	-24.3	-31.5	25.4	-11.2	-18.3	36.5	50	1,100
12	0.56	-0.500	17.3	-17.8	-24.9	10.7	-24.3	-31.5	23.9	-11.2	-18.3	35.1	56	667
0	0.56	-0.500	17.3	-17.8	-24.9	10.7	-24.3	-31.5	23.9	-11.2	-18.3	35.1	30	0

Wind Calculations						
Level	Force (k)		Shear (k)		Overturning Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	39.9	20.3	39.9	20.3	6261	3179
16	75.7	38.4	115.5	58.6	16753	8499
15	68.4	34.6	183.9	93.2	24646	12494
14	61.3	31.0	245.2	124.2	30403	15402
13	57.5	29.0	302.7	153.2	34807	17622
12	60.1	30.3	362.7	183.5	38450	19453
11	59.3	29.9	422.1	213.4	40517	20483
10	55.5	27.9	477.6	241.3	41549	20989
9	57.8	29.0	535.4	270.2	41761	21079
8	56.9	28.4	592.3	298.7	40274	20309
7	53.0	26.4	645.2	325.1	38069	19179
6	54.8	27.2	700.1	352.3	35003	17616
5	53.4	26.4	753.5	378.8	30140	15150
4	49.2	24.2	802.7	403.0	24882	12492
3	50.0	24.4	852.6	427.4	18758	9403
2	**55.6**	27.0	908.2	454.4	10898	5453
				Total	473173	238804

Frame Member Sizes

	Frame 1			Frame 2			Frame 3		
Level	Beam	Column	Braces	Beam	Columns	Braces	Beams	Columns	Braces
Roof	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
16	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
15	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
14	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
13	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
12	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
11	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
10	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
9	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
8	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
7	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
6	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
5	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
4	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
3	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
1	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8

	Frame 4			Frame 5			Frame 6		
Level	Beam	Column	Braces	Beam	Columns	Braces	Beams	Columns	Braces
Roof	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
16	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
15	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
14	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
13	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
12	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W18x50	W14x82	HSS6x6x5/8
11	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
10	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
9	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
8	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
7	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W18x50	W14x109	HSS7x7x5/8
6	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
5	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
4	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
3	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8
1	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W18x50	W14x159	HSS8x8x5/8

	Frame 7			Frame 8			Frame 9		
Level	Beam	Column	Braces	Beam	Columns	Braces	Beams	Columns	Braces
Roof	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2
16	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2
15	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2
14	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2
13	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2
12	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2	W14x43	W14x82	HSS6x3x1/2
11	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2
10	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2
9	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2
8	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2
7	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2	W14x43	W14x109	HSS6x4x1/2
6	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2
5	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2
4	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2
3	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2
2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2
1	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2	W14x43	W14x159	HSS6x6x1/2

GIRDER SLAB CALCULATIONS

Typical Floor (2nd - 16th floor)

DEAD LOAD: 60 psf
PARTITION LOAD: 20 psf
LIVE LOAD: 40 psf
TOPPING LOAD: 25 psf (after grout has cured)

ALLOWABLE $\Delta_{LL} = L/360$

$$LL_{red} = \left(1 - \left(\frac{\%\,red}{100}\right)\right)(LL)$$

100 (1-

PRECOMPOSITE

$$M_{DL} = \frac{(\text{PLANK SPAN})(DL/1000)(\text{DB SPAN})^2}{8} < M_s\ cap$$

$$\Delta_{DL} = \frac{5(\text{PLANK SPAN})(DL/1000)(\text{DB SPAN})^4(1728)}{384(I_s)(29000)}$$

COMPOSITE

$$M_{sup} = (\text{PLANK SPAN})(\text{PART LOAD} + \overset{\text{REDUCED}}{\text{LIVE LOAD}} + \text{TOPPING}/1000)(\text{DB SPAN})^2/8$$

$$M_{TL} = M_{DL} + M_{sup}$$

$$S_{req} = (M_{TL})(12\ \text{in/ft})/0.6(50) < S_t$$

$$\Delta_{sup} = \frac{5(\text{PLANK SPAN})(P.L + L.L + T.L/1000)(\text{DB SPAN})^4(1728)}{384(I_t)(29000)} < L/360$$

COMP. STRESS.

$$N_{value} = \frac{E_{st}}{E_{con}} = \frac{29000}{57,000\sqrt{4000}} = 8.04$$

$$S_{tc} = N(S_t)$$

$$f_c = M_{sup}(12\ \text{in/ft})/S_{tc}$$

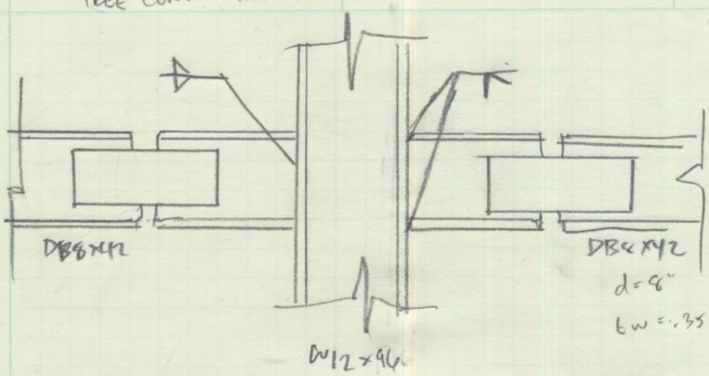
$$F_c = 0.45(\text{Grout } f'c) > f_c$$

BOTTOM FLANGE TENS. STRESS

$$f_b = \frac{M_{DL}(12\ \text{in/ft})}{S_{b\ st}} + \frac{M_{sup}(12\ \text{in/ft})}{S_{b\ trans.}}$$

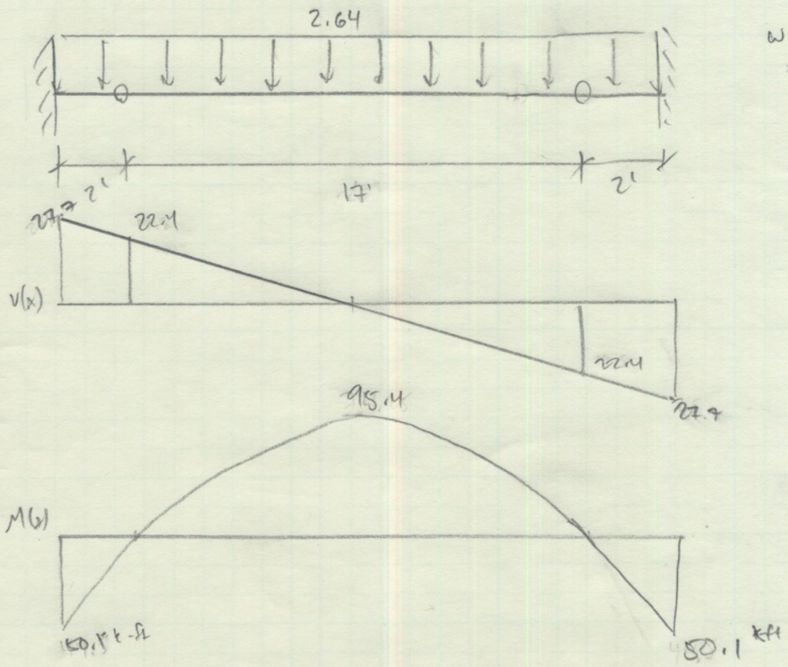
$$F_b = 0.9(50).$$

TREE CONNECTION



Level 2

TRIB WIDTH = 22'



$w = DL + LL$
$= 80 + 40$
$= 120\ psf\ (22') =$
$= 2640\ PLF$

Need WT8 to resist 44.5 k-ft

WT8 x 22.5
$d = 8.07"$
$t_w = 0.345$
$b_f = 7.04$
$t_f = .565$
$S_x = 37.8\ in^3$

$$S_{req} = \frac{(50.1\ ^{ft\text{-}k})(12\ ^{in/ft})}{(0.6 \times 50\ ksi)} = 20.0\ in^3 < 37.8\ in^3 \therefore \underline{\underline{OK}}$$

Tree Connection

Bottom Flange Tension Stress

$\frac{50\ (12'\text{-}ft)}{37.8} = 15.87\ ksi < F_c = 45\ ksi \quad \therefore \underline{OK}$

Shear

$f_v = \frac{27.7^k}{(0.345)(7.5)} = 10.7\ ksi$

$F_v = 0.4(50ksi) = 20\ ksi > 10.7\ ksi \quad \therefore OK$

Weld Design

· Bevel Top & Bottom as per G-S Design Guide

Fillet on Both sides.

Table J2.5 AISC 13^{th} Ed.

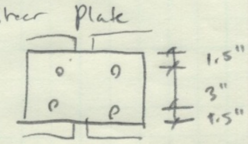
Tension normal to weld axis
Compression normal to weld axis
> strength of joint is controlled by base metal

Max size of fillet: $1/4"$ ⇒ controlled by web t_w of WT8x21.5

WT8x21.5   W12x72

$t_w = 0.345"$ $t_f = 0.67"$

Shear Plate



$V = 27.7^k$

Table 10-9a

Use 2 row bolts - $3/4"$ d A325-N

$7/16"$ plate ⇒ 28.5^k (ASD)

A325 Type N

Tree Connection

Use 9" x 6" x 7/16" plate

Check Bearing & 7 i.o. of DB. $t_w = 0.345$

Bearing: $2.4(65^{ksi})(1")(.345) = 53.42^{k}$

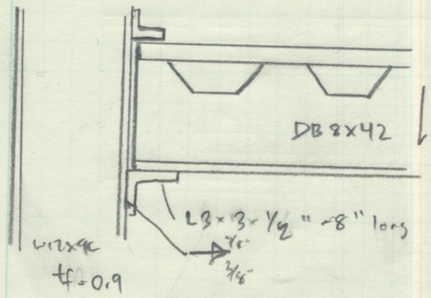
Edge: $1.2(65^{ksi})(1.5 - 0.5(1" + 1/16)](.345) = 26.1^{k}$

Other = $1.2(65^{ksi})(3 - 0.5(1 - 1/16)](.345) = 64.5^{k}$

$R_n/\Omega = 53 + 26.1 = 79/2.0 = 39.55 > 27.7^{k}$ ok

Seat Connection

Unstiffened seat connection



A7 SECOND FLOOR

DB8x42 (W12x53 parent beam)

$b_f = 10"$

$t_f = 0.575$

$d = 8"$

$t_w = 0.345$

$k_{det} = 1\frac{1}{2}$

$k_{des} = 1.24$

$V_u = 22.4^k$

Find N_{min}

$N = 4 - 0.75 = 3.25" = N_{prov.}$

Web Yielding

$$N_{min} = \frac{R_u}{1.0 F_y t_w} - 2.5 k_{des}.$$

$$= \frac{22.4}{1.0(50)(0.345)} - 2.5(1.24) = -1.80 < k_{det} \therefore \text{Use } k_{det} = 1\tfrac{1}{2}"$$

Web Crippling assume $N/d \le 0.2$

$$N_{min} = \frac{d}{3}\left(\frac{t_f}{t_w}\right)^{1.5}\left[\frac{R_u}{0.75(0.4)t_w^2}\sqrt{\frac{t_w}{EF_{yw}t_f}} - 1\right]$$

$$= \frac{8}{3}\left(\frac{0.575}{0.345}\right)^{1.5}\left[\frac{22.4}{0.75(0.4)(.345)^2}\sqrt{\frac{0.345}{29000(50)(.575)}} - 1\right]$$

$$= -2.4 < k_{det} \therefore \text{use } k_{det}$$

$k_{detailing} = 1.5" < 3.25" \therefore$ Use $N_{min} = 1.5"$

$1.25/8" = 0.156 < 0.2$ OK

- Seat Angle Flexure

$$e = \frac{1.25}{2} + 0.75 - 5/8 - 3/8 = 0.375"$$

$$\phi R_n = \frac{0.9 F_y L_a t_a^2}{4e} = \frac{0.9(36)(8")(1/2")^2}{4(0.375)} = 43.2 > 22.4^k \text{ OK}$$

Seat Connection

- Angle Shear Yielding

$k = 0.0$

$\ell = 3/4 + N/2 = 3/4 + 1.5/2 = 1.5$ in

$a = e/\ell = 1.5/3 = 0.5$

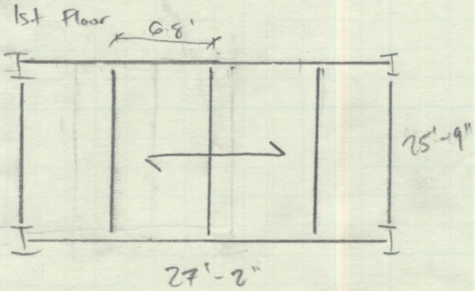
Find $C = 2.29$

$$D_{min} = \frac{\Omega R_u}{C C_1 \ell} = \frac{2.0(22.4)}{(2.29)(1.0)(3)} = 6.00 \text{ } 1/16^{ths} \Rightarrow$$

min weld $= 3/8''$

Composite Beam Design

1st Floor



A992 Steel

$F_y = 50$ ksi

Composite Beam

ASD

Live load = 100 psf

Dead load = SDL = 20 psf

DECK = 2.1 psf

Concrete = 50 psf

Beams = Assume 26 plf

Deck : USD 2" Lock Floor, 20 GAUGE

4" conc. (TOTAL DEPTH = 6") $f'_c = 3$ ksi

MAX UNSHORED SPANS, 3 SPANS => 7.85' > 6.8' ∴ OK

CONSTRUCTION LOADING

BEAMS SPANNING 26' ⊥ TO DECK

$W_{LL\ cons} = 20$ psf

$W_{DL} = (50 + 2.1\ \text{psf})(6.8') + 26\ \text{plf} = 380\ \text{plf}$.

$W_{TOTAL} = 380\ \text{plf} + (20\ \text{psf})(6.8') = 516\ \text{plf}$

$$M = \frac{516\ \text{plf}\ (26')^2}{8} = 43{,}602\ \text{lb-ft} = 43.6\ \text{k-ft}$$

Try W12x19 ; $\frac{M_r}{\Omega} = 61.6$ k-ft , $I_x = 130\ in^4$

Check Deflection during construction

$$\Delta_{cons} = \frac{5}{384}\frac{wl^4}{EI} = \frac{5}{384}\frac{(.516\ \text{klf})(26)^4(1728)}{(29{,}000)(130)}$$

$\Delta_{const} = .70"$

$$L/360 = \frac{(26)(12)}{360} = 0.87" > \Delta_{const} \therefore OK$$

Composite Beam

Composite Action

Beam ⇒ assume $a = 1"$

$y_2 = 6 - \frac{1}{2} = 5.5"$

$w_D = (20\,psf + 2.1 + 50\,psf)(6.8') + 26\,plf = 431.9\,plf$

Live Load Reduction

$L = L_0 \left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$ $K_{LL} \Rightarrow$ INT BEAM = 2

$A_T = 6.8' \times 26' = 176.8\,ft^2$

$= 100\,psf \left(0.25 + \frac{15}{\sqrt{2(176.8)}}\right)$

$= 100\,psf\,(1.04)$

No REDUCTION

$W_{TOTAL} = 431.9\,plf + (100\,psf)(6.8') = 1111.9\,plf$

$M = \frac{(1111.9)(16)^2}{8} = 35.6\,k\text{-}ft$

W10x22 PNA, BFL (5.5) $M_p/\Omega_p = 113\,^{k\text{-}ft}$

$b_{eff} \le \frac{1}{4}$ span $= \frac{26' \cdot 12"}{4} = 78"$ ⇐ controls

$\le$ spacing $= 6.8' \times 12" = 81.6"$

$C_{TOT} = C_c + C_f =$

$\Sigma Q_n = 117$

$a = \frac{117^k}{(0.85)(3)(60)} = 0.765 < 1"$

$y_2 = 6 - \frac{0.765}{2} = 5.62$

STUDS TABLE 3-21

Weak studs per rib 1 ⇒ $\phi = 3/4"$

$\frac{2\Sigma Q_n}{Q} = \frac{2(117)}{17.2} = 13.6\,k$

Use 14 - 3/4" ϕ studs

Composite Beam

Check LL Deflection

$$\Delta_{LL} = \frac{5}{384}\frac{(0.1)(26)^4(1728)}{29000\,(311)}$$

$I_{LB} = 311$ from table 3-20

$$\Delta_{LL} = 0.114''$$

$L/360 = 0.533 > 0.114''$ ∴ ok

$$0.87'' = \frac{5}{384}\frac{(0.1)(26)^4(1728)}{29000\,(I_{LB})}$$

required I

I_{LB} for LL $= 40.75\ in^4$

$M = 43.6'^{k}$

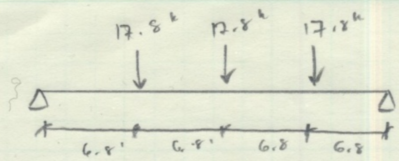
I_{LB} for Δ_{const} $= 2102\ in^4$

I_{LB} Δ_{total} $= 168\ in^4$

RAM Results:

W12 x 14 (18 studs)

GIRDER



$M_{max} = (17.8^k)(6.8') \cdot 121.^{'-k}$

Solve for deflection and the check strength

$\Delta_{total} = \frac{Pa}{24EI}(3l^2 - 4a^2)$

$l/240 = 1.36"$

$1.36" = \frac{17.8^k(81.6")}{24(29000)I}\left(3(326.0)^2 - 4(81.6)^2\right)$

$I = 448 \text{ in}^4$

$I_{LB} \geq 448 \text{ in}^4$

$y_2 = 5.5"$

Try 14x22 PNA · BFL (5.5) $M_p/\Omega = 153$ $I_{LB} = 523 \text{ in}^4$

$\Sigma Q_n = 157$

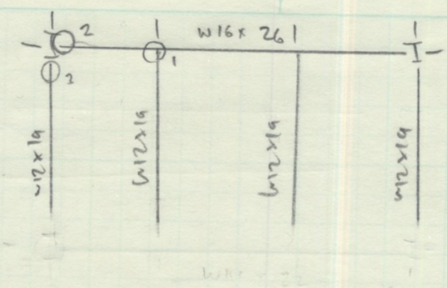
$a = \frac{157}{(0.85)(3)(60)} = 1.03$

$y_2 = 6 - \frac{1.03}{2} = 5.485$

Use $y_2 = 5 \Rightarrow$ $M_p/\Omega = 149$ $I_{LB} = 497 \text{ in}^4$

RAM ⇒ 14x22 w/ 30 studs

Connection

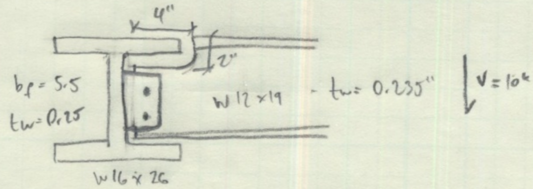


Connection #1 : Sheer Tab

#2 : Welded (to column) / bolted single angle

#3 : Bolted (to beam) / welded (to Girder) Double Angle

#1



Bolts : 3/4" ϕ A325-N

Plate : A36

$$V = \frac{(1111.9\ plf)(16')}{2} = 10^k$$

Table 10-9a

$n = \frac{9}{15.9} = 0.$

Use 2 - 3/4" ϕ A325N bolts

$\frac{1}{4}$" Plate $\Rightarrow 16.3^k$

$L = 5\frac{1}{2}$" $a = 2\frac{1}{2}$" $L_{eh} - 2d_b = 1.5$" $\Rightarrow$ 6" x 5" plate

weld size $.3/4\ t_p \Rightarrow 3/16$"

Coped Beam Flexural Strength

Buckling $\frac{c}{d} = \frac{4}{12.2} = 0.33$

$f = 2(.33) = 0.66$

$\frac{c}{h_0} = \frac{4}{10.2} = 0.39 \le 1.0$

$k = 2.2\left(\frac{10.2}{4}\right)^{1.65} = 10.31$

$\frac{F_{cr}}{\Omega} = \frac{26000}{1.67}\left(\frac{.20}{9.9}\right)^2(.33)(10.31)$

$= 21.62 ksi$

$S_{net} = 6.01\ in^2$

$\frac{50}{1.67} = 29.9\ ksi > F_{cr} = 21.62\ ksi$ ok

$M_n/\Omega = \frac{1}{2.00}(65^{ksi})(4.71\ in^3) = 153\ in^k$

$M_u = (9^k)(4.5") = 40.5\ in^k$ ok

Correction .

Block Shear of Beam

$\frac{R_n}{\Omega} = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

$\Omega = 2.00$ (ASD)

$L_{eh} = a - 0.5" = 2"$

$t_w = 0.235"$

(2) 3/4" ϕ BOLTS

F_u 65ksi $F_y = 50$ ksi

Table 9-3a Tension Rupture

$\frac{F_u A_{nt}}{t\Omega} = 50.8$ kip/in

Table 9-3b Shear Yielding, $n=2$, $L_{ev} = 2"$

$0.6 \frac{F_y A_{gv}}{t\Omega} = 75$ kip/in

Table 9-3c Shear Rupture

$\frac{0.6 F_u A_{nv}}{t\Omega} = 71.9$ kip/in

$\frac{R_n}{\Omega} = 71.9$ k/in $(0.235) + 50.8$ kip/in $(.235) \leq 75(.235) + 71.9(.235)$

$28.83 \leq \underline{\underline{34.5215}} > 9^k$ ok

Bearing : $2.4 F_u d_b t_w = 2.4(65)(3/4")(0.235) = 27.495$

Edge : $1.2 F_u L_c t = 1.2(65)(1.5 - 0.5(3/4 + 1/16))(0.2355) = 20.05$

Other : $1.2 F_u L_c t = 1.2(65)(3 - 13/16)(0.235) = 40.09$

$R_n/\Omega = [27.495 + 20.05]/2.00 = 23.77 > 9^k$

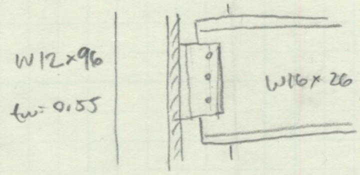
$\therefore$ ok

Size weld

$t_{weld,min} = \frac{1}{4} \cdot (5/8) = 0.16 = 2.5/16"$

use 3/16" weld both ends.

Connect #2



$V = 26$ kips

BOLTED / WELDED SINGLE ANGLE

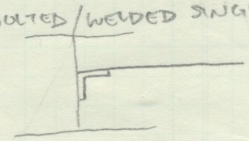


Table 8-10

$R_n = CC_1 Dl$

$kl = 4$

$l = 11''$ $\Rightarrow k = 0.4$

$t_w/2 = 0.55/2 = 0.275$

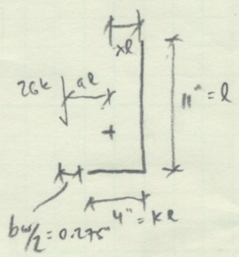
FROM TABLE, $x = 0.057$

$xl = 0.057(11) = 0.627$

$al = 4 + 0.275 - 0.627 = 3.65$

FROM TABLE 10-11

Use L4x4x3/8 x11"



FROM TABLE 8-4

C = 3.02 from interpolation

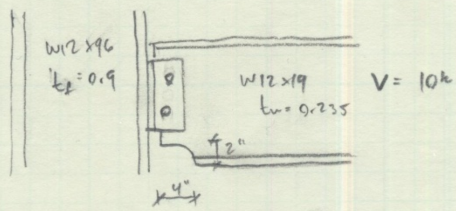
$$D_{min} = \frac{\Omega P_a}{C C_1 l} = \frac{2.00(26k)}{(3.02)(1.0)(11'')} = 1.565/16 \Rightarrow \text{use } 3/16'' \text{ weld.}$$

Min weld = 0.1875 . OK

TABLE J2.4

Max weld = 5/16" > 3/16" OK

Connection #3



Coped Beam flexural strength checked for connection #1

Table 10-1 2 rows A325 Type N w/ 1/4" Angle => 32.6 kips

1/4" thick angle 6" long.

L 3 x 3 x 1/4" - 6"

w/ n = 2, L = 6",

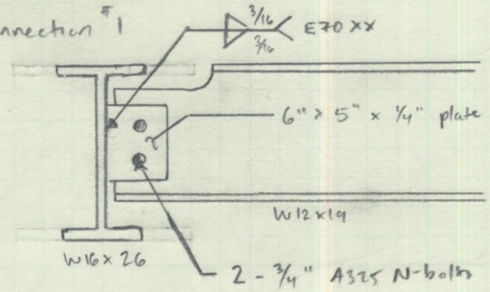
Weld design case 11, welds B (70 ksi)

size 3/8" => min supp. thickness = 0.286 < 0.9 ok

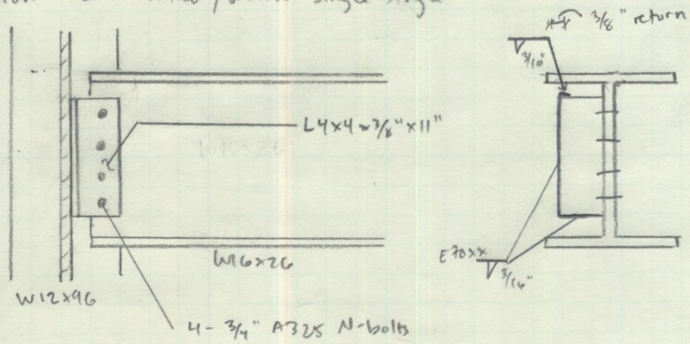
R_n/Ω = 21.9k

3" angle width

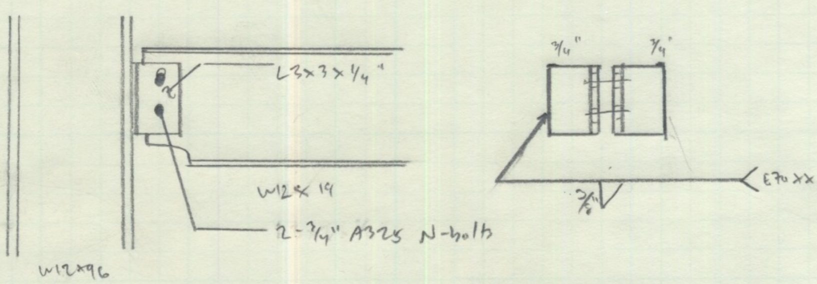
Connection #1



Connection #2 Bolted / welded Single Angle



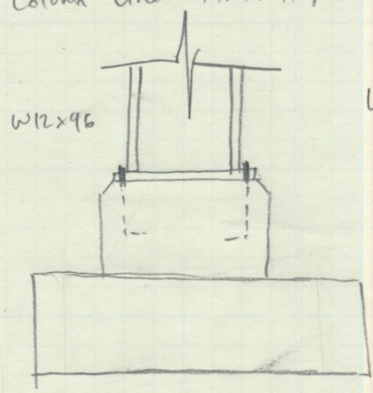
Connection #3 Bolted Welded Double Angle.



Footing Design

Bearing Capacity $\Rightarrow$ 12000 psf

Column Line 79.46 ft, 35.25 ft



Load @ Base = 914.85 k
Col: W12x96
$f'_c = 4$ ksi
A36 Base Plate

Base Plate Design

Assume $A_1 = A_2$

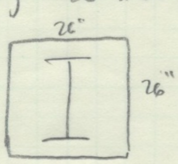
$P_p / \Omega_c > P_a \quad P_p = P_a \Omega_c$

$P_p = 0.85 f'_c A_1 \quad \Omega_c = 2.5$

$$A_1 = \frac{916.85^k (2.5)}{0.85\ (4\ \text{ksi})} = 674\ \text{in}^2 \approx 26" \times 26"\ \text{plate}$$

Try 26" x 26" plate

W12x96
$b_f = 12.2$
$d = 12.7$



$$m = \frac{N - 0.95d}{2}$$

$$= \frac{26" - 0.95(12.7")}{2} = 6.97"$$

Assume $\lambda = 1.0$

$$n = \frac{B - 0.8 b_f}{2} = \frac{26 - 0.8(12.2)}{2} = 8.12"$$

$$n' = \sqrt{\frac{d b_f}{4}} = \frac{\sqrt{(12.7)(12.2)}}{4} = 3.11"$$

Footing

$$t_{min} = l\sqrt{\frac{3.33 P_a}{F_y BN}} = 8.12'' \sqrt{\frac{3.33(916.85^k)}{36(26)(26)}} = 2.87''$$

Use $t = 3''$.

Base Plate = $\underline{26'' \times 26'' \times 3''}$

Footing Design

$$q_a \geq \frac{P}{A_{footing}} \Rightarrow 12ksf \geq \frac{916.85^k}{B^2}$$

$$B \geq 7.3' \Rightarrow \text{use } B = 7.5'$$

CRIT FACE $l = \dfrac{7.5 - 26''\left(\frac{1}{12''}\right)}{2} = 3.41'$

$$M_a = \frac{12ksf\,(3.41')^2}{2} = 69.7^{1k}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s(60)}{0.85(4)(12)} \Rightarrow a = 1.5A_s$$

$\Omega = 1.67$ $\quad M_a = \dfrac{M_n}{\Omega} = \dfrac{A_s f_y (d - a/2)}{\Omega} \Rightarrow 69.7(12) = \dfrac{A_s(60)\left(26.7 - \frac{1.5A_s}{2}\right)}{1.67}$

$\Omega = 2.00$ $\quad V_c = \dfrac{4\sqrt{f'_c}}{\Omega} = \dfrac{4\sqrt{4000}}{2.0} = 126.5$ psi

$$d^2\left(V_c + \frac{q}{4}\right) + d\left(V_c + \frac{q}{2}\right)w = \frac{q}{4}(BL - w^2)$$

$$d^2\left(126.5 + \frac{80.1}{4}\right) + d\left(126.5 + \frac{80.1}{2}\right)26 = \frac{80.1}{4}\left((108)^2 - 26^2\right)$$

$$146.5d^2 + 4330.3d - 220034.7 = 0$$

$$d = 26.7 \text{ in}$$

$$h = d + 3''\text{clear} + d_b$$

$$= 26.7 + 3 + 1.128 = 30.8'' \Rightarrow \text{use } 32''$$

Footing

$836.4 = 959.3 A_s - 27 A_s^2$

$A_s = 0.894$

Use #9 @ 12" ⇒ 1.00 in²/ft

$\rho = \frac{A_s}{bh} = \frac{1.00 \text{ in}^2}{(12")(32")} = 0.0026 > 0.0018 \therefore \underline{ok}$

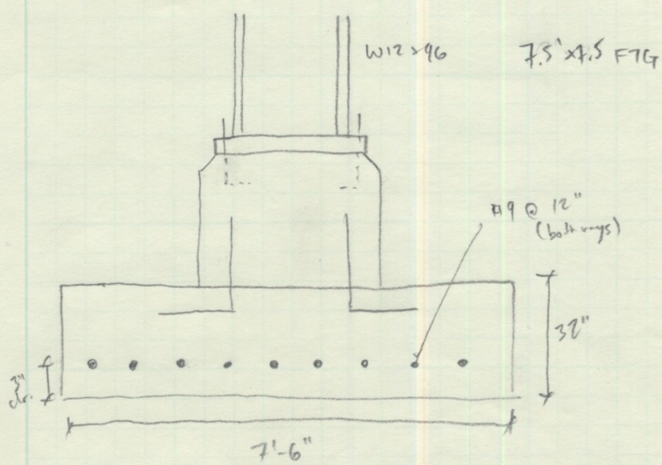
$a = 1.5(1.0) = 1.5$

$c = \frac{1.5}{0.85} = 1.76"$

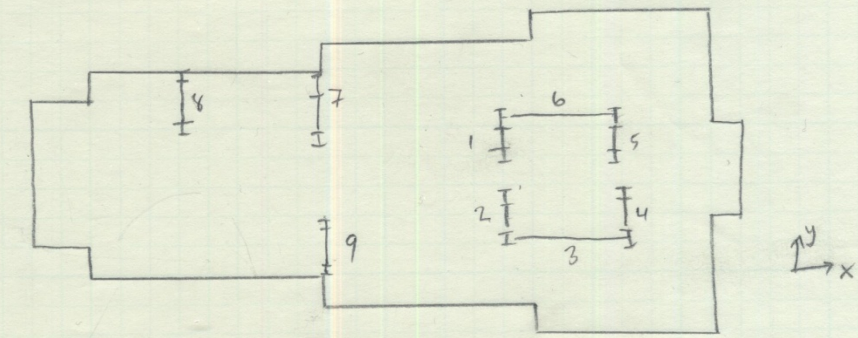
$\varepsilon_s = \frac{0.003}{1.76}(32 - 1.76) = 0.052 > 0.005 \therefore \underline{ok}$

$\frac{B_n}{\Omega} = \frac{0.85 f'_c A_p}{\Omega} = \frac{(0.85)(4)(26 \text{ in})^2}{1.5} = 919.4 > P = 914.85^k$

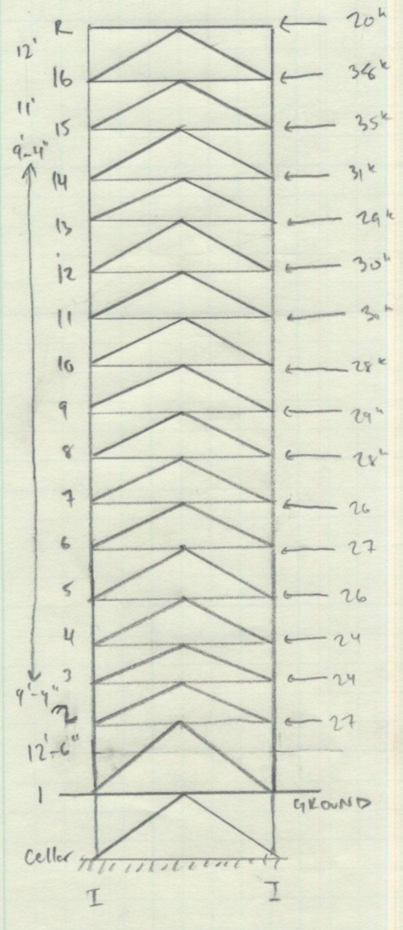
$A_{s,min} = 0.005 A_{pier} = 0.005(26)^2 = 3.38 \text{ in}^2 \therefore \underline{ok}$



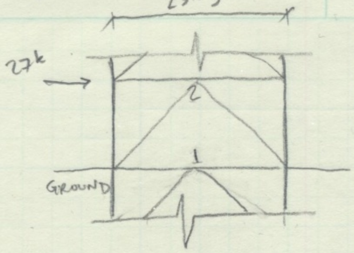
LATERAL FRAME DESIGN



FRAME 6



LAT. FRAME DESIGN



At first floor:

GRAV AXIAL
DL = 352 k
LL = 88 k

WIND AXIAL
$= \frac{5453'^{-k}}{23.25'} = 234.5^k$
from Spreadsheet

$D + W = 352 + 234.5^k = 586.5^k$

$D + 0.75L + 0.75W = 352 + .75(88) + .75(234.5) =$

Controls $\Rightarrow$ $\underline{\underline{594^k}}$

SIZE W14×159 $\quad I_x = 1900$

$L_b = 12.5'$

$$F_e = \frac{\pi^2 (29000)}{\left(\frac{0.7(12.5' \times 12''/')}{8.2}\right)^2} = 1745 \text{ ksi}$$

$$F_{cr} = \left[0.658^{\frac{50}{1745}}\right] 50 = 49 \text{ ksi}$$

$$\frac{P_n}{\Omega} = \frac{F_{cr} A_g}{\Omega} = \frac{49(46.7 in^2)}{1.67} = 1370 \text{ k}$$

$$\frac{M_n}{\Omega} = 1032 \text{ k-ft}$$

$$M = \frac{27^k \times 12.5'}{4} = 84.4'^k$$

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}}$$

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta h} = 1.0\left[\frac{452^k(12.5 \times 12)}{0.03}\right] = 226000$$

$$B_2 = \frac{1}{1 - \frac{1.6(594)}{226000}} = 1.0$$

$M_r = 84.4'^k$

$P_r = 594^k$

$$\frac{P_r}{P_c} = \frac{594}{1370} = 0.434 > 0.2 \quad (H1\text{-}1a)$$

$$H1\text{-}1a = \frac{P_r}{P_c} + \frac{8}{9}\frac{M_{cr}}{M_{cx}} = 0.434 + \frac{8}{9}\left(\frac{84.4}{1032}\right) = 0.507 < 1 \therefore \underline{ok}$$