Dulles Town Center Building One

Dulles, Virginia

THESIS REPORT

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April 7, 2009

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 2^{nd} 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

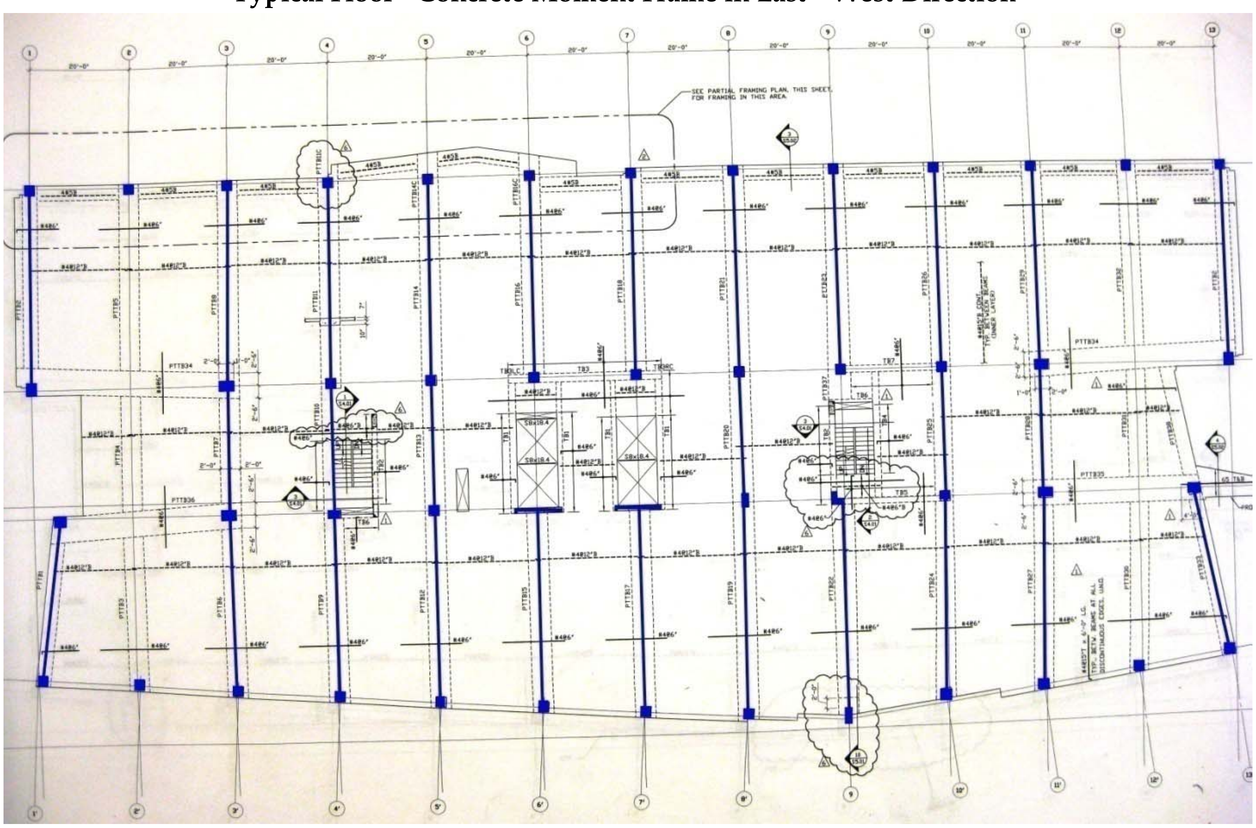

Figure 2

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

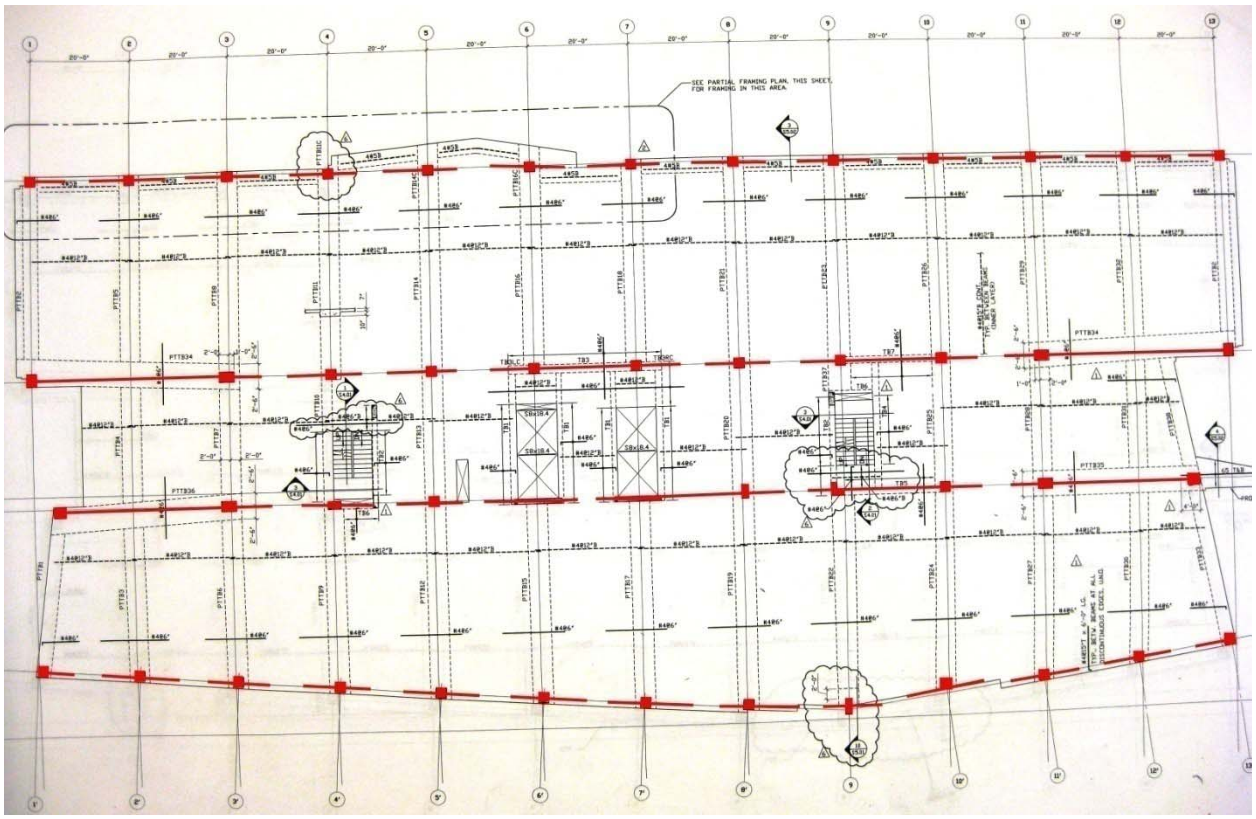

Figure 3

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

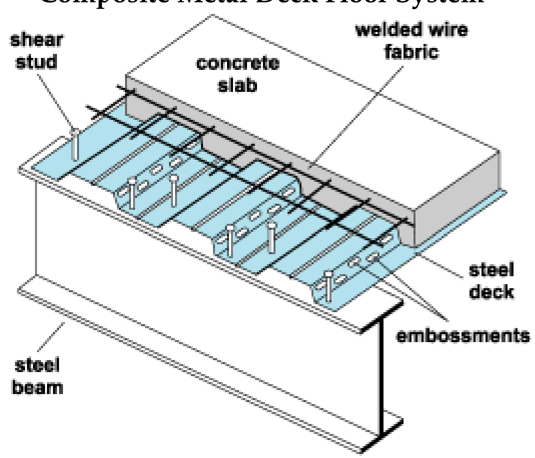

Figure 4

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

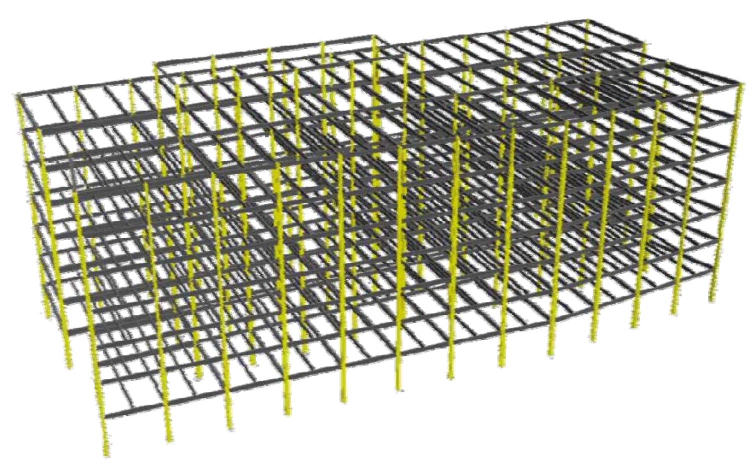

Figure 5

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

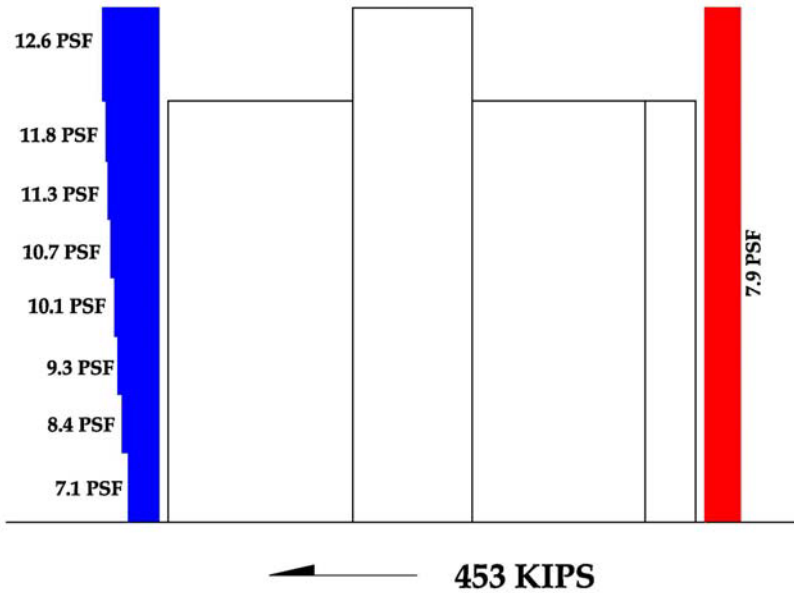

Figure 6

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

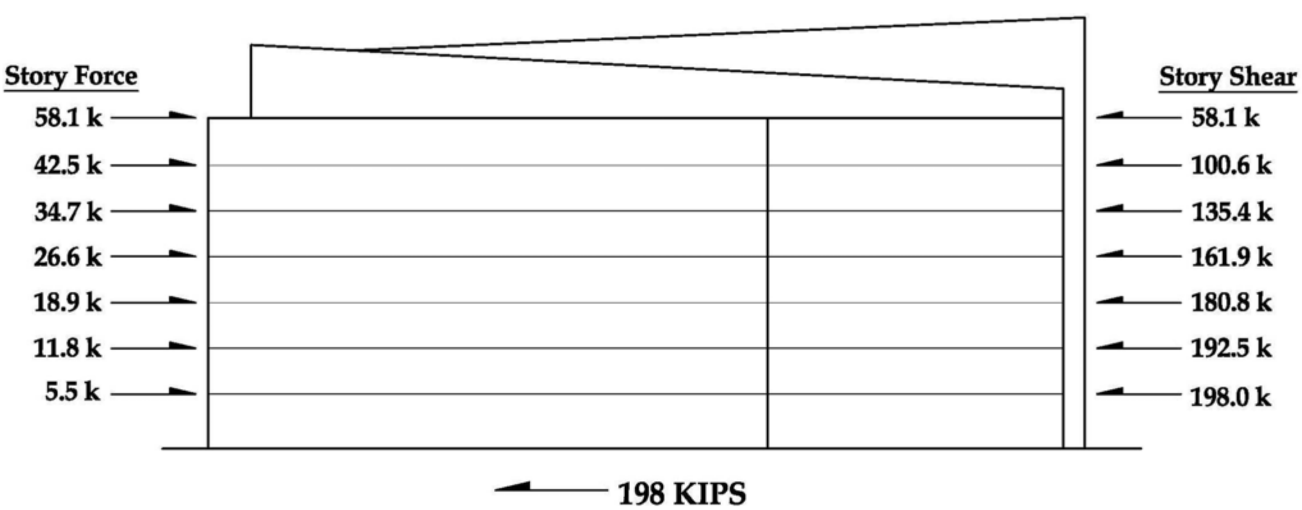

Figure 7

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

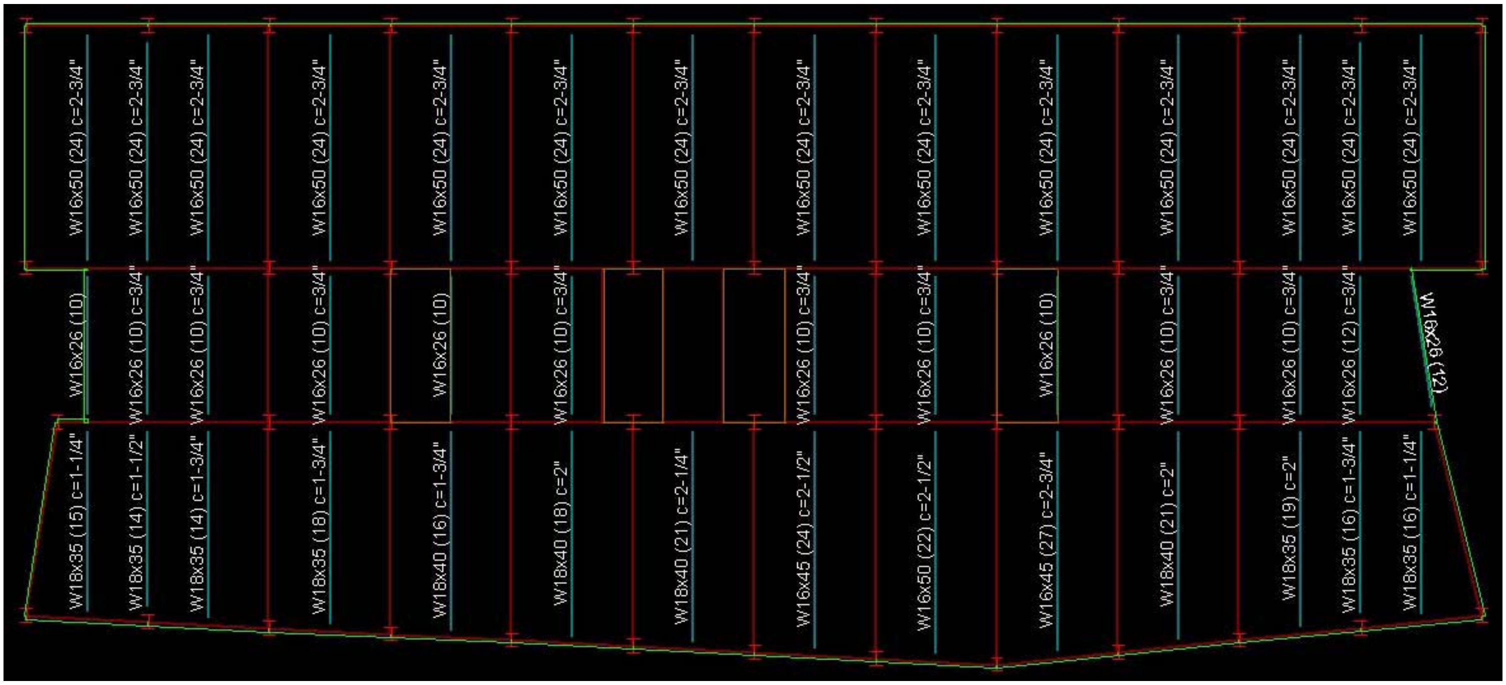

Figure 8

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

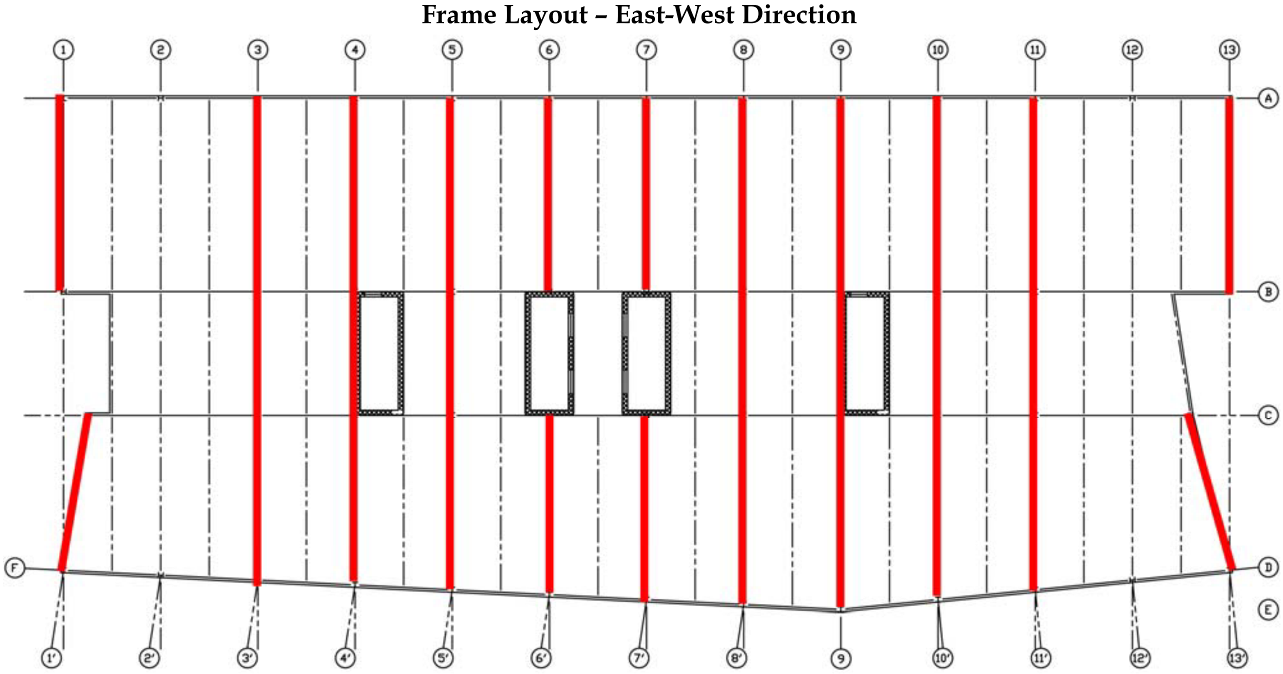

Figure 9

Frame Layout – North-South Direction

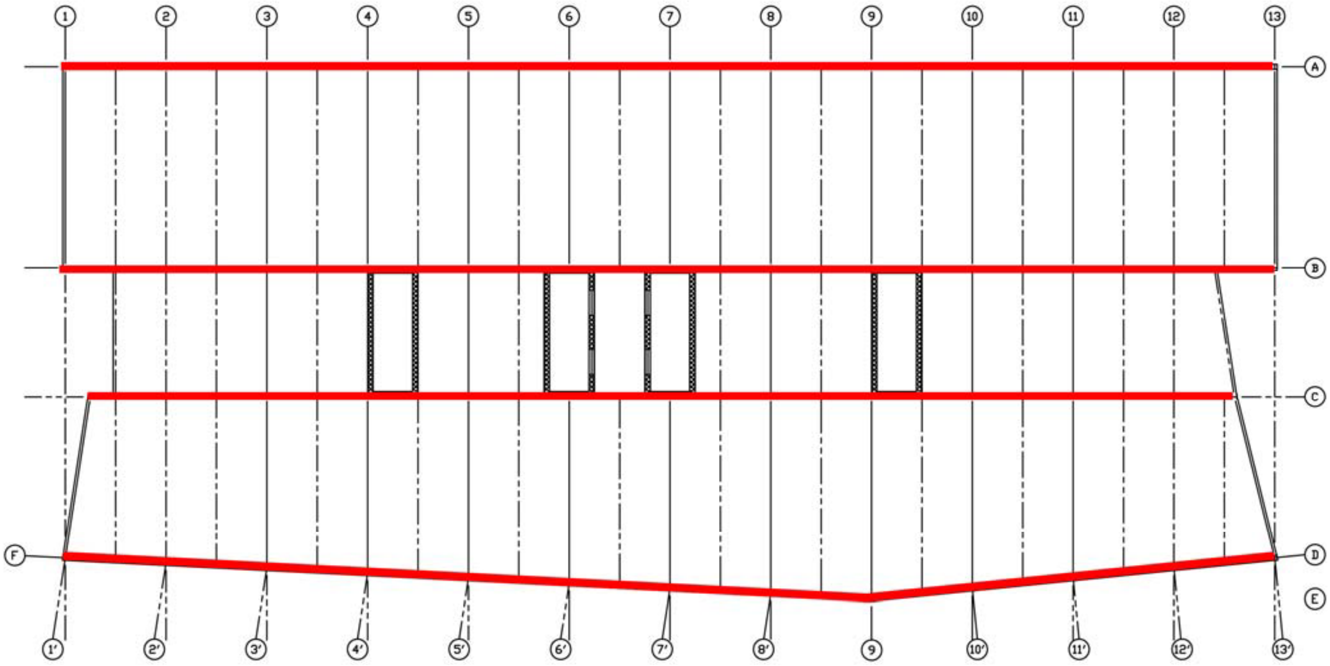

Figure 10

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

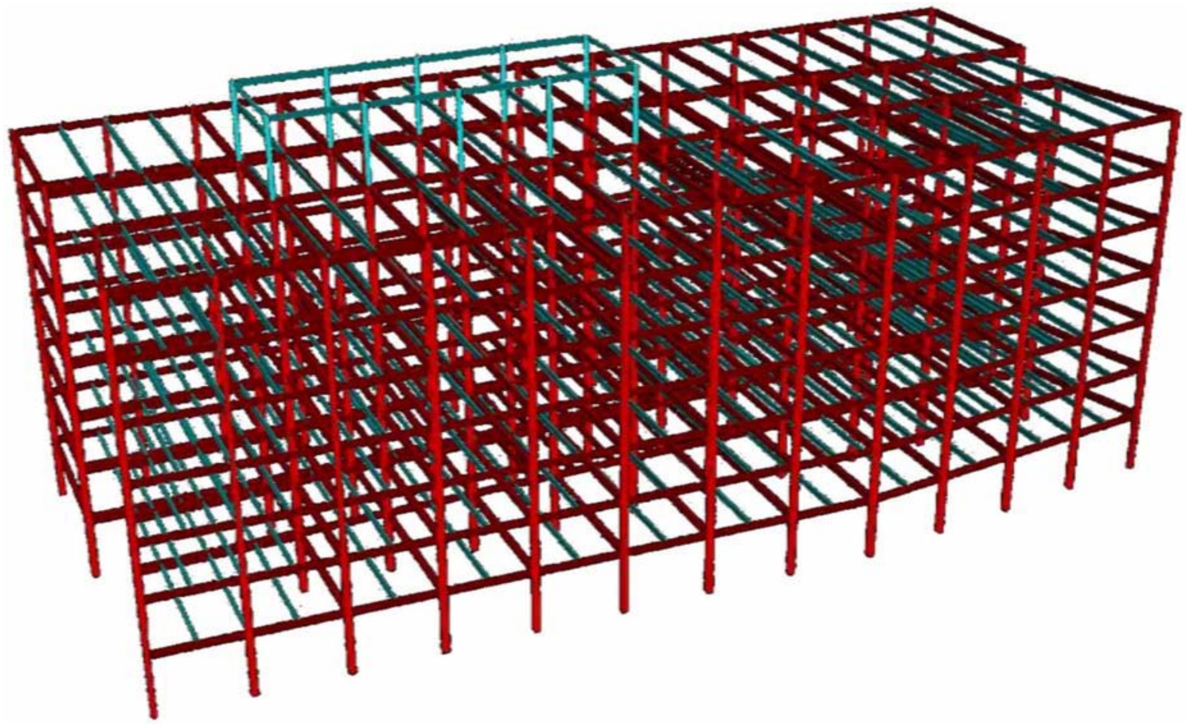

Figure 11

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.

Foundation – Section View

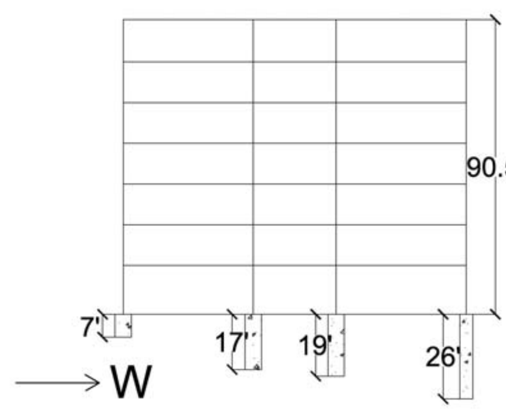

Figure 12

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.

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) $\Delta_{WIND} = h/400$			Total Drift (in)	Allowable Total Drift (in) $\Delta_{WIND} = h/400$		
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

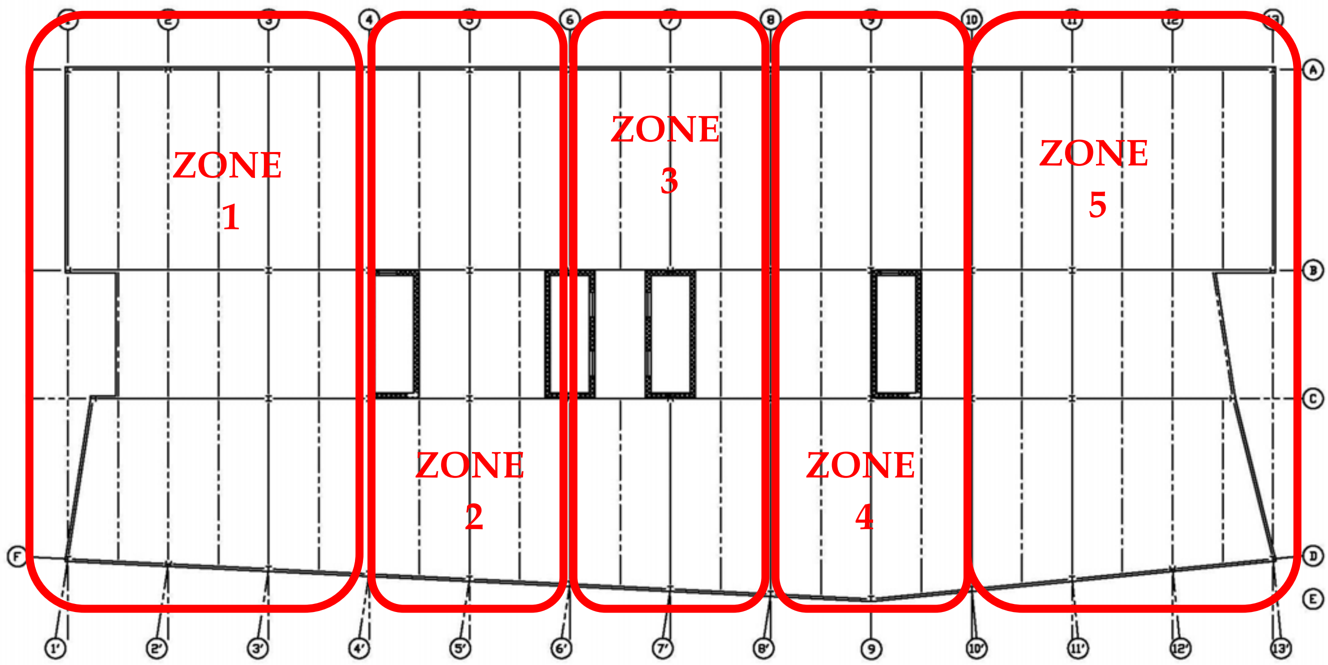

Figure 14

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

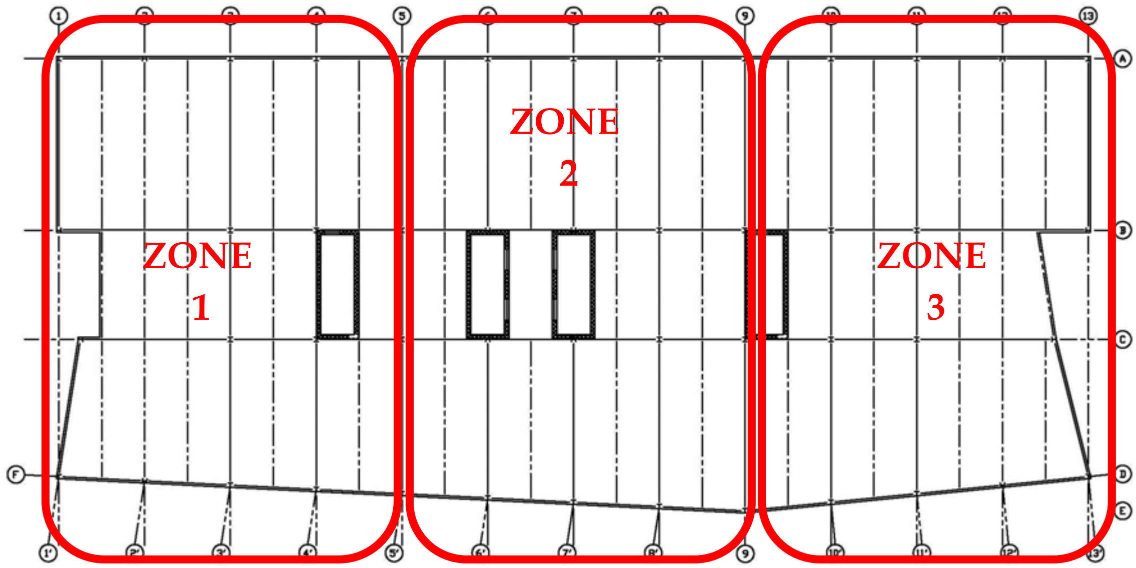

Figure 15

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

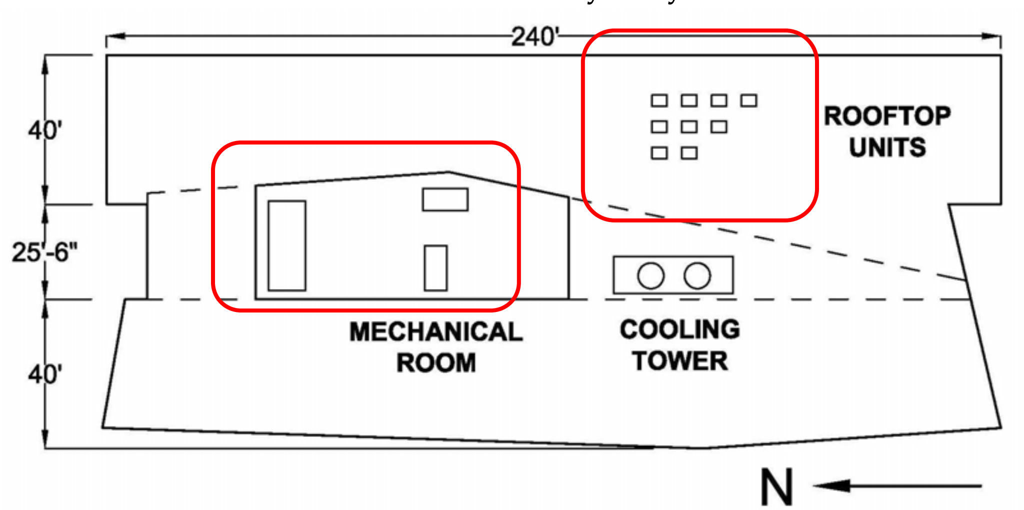

Figure 16

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

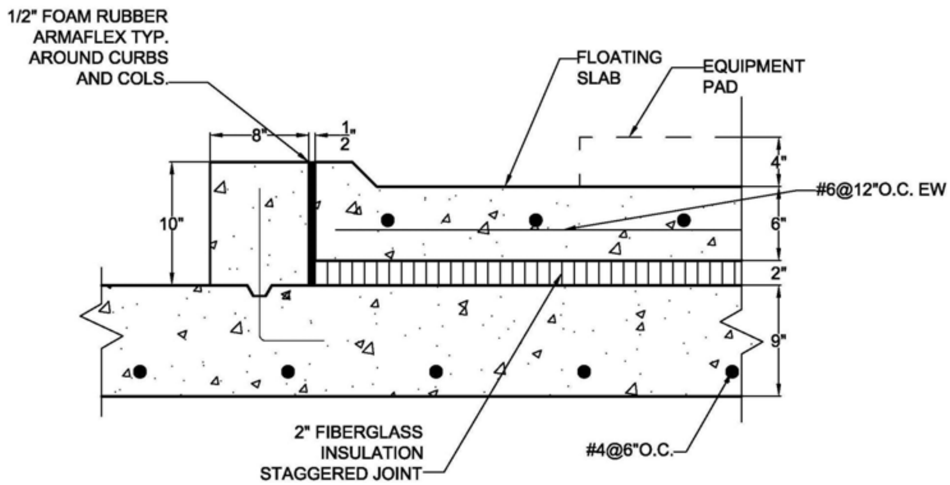

Figure 17

Mechanical Room – Proposed Floor System

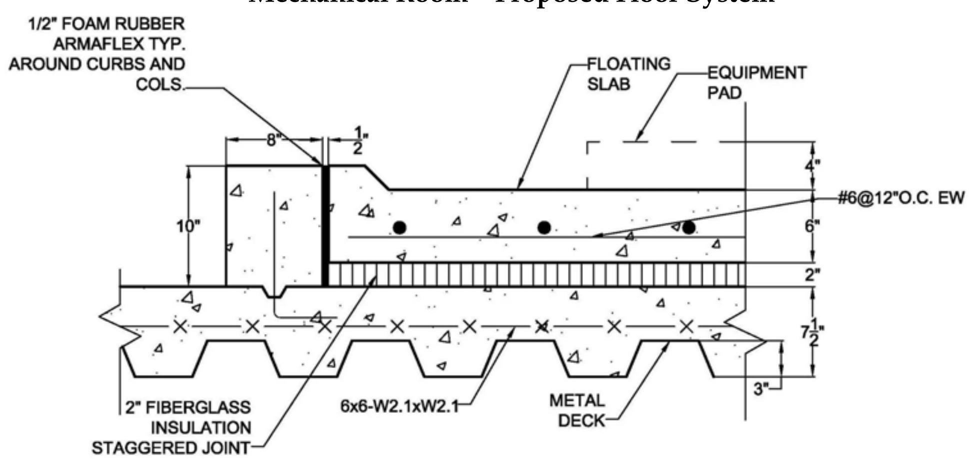

Figure 18

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

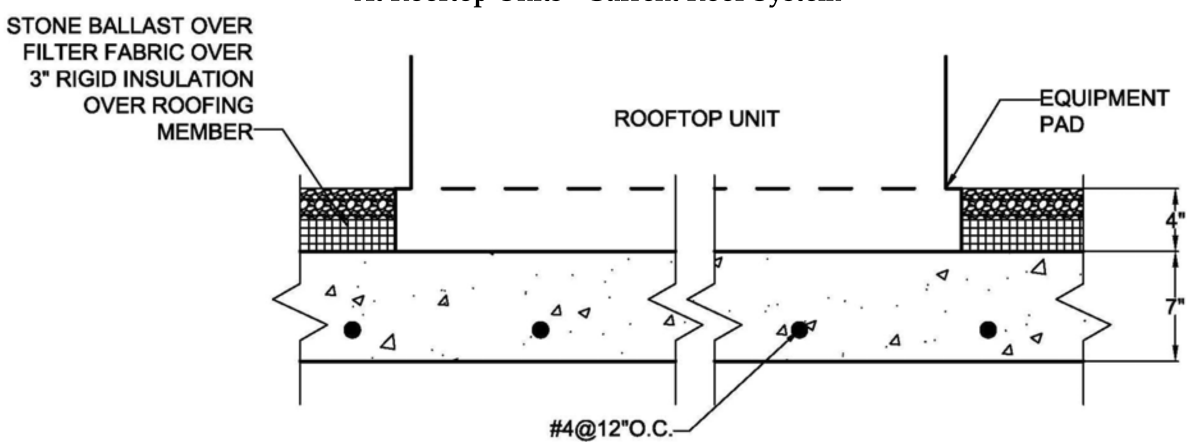

Figure 19

At Rooftop Units – Proposed Roof System

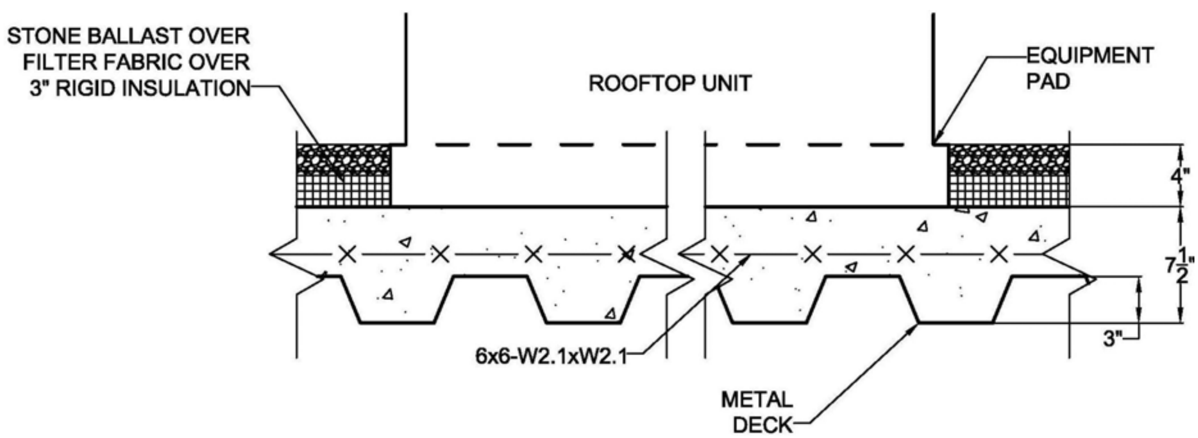

Figure 20

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 $10\log(a_2/S)$	-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.

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