Dulles Town Center Building One

Dulles, Virginia

THESIS REPORT

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Dulles Town Center Building One

Dulles, Virginia

Project Team

Owner: Lerner Enterprises
Architect: SmithGroup
Structural: SK&A
Civil: Dewberry & Davis
MEP: KCF/SHG Inc.
Construction: Tompkins Builders, Inc.

Building Statistics

Size: 202,110 sq. ft.
Height: 7 Floors Above Grade
1 Floor Below Grade
118 ft. to top of architectural fin
Occupancy: Commercial/Office
Build Dates: Fall 2000-Spring 2002
Cost: Withheld By Owner
Delivery Method: Design-Bid-Build

Architectural

On 12.37 acre lot at the intersection of RTE 7 and RTE 28
Precast Concrete with Curtain-Wall Systems
Open Floor Plan
Typical Floor-to-Floor Height: 12′-6″
Roof is Stone Ballast over Filter Fabric over 3″ Rigid Insulation over Roofing Membrane on top of Roof Slab

Structural

Post-Tension Beams with Non-Post-Tension One-Way Slab System
Slab-On-Grade and Caisson Foundation System
Lateral Forces taken by Eccentric Braced Frames and Ordinary Concrete Moment Frames
Typical Bay Size: 40′ x 20′

MEP

Single-Zone Self-Contained A/C Units (1 per floor)
Condenser Water System, both Open- and Closed-Loop Systems
Building Powered by 1500 kVA Transformer via 12-Duct Bank
Main Electrical Room Houses 4000 A, 480/277 V Switchboard
Typical Lighting at building core is Recessed Down Lighting with few Wall-Mounted Luminaires

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Executive Summary

Dulles Town Center Building One, or DTC One, is located in Dulles, Virginia; five minutes north of Dulles International Airport and 25 miles outside of Washington, D.C. It consists of seven stories of office space above grade and one story below grade that includes rentable space, storage, mechanical rooms, a loading area, a trash room, building service offices, and a workout space. The building is approximately 202,000 square feet and reaches a total height of 118 feet above grade. The building has an open floor plan and an average floor-to-floor height of 12'-6" making it ideal for office space.

The following report investigates and discusses the effects of redesigning the gravity and lateral systems of DTC One from concrete to steel. The structure currently utilizes a post-tensioned beam one-way concrete slab gravity system along with ordinary reinforced concrete moment frames. The steel system investigated in this report is a composite metal deck system with ordinary steel moment frames. With this change in material, a comparison of the cost and duration of construction between the two systems was made to determine if there would be a time or monetary benefit to the steel redesign. An acoustics study was conducted, as well, to the floor and roof systems separating the penthouse and roof from the 7th floor, respectively. They will be analyzed to determine if the decrease in concrete thickness within the floor slab used in the system will allow noise from the mechanical equipment above to disturb the office space below.

The structural system was originally designed using BOCA National Building Code, 1996, along with other old and outdated codes. The steel redesign of DTC One was conducted in accordance with current codes such as IBC 2006 and ASCE 7-05. To help with column and lateral system designs, a model was constructed in RAM and was used to help size members and keep the building within serviceability guidelines. Composite beams and other east-west beams were designed to be W18's in an effort to keep the floor-to-ceiling height at the current 9', but to no avail. The long spans and heavy wind loads caused the W18's to be large and, as a result, have depths larger than 18". W16's and W21's were also used within the structure, mainly in the interior moment frames running from north to south and in the roof system. Columns sized to be W14's were spliced every other floor in order to save time in construction and were used to take gravity and lateral loads and take them down to the already existing caisson foundation system.

The construction management study that was performed enabled both systems to be compared based on their cost and duration of construction. The cost analysis was made using R.S Means and yielded an estimated cost of $5.3 million for the steel structural system. The concrete structure turned out to be less than that with an estimated cost of $4.9 million. To offset the increase in cost, however, the steel structural system was erected more than a year faster than that of the existing concrete system. As for the acoustics study, the results indicated that there were no problems with sound penetration in the 7th floor office space induced by mechanical equipment on the roof and in the penthouse.

Introduction

DTC One project consists of seven stories of office space above grade and one story below grade that includes rentable space, storage, mechanical rooms, a loading area, a trash room, building service offices, and a workout space. It is located in Dulles, Virginia; five minutes north of Dulles International Airport and 25 miles outside of Washington, D.C. The building's architectural use of precast concrete and glass curtain-wall have helped set the tone for the modernist themes conveyed along the Route 28 corridor. At night, this building is one of the most recognizable buildings along Route 28 with its linear neon focal points.

The building is approximately 202,000 square feet and reaches a total height of 118 feet above grade. The building has an open floor plan and an average floor-to-floor height of 12'-6" making it ideal for office space. The floor framing system is a post-tension concrete beam and non-post-tension one-way slab system. This allows for long 40 foot spans making a typical bay 20 feet by 40 feet. The lateral force resisting system is made up of ordinary concrete moment resisting frames in both the east-west and north-south directions.

The following thesis report will discuss the effects and potential cost benefits of redesigning the gravity and lateral systems of DTC One from a concrete system to a steel system. The gravity system will go from a post-tension concrete beam and non-post-tension one-way slab floor framing system to a composite metal deck floor system and the lateral system will change from ordinary reinforced concrete moment frames to ordinary steel moment frames. A comparison of the project schedule and cost of both systems will then be made. An acoustics study will also be conducted on the floor system separating the roof and penthouse from the 7th floor to determine if the mechanical equipment above will disturb the office space below with the decrease in concrete used for the slab.

Basic Building Information

General Building Data

Building Name: Dulles Town Center Building One
Building Location: 21000 Atlantic Boulevard, Dulles, VA
Building Occupants: Harris Corporation, C2 Profile and Trex
Building Function and Occupancy: Commercial/Office – Use Groups B and A-3
Building Size: 202,110 square feet
Number of Stories above Grade: 7
Height of Building above Grade: 118′
Type of Construction: 2A modified to 2B
Dates of Construction: Fall 2000 – Spring 2002
Delivery Method: Design-Bid-Build

Project Team

Owner: A DEVELOPMENT OF LERNER ENTERPRISES

Architect: SMITHGROUP architecture engineering interiors planning

Structural Engineer: SK&A Consulting Structural Engineers

MEP Engineer: KCF/SHG Inc.

Civil Engineer: Dewberry

General Contractor: TOMPKINS BUILDERS, INC.

Governing Building Codes Used for Initial Design

- Virginia Uniform Statewide Building Code
- BOCA National Building Code, 1996
- International Mechanical Code, 1996
- International Plumbing Code, 1995 plus 1996 Supplement
- CABO ANSI A-117
- National Electrical Code, 1996

Existing Conditions

Site

The building is located at one of the most visible spots in Northern Virginia, where Route 7 meets Route 28. To the north there is a 679 spot parking lot. To the east is Atlantic Boulevard, on which both entrances to the site are found, one at the northeast corner of the site and one near the building entrance on the east side. To the west is Route 28, one of the major roadways in Northern Virginia. The site is 12.37 acres and generally slopes from northeast to southwest. Nearby structures include the Dulles Town Center Mall and its surrounding restaurants, stores and shopping centers.

Architecture

The building is split architecturally into three pieces. To the east there is a rectangular precast concrete "box" seven stories high with cut-out windows which opens to the ground level and houses office space and a lobby. The color of concrete plays off the color of the Dulles Town Center Mall located to the east. To the west there is a polygonal shape encased solely of glass that also houses office space and comes down to the cellar which has a precast concrete façade. On the 7th floor of this façade there is a box-like form protruding from the flat glass wall. This is used to break up the monotonous façade. Slicing through the two main building components is an architectural fin covered in corrugated metal panels that progress into galvanized metal paneling. This not only holds the building's core, such as central corridors, bathrooms, and elevator shafts, but also masks the mechanical penthouse and hides the cooling towers and other mechanical equipment on the roof. There are also neon lights, a blue one on the south face and orange ones on the east and west faces, that extend from the roof to the ground floor to show off the building's verticality and catch the attention of drivers at night. A view of the north-eastern façade is located to the left.

North-East Elevation

Figure 1

Building Envelope

The middle of the east facing façade consists of a curtain wall system of blue reflective insulating glass framed in aluminum mullions from the 2nd floor to the roof, with the ground floor being clear low-E glass at the entrance. On either side of this curtain wall there is precast concrete wall with ribbon windows made of evergreen-colored low-E insulating glass over architectural precast panels. The west facing façade is comprised entirely of a curtain wall system. There is field curtain wall made up of blue reflective insulating glass and then two accented curtain walls of 1″ thick evergreen low-E insulating glass. Both field and accented curtain walls are framed in aluminum mullions and supported by the concrete structural system. The entire system extends from the ground floor to the roof and is bordered by insulated metal paneling. At the cellar level the façade changes to precast concrete panels. The north and south faces are generally the same as the two main facades. Each consists of precast concrete with ribbon windows, curtain wall, and steel panels. The roof is a post-tensioned beam and non-post-tensioned one-way slab system.

Building Systems

Mechanical System

Each floor houses a variable air volume self-contained air conditioning unit. Supply ducts for the cellar are 60″ x 18″, while the rest of the floors are supplied by 72″ x 20″ ducts. The cellar also holds a single zone self-contained air conditioning unit, which through a 48″ x 14″ supply duct heats and cools the lobby. Plasma televisions in the main lobby each have their own exhaust/cooling fan with an operating capacity of 78 cfm. The elevator room has a self-contained water-cooled air conditioning unit which is 4 nominal tons. The stairwells are pressurized and the lavatories are vented through the roof.

The condenser water system is made up of both open and closed loop systems. The open loop consists of a 530-ton double-cell induced draft cooling tower and two cooling tower pumps connected to a plate type heat exchanger. The closed loop consists of three condenser water pumps connected to a heat exchanger which supplies condenser water to the self-contained units. This setup also has a waterside economizer system, which allows cooler water from the cooling tower through the heat exchanger to cool the building when outside air temperatures are cool enough.

Lighting /Electrical System

Corridors in the cellar use recessed fluorescent light fixtures and down-lighting. The main lobby is predominantly illuminated by recessed and surface mounted cathode ray tube fixtures. A typical floor's elevator lobby is lit by recessed down-lighting and wall washers. Building One was designed as a tenant specific building, therefore lighting within each office space varies by tenant. The typical office lighting is recessed fluorescent lighting.

Outdoor lighting consists of up-lighting, down-lighting, and accent lighting. There is up-lighting at the base of the building on small trees and spots of the building that do not have neon accents. The architectural fin on the roof and roof overhang are also illuminated by up-lighting. Typical down-lighting is only located at the main entrance into the building. Cold cathode neon light accents stretching the height of the building can be found on the south and west elevations giving the building prominence along Route 28.

Power to DTC One is supplied by a 1500 kVA Virginia Power transformer through a 12-duct bank. The building's main electric room, located in the cellar, houses a 4000 A, 480/277 V switchboard. A 250 kVA/200 kW, 480/277V emergency generator, three minor transformers, and various panelboards can also be found in the cellar. Five sets of four 2000 A #600 kCMil wires make up the feeder which runs from the main switchboard to bus mounted 175 A circuit breakers on floors one through seven. The electricity used by tenants then goes through 112.5 kVA, 480/208/120 V transformers into panelboards.

Security

A security guard is posted at the front desk in the lobby and monitors the security cameras to insure the safety of tenants during work hours. Proximity cards are also a security measure taken. They are required by all persons to enter the building after working hours, access the exercise room and first floor stair entrances. They are also needed to run the elevators once inside. There is a hands free phone in the exercise room in case of emergencies along with panic switches in the locker rooms. Other safety precautions can be found at the loading dock doors and main entrance. Motion detectors, closed-circuit television cameras, emergency alert sirens, and electrical locks are located at these areas to keep a check on traffic flow in and out of the building.

Fire Protection

A combination Class I standpipe/wet fire sprinkler system with 2 ½" fire department valves and automatic fire sprinklers provide 100% coverage to the building. The sprinklers will be both concealed and exposed pendent sprinklers. The fire alarm system is a solid-rate, multiplex, addressable type with a voice evacuation system. Walls surrounding stairwells, elevator shafts and electrical rooms have 2-hour fire ratings. Tenant space separation and columns supporting more than one floor or the roof have a 1-hour fire rating. Floor and roof construction and structural members supporting walls have a 2-hour fire rating.

Building Transportation

The vertical transportation system is comprised of 2 elevators located in the building's core. Each car is 6'-8" wide and 5'-3" deep. Each emits 13406 Btu/hr.

Telecommunications

There is a service alcove with a telephone closet within the building core on each floor with both 2000A, 480/277 V, 3 PH, 4 W and 1600 A, 480/277 V, 3 PH, 4 W bus ducts. All other telecommunication networks are set up individually by the tenants.

Codes and Standards

At the time Dulles Town Center Building One was being designed, the permissible codes used for design were the 1996 Building Officials and Code Administrators International, Inc. (BOCA) National Building Code, which references American Society of Civil Engineers (ASCE) 7, and the Virginia Uniform Statewide Building Code. Concrete was designed using American Concrete Institute (ACI) 318 and steel design references the American Institute of Steel Construction (AISC) "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings".

Materials

Concrete

Floor System	f'_c = 5,000 psi
Columns	f'_c = 4,000 psi /5,000 psi
Penthouse roof slab	f'_c = 4,000 psi
Beams	f′c = 4,000 psi
Slab on grade	f'_c = 3,500 psi
Walls and piers	f'_c = 3,000 psi
Caissons	f'_c = 3,000 psi
Grade beams	f'_c = 3,000 psi
Other	f'_c = 3,000 psi

Reinforcement

Welded Wire Fabric	ASTM A185
Reinforcing bars	ASTM A615, Grade 60
Column and pier ties	ASTM A615, Grade 40

Structural Steel

Steel Pipe	ASTM A53, Grade B
Steel Tube	ASTM 500, Grade B
Other	ASTM A36

Existing Structural System

Floor System

The typical floor is a post-tensioned beam and non-post-tensioned one-way slab system. The 7″ thick slab is of normal weight with continuous edge drops that are 3′ wide and 5 ½″ deep along the east face to help support the precast concrete and ribbon window façade. A typical bay is 20′x 40′ with a typical beam length of 40′. Slab reinforcement consists of #4 top bars spaced at 6″ on center and #4 bottom bars at 12″ on center. Reinforced concrete beams are located at stairwells and elevator shafts.

Lateral System

The lateral resistance system in the east-west direction, as seen in *Figure 2*, is comprised predominantly of concrete moment frames. The typical beams are post-tension concrete sized at 17″deep and 48″ wide. The typical columns are reinforced concrete and are 24″x 24″.

Typical Floor - Concrete Moment Frame in East – West Direction

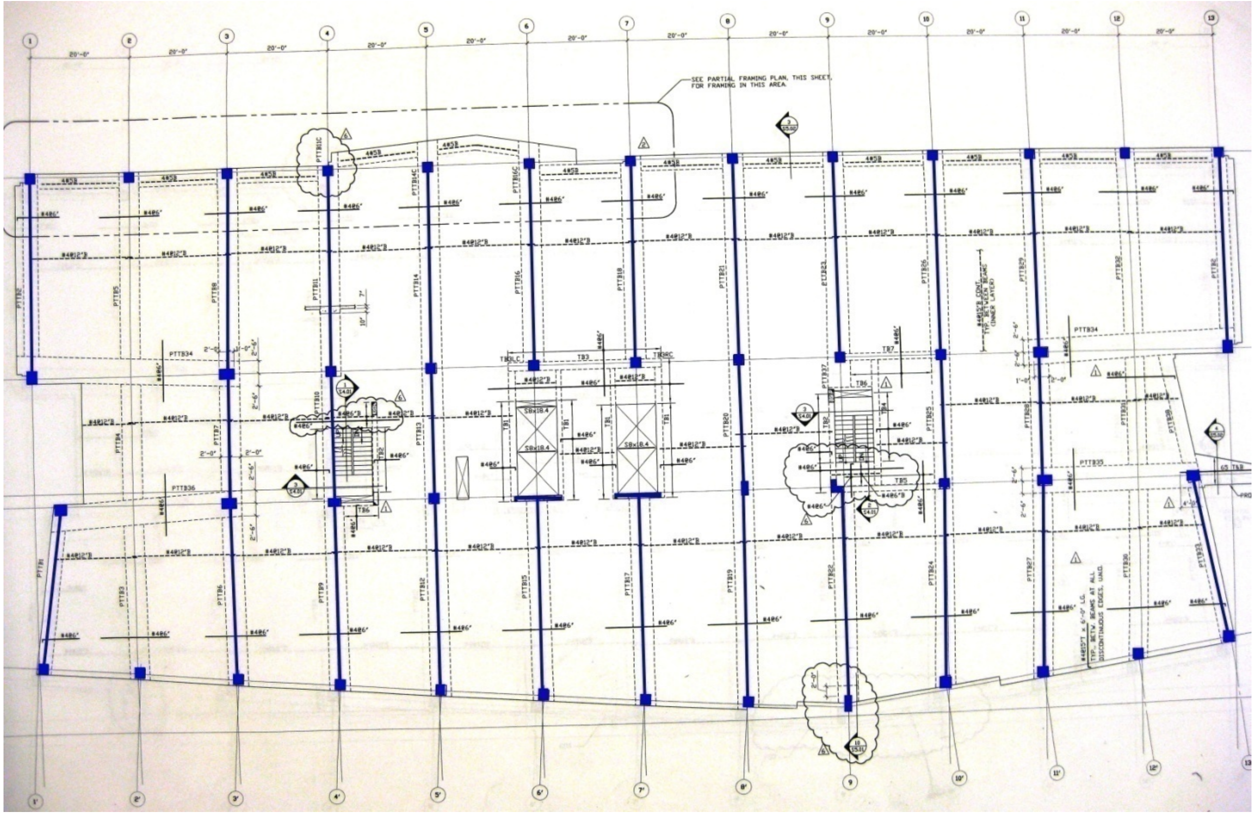


Figure 2

The north-south lateral system, seen in *Figure 3*, is also made up of concrete moment frames. The middle frames have large 24″x 60″ post-tensioned beams, shown as solid lines, at the frame-ends with the floor slab working laterally throughout the rest of the frame, shown with dashed lines, on typical 24″x 24″ reinforced concrete columns. The exterior frames use the 7″ slab, along with a 36″x 5 ½″ drop panel along the frame at plan north, with typical 24″x 24″ reinforced columns for lateral resistance.

Typical Floor - Concrete Moment Frame in East – West Direction

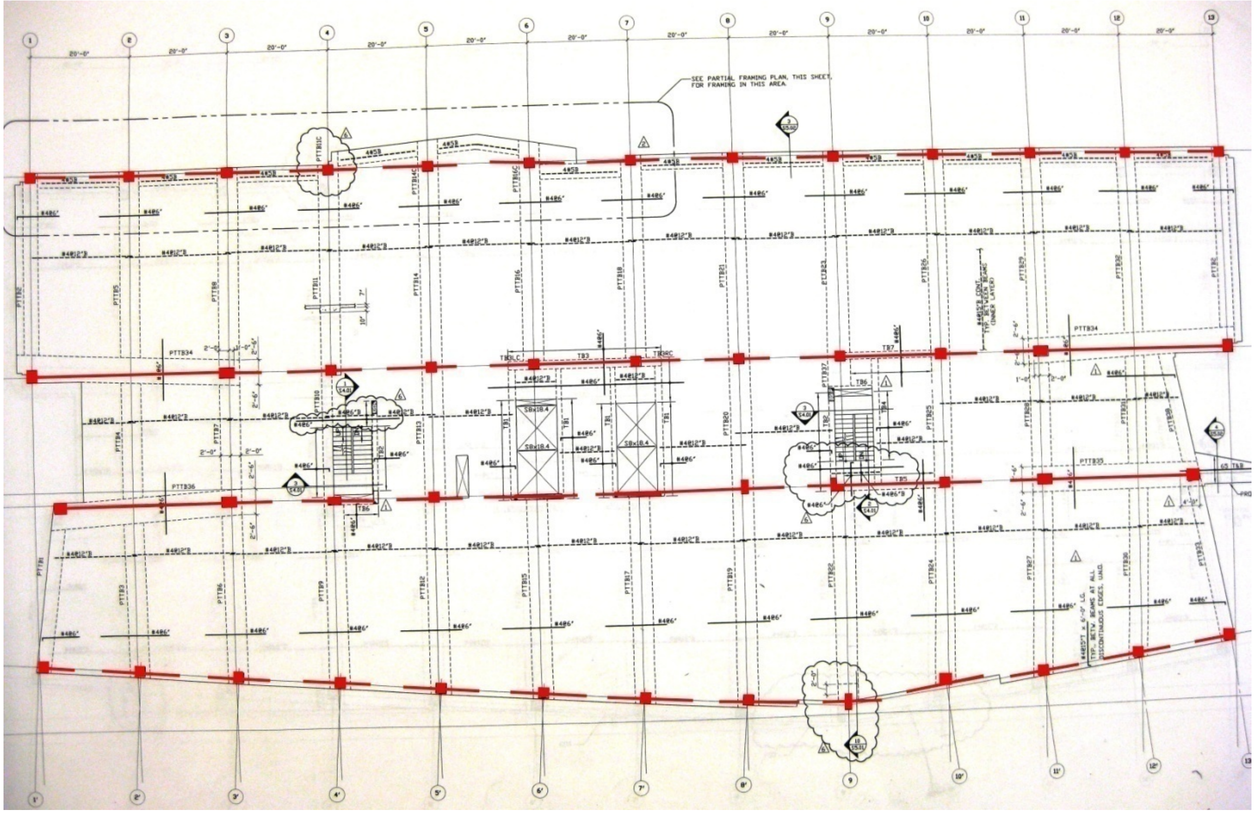


Figure 3

Foundation

The foundation system consists of a slab on grade with strap beams and caissons. The slab is 5″ thick and reinforced with 6x6 – W2.0xW2.0 welded wire fabric. It sits on a 6 mil. polyethylene vapor barrier over 6″ of washed, crushed stone. Strap beams ranging from 24″x 36″ to 48″x 48″ rest on a 2′-0″ thick foundation wall to help support the slab at grade changes. The cast-in-place caissons are capped with reinforced concrete and have shaft diameters that range from 30″ to 75″.

Roof System

The typical roof system also consists of a post-tension beam and non-post-tension one-way slab system. This typical roof system is just like the typical floor system in thickness, reinforcement, bay size, and beam length. Slab areas that support mechanical equipment, however, are 9″ thick and have #5 top bars at 8″ on center and #4 bottom bars at 6″ on center. The penthouse roof differs with its 8″ thick slab and #6 top bar- and #5 bottom bar-reinforcement at 12″ on center.

Columns

The vertical supporting elements are reinforced rectangular concrete columns with widths that range from 1′-0″ to 9′-2″. These 12″ x 110″ columns help support the stairwell and could act as small shear walls. Vertical reinforcement ranges in size from #8 to #11 rebar with #3 horizontal stirrups. The typical column is 24″ x 24″ with reinforcement consisting of (8) #8 vertical rebar, (3) #3 stirrups spaced at 3″ on center, and a hooked dowel extending 2′-6″minimum into the floor slab. These columns are also used for lateral resistance.

Problem Summary

Problem Statement

Concrete structural systems require long erection times due to curing time, shoring and re-shoring, and other labor intensive-related delays. Steel structures require much less time to erect, which could save money for the owner. They do, however, increase floor depth and increase overall building height. In Technical Report II, it was concluded that the current post-tensioned beam non-post-tensioned one-way slab system was optimal. Nonetheless, the composite metal deck system was found to be the next most efficient floor system. A composite metal deck structural system will be investigated to see if construction costs decrease while keeping the building under the maximum building height allowed by Loudoun County, Virginia. This new system will also decrease the roof slab thickness from 9" to 7 ½". The mechanical equipment located on the roof and in the penthouse could cause noise loud enough to penetrate the 7th floor office space. If this is the case, additional sound absorbing material will be required raising the cost of the 7th floor ceiling materials.

Proposed Solution

Floor System

The proposed floor system to be investigated and applied will be a composite metal deck system supported by steel members. It is a way to get the benefits of both steel and concrete into one floor system. The composite steel decking not only acts as permanent formwork, but also provides composite interlocking with the concrete to serve as reinforcement for the concrete slab.

After performing initial calculations in Technical Assignment II, members no larger than W18's were chosen to carry 3", 19 gage metal decking with a 7 ½" total slab depth. This makes the total floor depth approximately 28 ½". Current local codes will be investigated to determine if the overall height of the building peaks over the maximum height.

The material and construction costs associated with the application of this system will be analyzed and compared to the current structural system. The composite metal deck system will most likely have a shorter erection time, but a longer lead time will be required to fabricate W Shapes. The initial fabrication, material, and transport costs may outweigh the time and costs saved during construction time. These topics will be discussed and compared later in the report.

Lateral System

In order to keep the unobstructed architecture and advertised open floor plan, room for braced frames and shear walls was not available. Therefore, a lateral resisting system consisting of steel moment frames will be investigated. The seismic and wind loads will be calculated using ASCE 7-05 and will be used to design the new steel system. The location of moment frames within this system will be determined by available space and torsion effects created by the seismic and wind loads.

Foundation System

The proposed steel structural system will be much lighter than that of the current concrete system and therefore causes the need for the foundation system to be analyzed. In Technical Assignment III, it was assumed by inspection that overturning and uplift did not affect the current system due to building weight and soil friction. This could also be the case with the steel structure, but overturning and uplift must be investigated to determine if the current caisson system needs to be redesigned to handle the lateral forces.

Solution Methods

Floor System

The floor system will be designed with assistance from Vulcraft's *Steel Roof and Floor Deck* Product Catalog. Initial beam and column sizes will be determined using the 13th Edition of AISC's Steel Construction Manual and a model generated in RAM Structural System. The RAM model will continue to assist in design and help analyze the proposed system. Hand calculations will be conducted to compare sizes of members determined by computer software. The live loads that will be used in the design process will be taken from Chapter 4 of ASCE 7-05.

Lateral System

As done in Technical Assignment III, the lateral system will be designed using ASCE 7-05. Chapter 2 will be used for load combinations, Chapter 6 will be used for wind loads, and Chapters 11, 12, and 22 for seismic loads. The number of moment frames required will be determined by loads, both direct and torsional, on each frame and member sizes. The RAM model will assist in the design of the proposed steel moment frames and will calculate story displacements. A Portal Frame analysis will then be performed to get moments caused by lateral loads to use during hand calculations. Again, the 13th Edition of AISC's Steel Construction Manual will be used to check member sizes.

Foundation System

Since gravity loads will not affect the current foundation system, the caissons will be investigated to see if they can withstand overturning moments caused by wind and seismic loads. Size reduction to decrease material costs will be investigated as well, if the opportunity is presented. Analysis will include the use of ACI 318-08.

Breadth Topics

Construction Management Breadth

A complete investigation of costs and construction methods will be performed in order to compare the alternate steel system to the current concrete system. The goal will be to make the construction process as efficient as possible. This will include coordinating when a necessary building material should be ordered, when it should be erected, installed or poured, and the man- and machine-power needed to perform such tasks. This will help when offsetting lead times and set-backs. A cost analysis will be used to illustrate the effects changing the structural system has on the construction management of the project. The detailed cost analysis will be performed using prices from the R.S. Means catalog.

Acoustics Study

With the introduction of a steel structural system to the current layout of Dulles Town Center Building One, the decrease in concrete used for the roof and penthouse floor may lead to noise problems in the prime office space of the seventh floor. This study will investigate sound transmission using references such as “Noise Control in Buildings” by Cyril M. Harris and “Architectural Acoustics” by M. David Egan to determine sound penetration and acoustical materials necessary to help with sound absorption. A cost comparison will be conducted upon completion and compared to the existing ceiling and floor system.

Design Goals

The goal of this depth study was to determine the feasibility of changing the structural system of Dulles Town Center Building One from a post-tensioned beam one-way concrete slab system with ordinary reinforced concrete moment frames to a composite steel system with ordinary steel moment frames. A composite metal deck system was chosen for the redesign in order to learn more about steel as a building material and to establish whether it is more advantageous than the current concrete system. Other goals that were kept in mind during the redesign of Dulles Town Center Building One are as follows:

- To respect the current column layout in order to maintain the large spans and open floor plan and to limit the impact on the building's architecture.
- To design the new composite metal deck system efficiently and effectively while limiting the total floor depth to 42", which would keep the typical floor-to-ceiling height at its existing 9'.
- To use RAM Structural System to perform preliminary designs of gravity and lateral members and use them with hand calculations to determine final member sizes.
- To keep story and building drift within the serviceability standard of H/400 for wind loads and under the code-enforced $.020h_{sx}$ for seismic loads.
- To establish a design that not only quickens the duration of construction, but also decreases material and construction costs.
- To preserve a working environment on the 7th floor free of sound disruption caused by mechanical equipment on the roof and in the penthouse.
- To abide by all necessary codes and standards during the structural system redesign.

Structural Depth

Introduction

DTC One was originally designed as a spec building, using a post-tensioned beam one-way concrete slab system to achieve the desired long spans. These long spans would allow the owner to market open floor plans to possible tenants. The redesign was chosen to be in steel due to steel's high tensile strength, short erection time, lower weight, and because concrete was the main focus of last semester's technical reports. Within the possible steel framing systems, the composite steel system, which is seen in ***Figure 4***, was chosen due to its ability to reach the necessary spans while keeping an acceptable total floor depth. The redesign will use the most current codes as activities stated in the proposed solution are addressed. Ultimately, the conclusions from this study will be used in comparison with the existing structure later in the report to determine if changing DTC One's structural system to composite metal decking would be feasible.

Composite Metal Deck Floor System

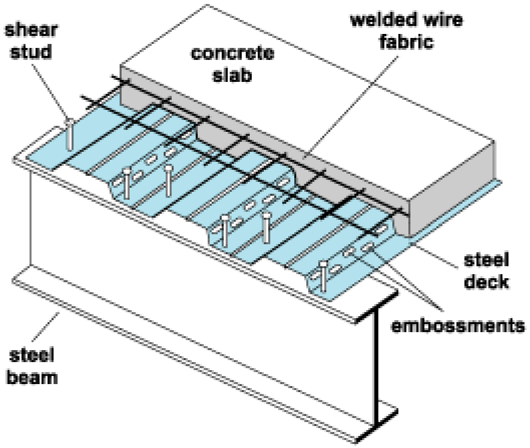


Figure 4

Codes and Standards

Necessary building codes were found in the 2006 International Building Code (IBC) and the American Society of Civil Engineers (ASCE) 7-05. Steel was designed referencing the 13th Edition of the American Institute of Steel Construction's (AISC) Manual for Steel Construction and AISC's Steel Design Guide 3: Serviceability Design Considerations for Steel Buildings (in the form of slides) while exploring camber. Corrugated steel deck sizes were determined using the Vulcraft Steel Roof and Floor Deck Product Catalog, which references the Steel Deck Institute's (SDI) standards and the American Iron and Steel Institute (AISI) specifications. The load combinations used during this redesign are as follows:

1. $1.4D$
2. $1.2D + 1.6L + .5L_r$
3. $1.2D + 1.6Lr + L$
4. $1.2D + 1.6W + L + .5L_r$
5. $1.2D + E + L + .2S$
6. $.9D + 1.6W$
7. $.9D + E$

Materials

Structural Steel

W-Shapes	ASTM A992
Shear Studs	ASTM A490
Base Plate	ASTM A572

Concrete

Slab on grade	$f'_c = 3{,}500$ psi
Slab on deck	$f'_c = 3{,}000$ psi
Walls and piers	$f'_c = 3{,}000$ psi
Caissons and grade beams	$f'_c = 3{,}000$ psi
Other	$f'_c = 3{,}000$ psi

Reinforcement

Welded Wire Fabric	ASTM A185
Reinforcing bars	ASTM A615, Grade 60

Design Procedure

Early on it was known that steel W-shapes would be able to span the long 40′ spans, so there was no need to reconsider the bay sizes or column grid. Live loads were determined from Chapter 4 of ASCE 7-05 and used to determine the metal deck needed to meet certain design criteria. Hand calculations were then performed to find initial sizes of the composite beams needed to support the deck. The computer software RAM Structural System was utilized to produce a typical floor plan and beam sizes designed by the program were compared to the hand calculations. To the right is a 3-D view of the RAM model used for this design. The beam sizes and number of shear studs from RAM closely resembled those found with hand calculations, which can be found in **Appendix A**. The beam depths, however, were too deep, so camber was investigated and used.

RAM Model

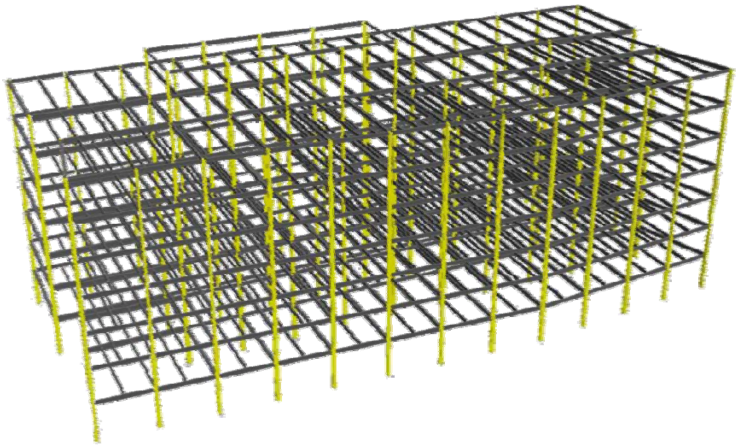


Figure 5

The lateral system and columns were the last of the structural components to be designed. As a result of a lack of space, the redesigned lateral system was to be kept as moment frames using an ordinary steel moment frame system. Some initial exterior beam sizes were calculated by hand and then checked with RAM. On the other hand, other beams and columns of the system were designed using RAM and then checked using hand calculations. Lateral design loads used for comparison were derived using methods from ASCE 7-05. Serviceability criteria and the foundation were checked last.

Design Loads

Gravity Loads

The gravity loads used in the redesign were taken from ASCE 7-05, product catalogs, existing building plans, and educated assumptions. Live loads were reduced as allowed by ASCE 7-05. A summary is provided in the following tables.

Dead Loads

Dead Loads (psf)	
Slab + Deck	75
Superimposed Ceiling	15
Precast Concrete Wall	93.75
Glass Ribbon Window	8
Curtain Wall	15
Metal Panels	3

Table 1

Live Loads

Live Loads (psf)	
Slab on Grade	100 psf
Mechanical Equipment	150 psf
Lobby and First Floor Corridors	100 psf
Office Space	80 psf
Corridors above 1st Floor	80 psf

Table 2

Roof Loads

Roof Loads (psf)	
Live	20
Mechanical	150 + 20
Snow	21

Table 3

Lateral Loads

Wind loads for Dulles Town Center Building One were determined using the Analytical Procedure found in Section 6.5 of ASCE 7-05. Wind loads were found to control strength design in the east-west direction. Variables used and calculations can be found in **Appendix B**. Below are the building's wind pressures in the east-west direction.

Wind Pressures in the East-West Direction

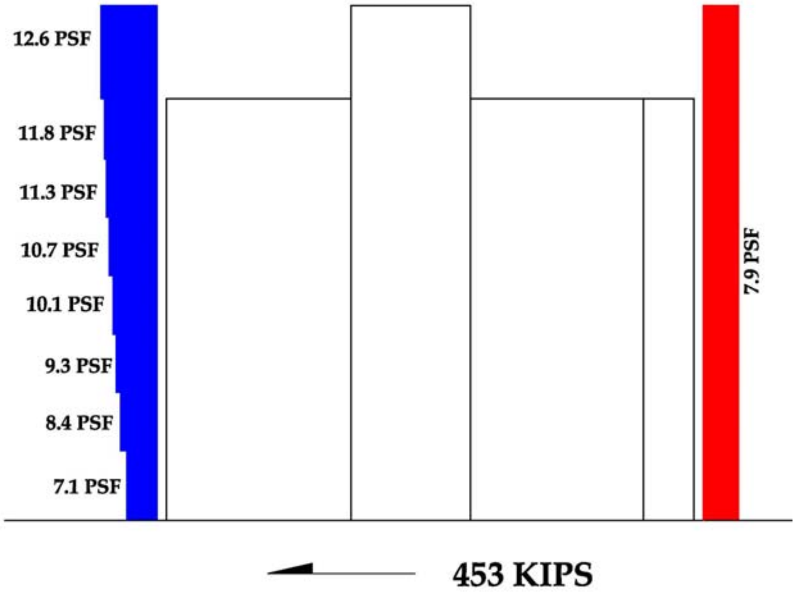


Figure 6

The seismic story forces and story shears, which control strength design in the north-south direction, can be found below in ***Figure 7***. Variables used and calculations can be found in **Appendix C**.

Seismic Loading – North-South Direction

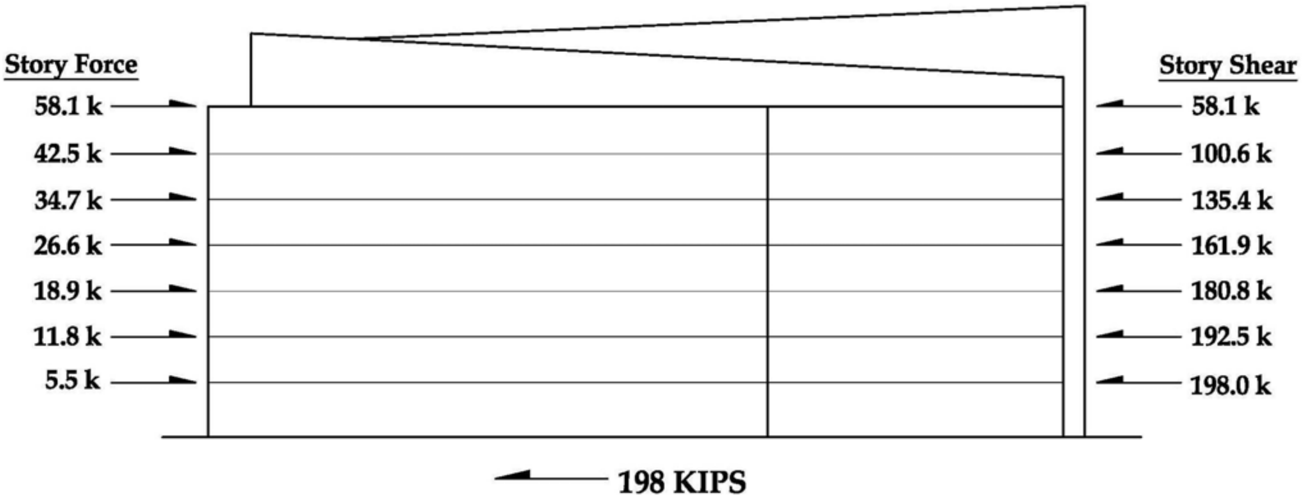


Figure 7

Design Process

Deck and Composite Beam

Research was conducted on metal decking to find if any advances in design strength have allowed spans to reach lengths of 20′ or more. The research was unsuccessful. Live loads were determined from Chapter 4 of ASCE 7-05 and, using 100 psf, a metal deck was chosen from the Vulcraft Product Catalog. The 2 hour fire rating ultimately controlled the slab thickness, whereas the gage of deck was determined by the deflection caused by live load. As a result, a 3″ 19 gage 3VLI deck was chosen with 7 ½″ of total slab depth and a recommended 6x6-W2.1xW2.1 welded wire fabric. This was also the case for the roof deck. The pages from the Vulcraft catalog can be found in **Appendix A**. Due to a limited maximum unshored clear span of 11′-6″, a mid-span infill beam was required within the 20′ span to support the perpendicularly laid deck.

Sizes for typical composite members and the required number of shear studs needed were then determined using Load and Resistance Factor Design (LRFD) methods and the AISC Steel Construction Manual. Members were designed using 1.2D and 1.6L and chosen based on moment capacities and the deflection limits listed below:

Live Load Deflection:	$\Delta_{LL} = L/360$
Total Load Deflection:	$\Delta_{TL} = L/240$
Pre-Composite Deflection:	$\Delta_{PC} = L/360$

RAM was then used to produce a typical floor plan. Floor plans with beam sizes can be found in **Appendix C**, along with column sizes. Beams incorporated in moment frames were designed by RAM and then compared to the hand calculations. The W24x55′s from RAM closely resembled those W21x62′s found with hand calculations. These sizes were unacceptable, however, due to their depths.

The solution was camber, which was investigated using AISC′s Steel Design Guide 3: Serviceability Design Considerations for Steel Buildings and RAM. Slides received from Dr. Louis Geschwindner gave an estimated cost of cambering a single member to be $30-$75. This was compared to the cost of the additional steel needed in the member for it to reach deflection requirements. From the slides, the cost of steel was approximately $0.40 per pound. Only the composite beams designed by RAM with and without camber were compared. At 40′ long, the additional 5 lbs. of the W24x55 would cost $5 more per beam, assuming each camber would cost the maximum $75 per beam. So, although the overall cost reduction due to camber was minimal, the 10″ depth decrease by using W16x50′s was well worth it. Other serviceability guidelines will have to be considered, as well, with the use of camber.

Below, ***Figure 8*** shows the typical composite beams in blue and their layout within the structural system. The size of the W shape is listed first, then the required number of shear studs in parentheses, and the camber applied to the beam last.

Typical Composite Beam Layout and Design

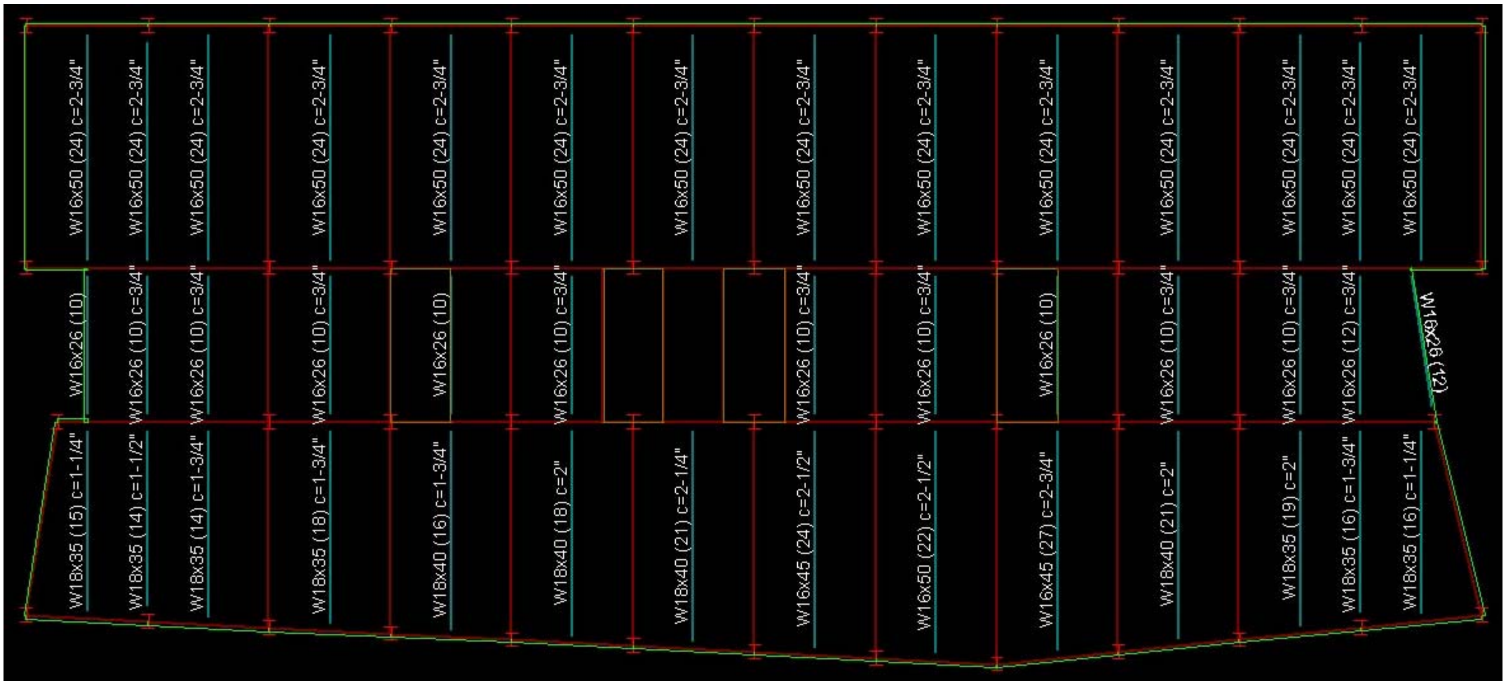


Figure 8

Lateral Framing

Multiple lateral systems were considered, such as braced frames, moment frames and shear walls. Unfortunately, due to the lack of space and the goal to maintain the current architectural design, there was no space within the floor plan to incorporate braced frames or shear walls. As a result, the redesigned lateral system would be ordinary steel moment frames with moment connections made up of flange welds and shear bolt connections. The lateral system was to be designed to withstand the lateral forces from wind in the east-west direction and seismic forces in the north-south direction. While doing this, the beams within the frames were limited W18 shapes in order to maintain the 9′ floor-to ceiling height. This was to maintain the architectural façade and evade any costs added if the building was to increase in height. Stairwell walls and elevator shafts were changed from 12″ thick cast-in-place concrete walls to 12″ fully grouted CMU block. They were assumed to only support gravity loads from the stairs and elevator equipment, which would be designed by others. Although these walls could offer some sort of lateral bracing, they were not included in this report's lateral frame analysis.

Using Equation 6-19, wind loads were computed and used to find direct story shears on each frame. Wind controlled strength design in the east-west direction with a base shear of 453 kips. This would ultimately govern beam and column design in the east-west direction.

Seismic loads were determined using the Equivalent Lateral Force Procedure found in Section 12.8 of ASCE 7-05. Base shear due to seismic loads was reduced significantly due to the large weight reduction. This base shear of 198 kips, however, still controlled strength design in the north-south direction. The building mass was symmetrical in the north-south direction, therefore there was no torsional shear added to the direct shear. A table of torsion constants can be found in **Appendix B**.

Based off the loads acquired through the ASCE 7-05 procedures, the number of frames needed and their layout was determined to be the same as the existing lateral system so as to keep lateral loads to each frame low in order to keep beam depths as shallow as possible. This allowed for building torsion to be checked. Due to the symmetrical layout of the frames, inherent torsion was kept very low in both directions and accidental torsion was assumed to be one. The small shear that was caused by torsion was then added to the direct shear to get a total shear on each frame. The diagram below and on the next page are moment frame layouts for both directions.

Frame Layout – East-West Direction

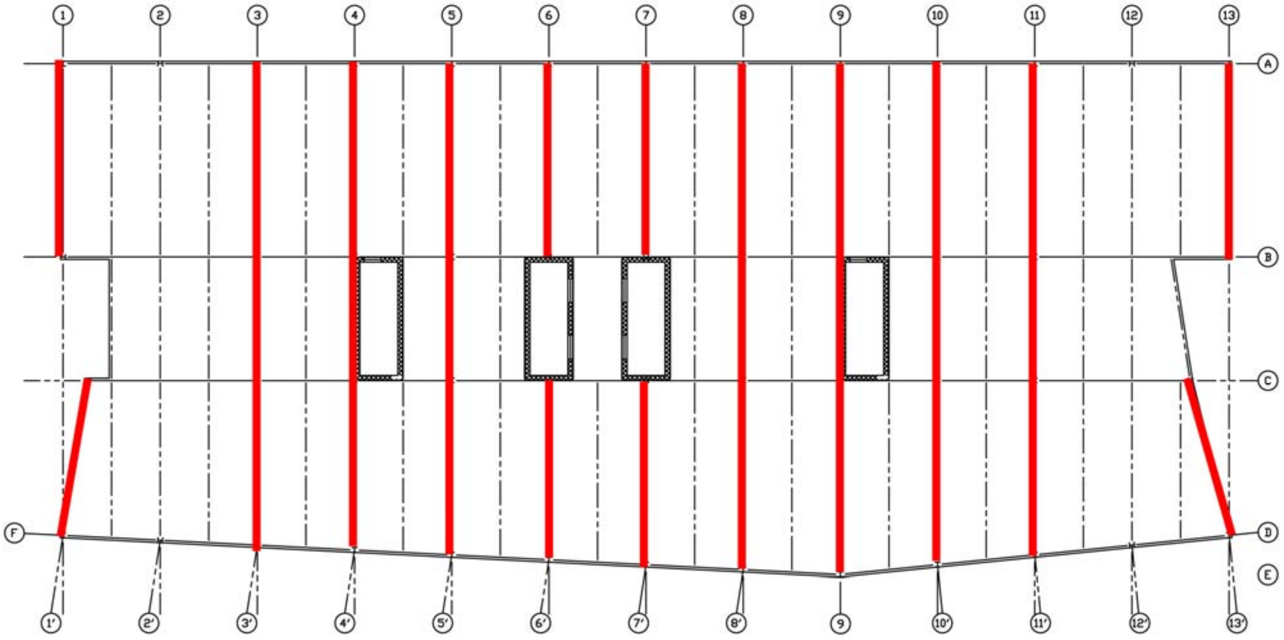


Figure 9

Frame Layout – North-South Direction

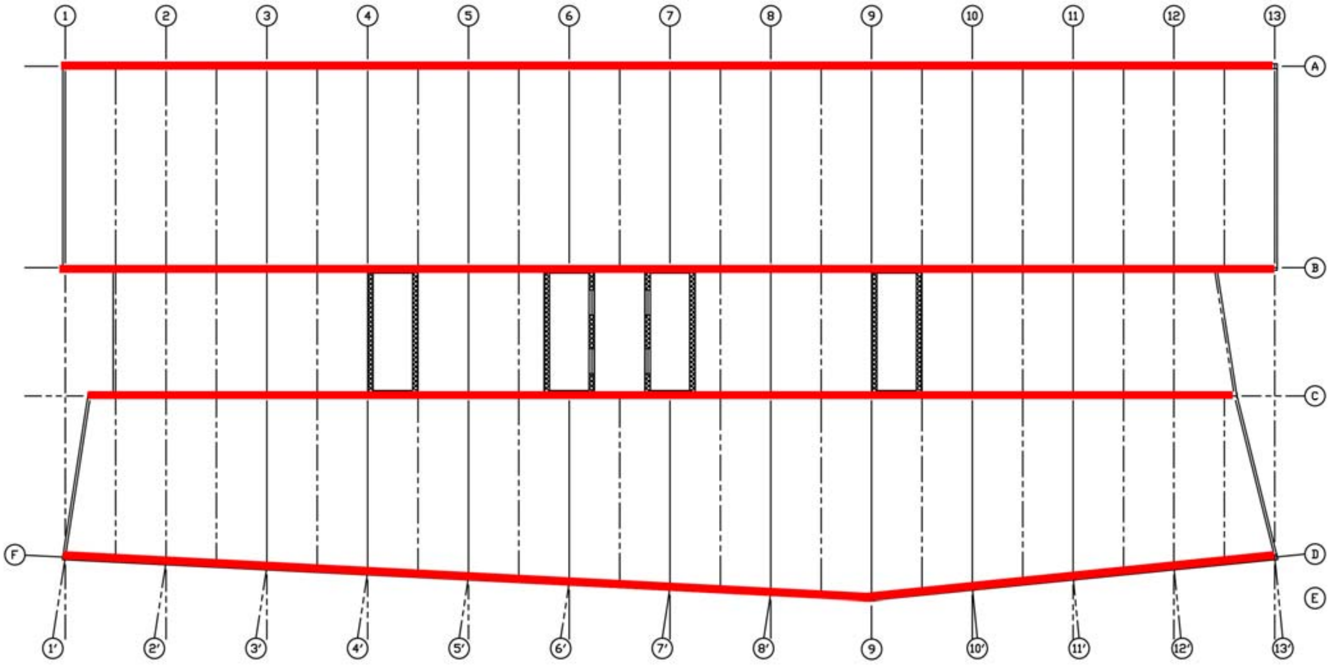


Figure 10

RAM was then used to design the moment frames. First, columns were placed with their strong axes in the east-west direction due to the geometry of the building and the large lateral forces caused by wind. RAM then designed the columns for gravity using AISC's 3rd Edition. Frame section views are located in **Appendix D** to show the sizes of all the columns. Next, the program analyzed lateral forces on the structure using code and load combinations taken from the 2006 IBC and ASCE 7-05.

In order for the steel redesign to be as efficient as possible, repetition of members was very important. After RAM completed its design, columns were then manually designed using the view/update command so that every two floors had the same W14 shape in any given column. This command also made sure the column was strong enough to withstand both axial and flexural forces acting on it. Beams were also manually designed following the design by RAM. This process was conducted so that the variance in frame member sizes in similar building areas was kept to a minimum. These manual designs were done in order to cut down on material costs for the structure and save time during the erection process. Floor plans of a typical framing plan, roof framing plan and the penthouse framing plan can be found in **Appendix D**.

The figure below shows a 3-D model of the moment frames in red and gravity members in blue.

RAM Model – Moment Frames and Gravity Members

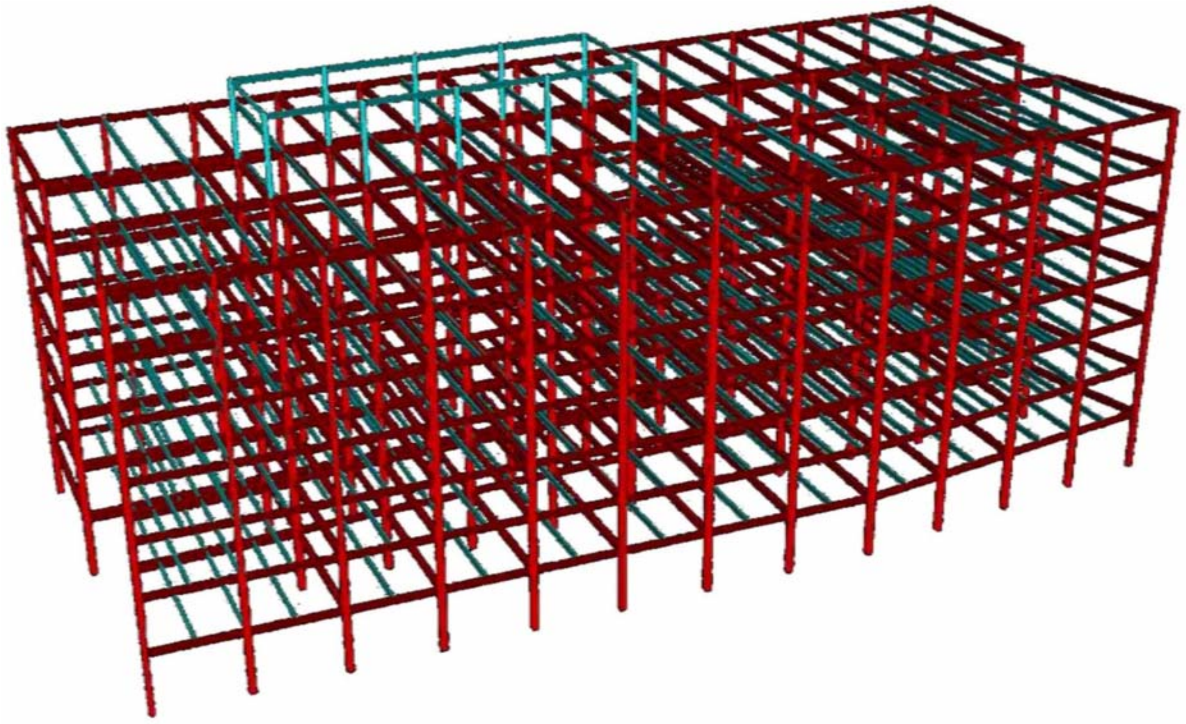


Figure 11

Strength checks on a column and girder were then performed. The portal frame analysis method was used to find moments and shears in the beams and columns incorporated in both east-west and north-south frames and gravity loads were brought down as normally done.

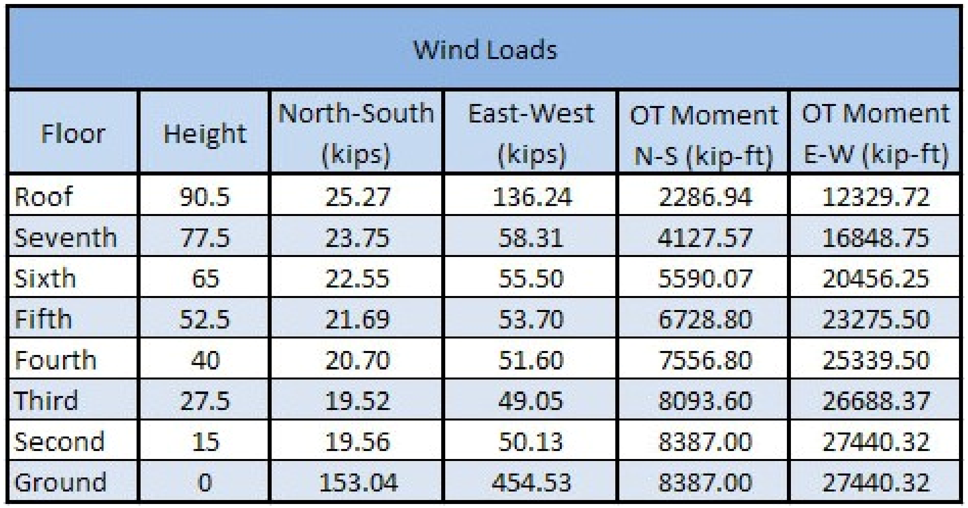
The girder strength check analyzed a 2nd floor exterior girder within the easterly north-south frame that supports the precast façade and was sized using LRFD methods and a deflection limit of L/500. The member was then compared to the member designed in RAM. The exterior girder calculated by hand used 1.2D + 1.0E + 1.0L due to seismic loading being in control of strength design. The result was a W16x50 shape. This was very close to the same girder designed by RAM, which was sized as a W16x57. Hand calculations for the 2nd story beam can be found in **Appendix C**.

The column strength check was performed on a 4th story interior column and used 1.2D+1.6Lr+L to determine the axial load. Live load was reduced wherever possible and in accordance with ASCE 7-05. Values obtained from Table 6-2 in the AISC Steel Construction Manual were then used to determine if the column was adequate. Hand calculations can be found in **Appendix B**.

Foundation

Due to the high wind forces and the reduced building weight, overturning moment had to be checked. Overturning moments caused by wind were determined in both directions, but only the east-west direction was checked for overturning. By inspection, seismic had no effect on the foundation from overturning moment. ***Table 4*** below shows the overturning moments due to wind.

Wind Loads and Overturning Moments



Wind Loads					
Floor	Height	North-South (kips)	East-West (kips)	OT Moment N-S (kip-ft)	OT Moment E-W (kip-ft)
Roof	90.5	25.27	136.24	2286.94	12329.72
Seventh	77.5	23.75	58.31	4127.57	16848.75
Sixth	65	22.55	55.50	5590.07	20456.25
Fifth	52.5	21.69	53.70	6728.80	23275.50
Fourth	40	20.70	51.60	7556.80	25339.50
Third	27.5	19.52	49.05	8093.60	26688.37
Second	15	19.56	50.13	8387.00	27440.32
Ground	0	153.04	454.53	8387.00	27440.32

Table 4

The building weights from the roof down to the basement were determined using live load reduction when possible. For uplift on the caissons, the load combination .9D + 1.6W was used. The resisting moment was significantly larger than that of the overturning moment. The compressions on an exterior and interior caisson were then checked using the cantilever method and load combination 1.2D + 1.6W + L + .5L_r. The total load on a single caisson on the governing exterior was 776 kips, which was less than the existing 796 kip load on the caisson. An interior caisson was also checked, resulting in a 995 kip axial load. The typical caisson carried a 1008 kip load, previously, therefore it worked for this load. These loads are too close to each other to even consider reducing caisson sizes. A section view of the caissons can be found to the right. A positive aspect of this analysis was that the existing intermediate caisson lines within the 40′ spans could be eliminated, reducing the foundation concrete by approximately 84 C.Y.

Foundation – Section View

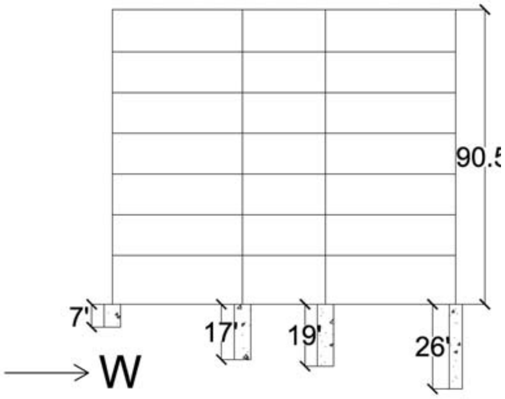


Figure 12

Serviceability

The final step was to determine if the steel building system met serviceability requirements and standards. The following are the two serviceability criteria considered for lateral drift and displacement.

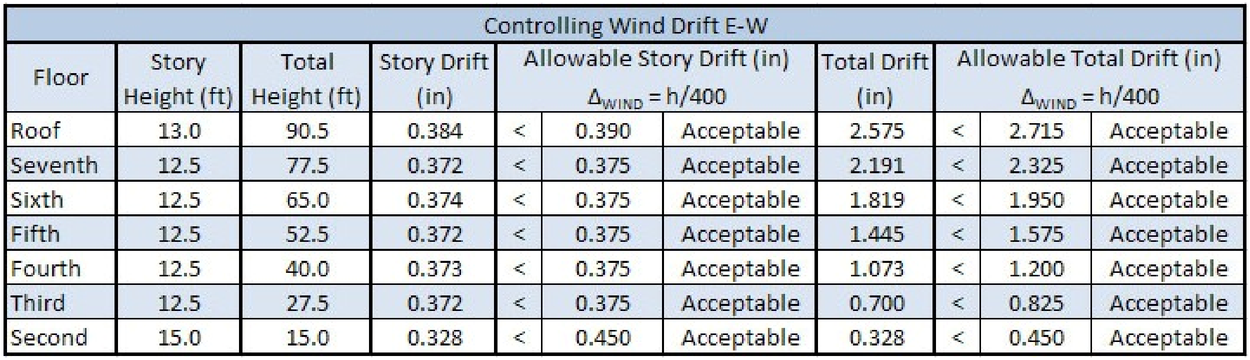
Wind: h/400
Seismic: $.020h_{sx}$

Drifts from both wind loads and seismic loads were obtained using RAM Frame. Wind drifts were used as calculated to determine if they met serviceability criteria, whereas seismic drifts were increased using the amplified level display found in Section 12.8 in ASCE 7-05, as seen below:

$$\delta x = \frac{C_d \; x \; \delta_{xe}}{I}$$

Serviceability did not control design in the north-south direction, but did control the design of the members within the east-west frames. It took many iterations of changing column and beams sizes to get the story displacements to meet serviceability requirements. Below is a table showing story displacements caused by wind in the east-west direction. Other drift tables can be found in **Appendix B** and **Appendix C**.

Wind Drift – East-West Direction



Controlling Wind Drift E-W									
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{WIND} = h/400			Total Drift (in)	Allowable Total Drift (in) Δ_{WIND} = h/400		
Roof	13.0	90.5	0.384	<	0.390	Acceptable	2.575	<	2.715	Acceptable
Seventh	12.5	77.5	0.372	<	0.375	Acceptable	2.191	<	2.325	Acceptable
Sixth	12.5	65.0	0.374	<	0.375	Acceptable	1.819	<	1.950	Acceptable
Fifth	12.5	52.5	0.372	<	0.375	Acceptable	1.445	<	1.575	Acceptable
Fourth	12.5	40.0	0.373	<	0.375	Acceptable	1.073	<	1.200	Acceptable
Third	12.5	27.5	0.372	<	0.375	Acceptable	0.700	<	0.825	Acceptable
Second	15.0	15.0	0.328	<	0.450	Acceptable	0.328	<	0.450	Acceptable

Table 5

Structural Depth Summary

Reasonable floor depth was accomplished using camber in the composite beams and multiple moment frames were used to lower lateral forces on beams and columns. The floor-to-ceiling height had to be dropped to 8’-9”, though, to allow for the extra beam depth. Seismic forces controlled strength design in the north-south direction and wind serviceability guidelines controlled design in the east-west direction. Designs found in RAM were compared to hand calculations and were found to be similar. It was also confirmed that the existing foundation was able to support the steel system’s loading while reducing necessary concrete by 84 C.Y.

Breadth Topics

Construction Management Breadth

One of the reasons for changing Dulles Town Center Building One from a concrete structure to a steel structure was to see if costs could be reduced due to a decrease in construction time and materials used. Within this section of the report, a detailed assessment of both systems will be made on the duration of construction as well as the material, labor, and equipment costs.

Site

As stated before, the building is located at one of the most visible spots in Northern Virginia, where Route 7 meets Route 28. The site's entrances are found to the east of the building along Atlantic Boulevard, which sees little to no traffic. One entrance is located at the northeast corner of the site and the other near the building entrance on the east side. The building, indicated in ***Figure 13,*** is located at the south end of this 12.37 acre site, therefore leaving the whole northern part of the site open for staging and lay down area. The general slope of the site is northeast to southwest, so runoff onto Route 28 must be considered during construction. The building sits at a comfortable distance away from Dulles Town Center Mall and its surrounding restaurants, stores and shopping centers, therefore noise from construction should not cause any problems.

Site Map

Figure 13

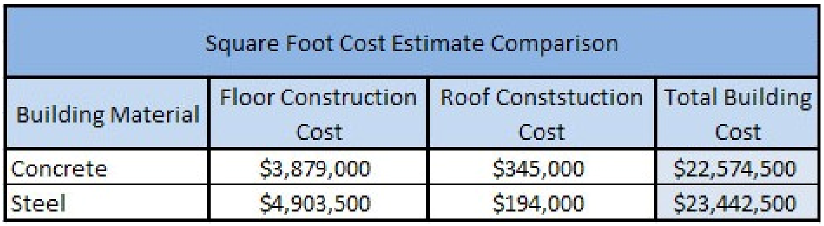
Construction Methods

The goal will be to make the construction process as fast and efficient as possible. Steel already will speed up erection time due to its ease of fabrication. Sizes were also inspected during the structural breadth and were changed manually to gain the benefits of member repetition. Member repetition cuts down on the number of different sections, which in turn cuts down on material costs, reduces field coordination time, and reduces the chance of a mistake during erection. Research was done on basic construction methods in the Northern Virginia area to determine how the concrete and steel structures would be erected. The result; both structural systems will be analyzed as being built using floor-to-floor construction. This involves constructing each building, in its entirety, floor by floor instead of in sections.

Costs

A detailed cost analysis was performed on both the existing concrete structure and the steel redesign. To get an idea of what the possible outcome would be, a square foot cost estimate was initially made for each building system using the 2009 R.S. Means Construction Cost Data online catalog. Parameters were set for location, city cost index, building area, building type, stories and building material. The program then calculated costs for the construction of both the substructure and superstructure, making many assumptions derived from a building model with very basic components. After analyzing each report, it was determined the total cost estimates had no significance in regards to this report. The cost of floor constructions, however, did seem to be a fair comparison of the different material costs. ***Table 6*** shows the floor and roof construction and final cost comparison between each structure. Semi-full reports can be found in **Appendix E** which show the materials taken into account for the floor and roof construction.

Square Foot Cost Estimate Comparison



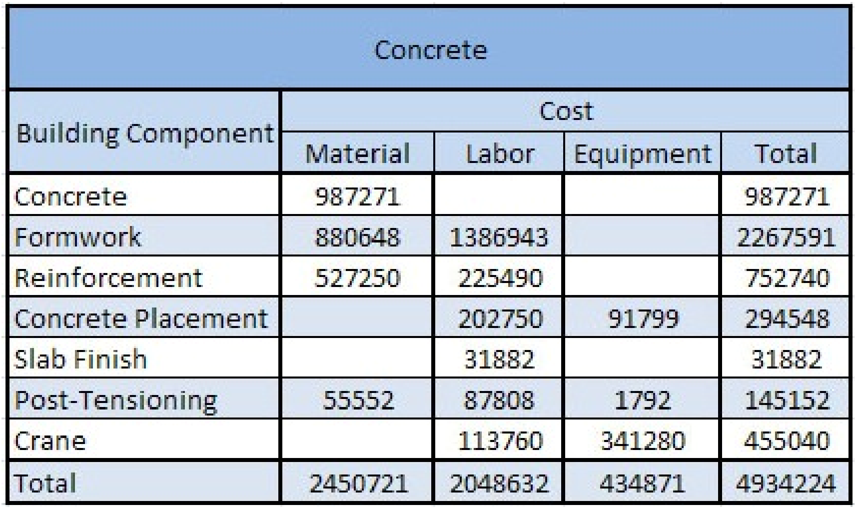
Square Foot Cost Estimate Comparison			
Building Material	Floor Construction Cost	Roof Conststuction Cost	Total Building Cost
Concrete	$3,879,000	$345,000	$22,574,500
Steel	$4,903,500	$194,000	$23,442,500

Table 6

To obtain a more detailed estimate, a more in-depth approach had to be taken. First the existing system had to be analyzed. Takeoffs for concrete and reinforcement had to be made in order to use R.S. Means to obtain prices for the building components. In regards to the concrete building, formwork, concrete, and reinforcement were considered when estimating column costs. The same were considered for floor slabs, except that floor finishing was required and therefore was also included in the pricing. When pricing the beams, formwork, concrete, reinforcement and post-tensioning were all taken into account. The steel redesign cost estimation consisted of concrete, slab finishing, welded-wire fabric, metal decking, W shapes, shear studs, and fireproofing. RAM was used for the takeoffs of weight for steel members and shear studs.

Once the unit-amount for each building component was determined, R.S. Means was used to price materials, labor costs, and equipment costs. Below you will find cost summaries of each system.

Cost Summary - Concrete



Concrete				
Building Component	Cost			
	Material	Labor	Equipment	Total
Concrete	987271			987271
Formwork	880648	1386943		2267591
Reinforcement	527250	225490		752740
Concrete Placement		202750	91799	294548
Slab Finish		31882		31882
Post-Tensioning	55552	87808	1792	145152
Crane		113760	341280	455040
Total	2450721	2048632	434871	4934224

Table 7

Cost Summary - Steel

Steel				
Building Component	Cost			
	Material	Labor	Equipment	Total
Steel Framing	3609375	10412	166320	3786107
Fireproofing	43719	47520	7440	98679
Metal Deck	677058	8180	81876	767114
Welded Wire Fabric	54188	52233		106421
Concrete	382456			382456
Concrete Placement		51310	18707	70017
Slab Finish		31811		31811
Crane		22800	68400	91200
Total	4766796	224266	342743	5333805

Table 8

Scheduling

Using the time acquired through the use of crew labor and unit-amounts, a schedule for each structural system was made. For the assumed construction of the existing DTC One, the building was divided into five zones. The amount of zones needed was due to the area limit of any single slab pour. ***Figure 14*** below shows the zones used.

Building Construction Zones – Existing Concrete Structure

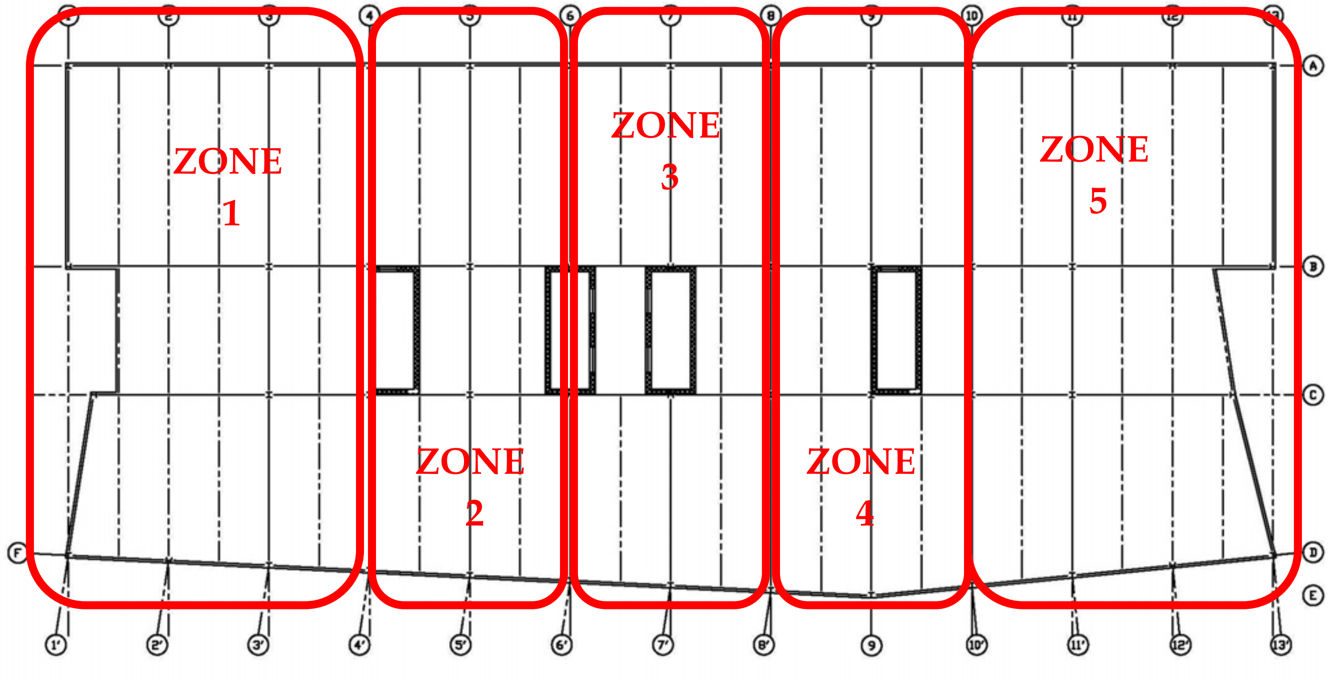


Figure 14

As stated before, this construction method, along with the method used for the steel structure, is a floor-by-floor method. That means the columns were formed, poured, and then cured before the slabs were formed, poured, and cured. To see the order of tasks completed, refer to **Appendix E** to see a full construction schedule. As a note, tasks shown in the schedule include curing time and therefore curing is not listed as its own task. Lead times are also not included because the only thing being analyzed is the construction time. The overall estimated construction duration was 474 days for the erection of the existing concrete system. This number, however seems a bit excessive and could be due to only using the number of crews provided in R.S. Means. If more crews were put on the job to hit time-consuming areas, like forming, the project would definitely move at a faster rate. The total cost would also go up as well.

Building construction zones were also need for the steel structure. Only three zones were needed for the erection of this system since the metal deck acts as the form and is stronger than plywood forms assembled on-site. Below you will find the three zones used for the steel building's estimated construction duration.

Building Construction Zones – Existing Concrete Structure

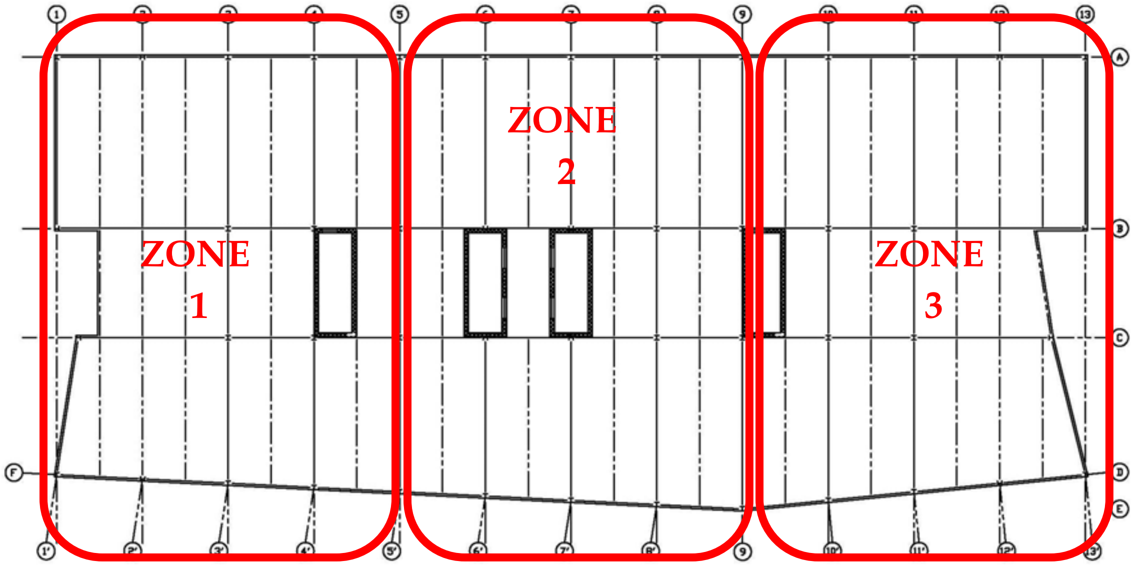


Figure 15

To see the order of tasks completed, refer to **Appendix E** to see a full construction schedule. As a note, tasks shown in the schedule include curing time and therefore curing is not listed as its own task. The overall estimated construction duration was 96 days.

Construction Management Summary

In using the more in-depth method of estimating, a more accurate comparison was made between the two building systems. The cost of the existing concrete structural system was estimated to be approximately $4.9 million. This turned out to be less than the composite steel system, which was estimated to be $5.3 million. The time it took the redesign to be erected, though, was more than a year faster. To the right is a summary of the results.

Cost and Time Comparison

Building System Comparison			
Post-Tensioned Beam One Way Slab System w/ Concrete Moment Frames		Composite Metal Deck System w/ Steel Moment Frames	
Costs		Costs	
Material	$2,450,721	Material	$4,766,796
Labor	$2,048,632	Labor	$224,266
Equipment	$434,871	Equpment	$342,743
TOTAL	$4,934,224	TOTAL	$5,333,805
Time		Time	
Days	474	Days	96

Table 9

Acoustical Breadth

With the introduction of a steel structural system to the current layout of Dulles Town Center Building One, the decrease in concrete thickness of the roof and penthouse floor may lead to noise problems in the prime office space of the seventh floor. This analysis will determine the sound pressure levels of the mechanical equipment located above the 7th floor and then calculate the sound transmitted, if any, into the office space below. It will then be determined if additional acoustical materials are necessary to keep the sound level within the preferred range of noise within the office space. Since Dulles Town Center Building One was originally designed as a spec building, this analysis was performed considering no finishes or ceiling systems. If, as a result, sound penetration does occur within the office space, a ceiling system could be designed to absorb it in addition to any noise emitted from the building systems running through it.

As seen in ***Figure 16*** the two areas of focus in this analysis are the spaces below the mechanical room and rooftop units. The area below the cooling tower can be neglected because it is known that it is a storage closet/small mechanical area in which noise penetration is acceptable.

Roof Floor Plan – Acoustically Analyzed Areas

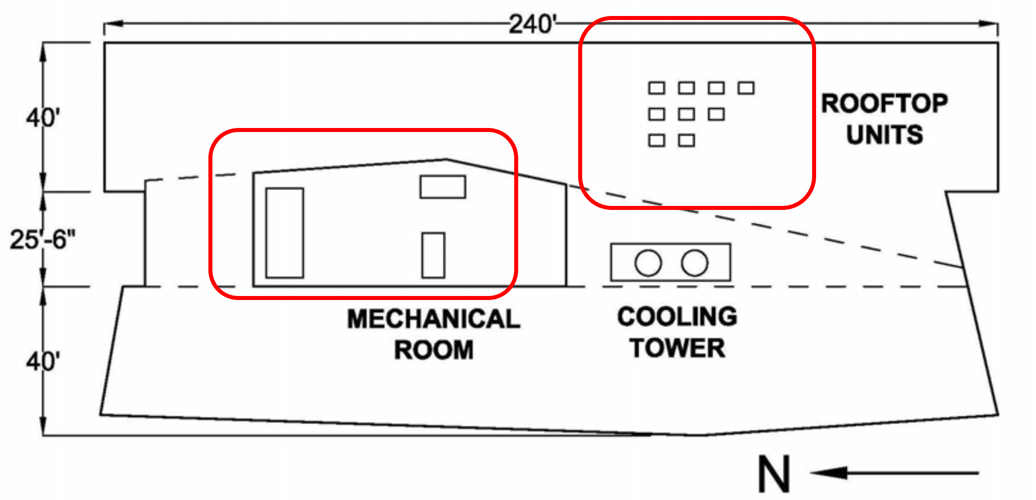


Figure 16

Sound pressure levels, background noise levels, absorption coefficients, and sound transmission coefficients were all found using "Architectural Acoustics" by M. David Egan and "Noise Control in Buildings" by Cyril M. Harris. These books were also referenced to analyze and design the floor systems separating the mechanical equipment and spaces of interest.

The first area analyzed for sound penetration was the area below the mechanical room. The current floor, as seen in ***Figure 17***, consists of a 6″ floating floor slab which is completely separated from the 9″ structural slab by a 2″ resilient underlayment of fiberglass insulation. This floor construction has high impact isolation effectiveness, so sound transmission, in this case, is minimal to none. The proposed floor system, as seen in ***Figure 18***, shows the metal deck and the 3″ reduction of thickness, acoustically speaking, in the structural slab due to the flutes. The ceiling is not shown because it is neglected during the analysis unless sound penetration is present.

Mechanical Room – Current Floor System

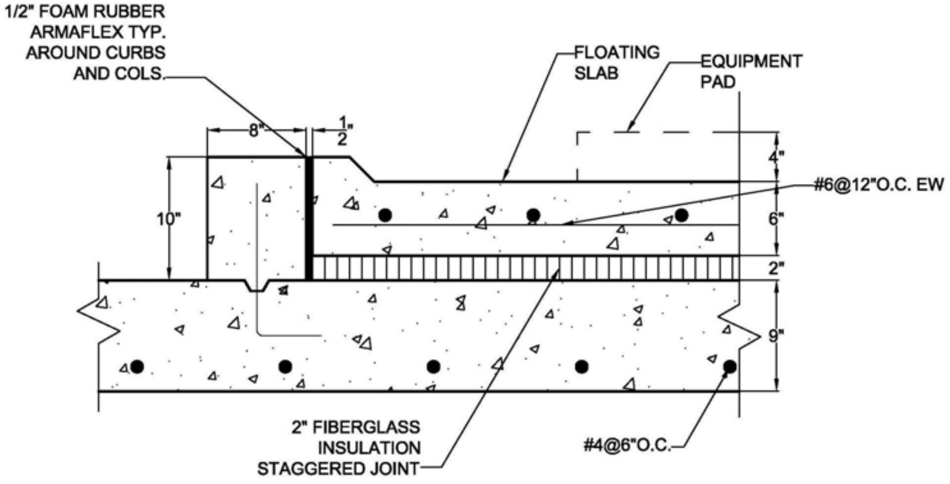


Figure 17

Mechanical Room – Proposed Floor System

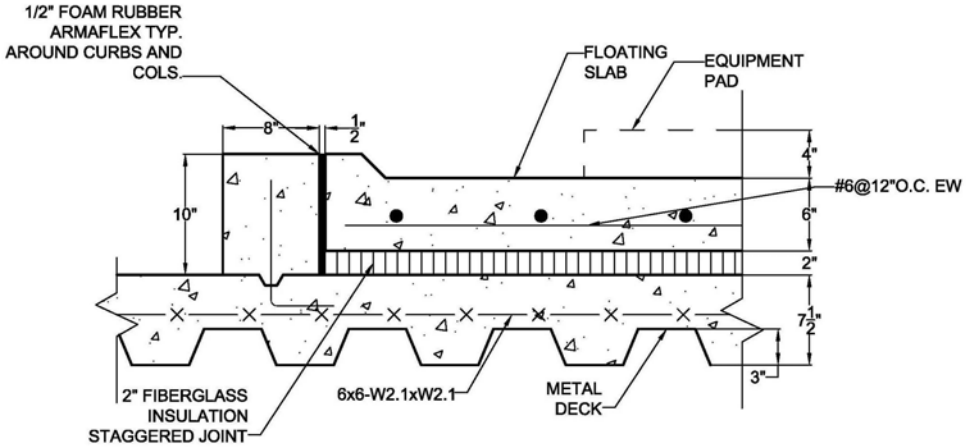
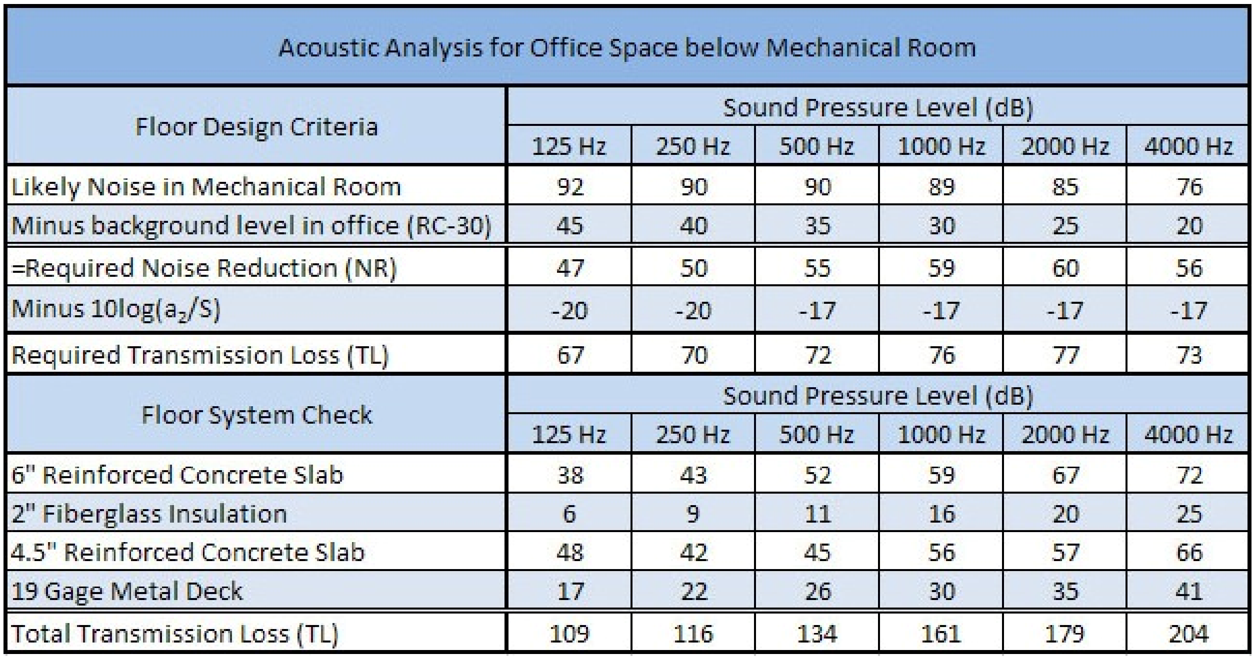


Figure 18

The following table shows that the office space beneath the mechanical room has no sound penetration from the equipment with the new floor system. The 10 ½" of total concrete thickness alone accounts for all the necessary transmission loss, therefore leaving the ceiling insulation and ceiling tile chosen by the tenant to require only enough absorbing capability to dampen sound from the building systems running through the ceiling. Partial calculations can be found in **Appendix F**.

Acoustic Analysis – Sound from Mechanical Room



Acoustic Analysis for Office Space below Mechanical Room						
Floor Design Criteria	Sound Pressure Level (dB)					
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Likely Noise in Mechanical Room	92	90	90	89	85	76
Minus background level in office (RC-30)	45	40	35	30	25	20
=Required Noise Reduction (NR)	47	50	55	59	60	56
Minus 10log(a_2/S)	-20	-20	-17	-17	-17	-17
Required Transmission Loss (TL)	67	70	72	76	77	73
Floor System Check	Sound Pressure Level (dB)					
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
6" Reinforced Concrete Slab	38	43	52	59	67	72
2" Fiberglass Insulation	6	9	11	16	20	25
4.5" Reinforced Concrete Slab	48	42	45	56	57	66
19 Gage Metal Deck	17	22	26	30	35	41
Total Transmission Loss (TL)	109	116	134	161	179	204

Table 10

The second area analyzed for sound penetration was the area below the rooftop units. The current roof, as seen in ***Figure 19,*** consists of a 7" structural slab. Stone ballast and rigid insulation also surround the equipment pad. Unlike the floor system analyzed previously, this roof construction only has fair impact isolation effectiveness, which means it is more likely to allow sound penetration. The proposed roof system, as seen in ***Figure 20,*** shows the metal deck and the 3" reduction of thickness, acoustically speaking, in the structural slab due to the flutes. The ceiling, again, is not shown because it is neglected during the analysis unless sound penetration is present.

At Rooftop Units - Current Roof System

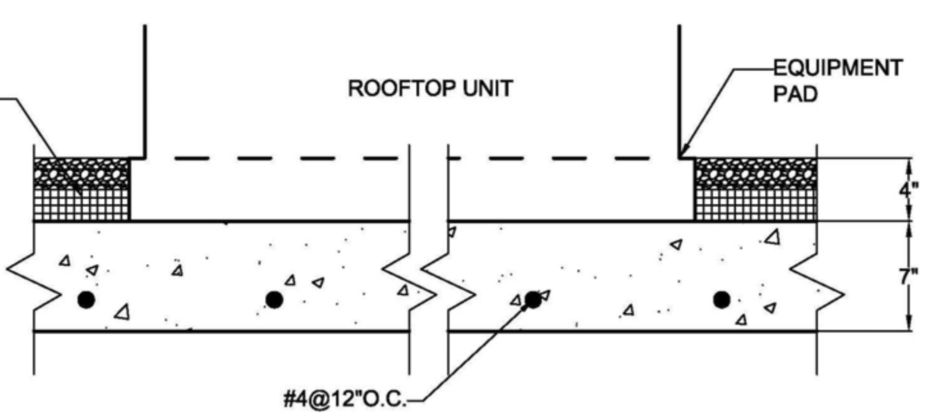


Figure 19

At Rooftop Units – Proposed Roof System

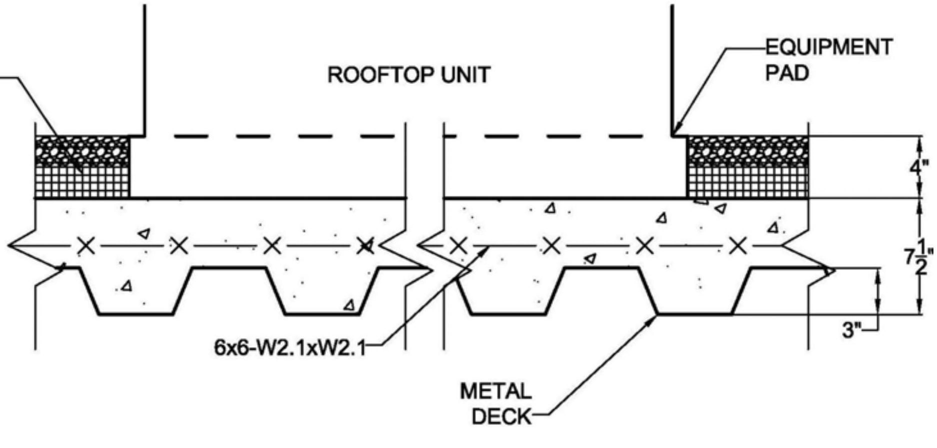
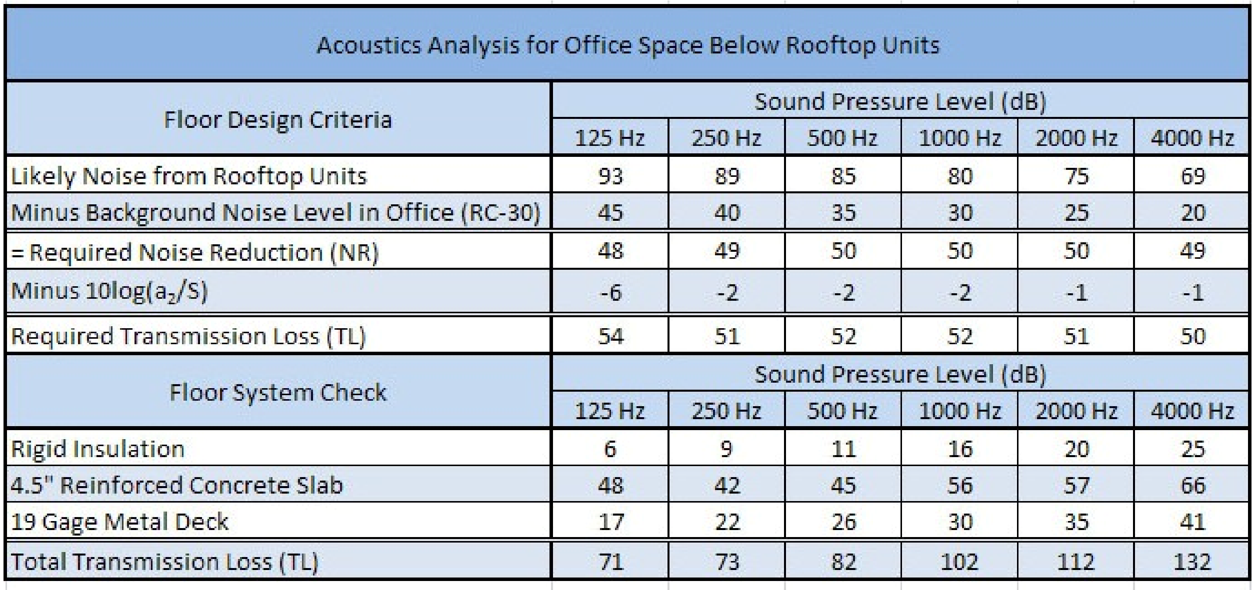


Figure 20

As ***Table 11*** shows, the office space beneath the rooftop units experiences no sound penetration from the equipment with the new roof system. The 4 ½" of concrete along with the metal deck are more than enough to absorb the sound from the mechanical units they support. The ceiling insulation and ceiling tile chosen by the tenant, therefore, are only required to absorb the sound produced by the building systems running through the ceiling. Partial calculations can be found in **Appendix F**.

Acoustic Analysis – Sound from Rooftop Units



Acoustics Analysis for Office Space Below Rooftop Units						
Floor Design Criteria	Sound Pressure Level (dB)					
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Likely Noise from Rooftop Units	93	89	85	80	75	69
Minus Background Noise Level in Office (RC-30)	45	40	35	30	25	20
= Required Noise Reduction (NR)	48	49	50	50	50	49
Minus 10log(a_2/S)	-6	-2	-2	-2	-1	-1
Required Transmission Loss (TL)	54	51	52	52	51	50
Floor System Check	Sound Pressure Level (dB)					
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Rigid Insulation	6	9	11	16	20	25
4.5" Reinforced Concrete Slab	48	42	45	56	57	66
19 Gage Metal Deck	17	22	26	30	35	41
Total Transmission Loss (TL)	71	73	82	102	112	132

Table 11

Acoustics Summary

After a thorough acoustics study of the roof and penthouse floor it has been concluded that there is no sound penetration in either area of interest. The machinery in the penthouse emits a maximum sound pressure of 92 decibels, or dB, which could penetrate the 7th floor office space. Background noise assumed to be in the office space is 45 dB, which means a required noise reduction of 48 is needed to keep sound from the penthouse from entering the 7th floor. The 10 ½"" of concrete alone from the floor slab and floating slab are enough to provide a transmission loss of 78 db, keeping mechanical noise out. The roof area that carries the rooftop units must keep 93 db of sound pressure from entering the office space. The 4 ½"concrete slab and metal decking provide a 71 db transmission loss, which is more than enough to buffer out the rooftop sound. So to reiterate, the sound caused by mechanical equipment on the roof and in the penthouse does not penetrate the 7th floor office spaces anywhere, which means there would be no extra costs for extra acoustical material.

Conclusions

This thesis study was conducted to investigate the feasibility of redesigning the current structural system of Dulles Town Center Building One out of steel. The main purpose was to see if construction time and building costs could be reduced in order to deliver a faster and cheaper structural system to the owner.

During the design it was imperative to keep the architecture as close to the original design as possible in order to avoid additional costs accrued due to extra façade or more permanent walls or structural members. Therefore, the beams spanning the open office space had to be able to reach 40′ and remain at or under and 18″ depth. This was necessary to maintain the 9′ floor-to-ceiling heights. When designed using standard code, however, the depths proceeded past the 18-inch goal so other measures had to be taken. Camber was researched and used on composite members, saving approximately $5 on each beam and 10″ on floor depth. Unfortunately, during the design of the moment frames in the east-west direction, serviceability guidelines forced the members to be as large as W18x130 making the depth of the beams total out at 19.3″. The ceiling had to be put at 8.75′ in order to preserve the current building height.

Even though the change in ceiling height is a small disadvantage, the use of steel provided many advantages as well. The structure's total weight was decreased by almost half and therefore reduced the seismic load on the building while also saving 84 C.Y. worth of concrete by getting rid of the intermediate caisson lines. Smaller columns were used in the redesign in the form of W14's. Shapes vary from W14x61 to W14x342 and are smaller than the existing typical 24″x24″ reinforced concrete columns. The redesign also shortened the construction duration through ease of construction and floor construction repetition.

Unfortunately, there are more disadvantages. The larger depth of the steel beams causes the total floor depth to increase from 42″ to approximately 45″, making the typical floor- to-ceiling height 8′-8″. In regards to construction, longer lead times could affect construction start dates and the prefabrication of steel members leads to less flexibility in design change later in the project. The cost per moment connection is also fairly expensive. The existing concrete system, in comparison to the steel system, was approximately $500,000 less, but takes more than double the amount of time to erect. This ultimately depends on crews used. The fluidity of design due to the repetition of floor construction is a big advantage in the field and limits mistakes.

In conclusion, after considering all the benefits and drawbacks of both structural systems, the result; it could be either. The project duration of the concrete seems to be a bit long, so a more in-depth analysis, along with a comparison on the amount of money saved on construction compared to the amount of money made from opening the building early, would be needed to make a more solidified decision on which building system is optimal.

Bibliography

Architectural Acoustics. Egan, M. David. Boston: McGraw-Hill, 1988.

ASCE Standard 7-05 Minimum Design Loads for Buildings and Other Structures. Virginia: ASCE/AEI, 2006.

Manual of Steel Construction, 13th Edition. American Institute of Steel Construction. Chicago: AISC, 2005.

Noise Control in Buildings. Harris, Ph.D., Cyril M. New York: McGraw-Hill, 1994.

R.S. Means Building Construction Cost Data 2009, 67th Edition. Waier, Philip R., R.S. Means, 2008.

Steel Design Guide 3: Serviceability Design Considerations for Steel Buildings. American Institute of Steel Construction. Slides from Dr. Louis F. Geschwinder.

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

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  - Prof. Robert Holland, Associate Professor of Architecture and Architectural Engineering,
  - Dr. Linda Hanagan, Associate Professor of Architectural Engineering,

- Other Assistance:
  - Dr. Louis F. Geschwindner, Professor Emeritus of Architectural Engineering,

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  - Michael Sladki at Cates Engineering, Ltd.,
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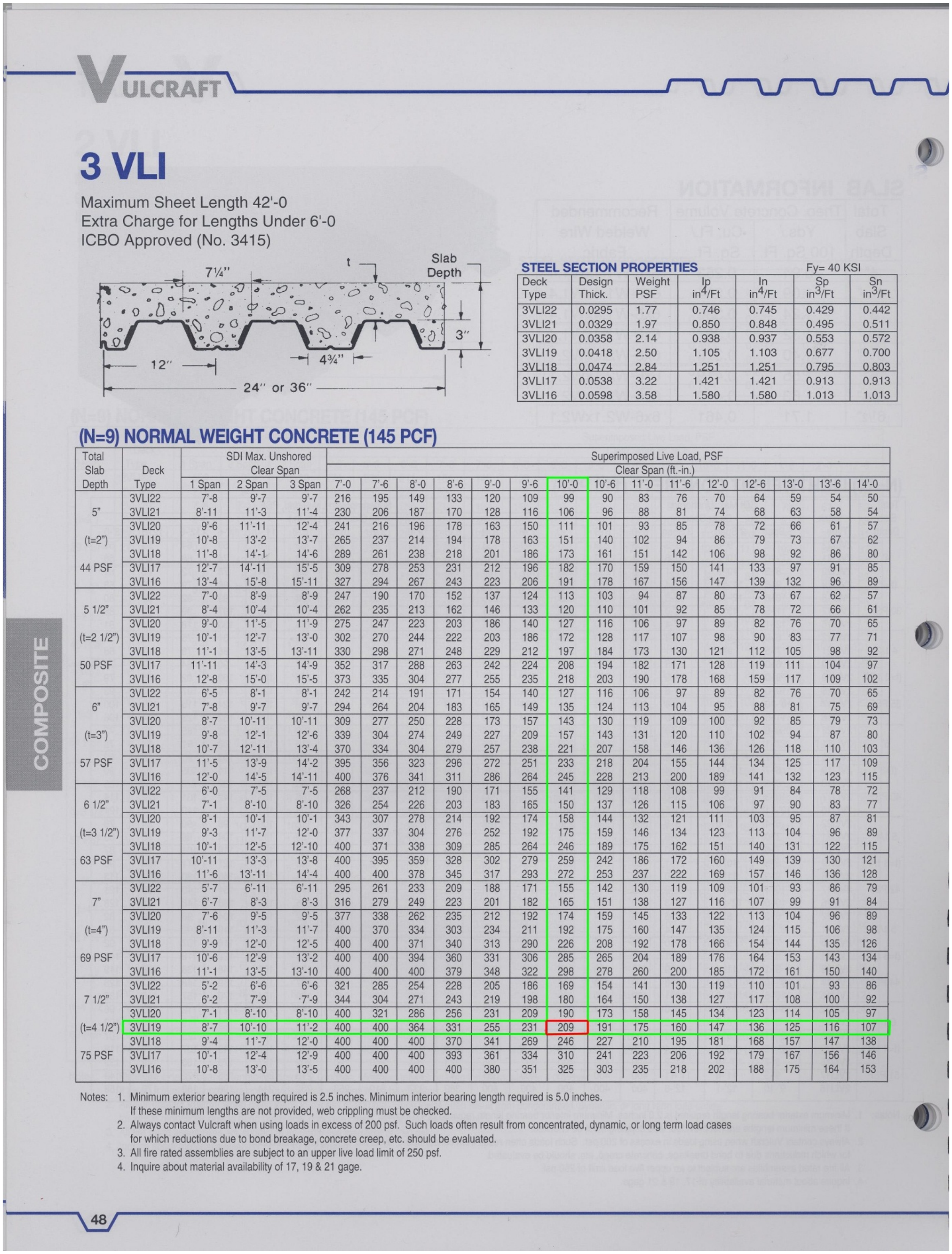
Appendix A

Floor System Design



3 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)

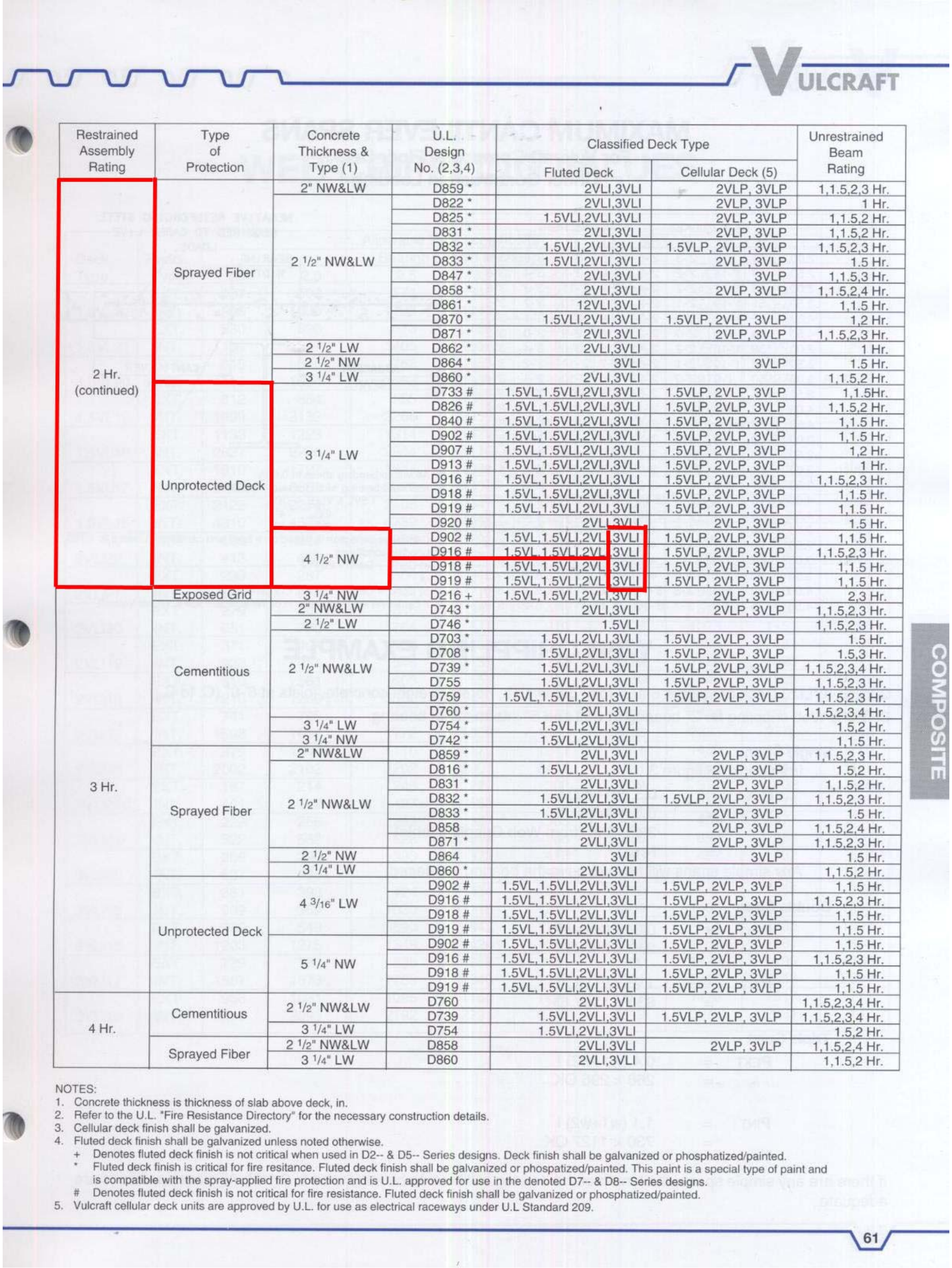
STEEL SECTION PROPERTIES Fy= 40 KSI

Deck Type	Design Thick.	Weight PSF	Ip in^{4}/Ft	In in^{4}/Ft	Sp in^{3}/Ft	Sn in^{3}/Ft
3VLI22	0.0295	1.77	0.746	0.745	0.429	0.442
3VLI21	0.0329	1.97	0.850	0.848	0.495	0.511
3VLI20	0.0358	2.14	0.938	0.937	0.553	0.572
3VLI19	0.0418	2.50	1.105	1.103	0.677	0.700
3VLI18	0.0474	2.84	1.251	1.251	0.795	0.803
3VLI17	0.0538	3.22	1.421	1.421	0.913	0.913
3VLI16	0.0598	3.58	1.580	1.580	1.013	1.013

(N=9) NORMAL WEIGHT CONCRETE (145 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0
	3VLI22	7'-8	9'-7	9'-7	216	195	149	133	120	109	99	90	83	76	70	64	59	54	50
5"	3VLI21	8'-11	11'-3	11'-4	230	206	187	170	128	116	106	96	88	81	74	68	63	58	54
	3VLI20	9'-6	11'-11	12'-4	241	216	196	178	163	150	111	101	93	85	78	72	66	61	57
(t=2")	3VLI19	10'-8	13'-2	13'-7	265	237	214	194	178	163	151	140	102	94	86	79	73	67	62
	3VLI18	11'-8	14'-1	14'-6	289	261	238	218	201	186	173	161	151	142	106	98	92	86	80
44 PSF	3VLI17	12'-7	14'-11	15'-5	309	278	253	231	212	196	182	170	159	150	141	133	97	91	85
	3VLI16	13'-4	15'-8	15'-11	327	294	267	243	223	206	191	178	167	156	147	139	132	96	89
	3VLI22	7'-0	8'-9	8'-9	247	190	170	152	137	124	113	103	94	87	80	73	67	62	57
5 1/2"	3VLI21	8'-4	10'-4	10'-4	262	235	213	162	146	133	120	110	101	92	85	78	72	66	61
	3VLI20	9'-0	11'-5	11'-9	275	247	223	203	186	140	127	116	106	97	89	82	76	70	65
(t=2 1/2")	3VLI19	10'-1	12'-7	13'-0	302	270	244	222	203	186	172	128	117	107	98	90	83	77	71
	3VLI18	11'-1	13'-5	13'-11	330	298	271	248	229	212	197	184	173	130	121	112	105	98	92
50 PSF	3VLI17	11'-11	14'-3	14'-9	352	317	288	263	242	224	208	194	182	171	128	119	111	104	97
	3VLI16	12'-8	15'-0	15'-5	373	335	304	277	255	235	218	203	190	178	168	159	117	109	102
	3VLI22	6'-5	8'-1	8'-1	242	214	191	171	154	140	127	116	106	97	89	82	76	70	65
6"	3VLI21	7'-8	9'-7	9'-7	294	264	204	183	165	149	135	124	113	104	95	88	81	75	69
	3VLI20	8'-7	10'-11	10'-11	309	277	250	228	173	157	143	130	119	109	100	92	85	79	73
(t=3")	3VLI19	9'-8	12'-1	12'-6	339	304	274	249	227	209	157	143	131	120	110	102	94	87	80
	3VLI18	10'-7	12'-11	13'-4	370	334	304	279	257	238	221	207	158	146	136	126	118	110	103
57 PSF	3VLI17	11'-5	13'-9	14'-2	395	356	323	296	272	251	233	218	204	155	144	134	125	117	109
	3VLI16	12'-0	14'-5	14'-11	400	376	341	311	286	264	245	228	213	200	189	141	132	123	115
	3VLI22	6'-0	7'-5	7'-5	268	237	212	190	171	155	141	129	118	108	99	91	84	78	72
6 1/2"	3VLI21	7'-1	8'-10	8'-10	326	254	226	203	183	165	150	137	126	115	106	97	90	83	77
	3VLI20	8'-1	10'-1	10'-1	343	307	278	214	192	174	158	144	132	121	111	103	95	87	81
(t=3 1/2")	3VLI19	9'-3	11'-7	12'-0	377	337	304	276	252	192	175	159	146	134	123	113	104	96	89
	3VLI18	10'-1	12'-5	12'-10	400	371	338	309	285	264	246	189	175	162	151	140	131	122	115
63 PSF	3VLI17	10'-11	13'-3	13'-8	400	395	359	328	302	279	259	242	186	172	160	149	139	130	121
	3VLI16	11'-6	13'-11	14'-4	400	400	378	345	317	293	272	253	237	222	169	157	146	136	128
	3VLI22	5'-7	6'-11	6'-11	295	261	233	209	188	171	155	142	130	119	109	101	93	86	79
7"	3VLI21	6'-7	8'-3	8'-3	316	279	249	223	201	182	165	151	138	127	116	107	99	91	84
	3VLI20	7'-6	9'-5	9'-5	377	338	262	235	212	192	174	159	145	133	122	113	104	96	89
(t=4")	3VLI19	8'-11	11'-3	11'-7	400	370	334	303	234	211	192	175	160	147	135	124	115	106	98
	3VLI18	9'-9	12'-0	12'-5	400	400	371	340	313	290	226	208	192	178	166	154	144	135	126
69 PSF	3VLI17	10'-6	12'-9	13'-2	400	400	394	360	331	306	285	265	204	189	176	164	153	143	134
	3VLI16	11'-1	13'-5	13'-10	400	400	400	379	348	322	298	278	260	200	185	172	161	150	140
	3VLI22	5'-2	6'-6	6'-6	321	285	254	228	205	186	169	154	141	130	119	110	101	93	86
7 1/2"	3VLI21	6'-2	7'-9	7'-9	344	304	271	243	219	198	180	164	150	138	127	117	108	100	92
	3VLI20	7'-1	8'-10	8'-10	400	321	286	256	231	209	190	173	158	145	134	123	114	105	97
(t=4 1/2")	3VLI19	8'-7	10'-10	11'-2	400	400	364	331	255	231	209	191	175	160	147	136	125	116	107
	3VLI18	9'-4	11'-7	12'-0	400	400	400	370	341	269	246	227	210	195	181	168	157	147	138
75 PSF	3VLI17	10'-1	12'-4	12'-9	400	400	400	393	361	334	310	241	223	206	192	179	167	156	146
	3VLI16	10'-8	13'-0	13'-5	400	400	400	400	380	351	325	303	235	218	202	188	175	164	153

Notes:
1. Minimum exterior bearing length required is 2.5 inches. Minimum interior bearing length required is 5.0 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.
4. Inquire about material availability of 17, 19 & 21 gage.



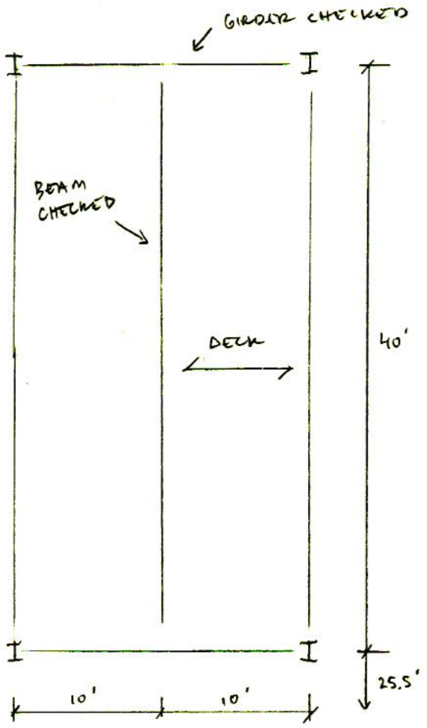
Restrained Assembly Rating	Type of Protection	Concrete Thickness & Type (1)	U.L. Design No. (2,3,4)	Classified Deck Type		Unrestrained Beam Rating
				Fluted Deck	Cellular Deck (5)	
2 Hr. (continued)	Sprayed Fiber	2" NW&LW	D859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	D822 *	2VLI,3VLI	2VLP, 3VLP	1 Hr.
			D825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			D831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			D832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			D847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			D858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			D861 *	12VLI,3VLI		1,1.5 Hr.
			D870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			D871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	D862 *	2VLI,3VLI		1 Hr.
		2 1/2" NW	D864 *	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	D860 *	2VLI,3VLI		1,1.5,2 Hr.
	Unprotected Deck	3 1/4" LW	D733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			D826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.
			D840 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D907 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			D913 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1 Hr.
			D916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D920 #	2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
		4 1/2" NW	D902 #	1.5VL,1.5VLI,2VL 3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D916 #	1.5VL,1.5VLI,2VL 3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D918 #	1.5VL,1.5VLI,2VL 3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D919 #	1.5VL,1.5VLI,2VL 3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
	Exposed Grid	3 1/4" NW	D216 +	1.5VL,1.5VLI,2VLI,3VLI	2VLP, 3VLP	2,3 Hr.
3 Hr.	Cementitious	2" NW&LW	D743 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	D746 *	1.5VLI		1,1.5,2,3 Hr.
		2 1/2" NW&LW	D703 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1.5 Hr.
			D708 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1.5,3 Hr.
			D739 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3,4 Hr.
			D755	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D759	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D760 *	2VLI,3VLI		1,1.5,2,3,4 Hr.
		3 1/4" LW	D754 *	1.5VLI,2VLI,3VLI		1.5,2 Hr.
		3 1/4" NW	D742 *	1.5VLI,2VLI,3VLI		1,1.5 Hr.
	Sprayed Fiber	2" NW&LW	D859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	D816 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5,2 Hr.
			D831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			D832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			D858	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			D871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW	D864	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	D860 *	2VLI,3VLI		1,1.5,2 Hr.
	Unprotected Deck	4 3/16" LW	D902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
		5 1/4" NW	D902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			D918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			D919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
4 Hr.	Cementitious	2 1/2" NW&LW	D760	2VLI,3VLI		1,1.5,2,3,4 Hr.
			D739	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3,4 Hr.
		3 1/4" LW	D754	1.5VLI,2VLI,3VLI		1.5,2 Hr.
	Sprayed Fiber	2 1/2" NW&LW	D858	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
		3 1/4" LW	D860	2VLI,3VLI		1,1.5,2 Hr.

NOTES:
1. Concrete thickness is thickness of slab above deck, in.
2. Refer to the U.L. "Fire Resistance Directory" for the necessary construction details.
3. Cellular deck finish shall be galvanized.
4. Fluted deck finish shall be galvanized unless noted otherwise.
    - \+ Denotes fluted deck finish is not critical when used in D2-- & D5-- Series designs. Deck finish shall be galvanized or phosphatized/painted.
    - * Fluted deck finish is critical for fire resitance. Fluted deck finish shall be galvanized or phospatized/painted. This paint is a special type of paint and is compatible with the spray-applied fire protection and is U.L. approved for use in the denoted D7-- & D8-- Series designs.
    - \# Denotes fluted deck finish is not critical for fire resistance. Fluted deck finish shall be galvanized or phosphatized/painted.
5. Vulcraft cellular deck units are approved by U.L. for use as electrical raceways under U.L Standard 209.

STRENGTH CHECK
TYPICAL COMPOSITE BEAM

1

TYPICAL OFFICE FLOOR



NORMAL WEIGHT CONCRETE
145 pcf , $f'_c = 4000$

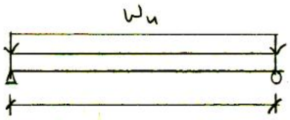
3" VULCRAFT , 3VLI 19 GAGE DECK
$t = 4.5"$ NWC 145 pcf $f'_c = 4000$ psi
75 psf , MAX. UNSHORED SPAN = 11'-2"

$$DL = \underset{\text{DECK}}{\underset{\uparrow}{75}} + \underset{\text{SUPER IMPOSED}}{\underset{\uparrow}{15}} = 90 \text{ psf}$$

LL = 80 psf PER ASCE 7-05

TRIB. AREA = (10)(40) = 400 ft²
INFL. AREA = $2A_T = 2(400) = 800$ ft²

$$LL = L_o\left(.25 + \frac{15}{\sqrt{800}}\right) = .78(80) = 63 \text{ psf}$$



w_u = DL = 90 psf (10) = .9 k/ft
LL = 63 psf (10) = .63 k/ft

$1.2D + 1.6L = 1.2(.9) + 1.6(.63) = 2.09$ k/ft

$$M_u = \frac{(2.09)(40)^2}{8} = 418 \text{ k-ft}$$

W/SW = 433 k-ft

DEFLECTION :

$$\Delta_{LL} \le \frac{(40\times12)}{360} = 1.33" = \frac{5(.63)(40')^4(1728)}{384(29000)(I_{REQ})} \Rightarrow I_{REQ} = 941 \text{ in}^4 \ (I_{LB})$$

$$\Delta_{TL} \le \frac{(40\times12)}{240} = 2" = \frac{5(1.53)(40)^4(1728)}{384(29000)(I_{REQ})} \Rightarrow I_{REQ} = 1520 \text{ in}^4 \ (I_{LB})$$

WITH SW → 1584 in⁴

PRE COMPOSITE DL: 75 psf (10) = .75 k/ft

$$\Delta_{PC} \le \frac{(40\times12)}{360} = 1.33" = \frac{5(.75)(40)^4(1728)}{384(29000)(I_{REQ})} \Rightarrow I_{REQ} = 1120 \text{ in}^4 \ (I_x)$$

w/SW → 1212 in⁴

USE A 21×62

$Y2 = 7.5" - 4"/2 = 5.5" \Rightarrow$ USE PNA ⑦ ; $\phi M_n = 540 > 418$ k-ft ∴ OK

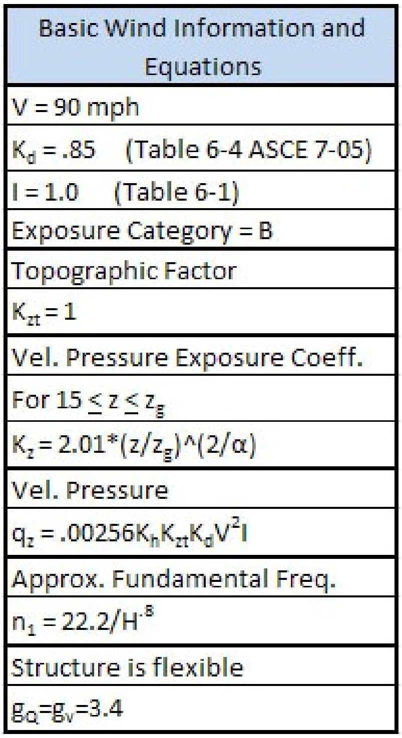
$\Sigma Q_n = 228$ $a = \frac{228}{.85(5.5")(120")} = .41" < 4"$ ∴ OK $I_{LB} = 2260 \text{ in}^4$ $I_x = 1330 \text{ in}^4$ ∴ OK

$Q_n = 17.2^k$ FOR 3/4" ϕ STUDS, DECK ⊥, WEAK ⇒ $228/17.2 = 13.3$ STUDS/SIDE ⇒ 14 PER SIDE

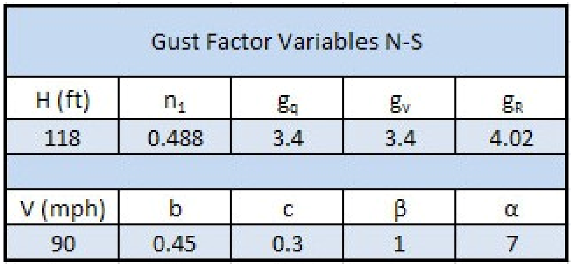
Appendix B

Wind

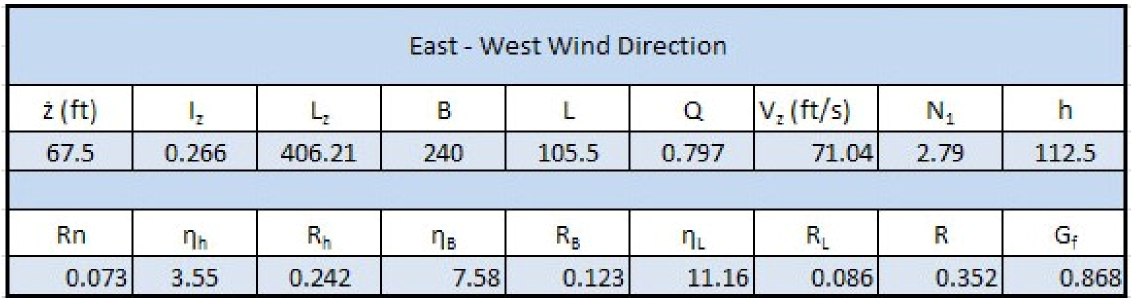
Wind Variable and Equation Tables



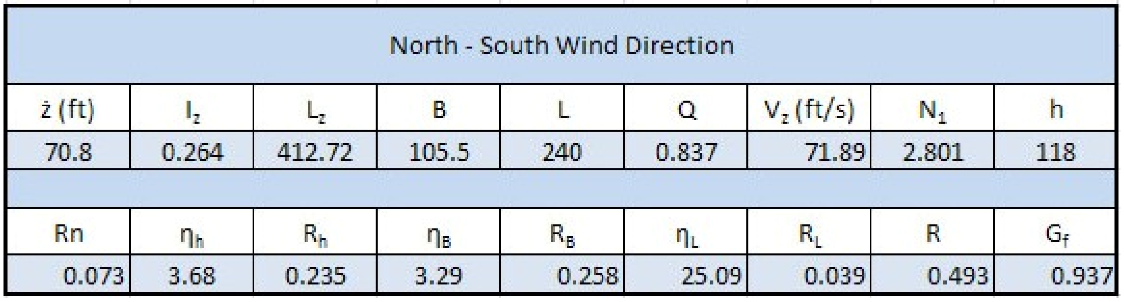
Basic Wind Information and Equations
V = 90 mph
K_d = .85 (Table 6-4 ASCE 7-05)
I = 1.0 (Table 6-1)
Exposure Category = B
Topographic Factor
$K_{zt} = 1$
Vel. Pressure Exposure Coeff.
For $15 \leq z \leq z_g$
$K_z = 2.01*(z/z_g)^\wedge(2/\alpha)$
Vel. Pressure
$q_z = .00256K_hK_{zt}K_dV^2I$
Approx. Fundamental Freq.
$n_1 = 22.2/H^{.8}$
Structure is flexible
$g_Q=g_v=3.4$



Gust Factor Variables N-S				
H (ft)	n_1	g_q	g_v	g_R
118	0.488	3.4	3.4	4.02
V (mph)	b	c	β	α
90	0.45	0.3	1	7

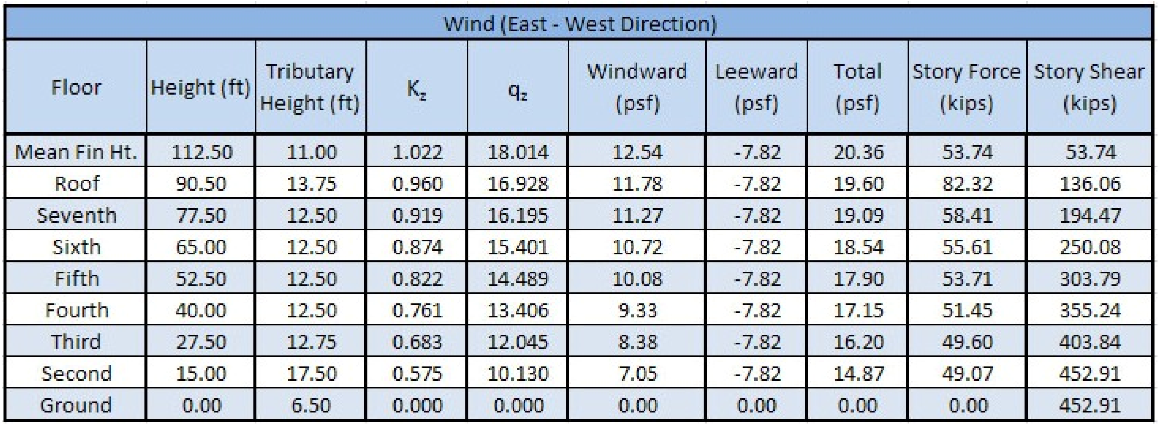


East - West Wind Direction								
ż (ft)	I_z	L_z	B	L	Q	V_z (ft/s)	N_1	h
67.5	0.266	406.21	240	105.5	0.797	71.04	2.79	112.5
Rn	η_h	R_h	η_B	R_B	η_L	R_L	R	G_f
0.073	3.55	0.242	7.58	0.123	11.16	0.086	0.352	0.868

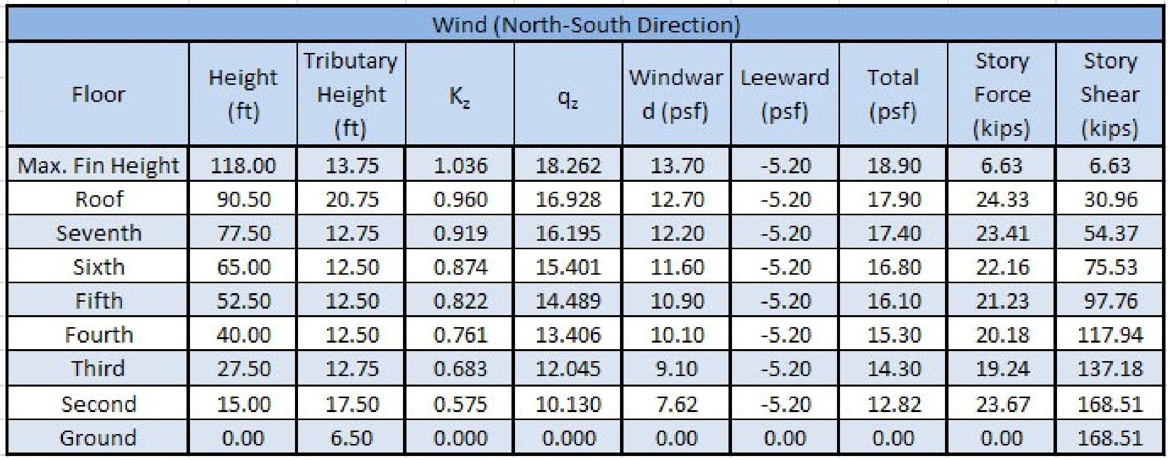


North - South Wind Direction								
ż (ft)	I_z	L_z	B	L	Q	V_z (ft/s)	N_1	h
70.8	0.264	412.72	105.5	240	0.837	71.89	2.801	118
Rn	η_h	R_h	η_B	R_B	η_L	R_L	R	G_f
0.073	3.68	0.235	3.29	0.258	25.09	0.039	0.493	0.937

Wind Force Tables and Diagram

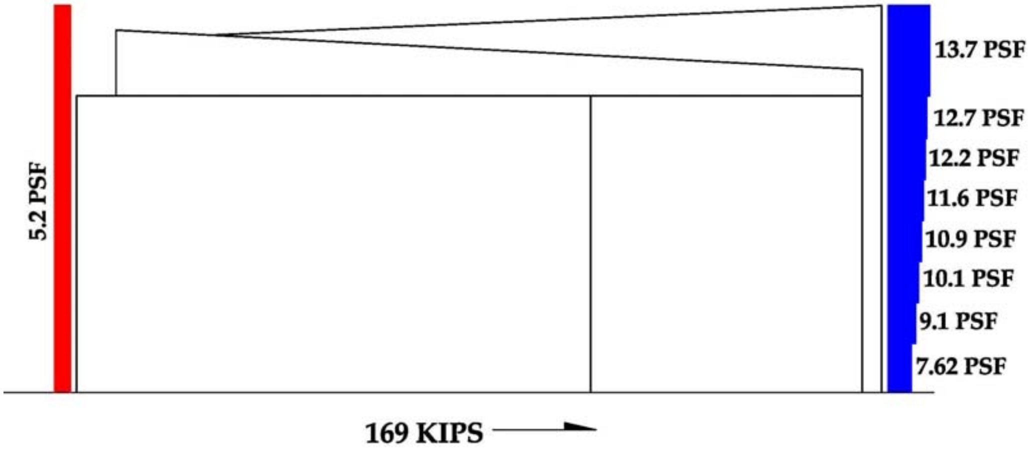


Wind (East - West Direction)									
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)
Mean Fin Ht.	112.50	11.00	1.022	18.014	12.54	-7.82	20.36	53.74	53.74
Roof	90.50	13.75	0.960	16.928	11.78	-7.82	19.60	82.32	136.06
Seventh	77.50	12.50	0.919	16.195	11.27	-7.82	19.09	58.41	194.47
Sixth	65.00	12.50	0.874	15.401	10.72	-7.82	18.54	55.61	250.08
Fifth	52.50	12.50	0.822	14.489	10.08	-7.82	17.90	53.71	303.79
Fourth	40.00	12.50	0.761	13.406	9.33	-7.82	17.15	51.45	355.24
Third	27.50	12.75	0.683	12.045	8.38	-7.82	16.20	49.60	403.84
Second	15.00	17.50	0.575	10.130	7.05	-7.82	14.87	49.07	452.91
Ground	0.00	6.50	0.000	0.000	0.00	0.00	0.00	0.00	452.91

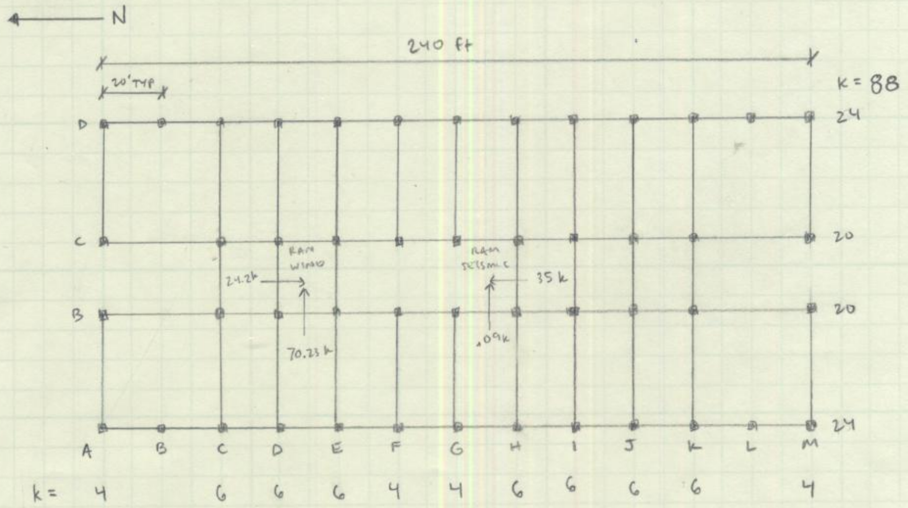


Wind (North-South Direction)									
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)
Max. Fin Height	118.00	13.75	1.036	18.262	13.70	-5.20	18.90	6.63	6.63
Roof	90.50	20.75	0.960	16.928	12.70	-5.20	17.90	24.33	30.96
Seventh	77.50	12.75	0.919	16.195	12.20	-5.20	17.40	23.41	54.37
Sixth	65.00	12.50	0.874	15.401	11.60	-5.20	16.80	22.16	75.53
Fifth	52.50	12.50	0.822	14.489	10.90	-5.20	16.10	21.23	97.76
Fourth	40.00	12.50	0.761	13.406	10.10	-5.20	15.30	20.18	117.94
Third	27.50	12.75	0.683	12.045	9.10	-5.20	14.30	19.24	137.18
Second	15.00	17.50	0.575	10.130	7.62	-5.20	12.82	23.67	168.51
Ground	0.00	6.50	0.000	0.000	0.00	0.00	0.00	0.00	168.51

Wind Pressures – North-South Direction



CENTER OF RIGIDITY



$\Sigma k = 58$

FINDING CENTER OF RIGIDITY

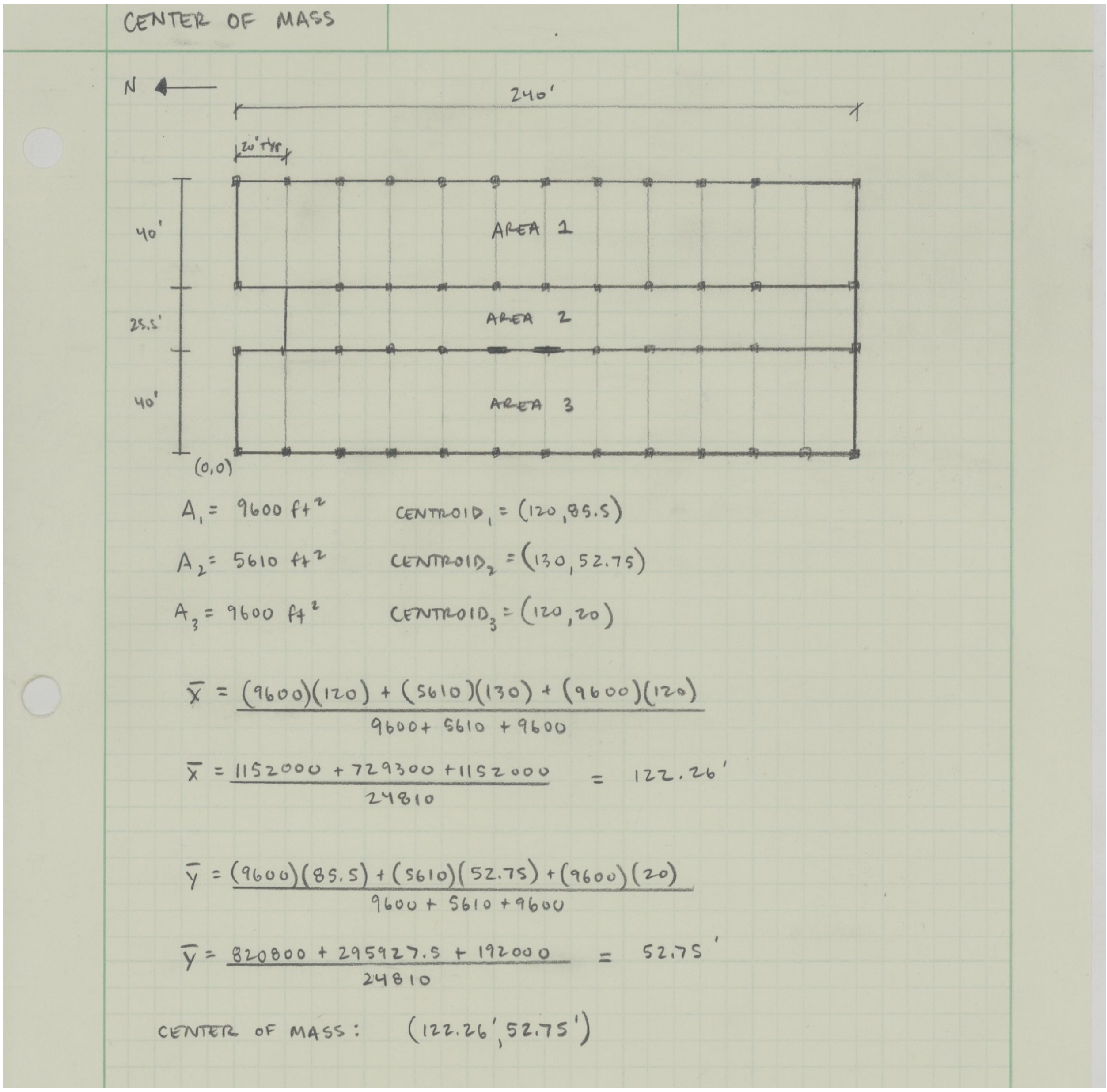
$$x_r = \frac{\Sigma k_{y_i} x_i}{\Sigma k_{y_i}}$$

$$x_r = \frac{[(4)(0) + 6(40) + 6(60) + 6(80) + 4(100) + 4(120) + 6(140) + 6(160) + 6(180) + 6(200) + 4(240)]}{4(4) + 7(6)}$$

$$= \frac{7000}{58} = 120.7'$$

$$y_r = \frac{[24(0) + 20(40') + 20(65.5) + 24(105.5)]}{2(24) + 2(20)} = 52.75'$$

CENTER OF RIGIDITY: (120.7', 52.75')



CENTER OF MASS

$A_1 = 9600 \text{ ft}^2$ CENTROID$_1$ = (120, 85.5)

$A_2 = 5610 \text{ ft}^2$ CENTROID$_2$ = (130, 52.75)

$A_3 = 9600 \text{ ft}^2$ CENTROID$_3$ = (120, 20)

$$\bar{x} = \frac{(9600)(120) + (5610)(130) + (9600)(120)}{9600 + 5610 + 9600}$$

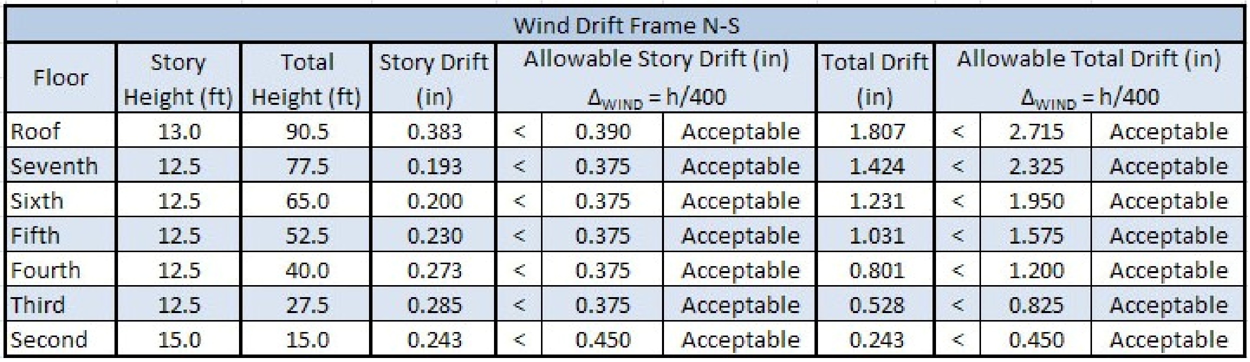
$$\bar{x} = \frac{1152000 + 729300 + 1152000}{24810} = 122.26'$$

$$\bar{y} = \frac{(9600)(85.5) + (5610)(52.75) + (9600)(20)}{9600 + 5610 + 9600}$$

$$\bar{y} = \frac{820800 + 295927.5 + 192000}{24810} = 52.75'$$

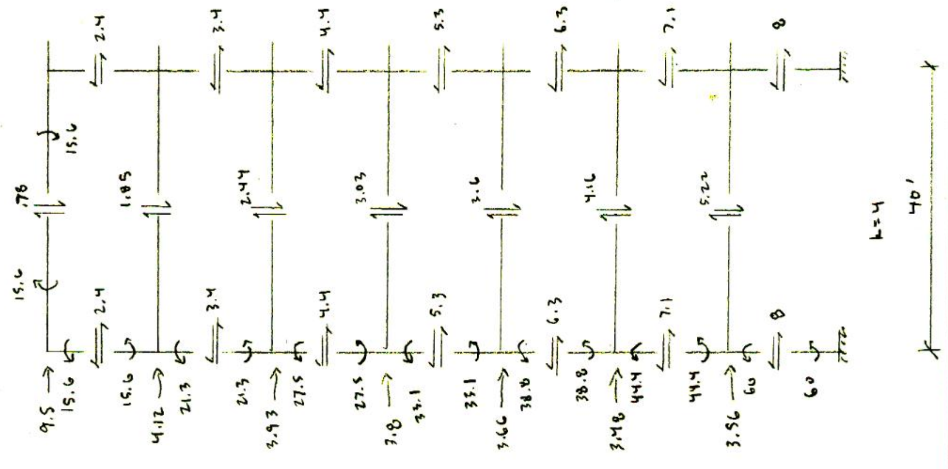
CENTER OF MASS: (122.26', 52.75')

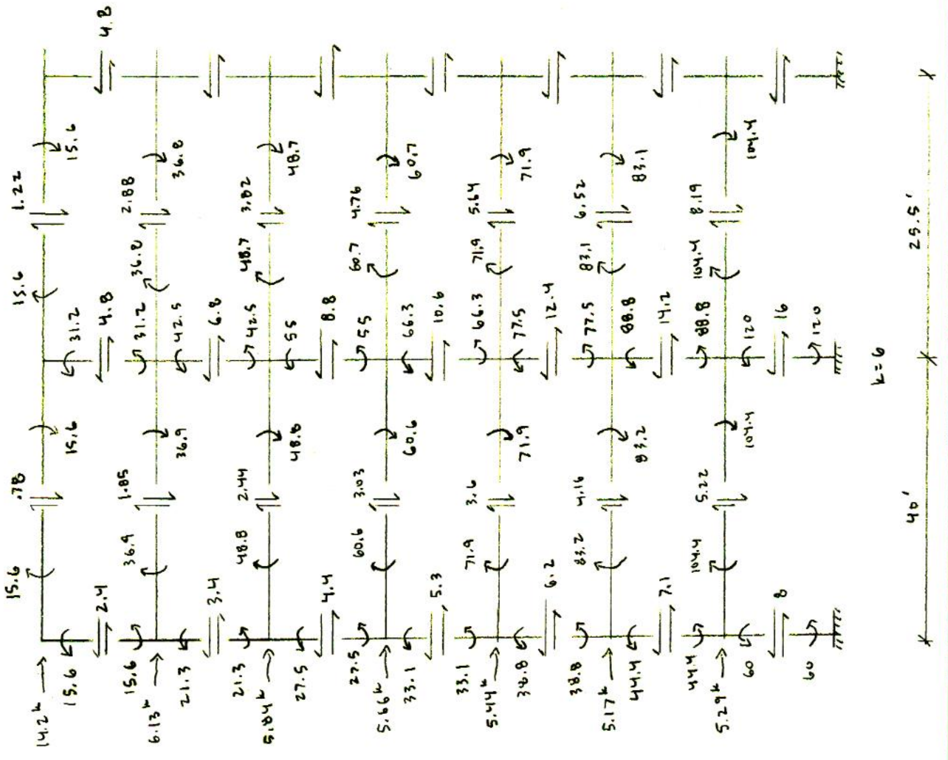
Torsion Constants							
Floor	Center of Mass		Center of Rigity		I_x (in^4)	I_y (in^4)	I_p (in^4)
	x_r (ft)	y_r (ft)	x_r (ft)	y_r (ft)			
Roof	122.26	52.75	120.70	52.75	384931	1103200	1488131
Seventh	122.26	52.75	120.70	52.75	384931	1103200	1488131
Sixth	122.26	52.75	120.70	52.75	384931	1103200	1488131
Fifth	122.26	52.75	120.70	52.75	384931	1103200	1488131
Fourth	122.26	52.75	120.70	52.75	384931	1103200	1488131
Third	122.26	52.75	120.70	52.75	384931	1103200	1488131
Second	122.26	52.75	120.70	52.75	384931	1103200	1488131



Wind Drift Frame N-S										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{WIND} = h/400$			Total Drift (in)	Allowable Total Drift (in) $\Delta_{WIND} = h/400$		
Roof	13.0	90.5	0.383	<	0.390	Acceptable	1.807	<	2.715	Acceptable
Seventh	12.5	77.5	0.193	<	0.375	Acceptable	1.424	<	2.325	Acceptable
Sixth	12.5	65.0	0.200	<	0.375	Acceptable	1.231	<	1.950	Acceptable
Fifth	12.5	52.5	0.230	<	0.375	Acceptable	1.031	<	1.575	Acceptable
Fourth	12.5	40.0	0.273	<	0.375	Acceptable	0.801	<	1.200	Acceptable
Third	12.5	27.5	0.285	<	0.375	Acceptable	0.528	<	0.825	Acceptable
Second	15.0	15.0	0.243	<	0.450	Acceptable	0.243	<	0.450	Acceptable

PORTAL FRAME ANALYSIS
WIND
E-W DIRECTION





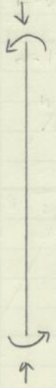
STRENGTH CHECK
FR5 COL. B.10

W14x145

$P_{L_r} = 111.35$ k
$P_{DL} = 58.95(4) = 235.8$
$P_{LL} = .42(52.4)(3) = 66$ k

RF => $D = 90(20)\left[\frac{40}{2} + \frac{26.5}{5}\right] = 58.95$ k
$L_r = (150+20)(20)[32.75] = 111.35$

7TH COL $D = 90(20)(32.75) = 58.95$ k
$LL = 80(20)(32.75) = 52.4$ k



$KL_x = .65(12.5) = 8.13'$
$KL_y = .65(12.5) = 8.13'$

6TH COL = 5TH COL => $D = 58.95$ k
$LL = 52.4$ k

TRIB $AREA_1 = 20' \times 32.75' = 655\ ft^2$
$K_{LL} = 4$ 3 FLOORS ABOVE

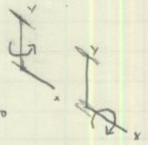
TRIB $AREA_3 = 3(655\ ft^2) = 1965\ ft^2$

LL REDUCTION

$LL = LL_0\left(.25 + \frac{15}{\sqrt{(4)(1965)}}\right) = .42 > .4 \therefore$ OK

Y I M_{EQ} APPLIED TO

x $\vdash\!\dashv$ M_{WIND} APPLIED TO



$P_u = 1.2D + 1.6L_r + L = 1.2(235.8) + 1.6(111.35) + 66 = 527$ k

$M_{u_x} = 1.6 M_{WIND} = 1.6(66.3$ k-ft$) = 106$ k-ft

$M_{u_y} = M_E = 23$ k-ft

$\boxed{I}$ $\frac{KL_x}{r_x/r_y} = \frac{8.13'}{1.59} = 5.1' \therefore KL_y$ CONTROLS

$\boxed{II}$

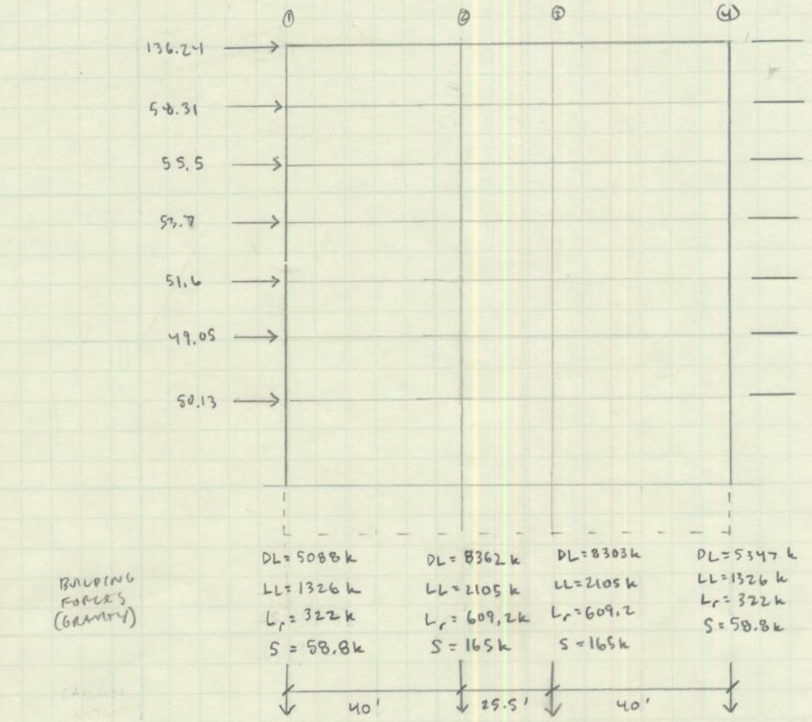
$p = .578E-3$
$b_x = .912\ E-3$
$b_y = 1.78\ E-3$

$pP = (.578E-3)(527) = .305 > .2$ (41-1a)

$pP + b_x M_{u_x} + b_y M_{u_y}$
$.305 + (.912E-3)(106$ k-ft$) + (1.78E-3)(23$ k-ft$) = .44 < 1 \therefore$ Ok

OVERTURNING MOMENT CHECK

GOVERNING CASE



USING CANTILEVER METHOD

$$\bar{x} = \frac{40A + 65.5A + 105.5A}{4A} = 52.75$$

$$I = 52.75^2A + 12.75^2A + 12.75A^2 + 52.75^2A = 5890A$$

USING FROM OVERTURNING MOMENT TABLE → (12329.72 k-ft)

RF $P_{1,4} = \frac{(12329,72)(52.75)A}{5890A} = 110.4$ $P_{2,3} = \frac{(12329.72)(12.75)A}{5890A} = 26.7$

FR7	(.009)(16848.75) = 151.6	33.7
FR6	(.009)(20456.25) = 184.1	40.9
FR5	(.009)(23275.5) = 209.5	46.6
FR4	(.009)(25339.5) = 228.1	50.7
FR3	(.009)(26688.37) = 240.2	53.4
FR2	(.009)(27470.32) = 247	54.9
	1370	307

TOTAL WEIGHT ON CAISSONS

(OUTER) 1.2(5347) + 1.6(1370) + (1326) + .5(322) = 10095/13 CAISSONS = 776 k < 796 k ∴ OK

EXISTING LOAD

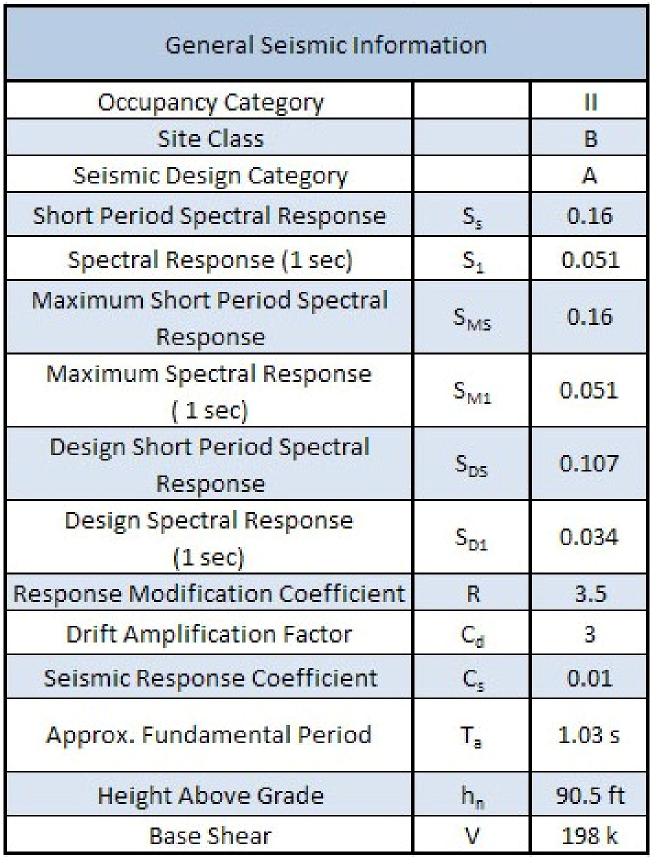
(INNER) 1.2(8362) + 1.6(306.9) + (2105) + .5(609) = 12935/13 = 995 k < 1008 k ∴ OK

EXISTING

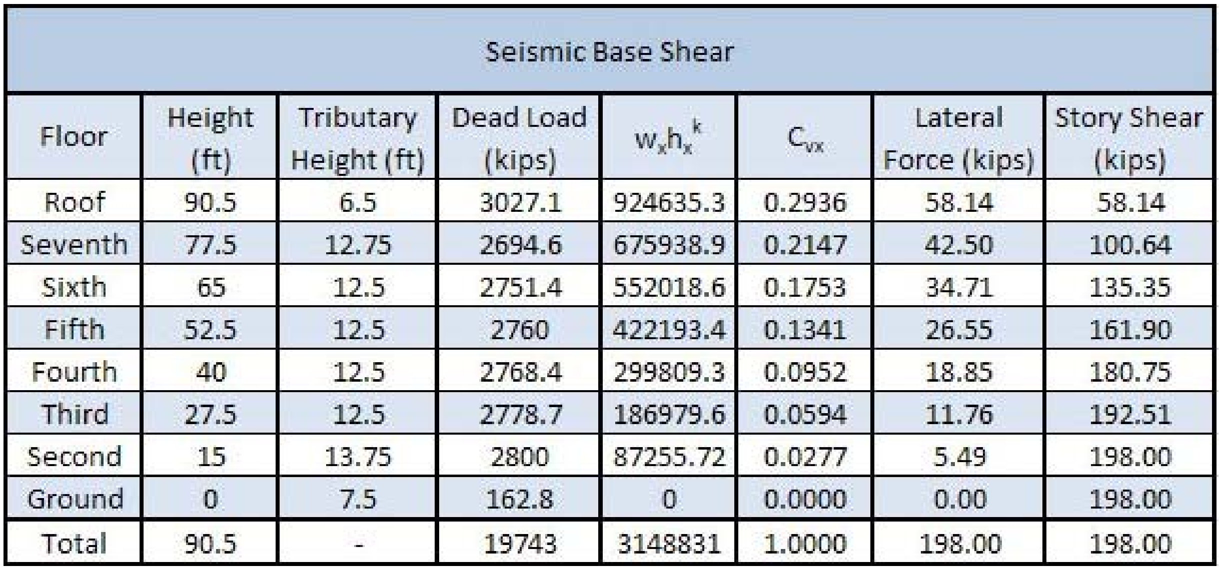
Appendix C

Seismic

Seismic Design Tables



General Seismic Information		
Occupancy Category		II
Site Class		B
Seismic Design Category		A
Short Period Spectral Response	S_s	0.16
Spectral Response (1 sec)	S_1	0.051
Maximum Short Period Spectral Response	S_{MS}	0.16
Maximum Spectral Response (1 sec)	S_{M1}	0.051
Design Short Period Spectral Response	S_{DS}	0.107
Design Spectral Response (1 sec)	S_{D1}	0.034
Response Modification Coefficient	R	3.5
Drift Amplification Factor	C_d	3
Seismic Response Coefficient	C_s	0.01
Approx. Fundamental Period	T_a	1.03 s
Height Above Grade	h_n	90.5 ft
Base Shear	V	198 k



Seismic Base Shear

Floor	Height (ft)	Tributary Height (ft)	Dead Load (kips)	$w_x h_x^k$	C_{vx}	Lateral Force (kips)	Story Shear (kips)
Roof	90.5	6.5	3027.1	924635.3	0.2936	58.14	58.14
Seventh	77.5	12.75	2694.6	675938.9	0.2147	42.50	100.64
Sixth	65	12.5	2751.4	552018.6	0.1753	34.71	135.35
Fifth	52.5	12.5	2760	422193.4	0.1341	26.55	161.90
Fourth	40	12.5	2768.4	299809.3	0.0952	18.85	180.75
Third	27.5	12.5	2778.7	186979.6	0.0594	11.76	192.51
Second	15	13.75	2800	87255.72	0.0277	5.49	198.00
Ground	0	7.5	162.8	0	0.0000	0.00	198.00
Total	90.5	-	19743	3148831	1.0000	198.00	198.00

Building Dead Loads

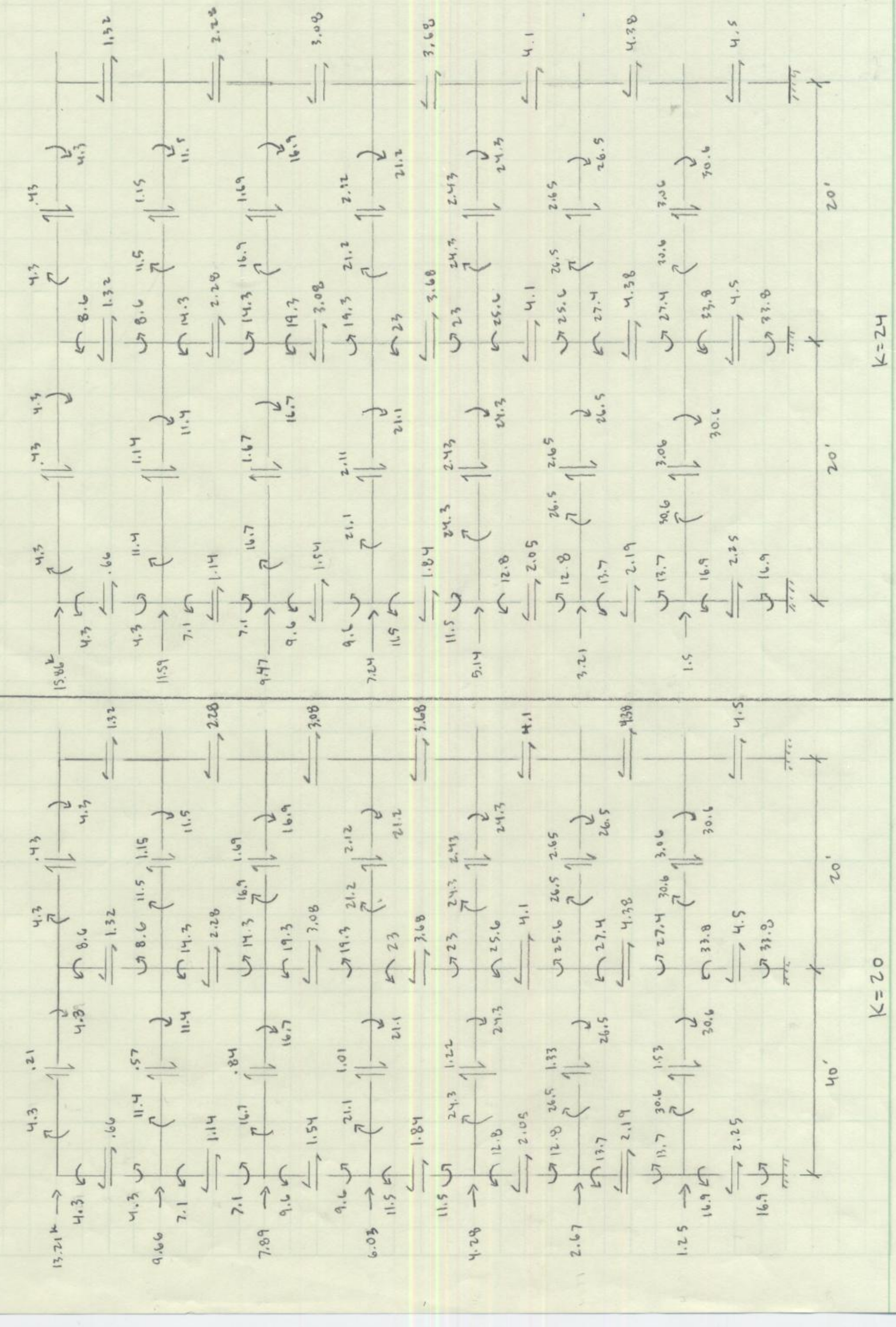
Level	Component	Weight (lbs)
Arch. Fin		
	Metal Panels	31700
	Steel Braces	108241
PH		
	Concrete + Deck	225000
	Steel Beams	4901
	Steel Columns	7572
	Superimposed	45000
Roof		
	Concrete + Deck	1893000
	Steel Beams	120897
	Steel Columns	13357
	Superimposed	375000
	Wall	202399
Floor 7		
	Concrete + Deck	1875000
	Steel Beams	146614
	Steel Columns	31619
	Superimposed	375000
	Wall	266400
Floor 6		
	Concrete + Deck	1903875
	Steel Beams	146614
	Steel Columns	36524
	Superimposed	380775
	Wall	283571
	SubTotal	8473059
Floor 5		
	Concrete + Deck	1903875
	Steel Beams	146614
	Steel Columns	45044
	Superimposed	380775
	Wall	283571
Floor 4		
	Concrete + Deck	1903875
	Steel Beams	146614
	Steel Columns	53563
	Superimposed	380775
	Wall	283571
Floor 3		
	Concrete + Deck	1903875
	Steel Beams	146614
	Steel Columns	63814
	Superimposed	380775
	Wall	283571
Floor 2		
	Concrete + Deck	1903875
	Steel Beams	146614
	Steel Columns	74064
	Other Steel	16969
	Superimposed	380775
	Wall	277523
Floor 1		
	Steel Columns	37032
	Wall	125802
	SubTotal	11269580

Total Building Weight = 19742639 lb = 19750 kips

Seismic Drift N-S

Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = .020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = .020h_{sx}$		
Roof	13.0	90.5	1.725	<	3.120	Acceptable	6.186	<	21.720	Acceptable
Seventh	12.5	77.5	0.570	<	3.000	Acceptable	4.461	<	18.600	Acceptable
Sixth	12.5	65.0	0.612	<	3.000	Acceptable	3.891	<	15.600	Acceptable
Fifth	12.5	52.5	0.711	<	3.000	Acceptable	3.279	<	12.600	Acceptable
Fourth	12.5	40.0	0.861	<	3.000	Acceptable	2.568	<	9.600	Acceptable
Third	12.5	27.5	0.915	<	3.000	Acceptable	1.707	<	6.600	Acceptable
Second	15.0	15.0	0.792	<	3.600	Acceptable	0.792	<	3.600	Acceptable

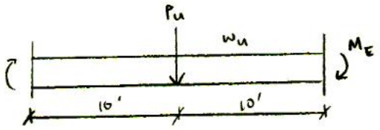
PORTAL FRAME ANALYSIS
SEISMIC
N-S DIRECTION



STRENGTH CHECK

GIRDER SUPPORTING PRECAST FACADE
2ND FLOOR EAST FACADE

EXTERIOR N-S FRAME
SEISMIC CONTROLS
LC: 1.2D + E + L + .2S



$P_D = 18k + 1.24k$
$P_L = 12.6k$

$M_E = 30.6$ k-ft [FROM PORTAL FRAME]
$w_u = (93.75_{psf})(6.5') + (8_{psf})(6') = .658$ k/ft

$$M_D = \frac{Pl}{8} + \frac{wl^2}{12} = \frac{(19.24)(20)}{8} + \frac{(.658)(20)^2}{12} = 48.1 + 21.93 = \pm 70 \text{ k-ft}$$

$$M_L = \frac{(12.6)(20)}{8} = 31.5 \text{ k-ft}$$

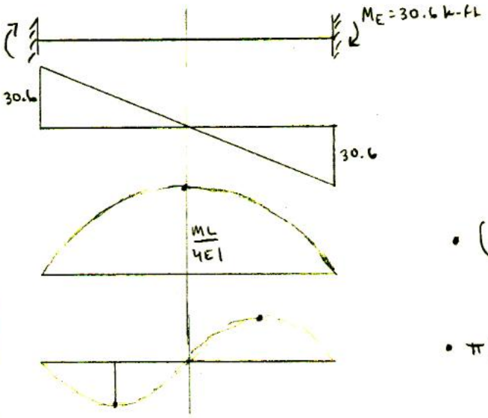
$$M_{u_L} = 1.2(70 \text{ k-ft}) + 1.0(30.6) + 1.0(31.5 \text{ k-ft}) = -146.1 \text{ k-ft}$$

$$M_{u_R} = 1.2(70) - 30.6 + 31.5 = 85 \text{ k-ft}$$

$$\Delta_{TL} = \frac{l}{500} = \frac{(20)(12)}{500} = .48''$$

$$I_{REQ} = \frac{(31.84)(20)^3(1728)}{192(29000)(.48)} + \frac{5(.658)(20)^4(1728)}{384(29000)(.48)} = 164.7 + 170.2 = 335 \text{ IN}^4$$

CONJUGATE BEAM METHOD FOR Δ_E



- $\left(\frac{M}{EI}\right)\left(\frac{L}{2}\right)\left(\frac{1}{2}\right) = \frac{ML}{4EI}$
- $\pi\left(\frac{L}{4}\right)\left(\frac{ML}{4EI}\right) = \frac{\pi ML^2}{16EI}$

$$I = \frac{\pi(30.6)(20)^2(1728)}{16(29000)(.48)} = 299 \text{ IN}^4$$

$$I_{REQ_{TOTAL}} = 335 \text{ IN}^4 + 299 \text{ IN}^4 = 634 \text{ IN}^4$$

NEED A W16x50 $\Rightarrow I = 659 \text{ IN}^4 > 634 \text{ IN}^4 \therefore$ OK

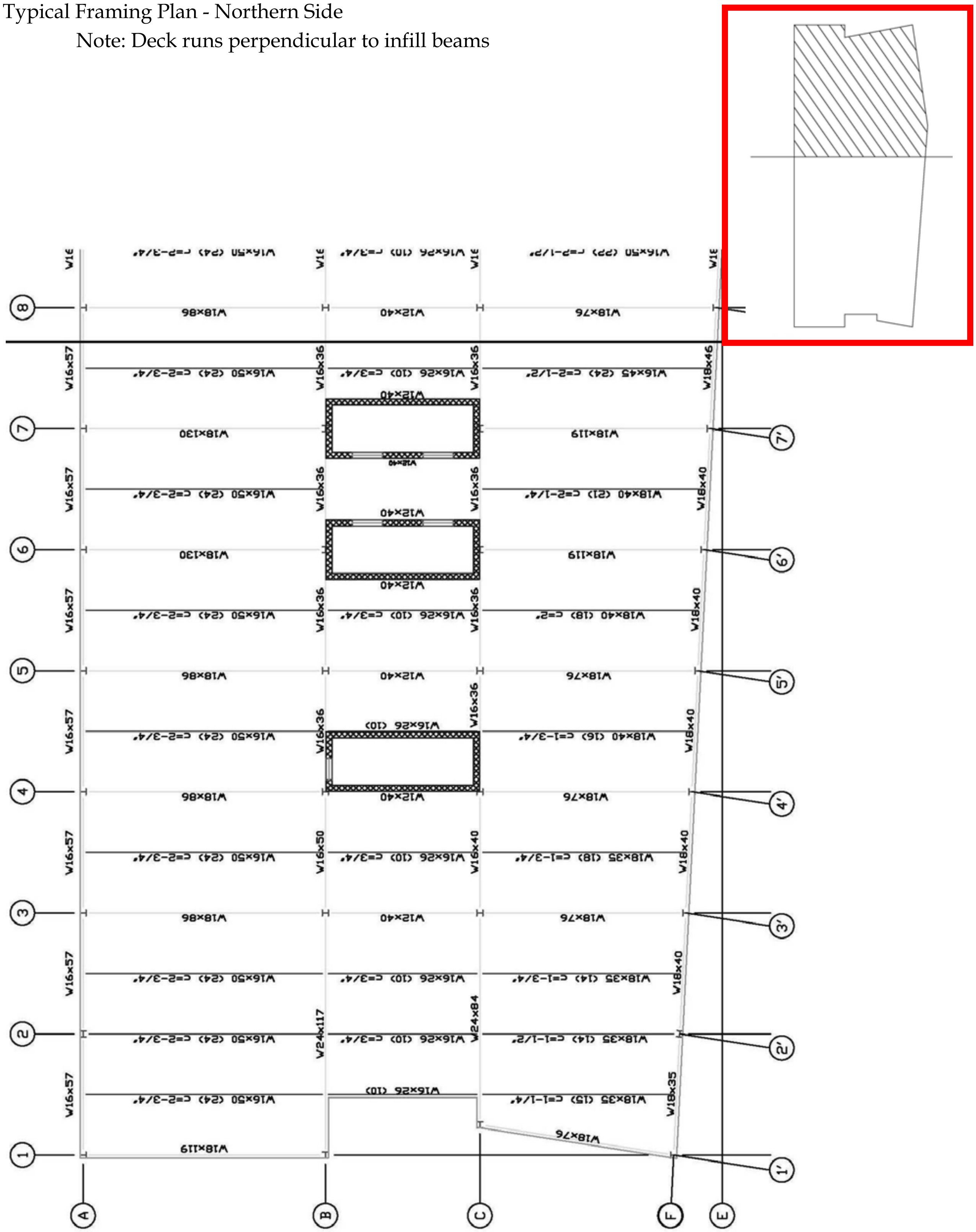
$\phi M_p = 345 \text{ k-ft} > M_{u_{MAX}} = 146.1 \text{ k-ft} \therefore$ OK

Appendix D

Floor Plans and Column Sizes

Typical Framing Plan - Northern Side

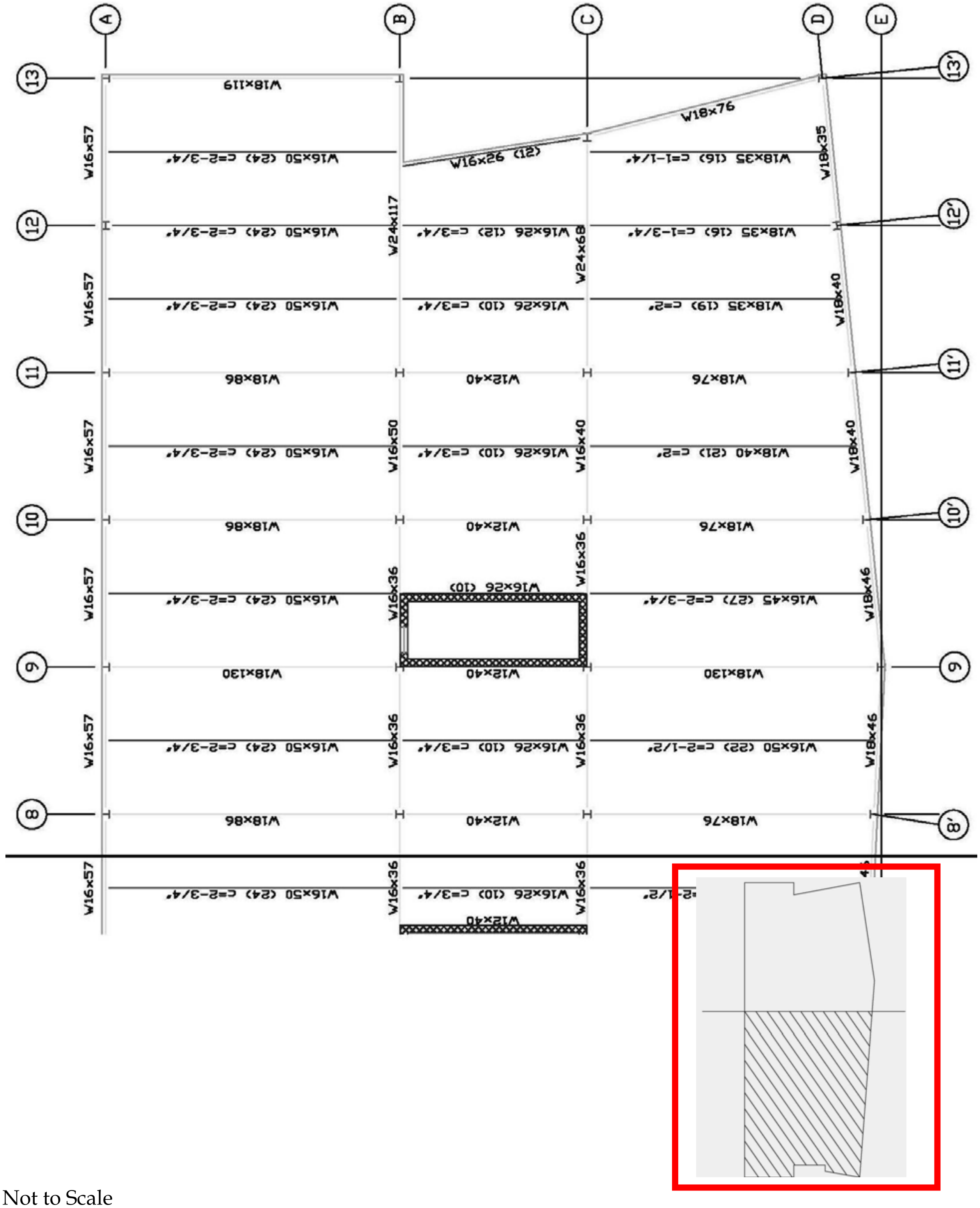
Note: Deck runs perpendicular to infill beams



Not to Scale

Typical Framing Plan – Southern End

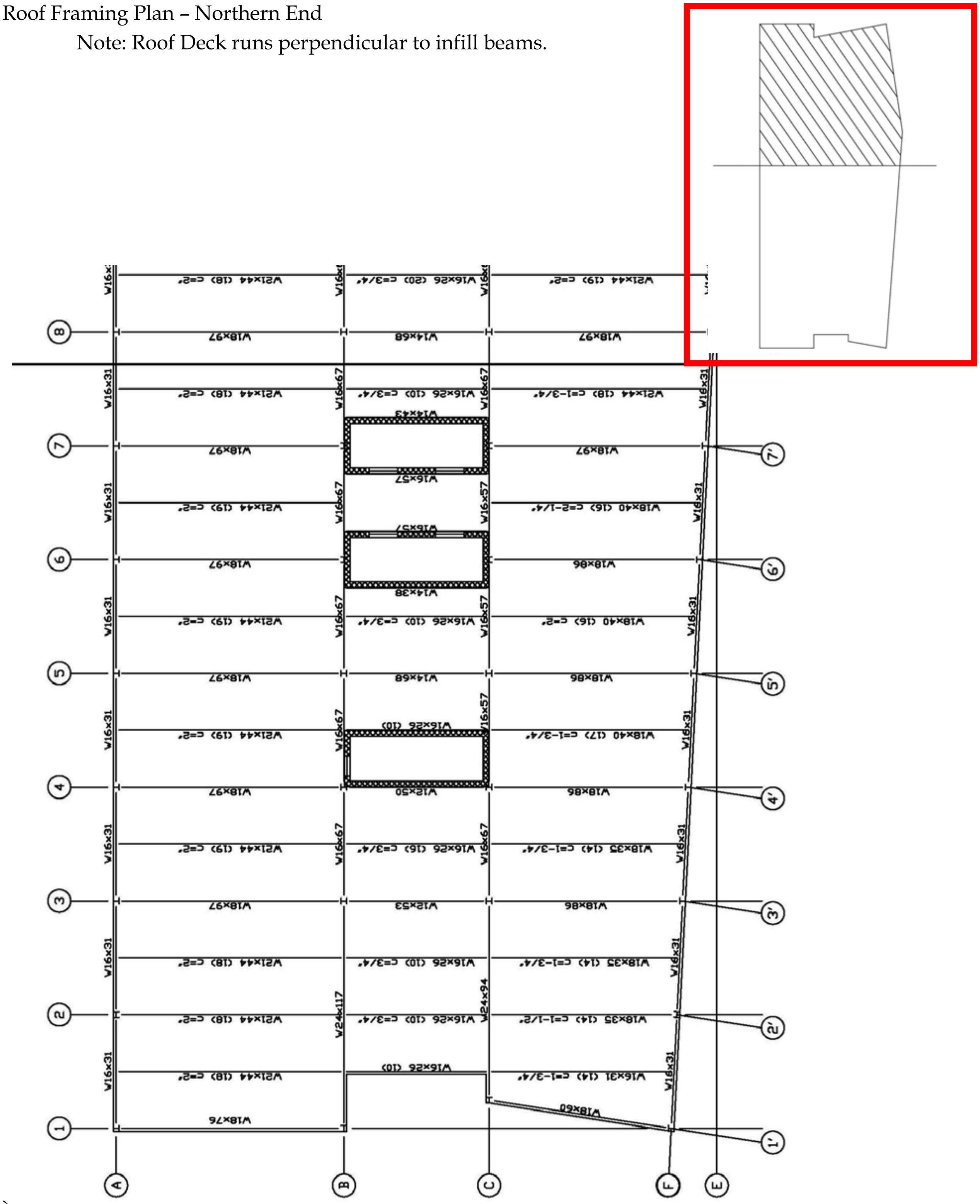
Note: Deck runs perpendicular to infill beams.



Not to Scale

Roof Framing Plan – Northern End

Note: Roof Deck runs perpendicular to infill beams.



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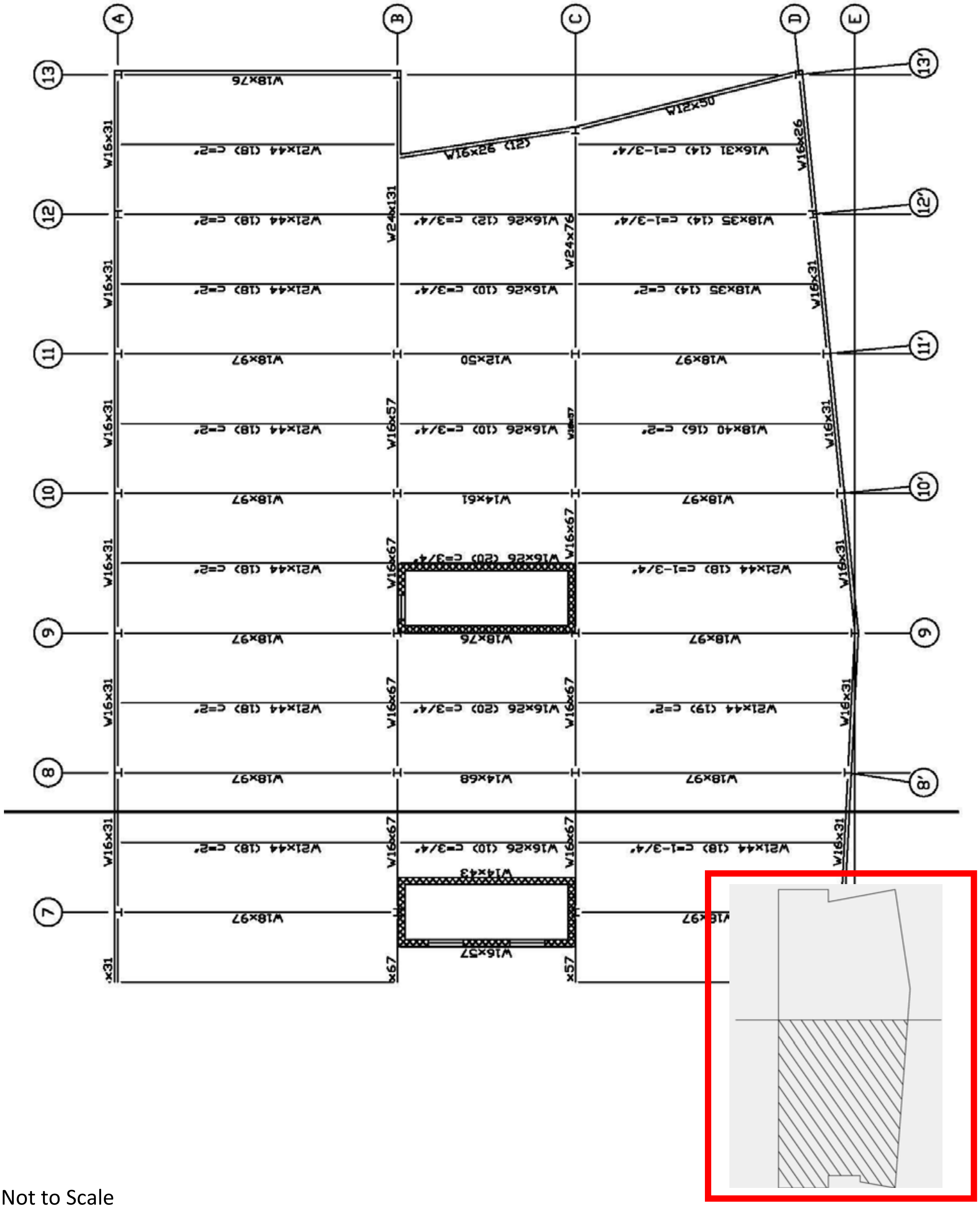
Not to Scale

Roof Framing – Southern End

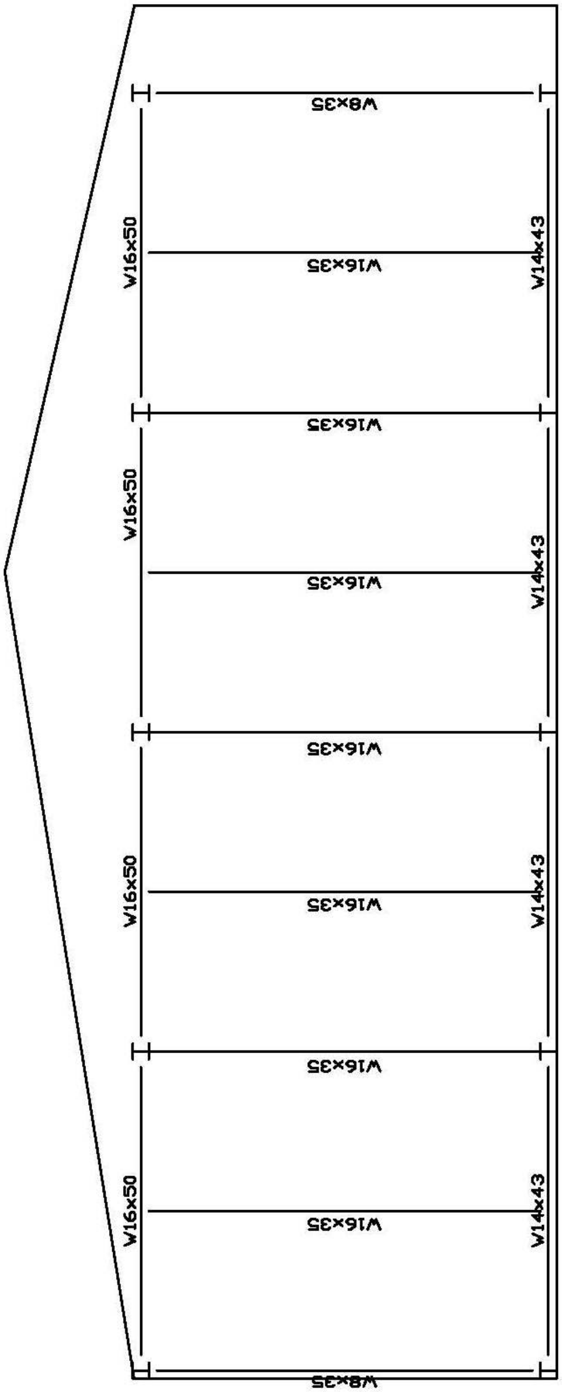
Note: Deck runs perpendicular to infill beams.

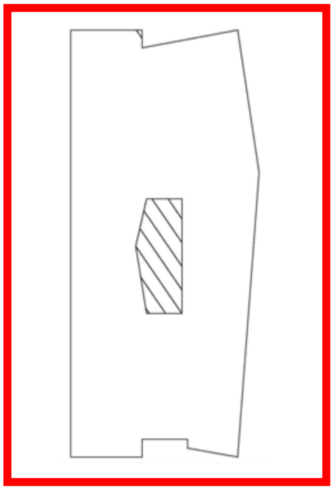


Not to Scale

Penthouse Framing Plan

Note: Roof Deck runs perpendicular to infill beams.





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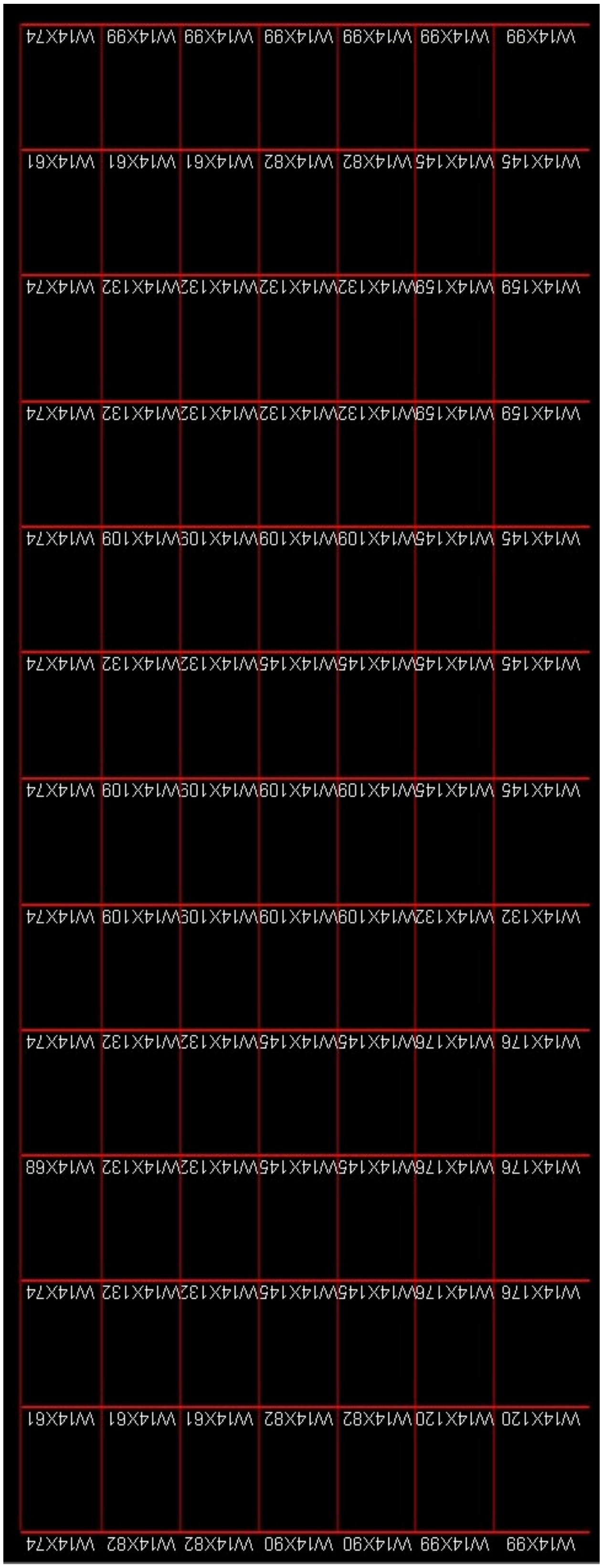
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W14X74 W14X132 W14X132 W14X132 W14X132 W14X159 W14X159
W14X74 W14X109 W14X109 W14X109 W14X109 W14X145 W14X145
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W14X74 W14X109 W14X109 W14X109 W14X109 W14X145 W14X145
W14X74 W14X109 W14X109 W14X109 W14X109 W14X132 W14X132
W14X74 W14X132 W14X132 W14X145 W14X145 W14X176 W14X176
W14X68 W14X132 W14X132 W14X145 W14X145 W14X176 W14X176
W14X74 W14X132 W14X132 W14X145 W14X145 W14X176 W14X176
W14X61 W14X61 W14X61 W14X82 W14X82 W14X120 W14X120
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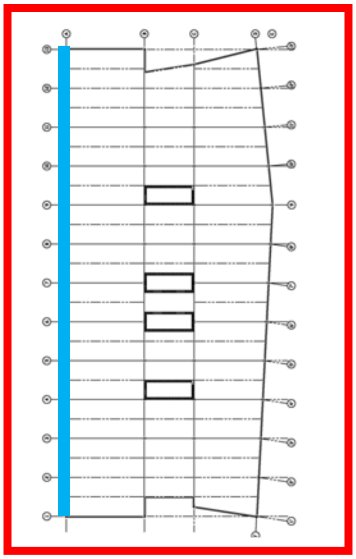
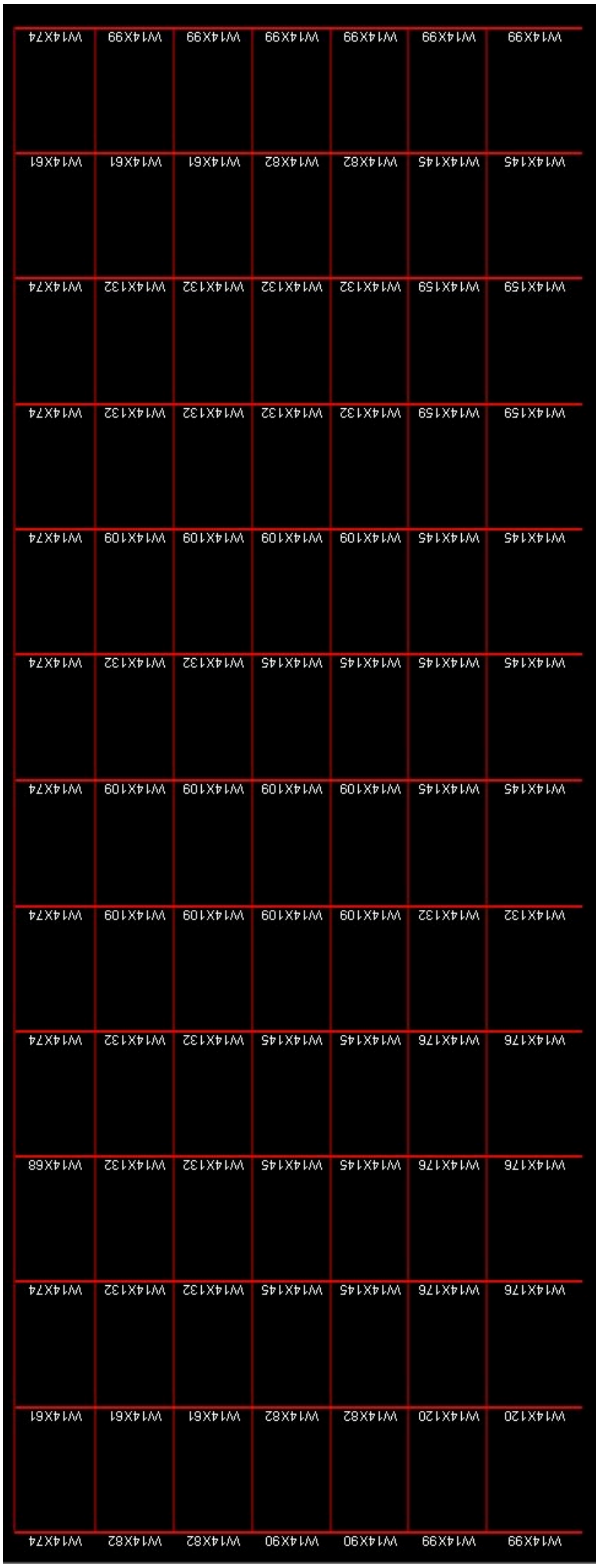
Frame A

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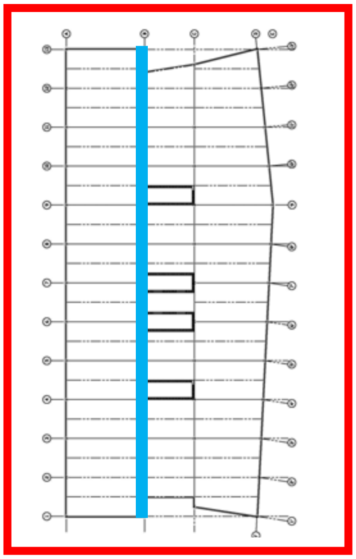
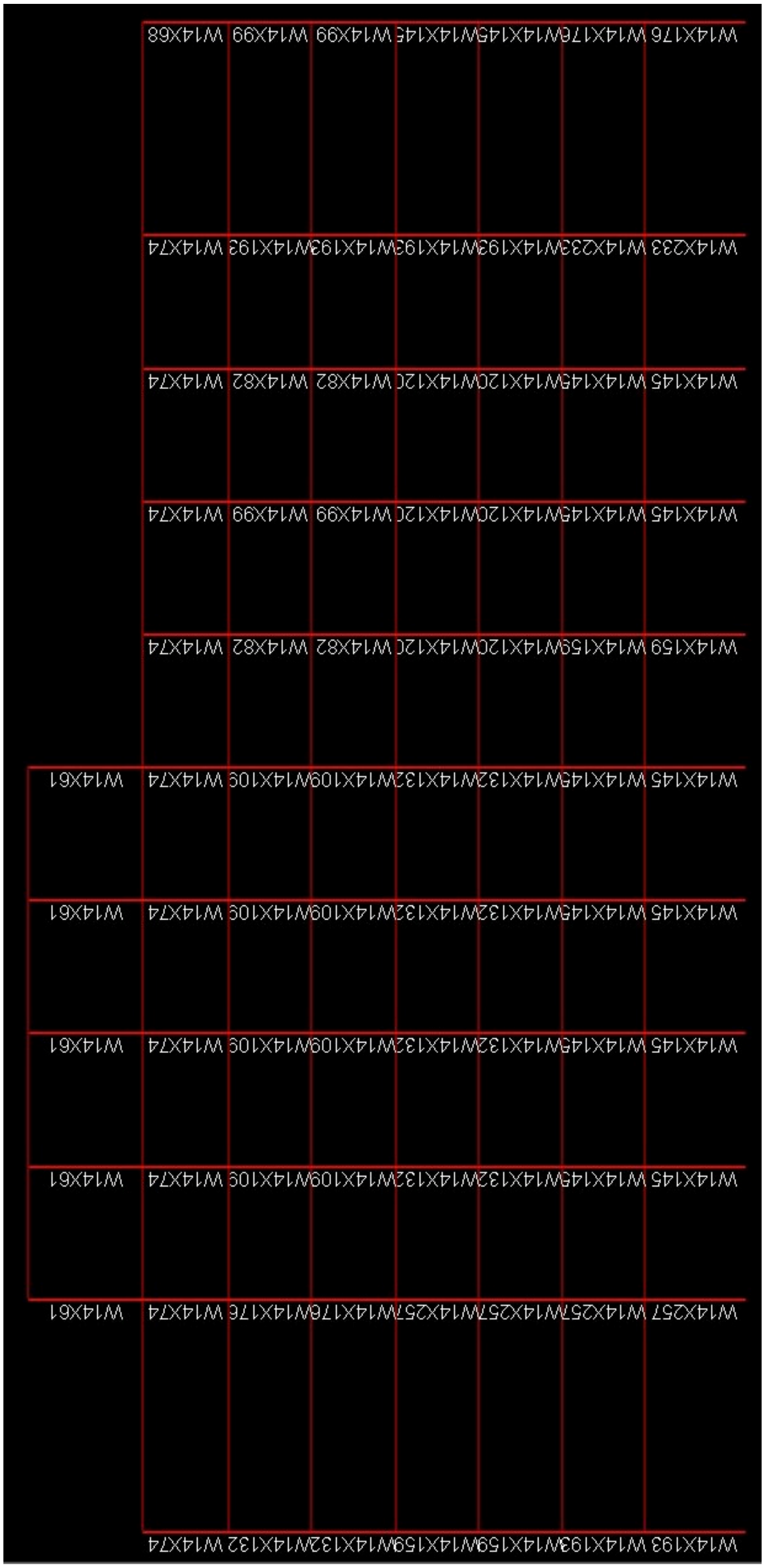
FRAME B

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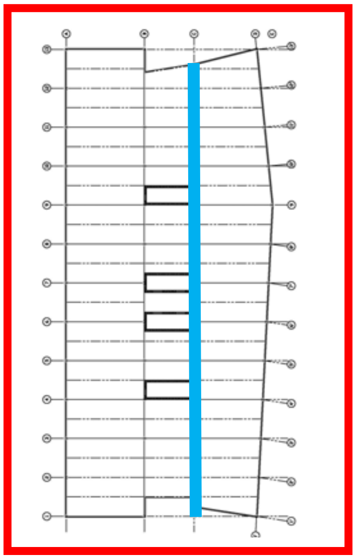
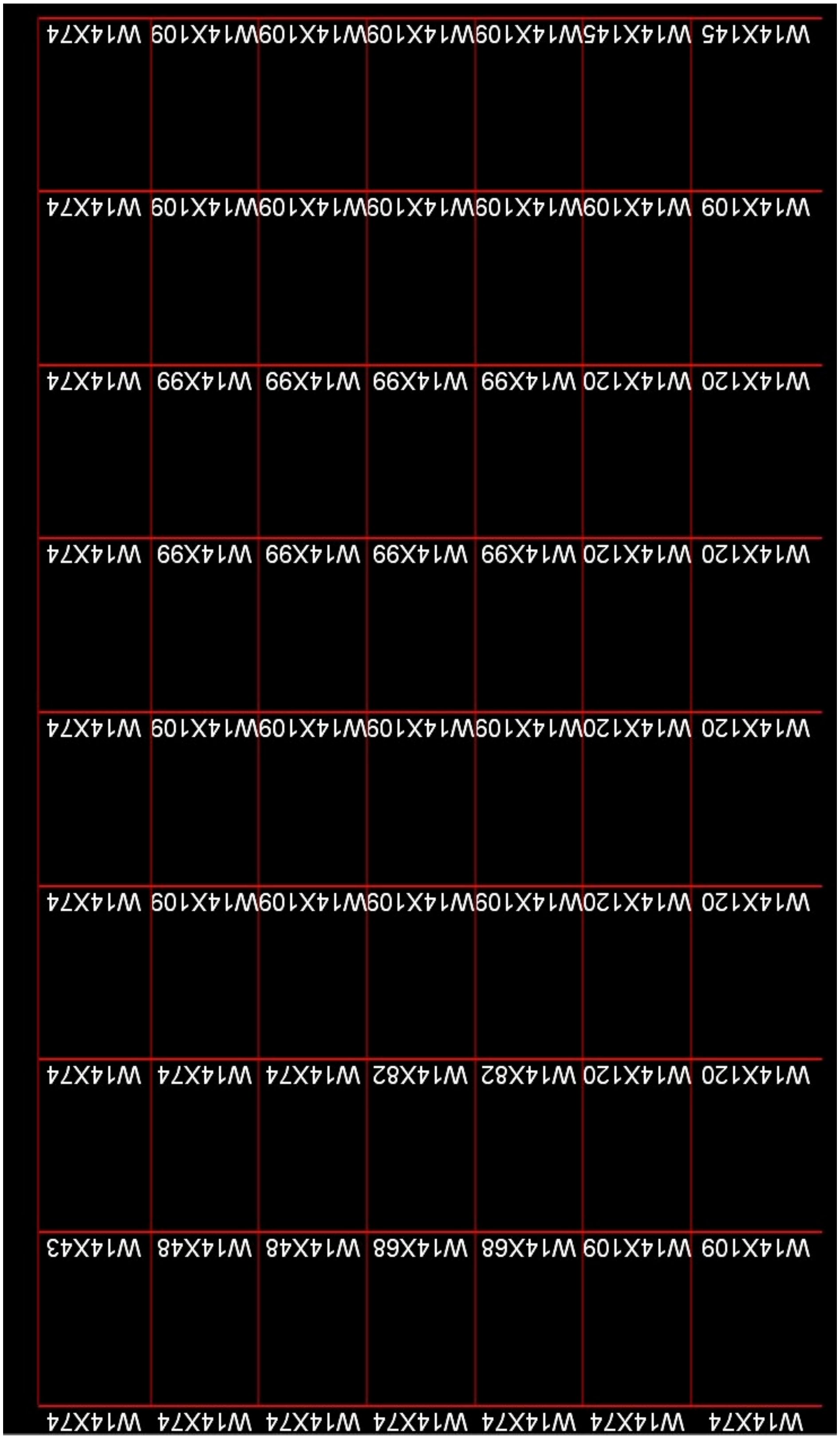
FRAME C

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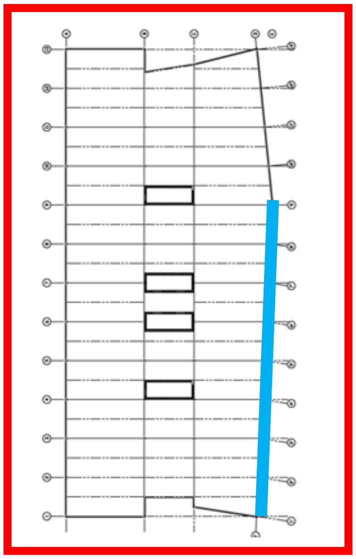
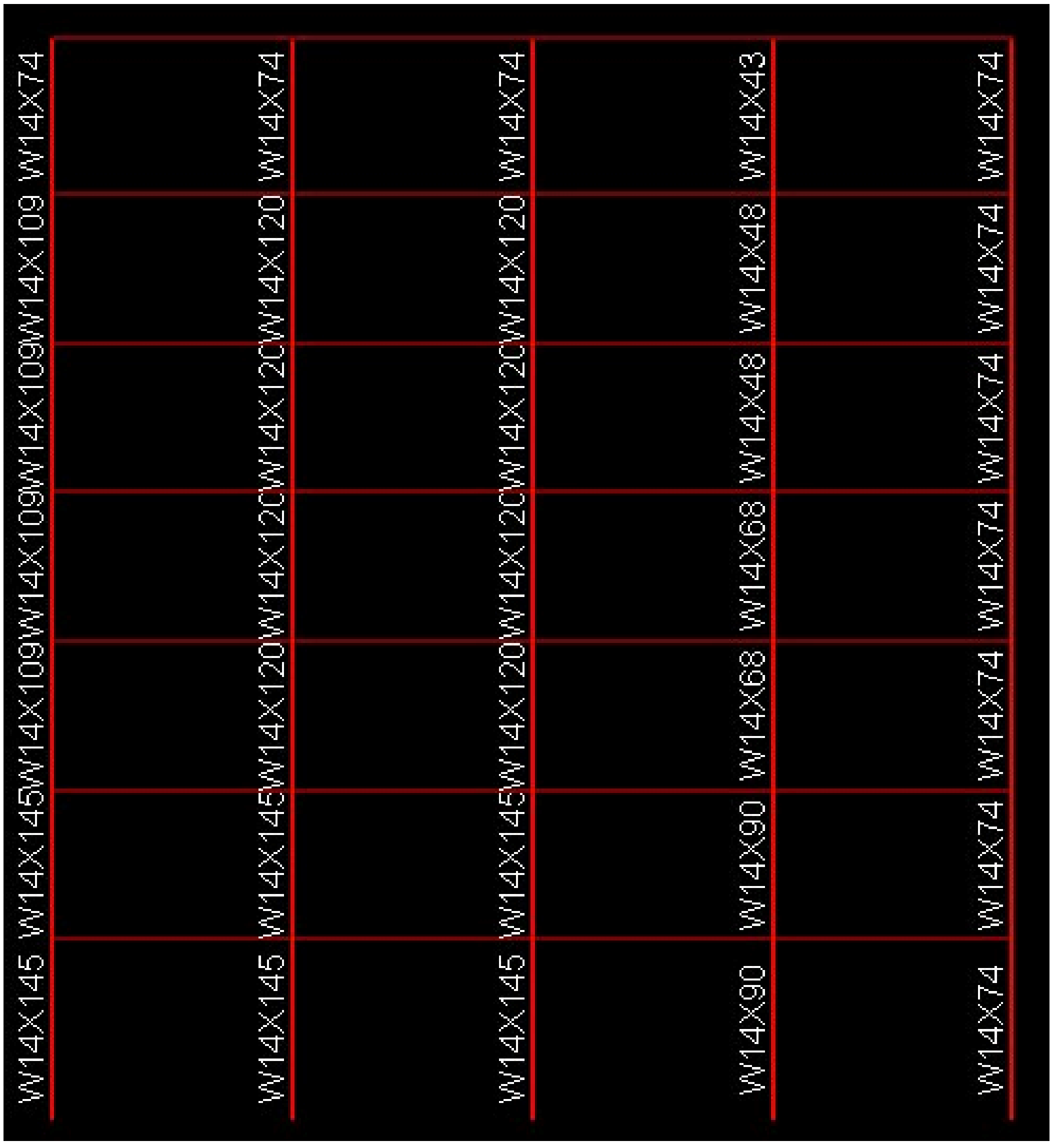
FRAME F

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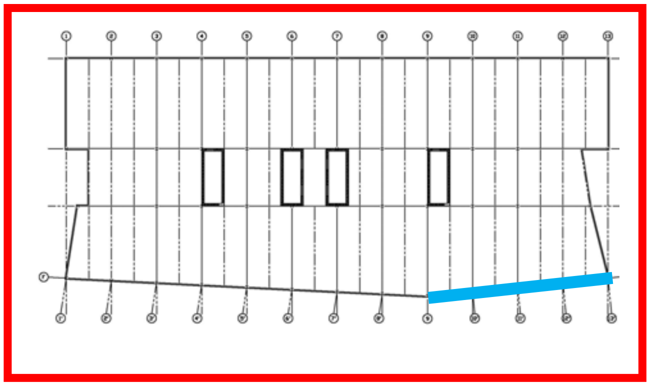
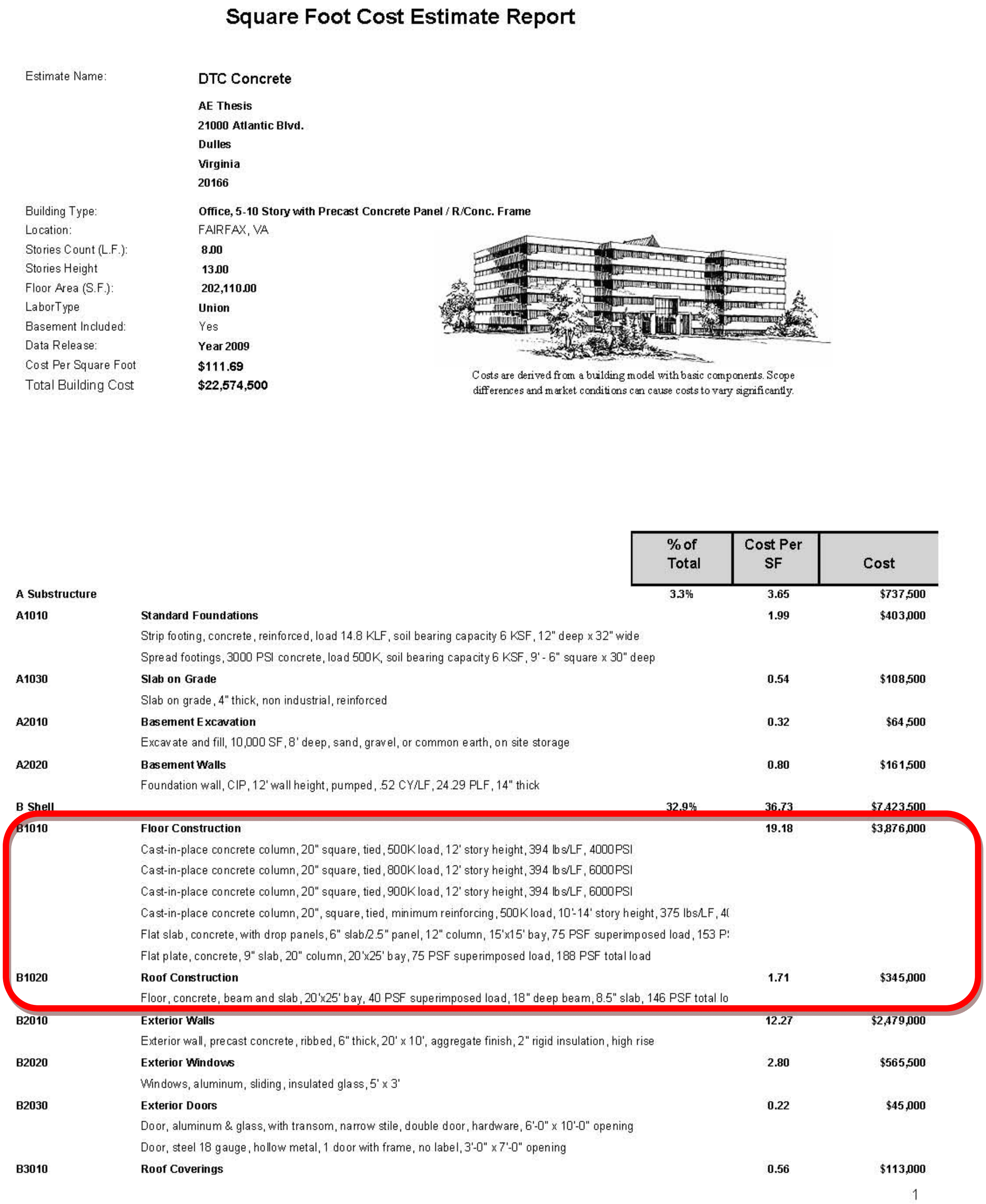


FRAME D

Appendix E

Construction Management Study

Square Foot Estimate Costs



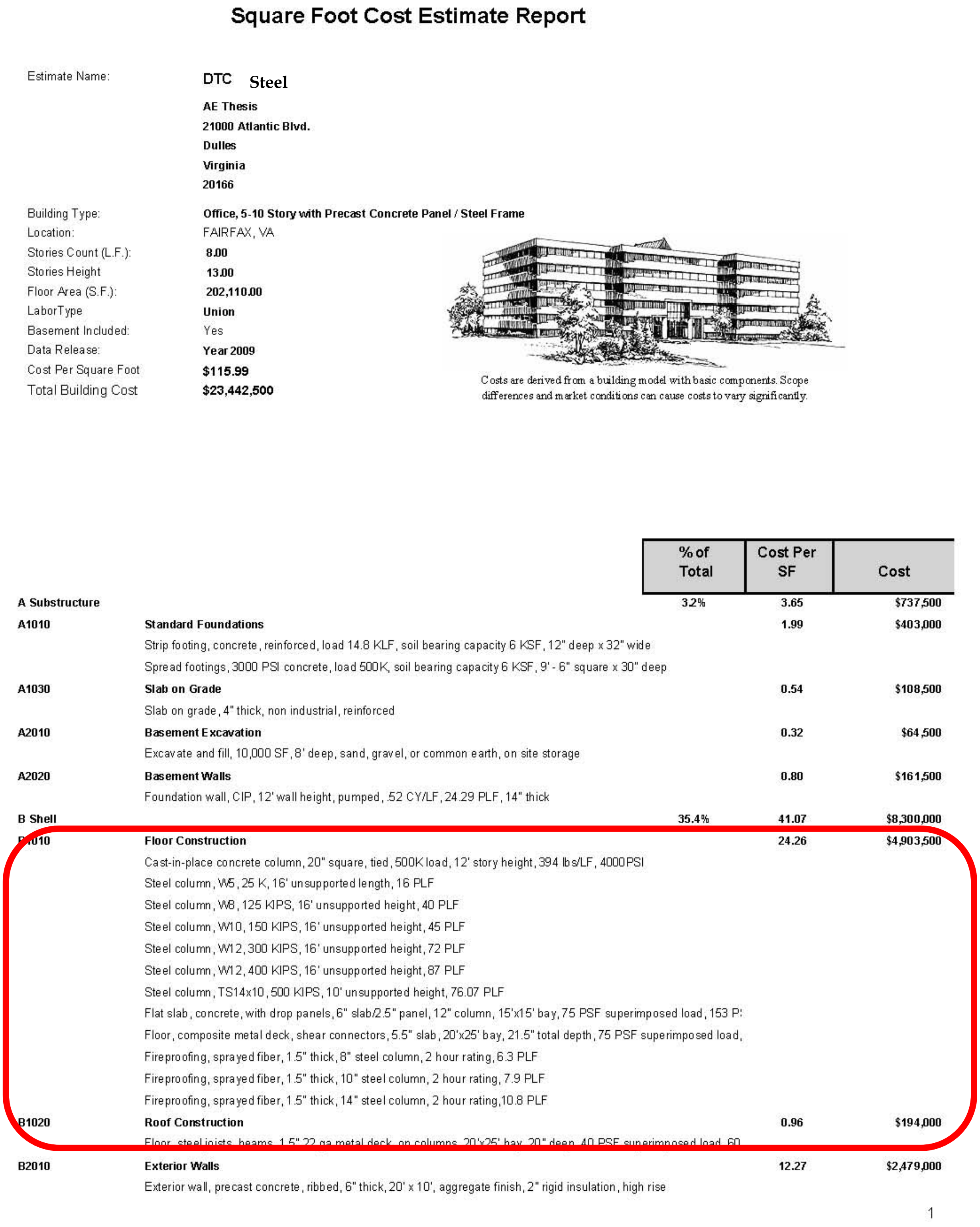
Square Foot Cost Estimate Report

Estimate Name:	DTC Concrete **AE Thesis** **21000 Atlantic Blvd.** **Dulles** **Virginia** **20166**
Building Type:	**Office, 5-10 Story with Precast Concrete Panel / R/Conc. Frame**
Location:	FAIRFAX, VA
Stories Count (L.F.):	**8.00**
Stories Height	**13.00**
Floor Area (S.F.):	**202,110.00**
LaborType	**Union**
Basement Included:	Yes
Data Release:	**Year 2009**
Cost Per Square Foot	**$111.69**
Total Building Cost	**$22,574,500**

Costs are derived from a building model with basic components. Scope differences and market conditions can cause costs to vary significantly.

		% of Total	Cost Per SF	Cost
A Substructure		3.3%	3.65	$737,500
A1010	**Standard Foundations**		1.99	$403,000
	Strip footing, concrete, reinforced, load 14.8 KLF, soil bearing capacity 6 KSF, 12" deep x 32" wide			
	Spread footings, 3000 PSI concrete, load 500K, soil bearing capacity 6 KSF, 9' - 6" square x 30" deep			
A1030	**Slab on Grade**		0.54	$108,500
	Slab on grade, 4" thick, non industrial, reinforced			
A2010	**Basement Excavation**		0.32	$64,500
	Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on site storage			
A2020	**Basement Walls**		0.80	$161,500
	Foundation wall, CIP, 12' wall height, pumped, .52 CY/LF, 24.29 PLF, 14" thick			
B Shell		32.9%	36.73	$7,423,500
B1010	**Floor Construction**		19.18	$3,876,000
	Cast-in-place concrete column, 20" square, tied, 500K load, 12' story height, 394 lbs/LF, 4000PSI			
	Cast-in-place concrete column, 20" square, tied, 800K load, 12' story height, 394 lbs/LF, 6000PSI			
	Cast-in-place concrete column, 20" square, tied, 900K load, 12' story height, 394 lbs/LF, 6000PSI			
	Cast-in-place concrete column, 20", square, tied, minimum reinforcing, 500K load, 10'-14' story height, 375 lbs/LF, 4			
	Flat slab, concrete, with drop panels, 6" slab/2.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 P			
	Flat plate, concrete, 9" slab, 20" column, 20'x25' bay, 75 PSF superimposed load, 188 PSF total load			
B1020	**Roof Construction**		1.71	$345,000
	Floor, concrete, beam and slab, 20'x25' bay, 40 PSF superimposed load, 18" deep beam, 8.5" slab, 146 PSF total lo			
B2010	**Exterior Walls**		12.27	$2,479,000
	Exterior wall, precast concrete, ribbed, 6" thick, 20' x 10', aggregate finish, 2" rigid insulation, high rise			
B2020	**Exterior Windows**		2.80	$565,500
	Windows, aluminum, sliding, insulated glass, 5' x 3'			
B2030	**Exterior Doors**		0.22	$45,000
	Door, aluminum & glass, with transom, narrow stile, double door, hardware, 6'-0" x 10'-0" opening			
	Door, steel 18 gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening			
B3010	**Roof Coverings**		0.56	$113,000

1



Square Foot Cost Estimate Report

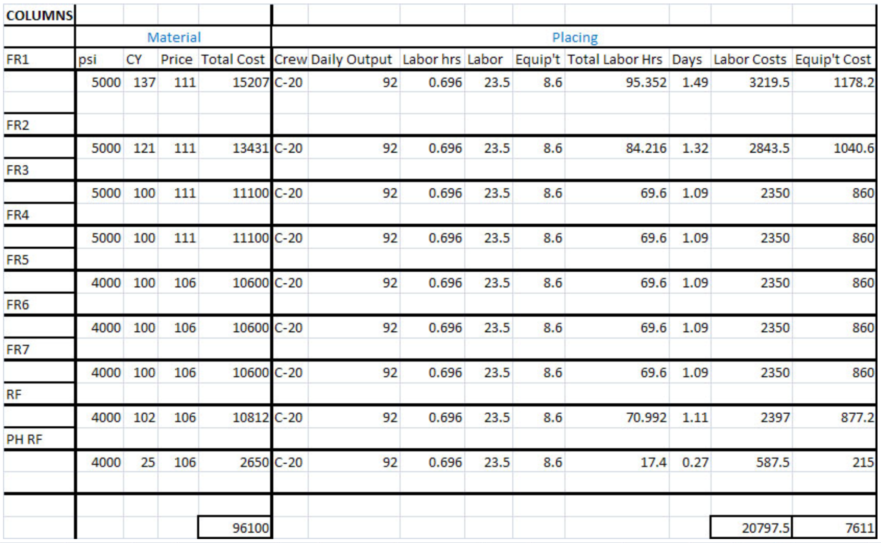
Estimate Name:	DTC Steel **AE Thesis** **21000 Atlantic Blvd.** **Dulles** **Virginia** **20166**
Building Type:	**Office, 5-10 Story with Precast Concrete Panel / Steel Frame**
Location:	FAIRFAX, VA
Stories Count (L.F.):	**8.00**
Stories Height	**13.00**
Floor Area (S.F.):	**202,110.00**
LaborType	**Union**
Basement Included:	Yes
Data Release:	**Year 2009**
Cost Per Square Foot	**$115.99**
Total Building Cost	**$23,442,500**

Costs are derived from a building model with basic components. Scope differences and market conditions can cause costs to vary significantly.

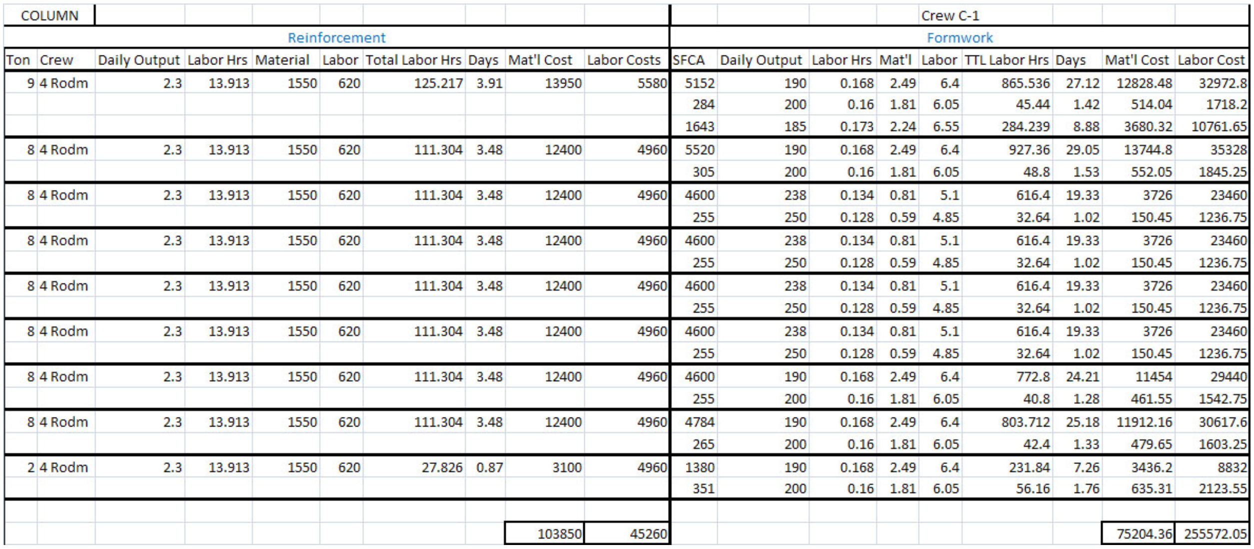
		% of Total	Cost Per SF	Cost
A Substructure		3.2%	3.65	$737,500
A1010	**Standard Foundations**		1.99	$403,000
	Strip footing, concrete, reinforced, load 14.8 KLF, soil bearing capacity 6 KSF, 12" deep x 32" wide			
	Spread footings, 3000 PSI concrete, load 500K, soil bearing capacity 6 KSF, 9' - 6" square x 30" deep			
A1030	**Slab on Grade**		0.54	$108,500
	Slab on grade, 4" thick, non industrial, reinforced			
A2010	**Basement Excavation**		0.32	$64,500
	Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on site storage			
A2020	**Basement Walls**		0.80	$161,500
	Foundation wall, CIP, 12' wall height, pumped, .52 CY/LF, 24.29 PLF, 14" thick			
B Shell		35.4%	41.07	$8,300,000
010	**Floor Construction**		24.26	$4,903,500
	Cast-in-place concrete column, 20" square, tied, 500K load, 12' story height, 394 lbs/LF, 4000PSI			
	Steel column, W5, 25 K, 16' unsupported length, 16 PLF			
	Steel column, W8, 125 KIPS, 16' unsupported height, 40 PLF			
	Steel column, W10, 150 KIPS, 16' unsupported height, 45 PLF			
	Steel column, W12, 300 KIPS, 16' unsupported height, 72 PLF			
	Steel column, W12, 400 KIPS, 16' unsupported height, 87 PLF			
	Steel column, TS14x10, 500 KIPS, 10' unsupported height, 76.07 PLF			
	Flat slab, concrete, with drop panels, 6" slab/2.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 P			
	Floor, composite metal deck, shear connectors, 5.5" slab, 20'x25' bay, 21.5" total depth, 75 PSF superimposed load,			
	Fireproofing, sprayed fiber, 1.5" thick, 8" steel column, 2 hour rating, 6.3 PLF			
	Fireproofing, sprayed fiber, 1.5" thick, 10" steel column, 2 hour rating, 7.9 PLF			
	Fireproofing, sprayed fiber, 1.5" thick, 14" steel column, 2 hour rating, 10.8 PLF			
B1020	**Roof Construction**		0.96	$194,000
	Floor, steel joists, beams, 1.5" 22 ga metal deck, on columns, 20'x25' bay, 20" deep, 40 PSF superimposed load, 60			
B2010	**Exterior Walls**		12.27	$2,479,000
	Exterior wall, precast concrete, ribbed, 6" thick, 20' x 10', aggregate finish, 2" rigid insulation, high rise			

1

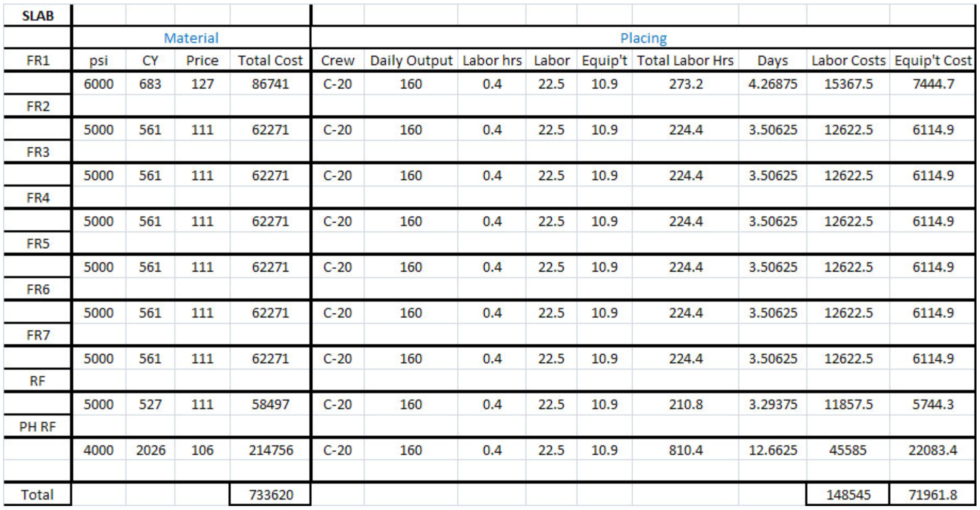
Pricing Tables - Concrete



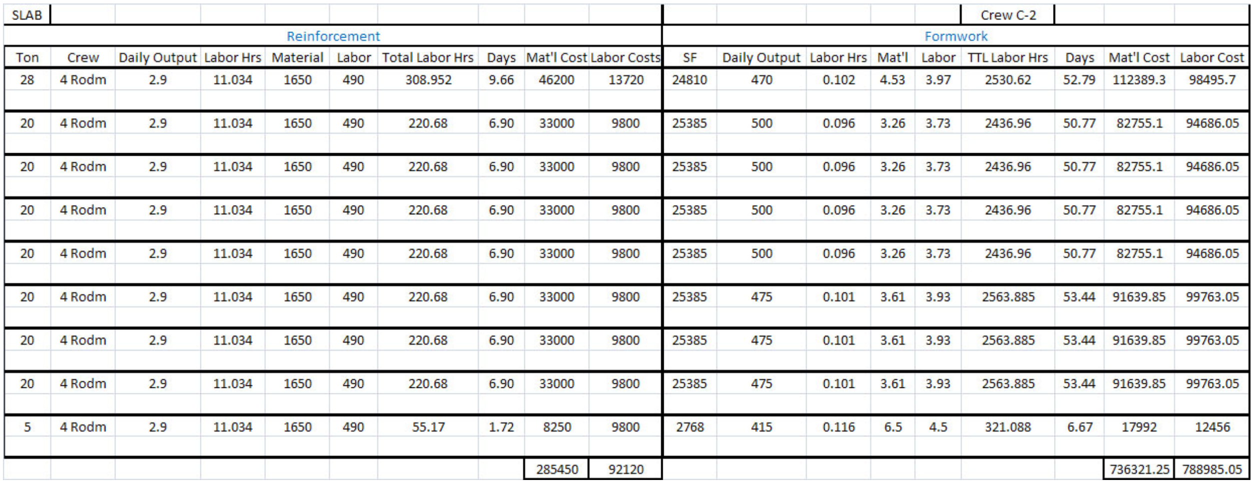
COLUMNS													
	Material				Placing								
FR1	psi	CY	Price	Total Cost	Crew	Daily Output	Labor hrs	Labor	Equip't	Total Labor Hrs	Days	Labor Costs	Equip't Cost
	5000	137	111	15207	C-20	92	0.696	23.5	8.6	95.352	1.49	3219.5	1178.2
FR2													
	5000	121	111	13431	C-20	92	0.696	23.5	8.6	84.216	1.32	2843.5	1040.6
FR3													
	5000	100	111	11100	C-20	92	0.696	23.5	8.6	69.6	1.09	2350	860
FR4													
	5000	100	111	11100	C-20	92	0.696	23.5	8.6	69.6	1.09	2350	860
FR5													
	4000	100	106	10600	C-20	92	0.696	23.5	8.6	69.6	1.09	2350	860
FR6													
	4000	100	106	10600	C-20	92	0.696	23.5	8.6	69.6	1.09	2350	860
FR7													
	4000	100	106	10600	C-20	92	0.696	23.5	8.6	69.6	1.09	2350	860
RF													
	4000	102	106	10812	C-20	92	0.696	23.5	8.6	70.992	1.11	2397	877.2
PH RF													
	4000	25	106	2650	C-20	92	0.696	23.5	8.6	17.4	0.27	587.5	215
				96100								20797.5	7611



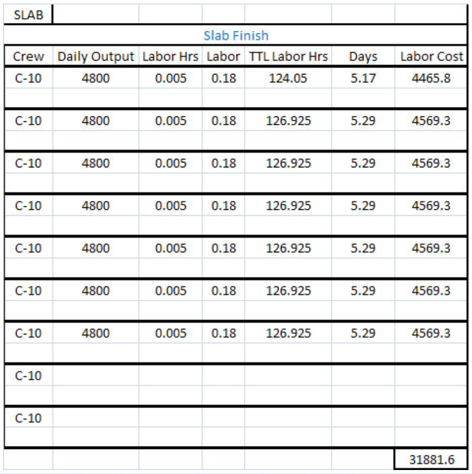
COLUMN																		
															Crew C-1			
Reinforcement										Formwork								
Ton	Crew	Daily Output	Labor Hrs	Material	Labor	Total Labor Hrs	Days	Mat'l Cost	Labor Costs	SFCA	Daily Output	Labor Hrs	Mat'l	Labor	TTL Labor Hrs	Days	Mat'l Cost	Labor Cost
9	4 Rodm	2.3	13.913	1550	620	125.217	3.91	13950	5580	5152	190	0.168	2.49	6.4	865.536	27.12	12828.48	32972.8
										284	200	0.16	1.81	6.05	45.44	1.42	514.04	1718.2
										1643	185	0.173	2.24	6.55	284.239	8.88	3680.32	10761.65
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	5520	190	0.168	2.49	6.4	927.36	29.05	13744.8	35328
										305	200	0.16	1.81	6.05	48.8	1.53	552.05	1845.25
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	4600	238	0.134	0.81	5.1	616.4	19.33	3726	23460
										255	250	0.128	0.59	4.85	32.64	1.02	150.45	1236.75
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	4600	238	0.134	0.81	5.1	616.4	19.33	3726	23460
										255	250	0.128	0.59	4.85	32.64	1.02	150.45	1236.75
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	4600	238	0.134	0.81	5.1	616.4	19.33	3726	23460
										255	250	0.128	0.59	4.85	32.64	1.02	150.45	1236.75
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	4600	238	0.134	0.81	5.1	616.4	19.33	3726	23460
										255	250	0.128	0.59	4.85	32.64	1.02	150.45	1236.75
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	4600	190	0.168	2.49	6.4	772.8	24.21	11454	29440
										255	200	0.16	1.81	6.05	40.8	1.28	461.55	1542.75
8	4 Rodm	2.3	13.913	1550	620	111.304	3.48	12400	4960	4784	190	0.168	2.49	6.4	803.712	25.18	11912.16	30617.6
										265	200	0.16	1.81	6.05	42.4	1.33	479.65	1603.25
2	4 Rodm	2.3	13.913	1550	620	27.826	0.87	3100	4960	1380	190	0.168	2.49	6.4	231.84	7.26	3436.2	8832
										351	200	0.16	1.81	6.05	56.16	1.76	635.31	2123.55
								103850	45260								75204.36	255572.05



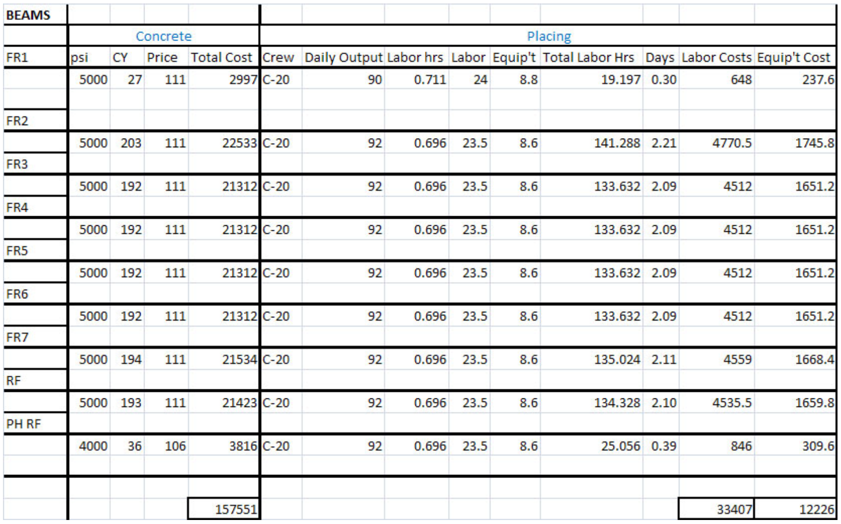
SLAB													
	Material				Placing								
FR1	psi	CY	Price	Total Cost	Crew	Daily Output	Labor hrs	Labor	Equip't	Total Labor Hrs	Days	Labor Costs	Equip't Cost
	6000	683	127	86741	C-20	160	0.4	22.5	10.9	273.2	4.26875	15367.5	7444.7
FR2													
	5000	561	111	62271	C-20	160	0.4	22.5	10.9	224.4	3.50625	12622.5	6114.9
FR3													
	5000	561	111	62271	C-20	160	0.4	22.5	10.9	224.4	3.50625	12622.5	6114.9
FR4													
	5000	561	111	62271	C-20	160	0.4	22.5	10.9	224.4	3.50625	12622.5	6114.9
FR5													
	5000	561	111	62271	C-20	160	0.4	22.5	10.9	224.4	3.50625	12622.5	6114.9
FR6													
	5000	561	111	62271	C-20	160	0.4	22.5	10.9	224.4	3.50625	12622.5	6114.9
FR7													
	5000	561	111	62271	C-20	160	0.4	22.5	10.9	224.4	3.50625	12622.5	6114.9
RF													
	5000	527	111	58497	C-20	160	0.4	22.5	10.9	210.8	3.29375	11857.5	5744.3
PH RF													
	4000	2026	106	214756	C-20	160	0.4	22.5	10.9	810.4	12.6625	45585	22083.4
Total				733620								148545	71961.8



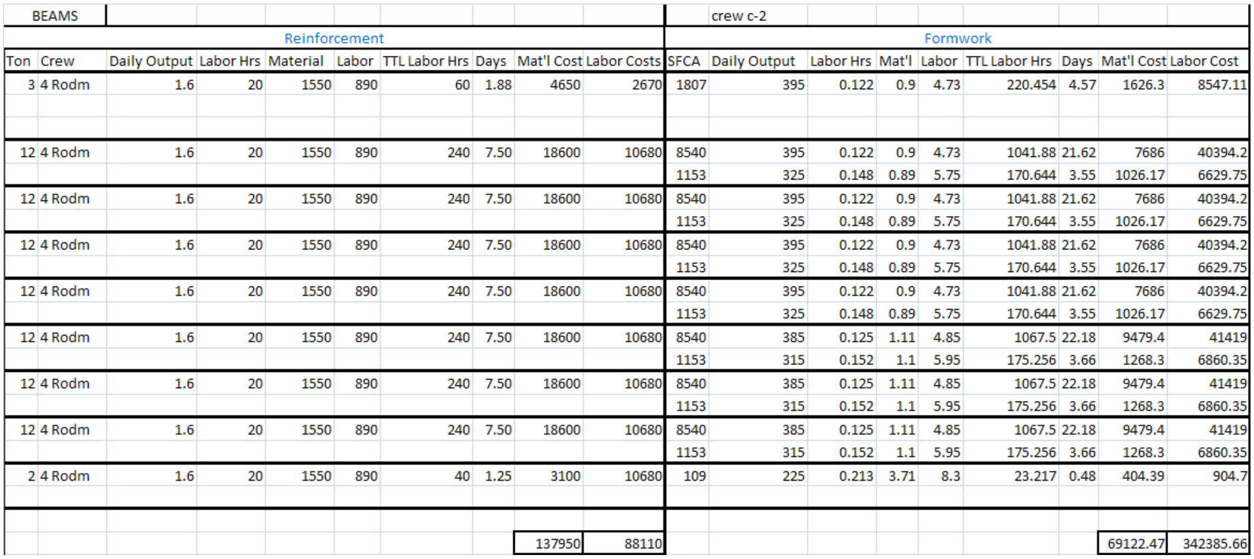
SLAB															Crew C-2			
				Reinforcement										Formwork				
Ton	Crew	Daily Output	Labor Hrs	Material	Labor	Total Labor Hrs	Days	Mat'l Cost	Labor Costs	SF	Daily Output	Labor Hrs	Mat'l	Labor	TTL Labor Hrs	Days	Mat'l Cost	Labor Cost
28	4 Rodm	2.9	11.034	1650	490	308.952	9.66	46200	13720	24810	470	0.102	4.53	3.97	2530.62	52.79	112389.3	98495.7
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	500	0.096	3.26	3.73	2436.96	50.77	82755.1	94686.05
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	500	0.096	3.26	3.73	2436.96	50.77	82755.1	94686.05
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	500	0.096	3.26	3.73	2436.96	50.77	82755.1	94686.05
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	500	0.096	3.26	3.73	2436.96	50.77	82755.1	94686.05
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	475	0.101	3.61	3.93	2563.885	53.44	91639.85	99763.05
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	475	0.101	3.61	3.93	2563.885	53.44	91639.85	99763.05
20	4 Rodm	2.9	11.034	1650	490	220.68	6.90	33000	9800	25385	475	0.101	3.61	3.93	2563.885	53.44	91639.85	99763.05
5	4 Rodm	2.9	11.034	1650	490	55.17	1.72	8250	9800	2768	415	0.116	6.5	4.5	321.088	6.67	17992	12456
								285450	92120								736321.25	788985.05



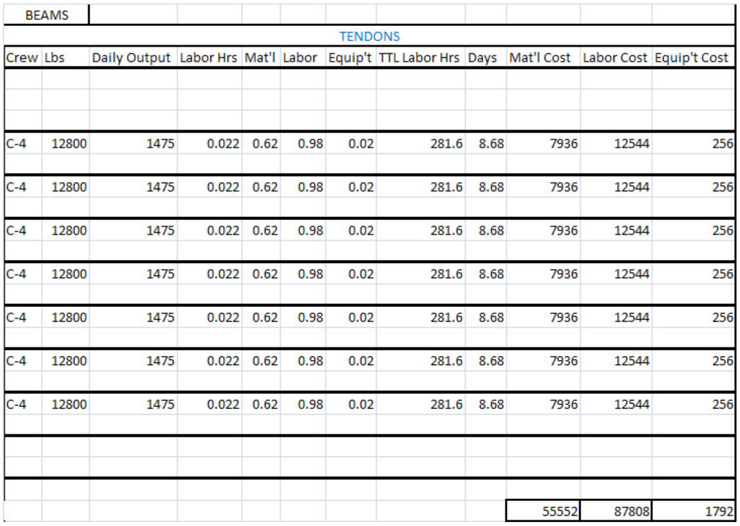
SLAB						
			Slab Finish			
Crew	Daily Output	Labor Hrs	Labor	TTL Labor Hrs	Days	Labor Cost
C-10	4800	0.005	0.18	124.05	5.17	4465.8
C-10	4800	0.005	0.18	126.925	5.29	4569.3
C-10	4800	0.005	0.18	126.925	5.29	4569.3
C-10	4800	0.005	0.18	126.925	5.29	4569.3
C-10	4800	0.005	0.18	126.925	5.29	4569.3
C-10	4800	0.005	0.18	126.925	5.29	4569.3
C-10	4800	0.005	0.18	126.925	5.29	4569.3
C-10						
C-10						
						31881.6



BEAMS													
		Concrete						Placing					
FR1	psi	CY	Price	Total Cost	Crew	Daily Output	Labor hrs	Labor	Equip't	Total Labor Hrs	Days	Labor Costs	Equip't Cost
	5000	27	111	2997	C-20	90	0.711	24	8.8	19.197	0.30	648	237.6
FR2													
	5000	203	111	22533	C-20	92	0.696	23.5	8.6	141.288	2.21	4770.5	1745.8
FR3													
	5000	192	111	21312	C-20	92	0.696	23.5	8.6	133.632	2.09	4512	1651.2
FR4													
	5000	192	111	21312	C-20	92	0.696	23.5	8.6	133.632	2.09	4512	1651.2
FR5													
	5000	192	111	21312	C-20	92	0.696	23.5	8.6	133.632	2.09	4512	1651.2
FR6													
	5000	192	111	21312	C-20	92	0.696	23.5	8.6	133.632	2.09	4512	1651.2
FR7													
	5000	194	111	21534	C-20	92	0.696	23.5	8.6	135.024	2.11	4559	1668.4
RF													
	5000	193	111	21423	C-20	92	0.696	23.5	8.6	134.328	2.10	4535.5	1659.8
PH RF													
	4000	36	106	3816	C-20	92	0.696	23.5	8.6	25.056	0.39	846	309.6
				157551								33407	12226

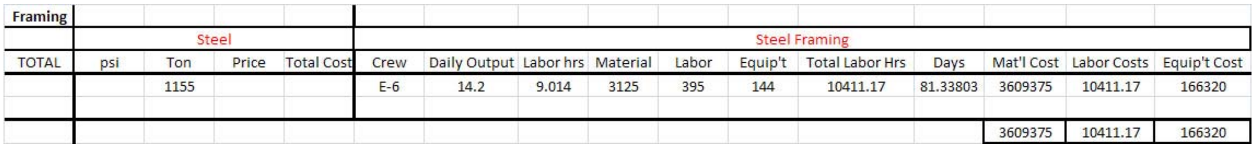


BEAMS											crew c-2							
				Reinforcement										Formwork				
Ton	Crew	Daily Output	Labor Hrs	Material	Labor	TTL Labor Hrs	Days	Mat'l Cost	Labor Costs	SFCA	Daily Output	Labor Hrs	Mat'l	Labor	TTL Labor Hrs	Days	Mat'l Cost	Labor Cost
3	4 Rodm	1.6	20	1550	890	60	1.88	4650	2670	1807	395	0.122	0.9	4.73	220.454	4.57	1626.3	8547.11
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	395	0.122	0.9	4.73	1041.88	21.62	7686	40394.2
										1153	325	0.148	0.89	5.75	170.644	3.55	1026.17	6629.75
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	395	0.122	0.9	4.73	1041.88	21.62	7686	40394.2
										1153	325	0.148	0.89	5.75	170.644	3.55	1026.17	6629.75
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	395	0.122	0.9	4.73	1041.88	21.62	7686	40394.2
										1153	325	0.148	0.89	5.75	170.644	3.55	1026.17	6629.75
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	395	0.122	0.9	4.73	1041.88	21.62	7686	40394.2
										1153	325	0.148	0.89	5.75	170.644	3.55	1026.17	6629.75
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	385	0.125	1.11	4.85	1067.5	22.18	9479.4	41419
										1153	315	0.152	1.1	5.95	175.256	3.66	1268.3	6860.35
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	385	0.125	1.11	4.85	1067.5	22.18	9479.4	41419
										1153	315	0.152	1.1	5.95	175.256	3.66	1268.3	6860.35
12	4 Rodm	1.6	20	1550	890	240	7.50	18600	10680	8540	385	0.125	1.11	4.85	1067.5	22.18	9479.4	41419
										1153	315	0.152	1.1	5.95	175.256	3.66	1268.3	6860.35
2	4 Rodm	1.6	20	1550	890	40	1.25	3100	10680	109	225	0.213	3.71	8.3	23.217	0.48	404.39	904.7
								137950	88110								69122.47	342385.66

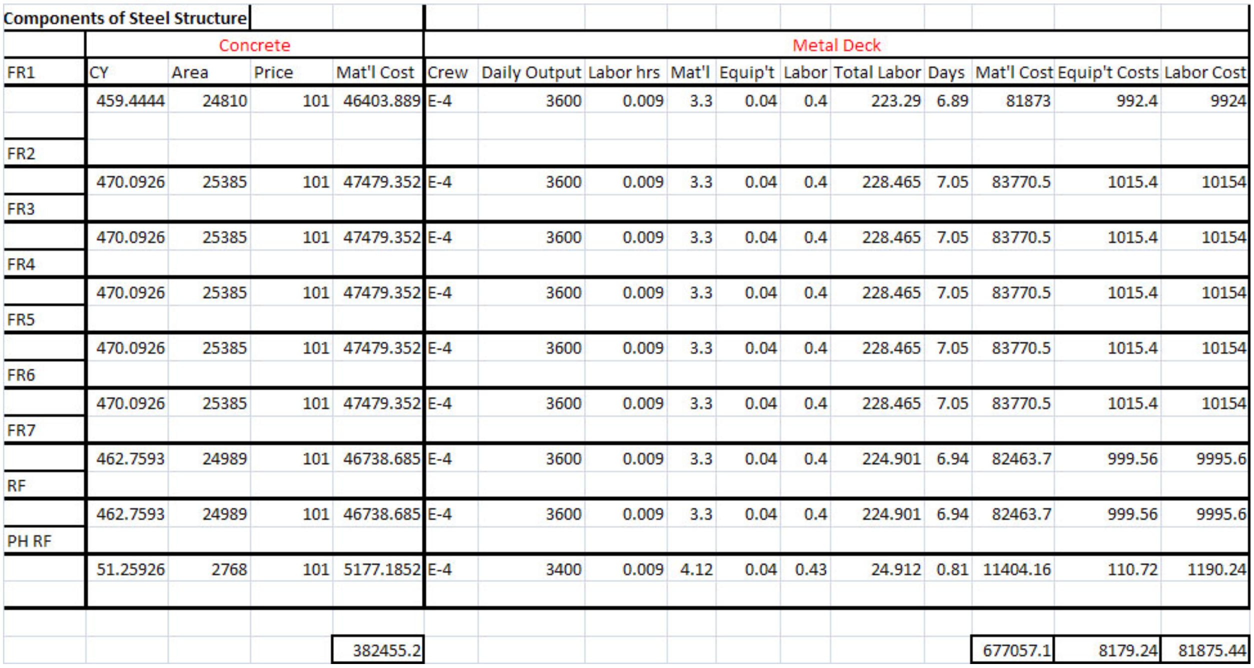


BEAMS											
						TENDONS					
Crew	Lbs	Daily Output	Labor Hrs	Mat'l	Labor	Equip't	TTL Labor Hrs	Days	Mat'l Cost	Labor Cost	Equip't Cost
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
C-4	12800	1475	0.022	0.62	0.98	0.02	281.6	8.68	7936	12544	256
									55552	87808	1792

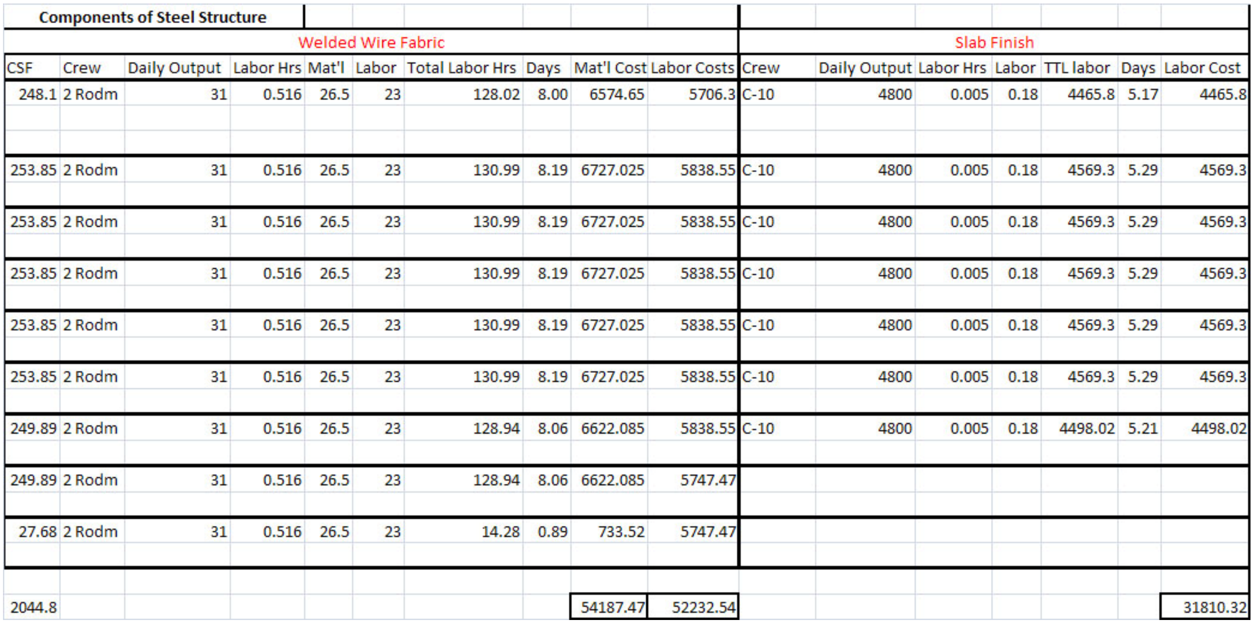
Pricing Tables - Steel



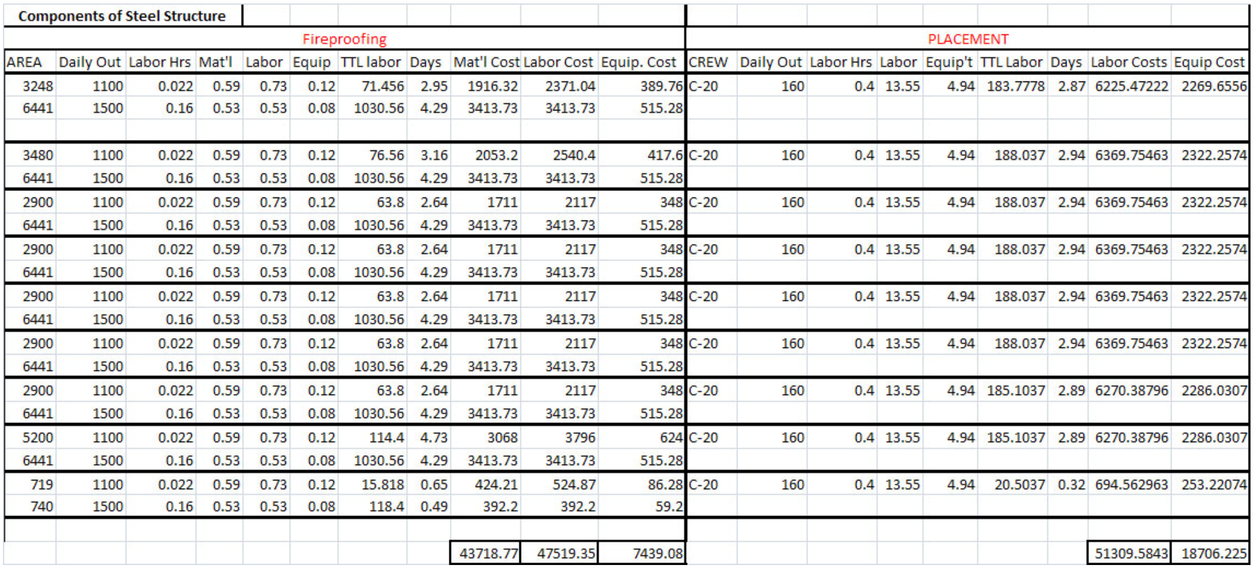
Framing															
	Steel				Steel Framing										
TOTAL	psi	Ton	Price	Total Cost	Crew	Daily Output	Labor hrs	Material	Labor	Equip't	Total Labor Hrs	Days	Mat'l Cost	Labor Costs	Equip't Cost
		1155			E-6	14.2	9.014	3125	395	144	10411.17	81.33803	3609375	10411.17	166320
													3609375	10411.17	166320



Components of Steel Structure															
	Concrete				Metal Deck										
FR1	CY	Area	Price	Mat'l Cost	Crew	Daily Output	Labor hrs	Mat'l	Equip't	Labor	Total Labor	Days	Mat'l Cost	Equip't Costs	Labor Cost
	459.4444	24810	101	46403.889	E-4	3600	0.009	3.3	0.04	0.4	223.29	6.89	81873	992.4	9924
FR2															
	470.0926	25385	101	47479.352	E-4	3600	0.009	3.3	0.04	0.4	228.465	7.05	83770.5	1015.4	10154
FR3															
	470.0926	25385	101	47479.352	E-4	3600	0.009	3.3	0.04	0.4	228.465	7.05	83770.5	1015.4	10154
FR4															
	470.0926	25385	101	47479.352	E-4	3600	0.009	3.3	0.04	0.4	228.465	7.05	83770.5	1015.4	10154
FR5															
	470.0926	25385	101	47479.352	E-4	3600	0.009	3.3	0.04	0.4	228.465	7.05	83770.5	1015.4	10154
FR6															
	470.0926	25385	101	47479.352	E-4	3600	0.009	3.3	0.04	0.4	228.465	7.05	83770.5	1015.4	10154
FR7															
	462.7593	24989	101	46738.685	E-4	3600	0.009	3.3	0.04	0.4	224.901	6.94	82463.7	999.56	9995.6
RF															
	462.7593	24989	101	46738.685	E-4	3600	0.009	3.3	0.04	0.4	224.901	6.94	82463.7	999.56	9995.6
PH RF															
	51.25926	2768	101	5177.1852	E-4	3400	0.009	4.12	0.04	0.43	24.912	0.81	11404.16	110.72	1190.24
				382455.2									677057.1	8179.24	81875.44



Components of Steel Structure																
				Welded Wire Fabric						Slab Finish						
CSF	Crew	Daily Output	Labor Hrs	Mat'l	Labor	Total Labor Hrs	Days	Mat'l Cost	Labor Costs	Crew	Daily Output	Labor Hrs	Labor	TTL labor	Days	Labor Cost
248.1	2 Rodm	31	0.516	26.5	23	128.02	8.00	6574.65	5706.3	C-10	4800	0.005	0.18	4465.8	5.17	4465.8
253.85	2 Rodm	31	0.516	26.5	23	130.99	8.19	6727.025	5838.55	C-10	4800	0.005	0.18	4569.3	5.29	4569.3
253.85	2 Rodm	31	0.516	26.5	23	130.99	8.19	6727.025	5838.55	C-10	4800	0.005	0.18	4569.3	5.29	4569.3
253.85	2 Rodm	31	0.516	26.5	23	130.99	8.19	6727.025	5838.55	C-10	4800	0.005	0.18	4569.3	5.29	4569.3
253.85	2 Rodm	31	0.516	26.5	23	130.99	8.19	6727.025	5838.55	C-10	4800	0.005	0.18	4569.3	5.29	4569.3
253.85	2 Rodm	31	0.516	26.5	23	130.99	8.19	6727.025	5838.55	C-10	4800	0.005	0.18	4569.3	5.29	4569.3
249.89	2 Rodm	31	0.516	26.5	23	128.94	8.06	6622.085	5838.55	C-10	4800	0.005	0.18	4498.02	5.21	4498.02
249.89	2 Rodm	31	0.516	26.5	23	128.94	8.06	6622.085	5747.47							
27.68	2 Rodm	31	0.516	26.5	23	14.28	0.89	733.52	5747.47							
2044.8								54187.47	52232.54							31810.32



Components of Steel Structure																				
Fireproofing											PLACEMENT									
AREA	Daily Out	Labor Hrs	Mat'l	Labor	Equip	TTL labor	Days	Mat'l Cost	Labor Cost	Equip. Cost	CREW	Daily Out	Labor Hrs	Labor	Equip't	TTL Labor	Days	Labor Costs	Equip Cost
3248	1100	0.022	0.59	0.73	0.12	71.456	2.95	1916.32	2371.04	389.76	C-20	160	0.4	13.55	4.94	183.7778	2.87	6225.47222	2269.6556
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
3480	1100	0.022	0.59	0.73	0.12	76.56	3.16	2053.2	2540.4	417.6	C-20	160	0.4	13.55	4.94	188.037	2.94	6369.75463	2322.2574
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
2900	1100	0.022	0.59	0.73	0.12	63.8	2.64	1711	2117	348	C-20	160	0.4	13.55	4.94	188.037	2.94	6369.75463	2322.2574
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
2900	1100	0.022	0.59	0.73	0.12	63.8	2.64	1711	2117	348	C-20	160	0.4	13.55	4.94	188.037	2.94	6369.75463	2322.2574
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
2900	1100	0.022	0.59	0.73	0.12	63.8	2.64	1711	2117	348	C-20	160	0.4	13.55	4.94	188.037	2.94	6369.75463	2322.2574
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
2900	1100	0.022	0.59	0.73	0.12	63.8	2.64	1711	2117	348	C-20	160	0.4	13.55	4.94	188.037	2.94	6369.75463	2322.2574
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
2900	1100	0.022	0.59	0.73	0.12	63.8	2.64	1711	2117	348	C-20	160	0.4	13.55	4.94	185.1037	2.89	6270.38796	2286.0307
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
5200	1100	0.022	0.59	0.73	0.12	114.4	4.73	3068	3796	624	C-20	160	0.4	13.55	4.94	185.1037	2.89	6270.38796	2286.0307
6441	1500	0.16	0.53	0.53	0.08	1030.56	4.29	3413.73	3413.73	515.28									
719	1100	0.022	0.59	0.73	0.12	15.818	0.65	424.21	524.87	86.28	C-20	160	0.4	13.55	4.94	20.5037	0.32	694.562963	253.22074
740	1500	0.16	0.53	0.53	0.08	118.4	0.49	392.2	392.2	59.2									
								43718.77	47519.35	7439.08								51309.5843	18706.225

Frame Takeoff

TOTAL STRUCTURE FRAME TAKEOFF

Floor Area (ft**2): 170585.7

Columns:

Wide Flange:

Steel Grade: 50

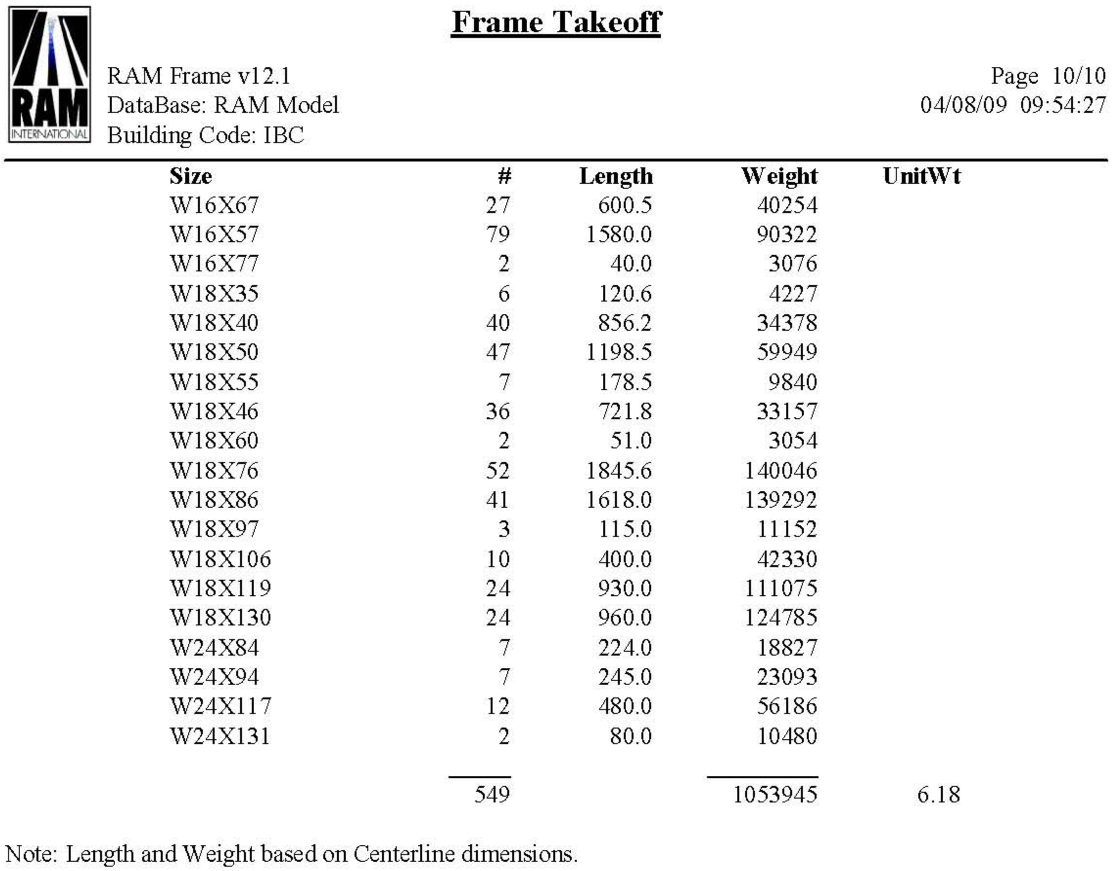
Size	#	Length ft	Weight lbs	UnitWt psf
W14X43	2	26.0	1115	
W14X48	4	50.0	2399	
W14X61	16	248.5	15136	
W14X90	4	52.5	4734	
W14X68	6	76.0	5172	
W14X99	22	280.0	27725	
W14X74	56	726.0	53854	
W14X82	16	200.0	16333	
W14X109	49	617.5	67238	
W14X120	31	402.5	48347	
W14X132	36	452.5	59742	
W14X145	43	580.0	84272	
W14X159	15	202.5	32179	
W14X176	10	135.0	23795	
W14X193	14	177.5	34306	
W14X211	4	50.0	10548	
W14X233	3	42.5	9906	
W14X257	7	92.5	23795	
W14X311	6	77.5	24103	
W14X342	2	27.5	9451	
	346		554150	3.25

Beams:

Wide Flange:

Steel Grade: 50

Size	#	Length ft	Weight lbs	UnitWt psf
W8X35	2	51.0	1787	
W14X43	4	80.0	3430	
W16X31	1	20.0	621	
W16X36	23	460.0	16592	
W16X40	72	1440.6	57843	
W16X45	9	180.0	8146	
W16X50	10	200.0	10004	



Frame Takeoff

RAM Frame v12.1
DataBase: RAM Model
Building Code: IBC

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04/08/09 09:54:27

Size	#	Length	Weight	UnitWt
W16X67	27	600.5	40254	
W16X57	79	1580.0	90322	
W16X77	2	40.0	3076	
W18X35	6	120.6	4227	
W18X40	40	856.2	34378	
W18X50	47	1198.5	59949	
W18X55	7	178.5	9840	
W18X46	36	721.8	33157	
W18X60	2	51.0	3054	
W18X76	52	1845.6	140046	
W18X86	41	1618.0	139292	
W18X97	3	115.0	11152	
W18X106	10	400.0	42330	
W18X119	24	930.0	111075	
W18X130	24	960.0	124785	
W24X84	7	224.0	18827	
W24X94	7	245.0	23093	
W24X117	12	480.0	56186	
W24X131	2	80.0	10480	
	549		1053945	6.18

Note: Length and Weight based on Centerline dimensions.

Gravity Beam Design Takeoff

RAM Steel v12.1
DataBase: RAM Model
Building Code: IBC

04/08/09 09:54:27
Steel Code: AISC LRFD

STEEL BEAM DESIGN TAKEOFF:

Floor Type: RF
Story Level 7
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W16X26	13	331.81	8671
W18X35	7	235.50	8254
W18X40	4	146.50	5882
W18X50	17	677.00	33864
	-----		-------------
	41		**56672**

Total Number of Studs = **672**

Floor Type: TYP
Story Levels 1 to 6
Steel Grade: 50

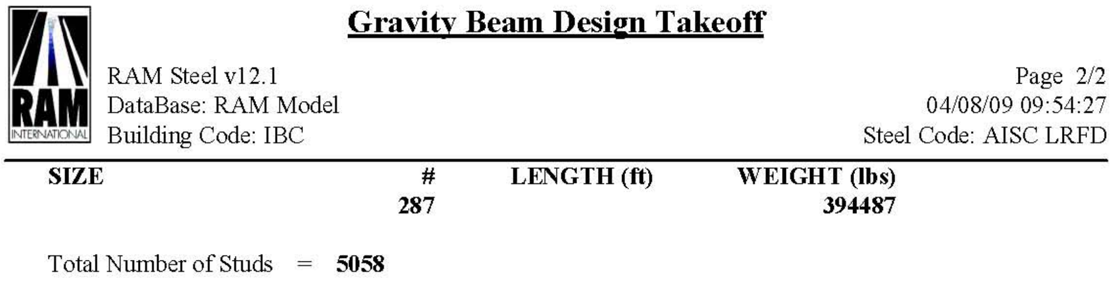
SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W16X26	13	331.81	8671
W18X35	7	235.50	8254
W18X40	4	146.50	5882
W16X45	2	77.50	3507
W16X50	15	599.50	29987
	-----		-------------
	41		**56303**

Total Number of Studs = **731**

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W16X26	91	2322.68	60699
W16X45	12	465.00	21044
W16X50	90	3597.00	179925
W18X35	49	1648.50	57778
W18X40	28	1025.50	41177
W18X50	17	677.00	33864
	-----		-------------



Gravity Beam Design Takeoff

RAM Steel v12.1
DataBase: RAM Model
Building Code: IBC

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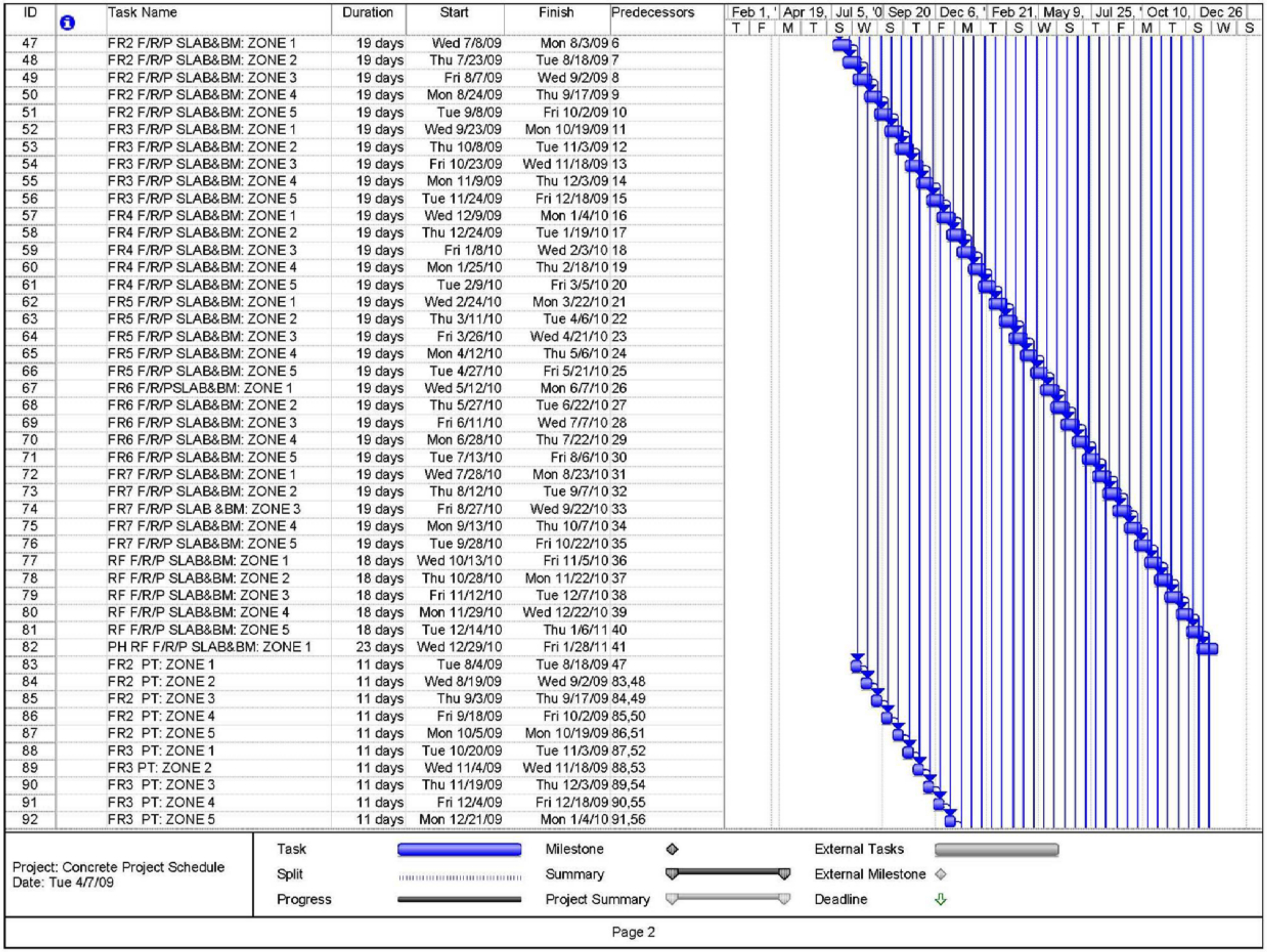
04/08/09 09:54:27
Steel Code: AISC LRFD

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
	287		**394487**

Total Number of Studs = **5058**

Scheduling - Concrete



ID	Task Name	Duration	Start	Finish	Predecessors
47	FR2 F/R/P SLAB&BM: ZONE 1	19 days	Wed 7/8/09	Mon 8/3/09	6
48	FR2 F/R/P SLAB&BM: ZONE 2	19 days	Thu 7/23/09	Tue 8/18/09	7
49	FR2 F/R/P SLAB&BM: ZONE 3	19 days	Fri 8/7/09	Wed 9/2/09	8
50	FR2 F/R/P SLAB&BM: ZONE 4	19 days	Mon 8/24/09	Thu 9/17/09	9
51	FR2 F/R/P SLAB&BM: ZONE 5	19 days	Tue 9/8/09	Fri 10/2/09	10
52	FR3 F/R/P SLAB&BM: ZONE 1	19 days	Wed 9/23/09	Mon 10/19/09	11
53	FR3 F/R/P SLAB&BM: ZONE 2	19 days	Thu 10/8/09	Tue 11/3/09	12
54	FR3 F/R/P SLAB&BM: ZONE 3	19 days	Fri 10/23/09	Wed 11/18/09	13
55	FR3 F/R/P SLAB&BM: ZONE 4	19 days	Mon 11/9/09	Thu 12/3/09	14
56	FR3 F/R/P SLAB&BM: ZONE 5	19 days	Tue 11/24/09	Fri 12/18/09	15
57	FR4 F/R/P SLAB&BM: ZONE 1	19 days	Wed 12/9/09	Mon 1/4/10	16
58	FR4 F/R/P SLAB&BM: ZONE 2	19 days	Thu 12/24/09	Tue 1/19/10	17
59	FR4 F/R/P SLAB&BM: ZONE 3	19 days	Fri 1/8/10	Wed 2/3/10	18
60	FR4 F/R/P SLAB&BM: ZONE 4	19 days	Mon 1/25/10	Thu 2/18/10	19
61	FR4 F/R/P SLAB&BM: ZONE 5	19 days	Tue 2/9/10	Fri 3/5/10	20
62	FR5 F/R/P SLAB&BM: ZONE 1	19 days	Wed 2/24/10	Mon 3/22/10	21
63	FR5 F/R/P SLAB&BM: ZONE 2	19 days	Thu 3/11/10	Tue 4/6/10	22
64	FR5 F/R/P SLAB&BM: ZONE 3	19 days	Fri 3/26/10	Wed 4/21/10	23
65	FR5 F/R/P SLAB&BM: ZONE 4	19 days	Mon 4/12/10	Thu 5/6/10	24
66	FR5 F/R/P SLAB&BM: ZONE 5	19 days	Tue 4/27/10	Fri 5/21/10	25
67	FR6 F/R/PSLAB&BM: ZONE 1	19 days	Wed 5/12/10	Mon 6/7/10	26
68	FR6 F/R/P SLAB&BM: ZONE 2	19 days	Thu 5/27/10	Tue 6/22/10	27
69	FR6 F/R/P SLAB&BM: ZONE 3	19 days	Fri 6/11/10	Wed 7/7/10	28
70	FR6 F/R/P SLAB&BM: ZONE 4	19 days	Mon 6/28/10	Thu 7/22/10	29
71	FR6 F/R/P SLAB&BM: ZONE 5	19 days	Tue 7/13/10	Fri 8/6/10	30
72	FR7 F/R/P SLAB&BM: ZONE 1	19 days	Wed 7/28/10	Mon 8/23/10	31
73	FR7 F/R/P SLAB&BM: ZONE 2	19 days	Thu 8/12/10	Tue 9/7/10	32
74	FR7 F/R/P SLAB &BM: ZONE 3	19 days	Fri 8/27/10	Wed 9/22/10	33
75	FR7 F/R/P SLAB&BM: ZONE 4	19 days	Mon 9/13/10	Thu 10/7/10	34
76	FR7 F/R/P SLAB&BM: ZONE 5	19 days	Tue 9/28/10	Fri 10/22/10	35
77	RF F/R/P SLAB&BM: ZONE 1	18 days	Wed 10/13/10	Fri 11/5/10	36
78	RF F/R/P SLAB&BM: ZONE 2	18 days	Thu 10/28/10	Mon 11/22/10	37
79	RF F/R/P SLAB&BM: ZONE 3	18 days	Fri 11/12/10	Tue 12/7/10	38
80	RF F/R/P SLAB&BM: ZONE 4	18 days	Mon 11/29/10	Wed 12/22/10	39
81	RF F/R/P SLAB&BM: ZONE 5	18 days	Tue 12/14/10	Thu 1/6/11	40
82	PH RF F/R/P SLAB&BM: ZONE 1	23 days	Wed 12/29/10	Fri 1/28/11	41
83	FR2 PT: ZONE 1	11 days	Tue 8/4/09	Tue 8/18/09	47
84	FR2 PT: ZONE 2	11 days	Wed 8/19/09	Wed 9/2/09	83,48
85	FR2 PT: ZONE 3	11 days	Thu 9/3/09	Thu 9/17/09	84,49
86	FR2 PT: ZONE 4	11 days	Fri 9/18/09	Fri 10/2/09	85,50
87	FR2 PT: ZONE 5	11 days	Mon 10/5/09	Mon 10/19/09	86,51
88	FR3 PT: ZONE 1	11 days	Tue 10/20/09	Tue 11/3/09	87,52
89	FR3 PT: ZONE 2	11 days	Wed 11/4/09	Wed 11/18/09	88,53
90	FR3 PT: ZONE 3	11 days	Thu 11/19/09	Thu 12/3/09	89,54
91	FR3 PT: ZONE 4	11 days	Fri 12/4/09	Fri 12/18/09	90,55
92	FR3 PT: ZONE 5	11 days	Mon 12/21/09	Mon 1/4/10	91,56

Project: Concrete Project Schedule
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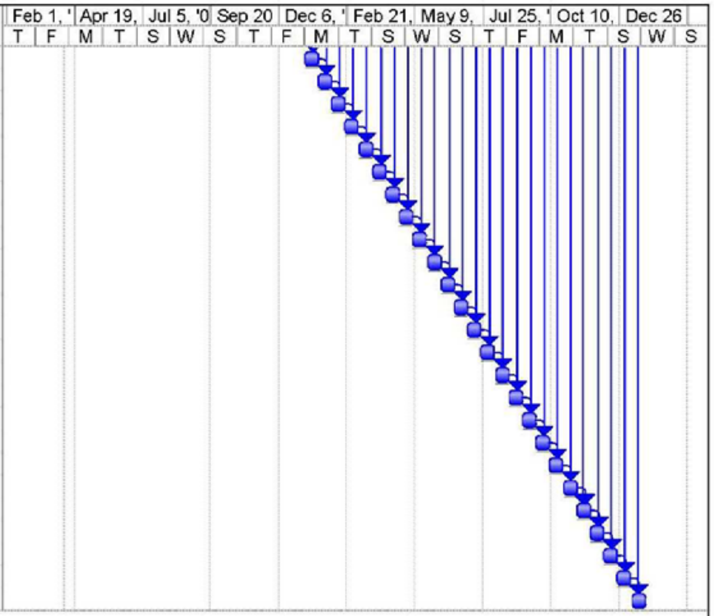
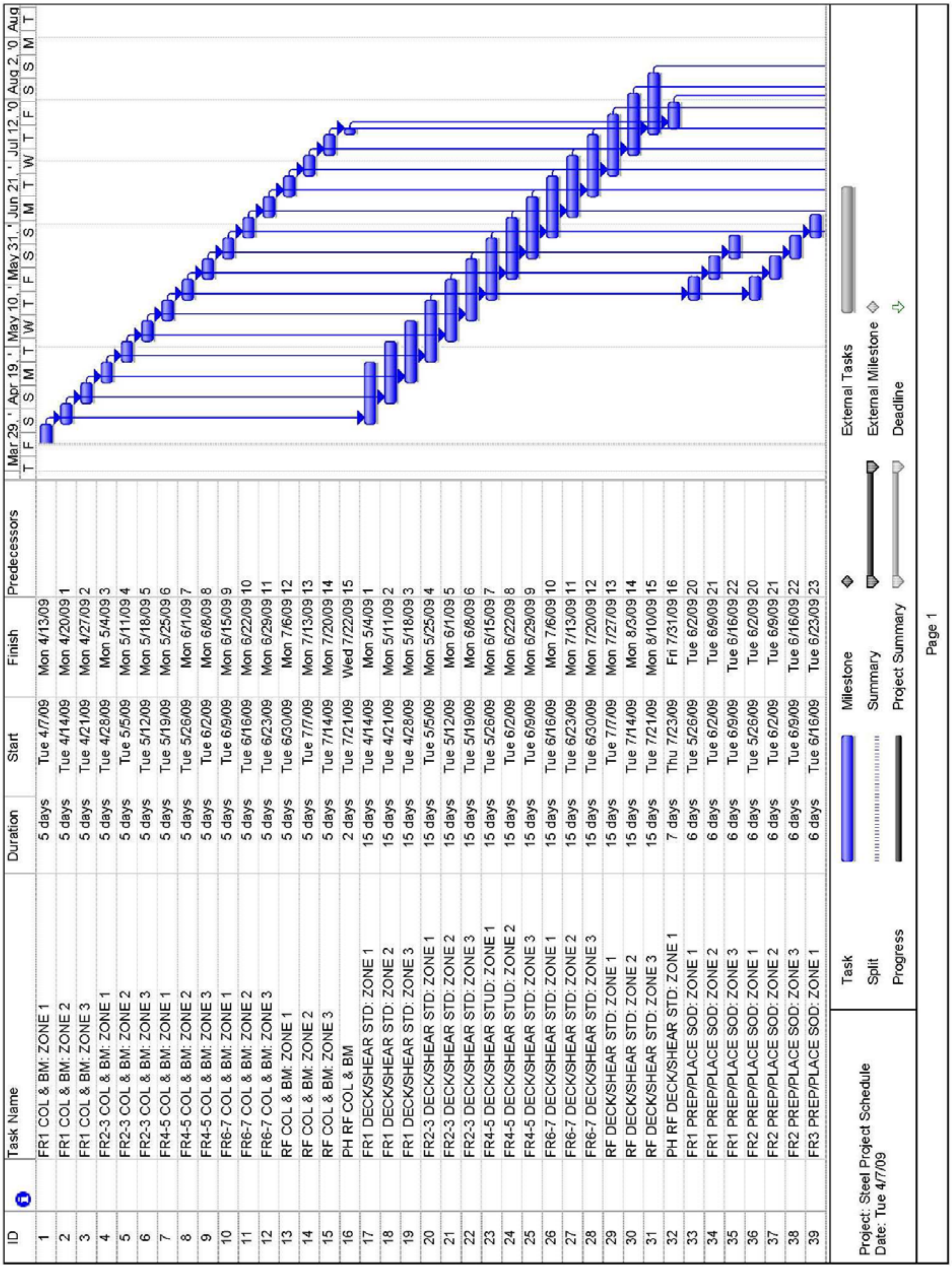
ID	ℹ	Task Name	Duration	Start	Finish	Predecessors
93		FR4 PT: ZONE 1	11 days	Tue 1/5/10	Tue 1/19/10	57,92
94		FR4 PT: ZONE 2	11 days	Wed 1/20/10	Wed 2/3/10	58,93
95		FR4 PT: ZONE 3	11 days	Thu 2/4/10	Thu 2/18/10	59,94
96		FR4 PT: ZONE 4	11 days	Fri 2/19/10	Fri 3/5/10	95,60
97		FR4 PT: ZONE 5	11 days	Mon 3/8/10	Mon 3/22/10	96,61
98		FR5 PT: ZONE 1	11 days	Tue 3/23/10	Tue 4/6/10	97,62
99		FR5 PT: ZONE 2	11 days	Wed 4/7/10	Wed 4/21/10	98,63
100		FR5 PT: ZONE 3	11 days	Thu 4/22/10	Thu 5/6/10	99,64
101		FR5 PT: ZONE 4	11 days	Fri 5/7/10	Fri 5/21/10	100,65
102		FR5 PT: ZONE 5	11 days	Mon 5/24/10	Mon 6/7/10	101,66
103		FR6 PT: ZONE 1	11 days	Tue 6/8/10	Tue 6/22/10	102,67
104		FR6 PT: ZONE 2	11 days	Wed 6/23/10	Wed 7/7/10	103,68
105		FR6 PT: ZONE 3	11 days	Thu 7/8/10	Thu 7/22/10	104,69
106		FR6 PT: ZONE 4	11 days	Fri 7/23/10	Fri 8/6/10	105,70
107		FR6 PT: ZONE 5	11 days	Mon 8/9/10	Mon 8/23/10	106,71
108		FR7 PT: ZONE 1	11 days	Tue 8/24/10	Tue 9/7/10	107,72
109		FR7 PT: ZONE 2	11 days	Wed 9/8/10	Wed 9/22/10	108,73
110		FR7 PT: ZONE 3	11 days	Thu 9/23/10	Thu 10/7/10	109,74
111		FR7 PT: ZONE 4	11 days	Fri 10/8/10	Fri 10/22/10	110,75
112		FR7 PT: ZONE 5	11 days	Mon 10/25/10	Mon 11/8/10	111,76
113		RF PT: ZONE 1	11 days	Tue 11/9/10	Tue 11/23/10	112,77
114		RF PT: ZONE 2	11 days	Wed 11/24/10	Wed 12/8/10	113,78
115		RF PT: ZONE 3	11 days	Thu 12/9/10	Thu 12/23/10	114,79
116		RF PT: ZONE 4	11 days	Fri 12/24/10	Fri 1/7/11	115,80
117		RF PT: ZONE 5	11 days	Mon 1/10/11	Mon 1/24/11	116,81



Project: Concrete Project Schedule
Date: Tue 4/7/09

Task ▬ Milestone ◆ External Tasks ▬
Split ┅ Summary ▬ External Milestone ◇
Progress ▬ Project Summary ▬ Deadline ⇩

Scheduling – Steel



ID	Task Name	Duration	Start	Finish	Predecessors
1	FR1 COL & BM: ZONE 1	5 days	Tue 4/7/09	Mon 4/13/09	
2	FR1 COL & BM: ZONE 2	5 days	Tue 4/14/09	Mon 4/20/09	1
3	FR1 COL & BM: ZONE 3	5 days	Tue 4/21/09	Mon 4/27/09	2
4	FR2-3 COL & BM: ZONE 1	5 days	Tue 4/28/09	Mon 5/4/09	3
5	FR2-3 COL & BM: ZONE 2	5 days	Tue 5/5/09	Mon 5/11/09	4
6	FR2-3 COL & BM: ZONE 3	5 days	Tue 5/12/09	Mon 5/18/09	5
7	FR4-5 COL & BM: ZONE 1	5 days	Tue 5/19/09	Mon 5/25/09	6
8	FR4-5 COL & BM: ZONE 2	5 days	Tue 5/26/09	Mon 6/1/09	7
9	FR4-5 COL & BM: ZONE 3	5 days	Tue 6/2/09	Mon 6/8/09	8
10	FR6-7 COL & BM: ZONE 1	5 days	Tue 6/9/09	Mon 6/15/09	9
11	FR6-7 COL & BM: ZONE 2	5 days	Tue 6/16/09	Mon 6/22/09	10
12	FR6-7 COL & BM: ZONE 3	5 days	Tue 6/23/09	Mon 6/29/09	11
13	RF COL & BM: ZONE 1	5 days	Tue 6/30/09	Mon 7/6/09	12
14	RF COL & BM: ZONE 2	5 days	Tue 7/7/09	Mon 7/13/09	13
15	RF COL & BM: ZONE 3	5 days	Tue 7/14/09	Mon 7/20/09	14
16	PH RF COL & BM	2 days	Tue 7/21/09	Wed 7/22/09	15
17	FR1 DECK/SHEAR STD: ZONE 1	15 days	Tue 4/14/09	Mon 5/4/09	1
18	FR1 DECK/SHEAR STD: ZONE 2	15 days	Tue 4/21/09	Mon 5/11/09	2
19	FR1 DECK/SHEAR STD: ZONE 3	15 days	Tue 4/28/09	Mon 5/18/09	3
20	FR2-3 DECK/SHEAR STD: ZONE 1	15 days	Tue 5/5/09	Mon 5/25/09	4
21	FR2-3 DECK/SHEAR STD: ZONE 2	15 days	Tue 5/12/09	Mon 6/1/09	5
22	FR2-3 DECK/SHEAR STD: ZONE 3	15 days	Tue 5/19/09	Mon 6/8/09	6
23	FR4-5 DECK/SHEAR STUD: ZONE 1	15 days	Tue 5/26/09	Mon 6/15/09	7
24	FR4-5 DECK/SHEAR STUD: ZONE 2	15 days	Tue 6/2/09	Mon 6/22/09	8
25	FR4-5 DECK/SHEAR STD: ZONE 3	15 days	Tue 6/9/09	Mon 6/29/09	9
26	FR6-7 DECK/SHEAR STD: ZONE 1	15 days	Tue 6/16/09	Mon 7/6/09	10
27	FR6-7 DECK/SHEAR STD: ZONE 2	15 days	Tue 6/23/09	Mon 7/13/09	11
28	FR6-7 DECK/SHEAR STD: ZONE 3	15 days	Tue 6/30/09	Mon 7/20/09	12
29	RF DECK/SHEAR STD: ZONE 1	15 days	Tue 7/7/09	Mon 7/27/09	13
30	RF DECK/SHEAR STD: ZONE 2	15 days	Tue 7/14/09	Mon 8/3/09	14
31	RF DECK/SHEAR STD: ZONE 3	15 days	Tue 7/21/09	Mon 8/10/09	15
32	PH RF DECK/SHEAR STD: ZONE 1	7 days	Thu 7/23/09	Fri 7/31/09	16
33	FR1 PREP/PLACE SOD: ZONE 1	6 days	Tue 5/26/09	Tue 6/2/09	20
34	FR1 PREP/PLACE SOD: ZONE 2	6 days	Tue 6/2/09	Tue 6/9/09	21
35	FR1 PREP/PLACE SOD: ZONE 3	6 days	Tue 6/9/09	Tue 6/16/09	22
36	FR2 PREP/PLACE SOD: ZONE 1	6 days	Tue 5/26/09	Tue 6/2/09	20
37	FR2 PREP/PLACE SOD: ZONE 2	6 days	Tue 6/2/09	Tue 6/9/09	21
38	FR2 PREP/PLACE SOD: ZONE 3	6 days	Tue 6/9/09	Tue 6/16/09	22
39	FR3 PREP/PLACE SOD: ZONE 1	6 days	Tue 6/16/09	Tue 6/23/09	23

Project: Steel Project Schedule
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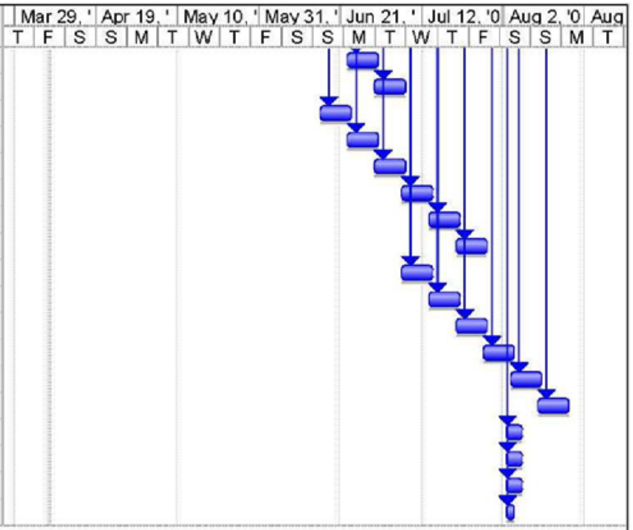
ID	ℹ	Task Name	Duration	Start	Finish	Predecessors
40		FR3 PREP/PLACE SOD: ZONE 2	6 days	Tue 6/23/09	Tue 6/30/09	24
41		FR3 PREP/PLACE SOD: ZONE 3	6 days	Tue 6/30/09	Tue 7/7/09	25
42		FR4 PREP/PLACE SOD: ZONE 1	6 days	Tue 6/16/09	Tue 6/23/09	23
43		FR4 PREP/PLACE SOD: ZONE 2	6 days	Tue 6/23/09	Tue 6/30/09	24
44		FR4 PREP/PLACE SOD: ZONE 3	6 days	Tue 6/30/09	Tue 7/7/09	25
45		FR5 PREP/PLACE SOD: ZONE 1	6 days	Tue 7/7/09	Tue 7/14/09	26
46		FR5 PREP/PLACE SOD: ZONE 2	6 days	Tue 7/14/09	Tue 7/21/09	27
47		FR5 PREP/PLACE SOD: ZONE 3	6 days	Tue 7/21/09	Tue 7/28/09	28
48		FR6 PREP/PLACE SOD: ZONE 1	6 days	Tue 7/7/09	Tue 7/14/09	26
49		FR6 PREP/PLACE SOD: ZONE 2	6 days	Tue 7/14/09	Tue 7/21/09	27
50		FR6 PREP/PLACE SOD: ZONE 3	6 days	Tue 7/21/09	Tue 7/28/09	28
51		FR7 PREP/PLACE SOD: ZONE 1	6 days	Tue 7/28/09	Tue 8/4/09	29
52		FR7 PREP/PLACE SOD: ZONE 2	6 days	Tue 8/4/09	Tue 8/11/09	30
53		FR7 PREP/PLACE SOD: ZONE 3	6 days	Tue 8/11/09	Tue 8/18/09	31
54		RF PREP/PLACE SOD: ZONE 1	4 days	Mon 8/3/09	Thu 8/6/09	32
55		RF PREP/PLACE SOD: ZONE 2	4 days	Mon 8/3/09	Thu 8/6/09	32
56		RF PREP/PLACE SOD: ZONE 3	4 days	Mon 8/3/09	Thu 8/6/09	32
57		PH RF PREP/PLACE SOD: ZONE 1	2 days	Mon 8/3/09	Tue 8/4/09	32



Project: Steel Project Schedule
Date: Tue 4/7/09

Task	▬	Milestone	◆	External Tasks	▬
Split	┈┈┈	Summary	▬	External Milestone	◇
Progress	▬	Project Summary	▬	Deadline	⇩

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Appendix F

Acoustics Study

ACOUSTICS BREADTH

AREA I

- LIKELY NOISE IN MECHANICAL ROOM (p 315', ARCH. ACOUST.)

SOUND PRESSURE LEVEL (dB)

	125 Hz	250	500	1000	2000	4000
MECH. ROOM.	92	96	90	89	85	76

- BACKGROUND NOISE LEVEL IN OFFICE (p 240, A.A.)

RC-30 → p 402 ROOM CRITERIA

RC-30	45	40	35	30	25	20

- FINDING a_2 (JUST USING THE SHARED FLOOR/CEILING ASSEMBY) p 239

S	α						
20'x25.5'	→ .01	.01	.02	.02	.02	.02	(p52) PAINTED CONCRETE
$a_2 = \Sigma S\alpha$	→ 5.1	5.1	10.2	10.2	10.2	10.2	

- CHECKING

~ 6" REINF. CONC	38	43	52	59	67	72
~ 2" FIBERGLASS INSUL.	6	9	11	16	20	25
~ 4" REINF. CONC.	48	42	45	56	57	66
~ METAL DECK	17	22	26	30	35	41

} p 204-205

AREA II

- LIKELY NOISE FROM ROOFTOP UNITS (p 315, ARCH. ACOUST.)

ROOFTOP UNITS	93	89	85	80	75	69

- BACKGROUND NOISE [SAME AS ABOVE]

- FINDING a_2 p (239)

S	α						
20x60	→ .25	.60	.65	.70	.75	.80	(p52) LOOSE GRAVEL
$a_2 = \Sigma S\alpha$	→ 300	720	780	840	900	960	

- CHECKING

~ 4" REINF. CONC	48	42	45	56	57	66
~ METAL DECK	17	22	26	30	35	41