FINAL REPORT

1776 Wilson Boulevard

Arlington Virginia

Joshua Urban

Structural Option

Advisor: Dr. Thomas Boothby

April 4th, 2012

1776 WILSON BOULEVARD

Joshua Urban | Structural Option | Arlington, VA

GENERAL DATA

SITE
Rosslyn/Ballston corridor

OCCUPANT
Up for lease

FUNCTION
Class A office with retail space

SIZE AND COST
249,000 SF
$63.5 Million

STORIES
5 above grade
3.5 below grade

CONSTRUCTION
May 2011- August 2012

DELIVERY
Design-bid-Build

PROJECT TEAM

Owner/Developer
Skanska USA

Architect
RTKL Associates Inc.

Structural
Tadjer-Cohen-Edelson

Lighting
CM Kling

Mechanical
Girard Engineering

Green Building
SD Keppler

STRUCTURAL SYSTEM

- High strength concrete structural system
- Two way post tensioned concrete slabs typically 8"-12" thick
- Reinforced concrete columns typically sized at 22x22 inches
- Shallow foundation consisting of a 4" thick concrete slab on grade poured over polyethylene and washed gravel
- Combination of reinforced ordinary concrete shear walls and reinforced concrete moment frames to resist lateral forces

LIGHTING/ELECTRICAL SYSTEM

- Integrated lighting control into one system
- Control system communicates with fully digital relay panels, digital switches, photocells, and various interfaces
- Voltage to equipment will be 480V (3 phase) and voltage to occupied spaces will be 208V (3 phase)

ARCHITECTURE

1776 Wilson Boulevard is a 5 story office building with retail space on the ground floor. 3.5 levels of underground parking are also included. Column layouts create flexible space for offices still looking for tenants and an open ground floor large enough for mezzanines. A generous amount of glazing decorates the facades along with pre cast concrete and masonry panels.

SUSTAINABILITY

Designed to be LEED Platinum, 1776 Wilson includes a 17,000 SF green roof, brownfield site redevelopment, solar photovoltaic system on the roof, and a program to educate future tenants on sustainability and design features.

MECHANICAL SYSTEM

- Central plant chilled water system located in the rooftop penthouse
- Office floors have one draw through VAV air handling unit with fan powered terminal devices installed in the ceilings
- Direct digital control designed to monitor selected environmental conditions
- Perimeter supplied with fan powered VAV boxes with electric reheat while the interior is supplied with cooling only VAV boxes

all images provided by Skanska USA

CPEP PAGE http://www.engr.psu.edu/ae/thesis/portfolios/2012/JDU111/index.html

Table of Contents

Executive Summary

1776 Wilson Boulevard is a 5 Story Class A office building with retail space. This report involves the relocation of this building to a new site in Oakland, California. After relocating, the existing post tensioned concrete structure will be analyzed and updated for new seismic loads. Upon the completion of this work, it was decided to redesign the building as a composite steel structure.

This entire process utilizes various Bentley RAM computer programs to aid in the analysis and redesigns. The first stages involved using RAM Structural System and RAM Concept to design and detail the existing concrete structure under new seismic loadings. Structural members were found undersized and bottom bar and edge reinforcing needed to be significantly increased to provide structural integrity to the system. Because of this is, it was decided to investigate a steel alternative.

A significant portion of the work involved redesigning the lateral force resisting system for the steel redesign and choosing an appropriate layout for the specially braced frames. These frames had to resist the increased seismic loads that were calculated after relocation. The new structure was found to have a base shear of 1006 k and an overturning moment of 54,575 ft-k. Using the centers of mass and rigidity as guidelines, a braced frame layout was designed utilized three interior special concentrically braced frames that minimized effects due to torsion. Other aspects taken into consideration during the design process included deflection criteria, vibration, and drift control. The braces were sized as W14x90 members.

A construction management breadth topic was studied to determine cost and schedule impacts of the relocation and subsequent redesigns. With a composite steel system, it was found that the schedule would be shortened by 10 working days and there would be a $1.1 million dollar cost savings on the superstructure.

Based on these results, as well as weighing the pros and cons of the two systems, it was decided that the redesigned composite steel structure would be the choice for the relocated 1776 Wilson Boulevard.

A sustainability breadth study was also performed to compare energy savings for each location and to make sure the LEED Platinum rating was maintained after relocation. It was found that the green roof system was less economical for Oakland than a white roof would be, an annual difference of $170 on energy costs was found. The PV solar panel array was found to be much more beneficial in the new climate, the addition of a green roof below the panels was studied and further increases in energy benefits were found. Heating and cooling loads for a south facing office were also performed and it was determined that the existing HVAC system would be oversized in the new climate. Taking everything into consideration, 1776 Wilson would maintain its LEED Platinum rating as well as provide significant energy benefit increases.

Acknowledgements

I'd like to start this report by giving thanks to the people who offered their support along the way. Without these individuals, this project would not have been possible.

Skanska USA

For sponsoring this project and providing me with access to all information needed.
Edward Szwarc
Joe Hammerstrom

Pennsylvania State University

The entire AE faculty for 4 excellent years of education
Specifically, my project advisor, Dr. Thomas Boothby

Last but not least, my friends and family for all their support and encouragement during the last 5 years.

Introduction

Located in the Rosslyn/Ballston corridor of Arlington Virginia, 1776 Wilson Boulevard will be a Class A office building with retail space and three and a half levels of below grade parking. Currently under construction, the building is to be built on a previously contaminated brownfield site that has been redeveloped. Scheduled to be finished in August of 2012, 1776 Wilson will contain approximately 249,000 SF. The lump sum construction contract is valued at 63.5 million dollars.

Fig. 1 Lobby Rendering

Designed by RTKL Associates, all 26,000 SF of retail space will be located on the ground floor and the upper four floors will contain 108,000 SF of flexible office space perfect for a building that is currently up for lease and the future tenants are currently unknown. 1776 Wilson will also include a three and half level parking garage which will be able to accommodate over 200 cars The retail space will have a high ceiling making tenant mezzanines possible. Most of the mechanical equipment will be located in a penthouse on top of the building. Besides the flexible office space, one of the most important interior aspects of the building is the luminous lobby that complements the generous amount of day lighting the building will receive. 1776 Wilson will also provide downtown convenience, located within walking distance of two Metro stations; several retail outlets and restaurants are within close proximity of the site.

Fig. 2 Green Roof Rendering

1776 Wilson Boulevard also goes beyond the norm for sustainability; the project is designed to be LEED Platinum. The numerous green features include a 17,000 SF green roof, photovoltaic solar panels on the roof, and an incentive program aimed at educating tenants on the sustainability features of the building.

Arlington County's C-0-2.5 zoning district includes the site of the finished building; this area generally designates commercial office buildings, hotels, and apartments. The upper floors will be considered separate mixed use occupancy while the parking levels are non-separated mixed use in accordance with building code. A generous amount of glazing helps create a well and naturally lit interior. Typical one inch thick windows with a U value ranging from 0.26 to 0.28 decorate the façades along with aluminum framed curtain walls. The rest of the façade features precast concrete and masonry panels. The roof consists of a combination of 10 and 12 inch thick post-tensioned slabs with roof pavers. The PV solar panels will add 6.6 to 6.8 psf to the roof dead load. In addition to the roof pavers, the roof will be insulated and covered by garden covering. Where roof pavers and garden covering are absent, elastomeric cementitious topped insulation is used.

Site Conditions

The site is essentially rectangular with approximate dimensions of 275 feet in the North to South direction and 125 to 200 feet in the East to West direction. This provides a total foot print area of approximately 45,500 SF. The existing site grades slope slightly from the North to the South. The surrounding area includes both residential and commercial buildings; the site itself was occupied by one to two story buildings before the project began.

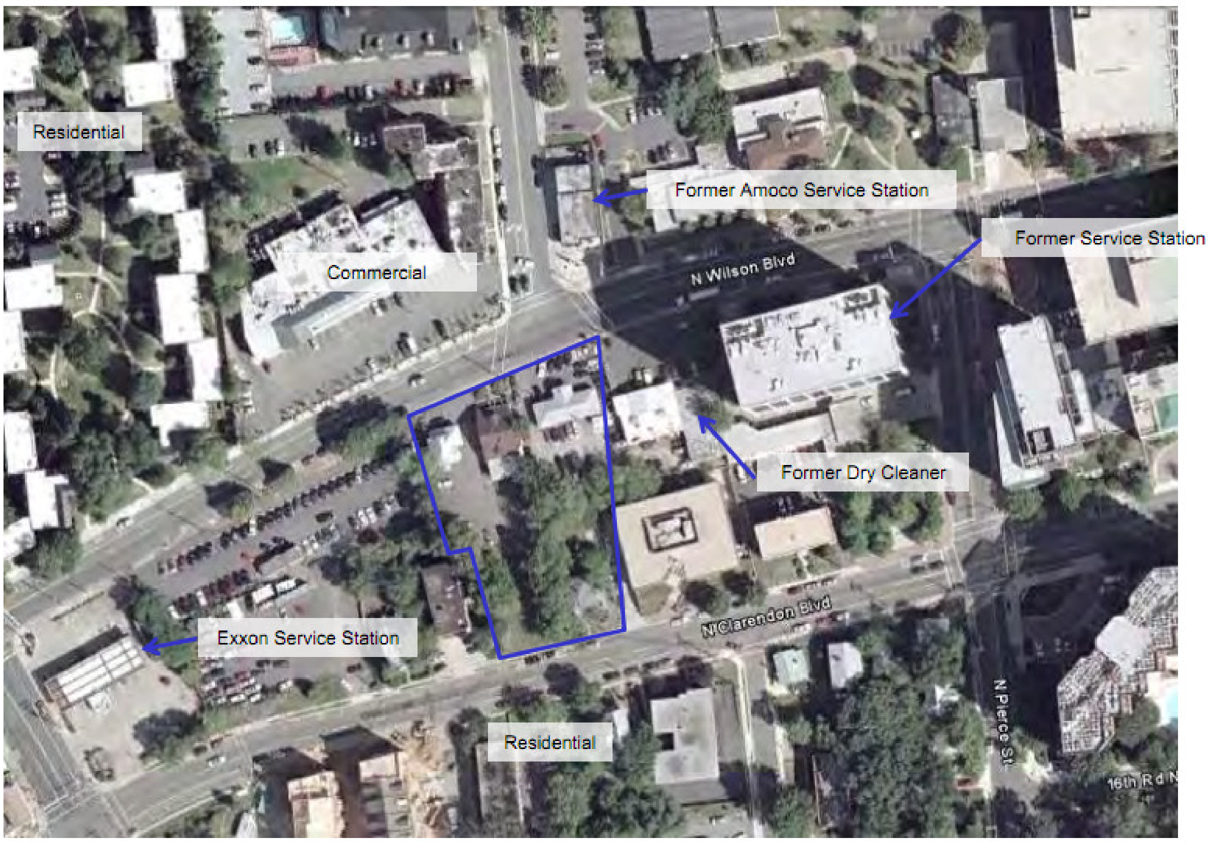


Fig. 3 Aerial View of Site

The results found in the geotechnical report for the project were based on nine soil borings. Ground cover at the site was variable and consisted of one of the following:

- 1-3 inches of asphalt with 1-21 inches of gravel below
- 2 inches of gravel
- 4 to 6 inches of top soil

Below the ground cover, a geotechnical report provided by ECS Mid-Atlantic done on the site divided the soil into three strata:

Stratum	Name	Description
I	Fill/Possible Fill	17-36 feet below site grades consisting of various amounts of sand, gravel, and clay
II	Natural Alluvial/Marine Solids	28-52 feet below site grades and under stratum 1, this stratum consists of poorly graded sand, clayey sand, and low plasticity clay with varying gravel content
III	Residual Soils/Weathered Rock	Below stratum 2 and consists of Micaceous silty sand with rock fragments.

Table 1 Soil Stratums

It was also known that this particular area has high groundwater flow. The ground water is to be controlled by a dewatering system that will need to be put in place during below grade construction.

1776 Wilson falls into Arlington's C-0-2.5 zoning district. This district is used for office buildings, commercial uses including retail, as well as hotels and apartments. The ratio of maximum office and/or commercial floor area to site area is 2.5:1. No office building is to exceed 12 stories, excluding penthouse spaces, by site plan approval. All penthouses are limited to one floor. Each plot is to have a minimum average width of 100 feet and a minimum area of 20,000 square feet.

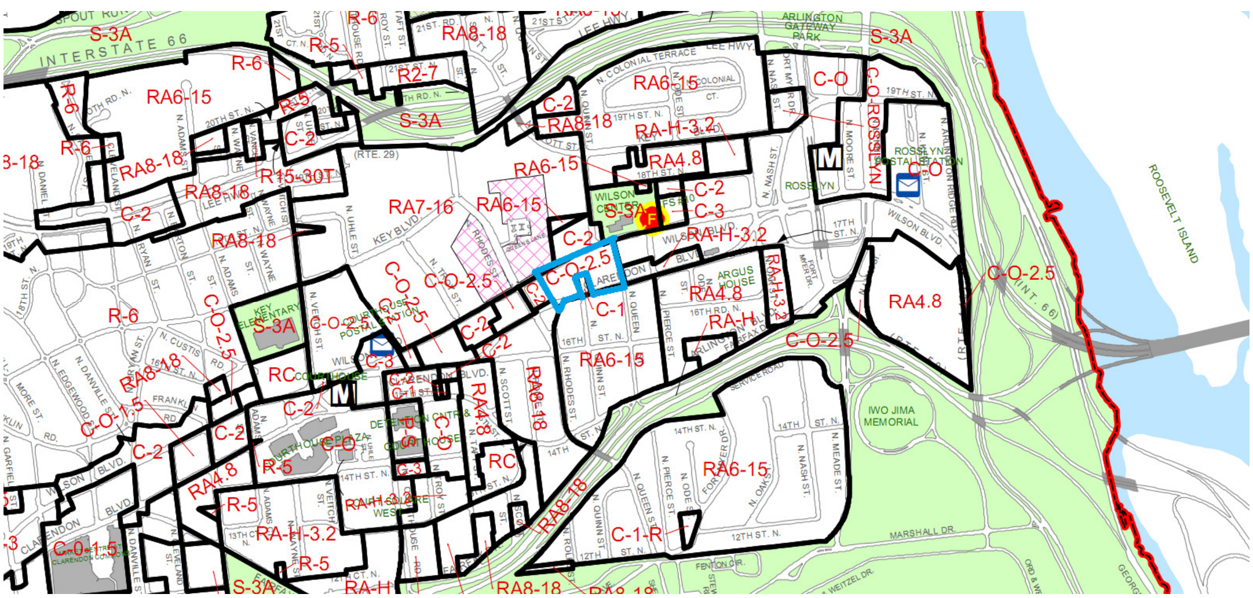


Fig. 4 Zoning Map for Arlington - The blue outline marks the district where 1776 Wilson is to be located

Structural System Overview

Foundation

The geotechnical report called for a shallow foundation system on the stratum one and two soils with a designed bearing capacity of 10,000 psf. The shallow system will consist of a 4 inch thick slab on grade with 6”x6”-8/8 W.W.F. lap mesh 6 inches in all directions and concrete footings. The slab is placed over 10 mil polyethylene and 6 inches of washed gravel. Control joints are located at 20 feet on center for all exterior slabs. Interior slabs are to be placed in 600 SF panels with control joints placed 30 feet on center. The interior slabs are also to be laid over a layer of vapor barrier which is placed on top of 6 inches of washed gravel. Groundwater on the site must be at least two feet below the foundation subgrade level. Since the soils in the area are soft and sensitive, precautions will need to be taken to ensure equipment doesn’t sink into the soil which would hinder the equipment’s usability.

All columns and walls have footings that are to penetrate at least one foot into undisturbed soil or compacted fill. All exterior footings must be at least 2’6” below the finished grade, this also holds true for footings in unheated spaces such as garages. The typical wall footing will be 12 inches deep and extend 6 inches past the face of the wall. Disturbed earth under footings will be replaced with 2000 psi concrete. The footings will be 4000 psi concrete and the slab on grade will be 5000 psi. Figure 5 shows details for control and construction joints as well as a partial foundation plan where the elevator core is located. The full foundation plan can be found in appendix A.

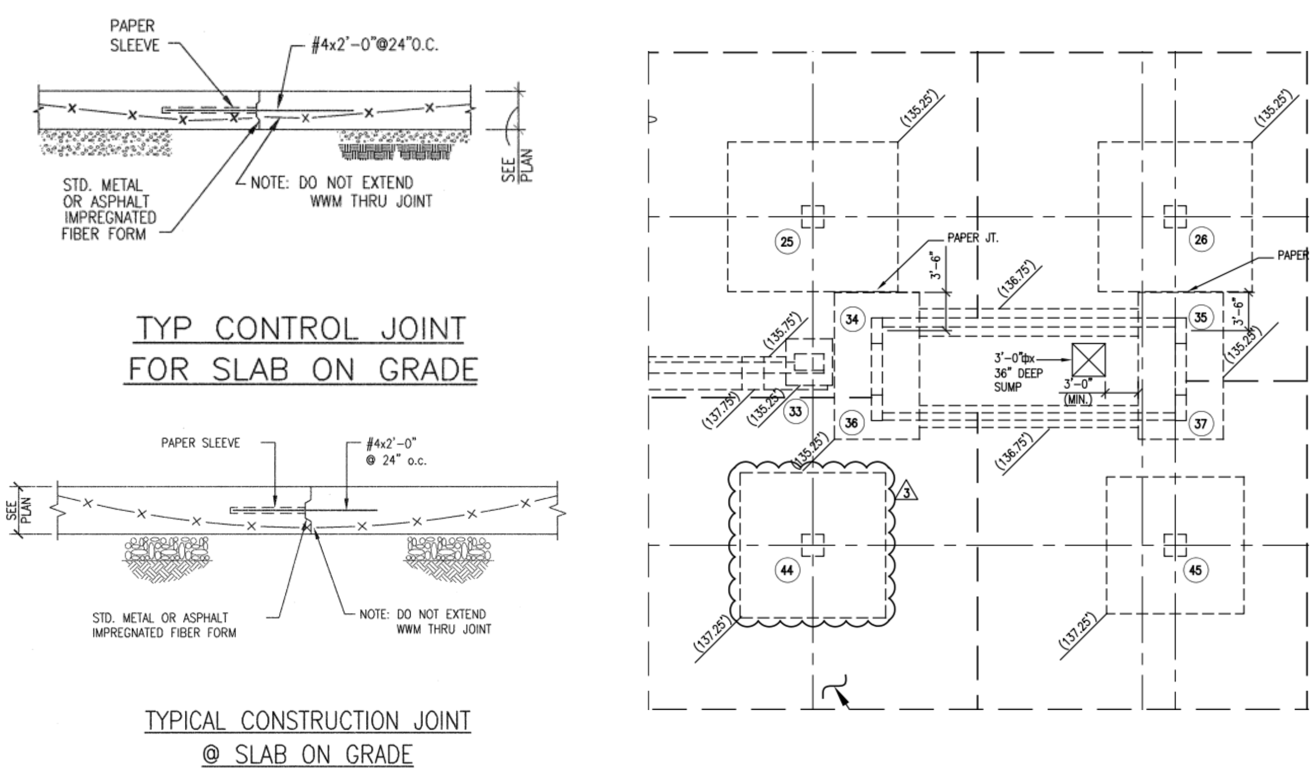


Fig. 5 Foundation Details and Partial Plan

Floor System

This project uses a high strength post tensioned concrete structure. Each floor consists of flat slabs typically ranging in thickness from 8 to 12 inches. Each slab has drop panels at the column locations that are typically 8”thick to 10” thick, much larger than typical drop panels. Select areas of the floor have continuous drop panels that serve as beams in between columns. Structural framing plans showing the column layouts and drop panels can be found in Appendix A. Post tensioning is put to use starting on the second floor and the column layouts create typical 30’ by 30’ bays with 30’ by 45’ bays also present. The high strength concrete used for the framing system of the building allows for these bays as well as reducing the total weight of the building, the typical strength is 6000 psi.

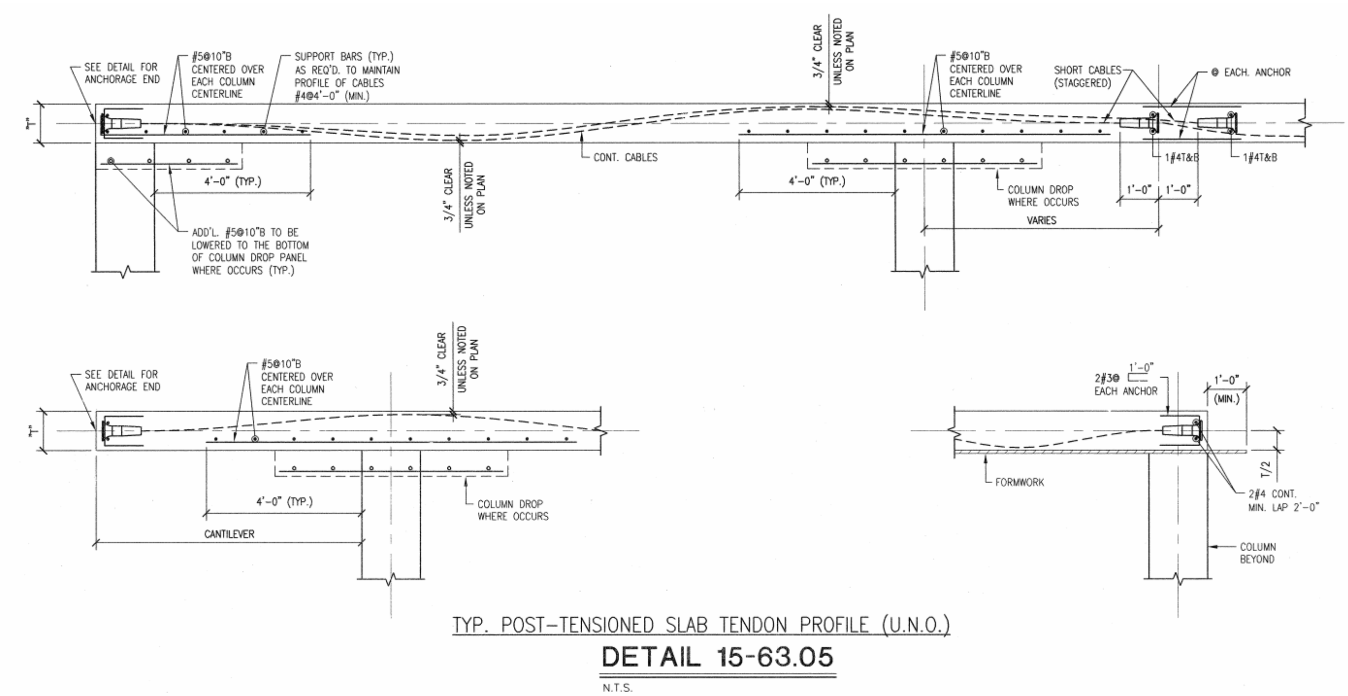


Fig. 6 Typical Post Tensioned Slab Tendon Profile

Roof System

The roof system of 1776 Wilson consists of 8 and 10 inch thick post tensioned two way slabs. The roof area is covered by either vegetation from the green roof, roof pavers, or a concrete wearing slab. Below the roof surface consists of filter fabric which is accompanied by a deck drainage mat where there is vegetation. Four inches of roof insulation is used as well as hot rubberized asphalt for the waterproofing assembly. The roof areas will see added load due to the solar panels and racking system, these will add 6.6 to 8 psf to the roof dead load.

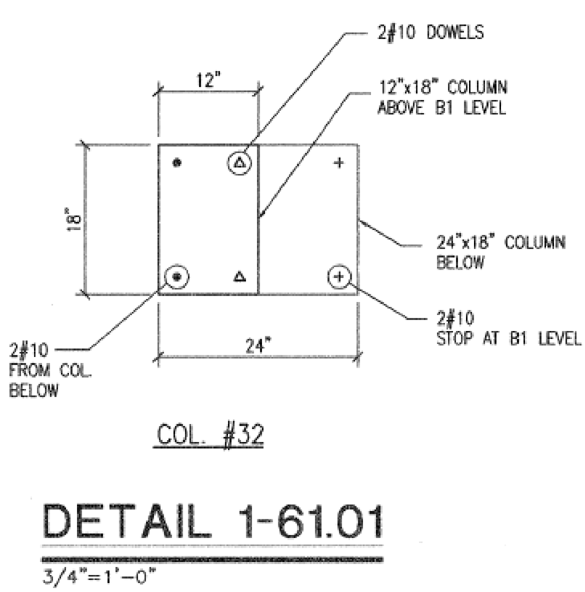
Columns

The column layouts of 1776 Wilson are uniform and create typical 30 feet by 30 feet bays, with some 30 feet by 45 feet bays. The reinforced concrete columns on the upper floors are typically 22x22 inch columns and 12x30 inch columns; the lower levels are typically 24x24 inch columns. Reinforcement ranges from #8 to #11 bars. High strength concrete is used to keep column sizes down and maximize open space. Typical column heights are 13.3’ on the 2nd, 3rd, and 4th floors. The fifth floor has columns that are an extra foot in length while the ground floor requires up to 28’ of open space on the south side of the building so there is sufficient room for the tenant mezzanines.

Floor	Sizes	Reinforcement	Compressive Strength (ksi)
5th	22x22, 12x30	4#10, 8#11, 4#9	Typically 5, some columns are 6
4th	22x22, 12x30	4#10, 8#10, 4#9	Typically 5, some columns are 6
3rd	22x22, 12x30	4#9, 4#10, 4#11, 8#10, 8#11	Typically 5, some columns are 6 and 8
2nd	22x22, 12x30	4#10, 4#11, 8#10, 12#11, 6#9	Typically 5, some columns are 6
1st	24x24, 12x30, 24x29 ¾*	4#11, 8#9, 8#10, 8#11 12#11,	Typically 8, some columns are 10
Basement Levels	24x24, 12x30, 32x18, 24x18, 12x18*	4#11, 12#11, 8#11, 4#10, 6#9, 8#9	Typically 8 at the B1 level, 6 below, some columns are 10

Table 2 Column Schedule Summary

*see following details



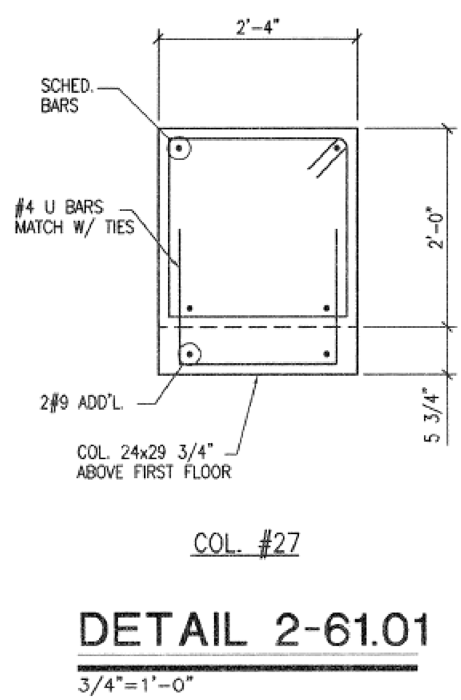


Fig. 7 Column Details

Lateral System

1776 Wilson Boulevard incorporates a combination of ductile reinforced concrete moment frames and reinforced concrete shear walls to resist lateral loads due to wind and seismic conditions. The shear walls are located around the elevator core of the building and provide aid to the moment frames up to and including the 3rd floor, the 4th and 5th floors are purely moment frames. This creates a dual system on the bottom three floors that share the lateral loads. Simplifications were made to the North façade for the wind analysis done. Both wind and seismic loads were calculated using ASCE 7-05 which incorporates how to include dual systems in the determination of lateral loads. In accordance with section 12.2.3, the lowest R value is to be used for the entire system in seismic load determination. Wind loads for dual systems are inherently more complicated so the assumption that the building is rigid in both directions, leading to a gust effect factor of 0.85, was used to simplify the calculations.

The lateral loads are distributed by relative stiffness. The loads will start at the roof diaphragm and travel to the floor diaphragm through the columns that make up the reinforced concrete moment frames. Due to the nature of this type of structural system, all columns serve as lateral members in helping resist the lateral loads. Because of this, there are essentially moment frames in both directions at each column line. The floor slabs themselves serve as the beams in this system with drop caps and continuous drops adding extra thickness. This extra thickness helps transfer the loads to the columns. Once the loads reach the third floor, a significant portion is drawn to the shear walls due to their high rigidity in comparison with the moment frames. The shear walls resist lateral loads and moments about their strong axis. They also are able to resist gravity loads from tributary members of the structure. Eventually, the lateral loads will travel to the foundation where they will be dispersed into the soil.

The shear walls are located in the same spot on each floor except the 4th and 5th floors where they are discontinued. The two shear walls oriented in the E-W direction are 12” thick while the longer shear wall oriented in the N-S direction is 10” thick. These three shear walls help lower the lateral loads the moment frames must resist. Due to torsion, the building tends to rotate around the taller northern half of the building. By attracting a larger portion of the lateral loads, the shear walls are able to help decrease the movement of the southern half of the structure. Figure 8 shows the locations of the shear walls.

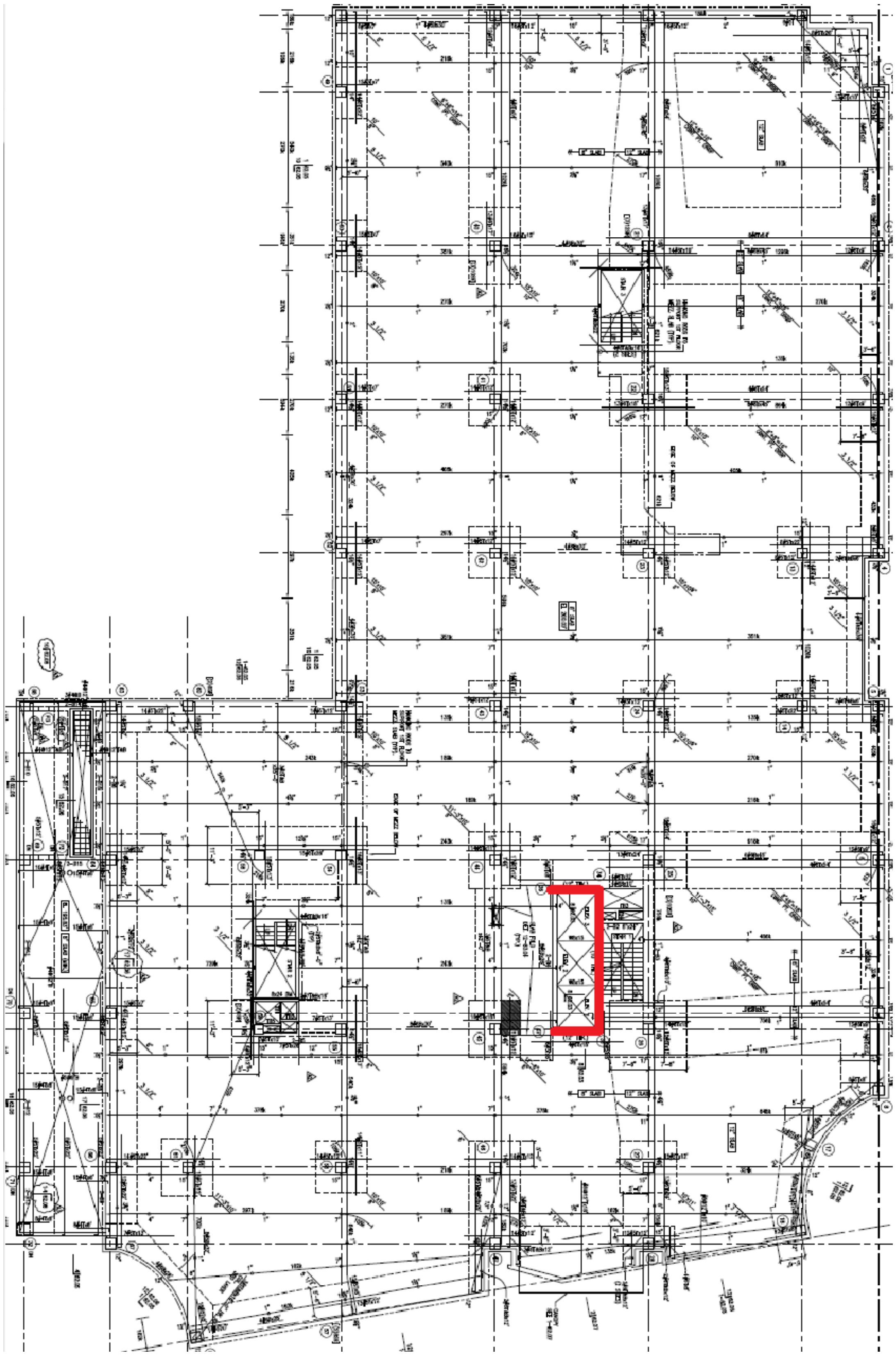


Fig. 8 2nd Floor Framing Plan: shear walls highlighted in red

Design Codes

The following documents were used and referenced throughout the progression of this project:

- ACI 318-08 Building Code Requirements for Concrete Buildings published by the American Concrete Institute
- ASCE 7-05/10 Minimum Design Loads for Buildings and Other Structures published by the American Society of Civil Engineers
- IBC 2006 International Building Code published by the International Code Council, Inc.
- 2009 ASHRAE Handbook Fundamentals published by the American Society of Heating, Refrigeration, and Air Conditioning Engineers

Other Reference Notes:

Refer to Appendix B for a list of other references used throughout this project

All images, unless otherwise noted and listed in Appendix B, were provided graciously by Skanska USA.

Materials

The following table summarizes the materials and their strengths that are used in the current design for 1776 Wilson Boulevard.

Structural Element	Strength
Footings, walls, and grade beams	F'c = 4 ksi
Framed floors, precast concrete units, and slab on grade	F'c = 5 ksi
Columns	F'c = 5,6,8, and 10 ksi
Light Weight Concrete	F'c = 3 ksi
Reinforcement Steel	ASTM-A615 Grade 60
Welded Wire Mesh	ASTM-A185

Table 3 Materials

Post Tensioned Concrete – tendons consist of steel strands that conform to ASTM A-416, Fpu = 270,000 psi. Tendons are stressed after reaching 75% design strength of concrete.

Masonry – concrete masonry units conform to ASTM C90 Grade 1, minimum f'm = 1500 psi. Above grade mortar will be Type S conforming to ASTM C270, below grade will be Type M, and veneer face brick will be Type N.

Design Loads

The design dead and live loads used for the designed building were listed on the drawings; ASCE 7-05 and IBC 2006 were mainly used in the design to arrive at these loads. For the analysis done during this project, loads were taken from ASCE 7-05 or assumed.

Occupancy	Design	ASCE 7-05
Office Lobbies and 1st Floor Corridors	100 psf	100 psf
Offices	50 psf + 15 for partitions	50 psf + 15 for partitions
Corridors Above First Floor	80 psf	80 psf
Roof	30 psf	20 psf
Stairs and Exit Ways	100 psf	100 psf
Storage	125 psf	125 psf
Fitness Center	100 psf	100 psf

Table 4 Live Load Summary

Floor	Design Load
Normal Weight Concrete	150 pcf
MEP/Ceilings	15 psf
Drop Panels	Same as normal weight concrete

Table 5 Floor Dead Loads

Roof	Design Load
Normal Weight Concrete	150 pcf
Solar Panels and Racking System	6.6-8 psf
Roof Paver, Insulation, and Waterproofing	24 psf
Vegetation	1-2 psf

Table 6 Roof Dead Loads

Snow Loads

The snow loads for this analysis were taken from ASCE 7-10 chapter 7. Initially, ASCE 7-10 was used for the lateral load determinations done for technical report 1. However, due to the complicated changes made to wind analysis, ASCE 7-05 was used to recalculate the lateral loads for the next technical report. The snow loads were not altered though and remain calculated based on ASCE 7-10. The ground snow load for the Arlington area was decreased from 30 psf to 25 psf in the transition from ASCE 7-05 to ASCE 7-10.

Snow Load Criteria	Value
Exposure Factor	Ce = 0.9
Thermal Factor	Ct = 1.0
Importance Factor	Is = 1.0
Ground Snow Load	Pg = 25 psf
Flat Roof Snow Load	Pf = 15.75 psf
Snow Density	17.25 lb/ft^3

Table 7 Snow Load Information

Snow drift calculations were also performed, please refer to Appendix D for more information.

Proposal

Problem Statement

On August 23rd, 2011, a 5.9 magnitude earthquake struck central Virginia and was felt from New York to Georgia. Geologists are now mapping a previously undiscovered fault line that was the source of the rare earthquake. Not only did this incident raise awareness that there could be a new fault line running through the Northeastern portion of the United States, but it also shed light on the fact that older building s in the area may not be structurally sufficient if more powerful earthquakes were to occur. 1776 Wilson Boulevard was not far enough in the construction process to see any damage or schedule delays due to the earthquake but that wasn't the case for some nearby structures in the process of being constructed. If more powerful earthquakes were to occur, how would 1776 Wilson fair upon completion? If the earthquake had struck a year earlier would the design have changed? These scenarios are the basis of the proposal for this project.

Problem Approach

To analyze the issue of stronger earthquakes and how 1776 Wilson would perform under such circumstances, it was decided that the building be moved to an area that needs to be designed more strictly and to higher loads in accordance with code. Specifically, Oakland California was chosen as the new location for the building. Despite a new fault line that is being currently mapped, it isn't likely that the Northeastern US will see earthquakes on the same magnitude as California would, and certainly it wouldn't happen as frequently. Nonetheless, going from one extreme to the other will show how efficient 1776 Wilson would be, how the design would change, and if a redesign would be likely.

It is anticipated that a switch to a steel structure will be necessary; this would greatly decrease the weight of the structure which directly relates to the seismic loads that need to be resisted. Before redesigning the building, however, the current concrete structure (as modeled for technical report three) will be relocated and modified to maintain structural integrity under much higher lateral loadings. If it is deemed necessary, an investigation into a steel redesign will be carried out.

Redesigning the structure in steel will come with its own host of problems that will need to be dealt with and addressed. A new column layout could potentially impact the interior architecture of 1776 Wilson, a new lateral system will need to be designed and analyzed which could also impact architectural features both interior and exterior, steel seismic provisions will need to be considered as well. Because Oakland is in a high hazard seismic region, special moment frames will be necessary and there will be cost and schedule impacts that will be worth looking into.

Breadth Topics

Construction Management

The redesigned concrete structure will result in changes to cost and schedule and, as anticipated, a redesigning of the structural system in steel will require its own cost and schedule analysis as well. These factors will be significant in the final decision on what system to use. Critical paths will be identified to pinpoint the most important tasks that the overall project duration depends on. This will be done by determining the available float for each task. MSProject will be used to layout the superstructure schedule. In the essence of time and preventing an overwhelming scope of work, below grade structural aspects will not be included. Once the schedules are completed, a cost analysis can be carried out. This will need to be done after the scheduling since the costs are dependent on the durations of each task.

The costs will be determined by performing take offs on different structural elements and using RS Means as the main source for cost data. Once the cost analysis is complete for each system, comparisons can be made and a final decision on which system to use can be reached.

Sustainability

One of the biggest selling points of 1776 Wilson Boulevard is its LEED Platinum rating. It is crucial that after the relocation and redesign that this rating is maintained. A large portion of the current LEED points come from site related aspects of the project such as being built on a redeveloped brownfield plot. The new location will need to provide similarities in these areas in order to keep those LEED points. The current design for 1776 Wilson also includes some energy saving features such as a solar photovoltaic panel layout on the highest roof as well as a 17,000 SF green roof. An energy analysis will need to be performed that includes these two features in order to compare the cost and energy benefits. This will also help determine if the LEED points in these areas would be maintained after the location switch.

The new location will also provide a new climate. This new climate will impact the HVAC system design. To get an idea on what kind of impact this might be, heating and cooling load calculations will be carried out for a south facing office and comparisons will be made to the loads for the building in Arlington Virginia. New HVAC equipment won’t be sized as a part of this proposal but the differences will help determine if any cost savings could be gained that could go towards more energy efficient equipment.

These heating and cooling loads will also be impacted by the window area to wall area ratio. Currently, a south facing office will have 63% glazing on the exterior wall. An architectural modification could be made here depending on the heating and cooling loads. This could also tie into the redesign of the new steel structure.

Criteria

In order to determine how successful this proposal is after all the research and analysis is complete, a set of criteria needed to be defined. The current project is set to open in August of 2012, if the opening is delayed the owner will have delayed profits and the tenants will need to wait longer before opening at their new location. The relocated structure will be on the same schedule so at least one of the redesigns, concrete or steel, must have a shorter duration time than the existing schedule. It is anticipated that the relocated concrete structure will have a longer schedule so the steel redesign will have a separate criteria for individual success. That criteria is cost. If the steel redesign is both faster and cheaper to construct, it will be seen as a successful redesign.

Cost is an important criteria for success because composite steel systems have cons in comparison with a post tensioned concrete building, especially an office building. These cons will be addressed later in this report but because of these extra design considerations it is important that a significant aspect such as cost is an advantage to the steel structure.

For the sustainability breadth topic, the main goal was to make sure 1776 Wilson Boulevard maintained its LEED Platinum rating. In terms of energy benefits, it was likely that the building would be more energy efficient in Oakland’s climate but other aspects of the LEED rating such as site related features could take away points and lower the LEED rating.

New Site Location

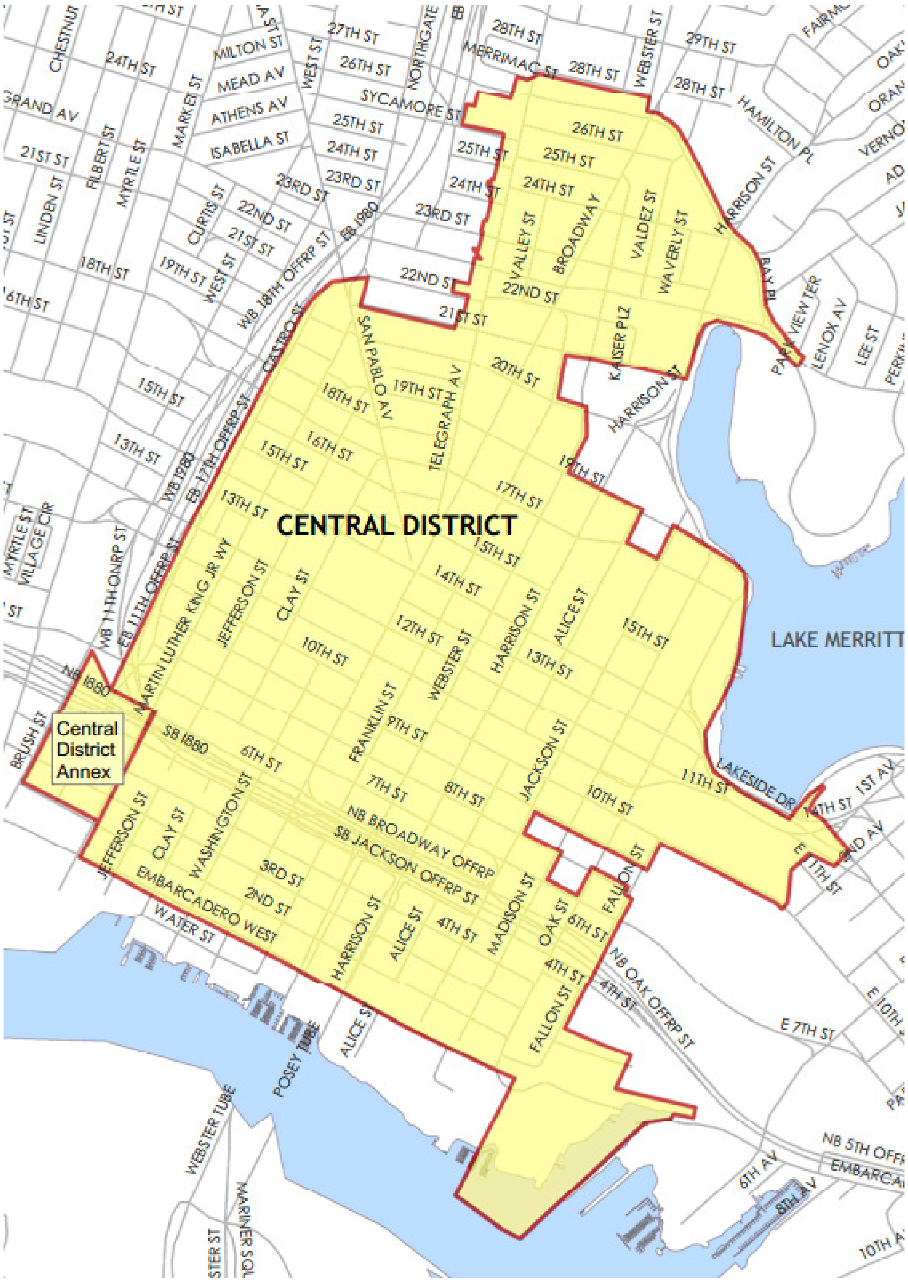


Fig. 9 Central District Project Area Map

The new site chosen for 1776 Wilson Boulevard is the Central District Project Area located in the San Francisco/Oakland metropolitan area.

This site was chosen because of various similarities to the current location of 1776 Wilson, some of which tie into the building's LEED Platinum rating.

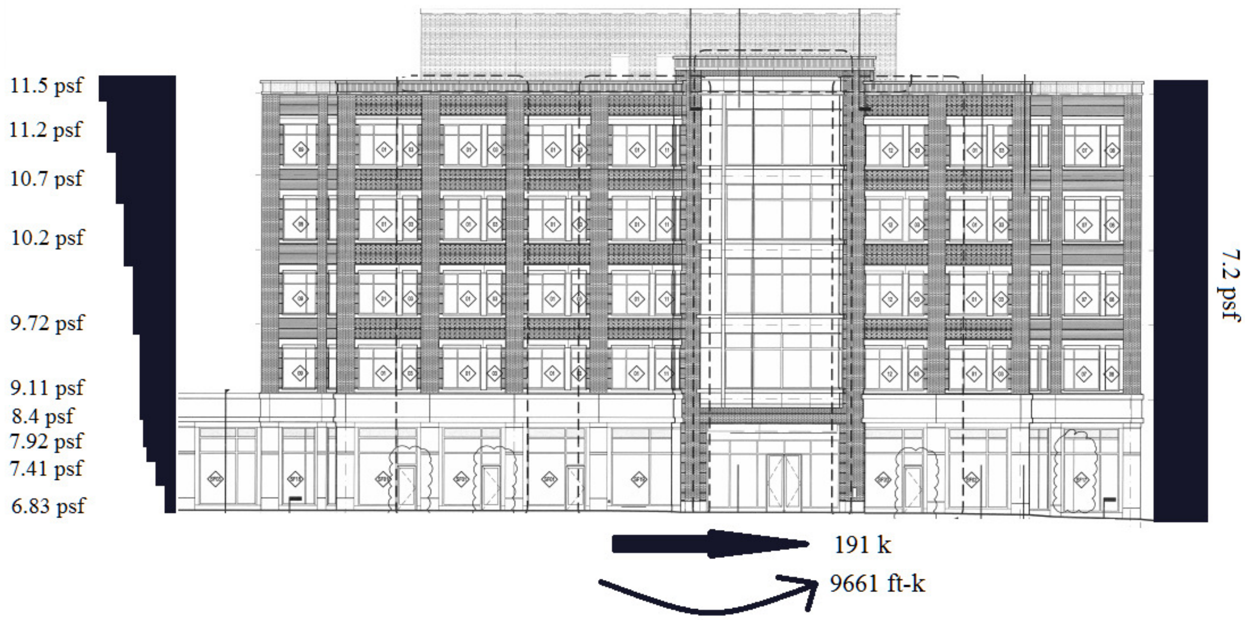
Figure 9 shows a map of the Central District Project Area. The area is undergoing a current redevelopment plan that aims to strengthen the area's status as an important office center. While Arlington has a similar redevelopment plan that's been in action for several years, this still holds as a nice comparison to the original site and an appropriate location for the new 1776 Wilson Boulevard.

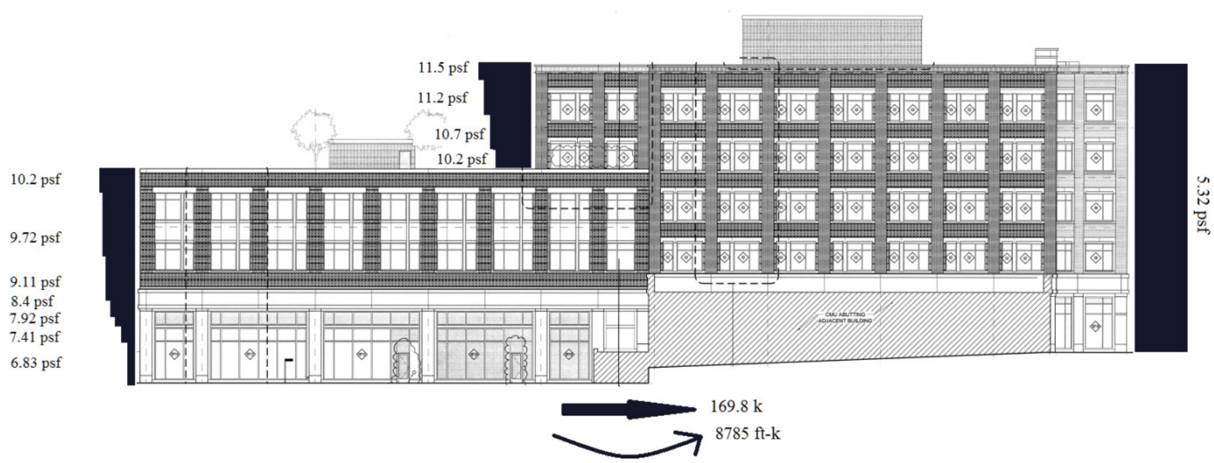
Refer to the sustainability breadth topic portion of this report for a more information on why this area was chosen as the site for the relocated 1776 Wilson Boulevard.

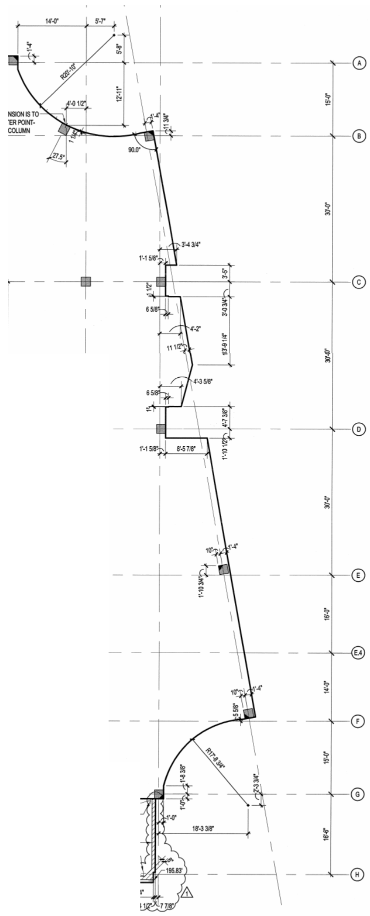
Structural Depth

Lateral Loads Summary

The following figures serve as a summary of the lateral loads that were calculated for the technical reports previously completed for this project.







d analysis


155.3 k
235 k
122.4 k
121.5 k
77.7 k
235 k
390.3 k
512.7 k
634.2 k
712 k
712 k
44110 ft-k

Seismic Loads Updated

1776 Wilson Boulevard will essentially be going from one extreme to the other in terms of seismic loads when it is relocated to Oakland California. Significant increases in the seismic loads will result from the move and they were calculated for both the steel and concrete redesigns. The following figures summarize the results of those calculations. Appendix C contains the original lateral load calculations performed for previous reports as well as the updated calculations.

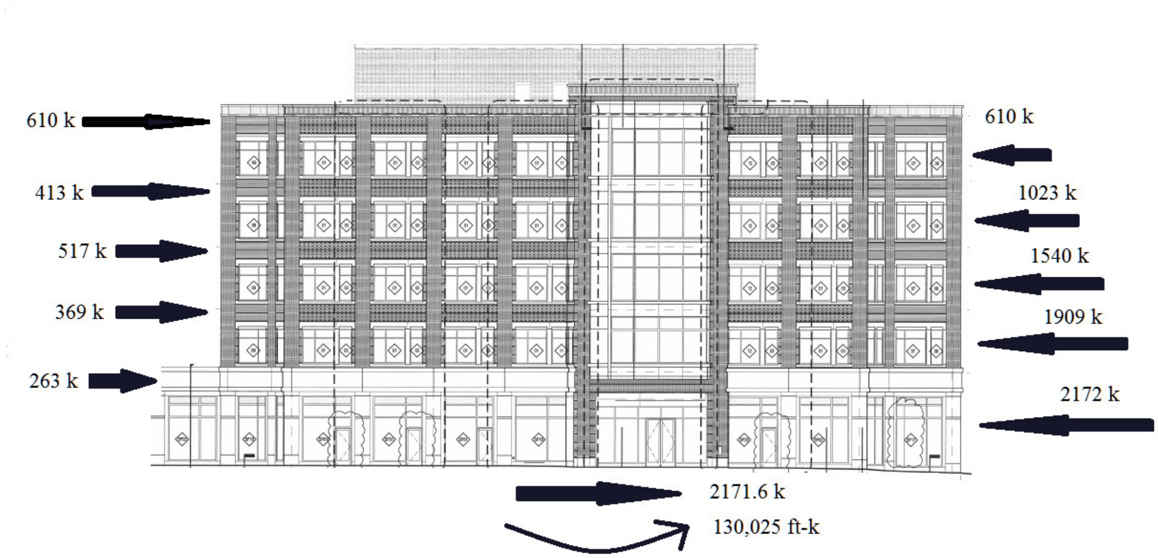


Fig. 14 Seismic Elevation for Concrete Redesign

Fig. 15 Seismic Elevation for Steel Redesign

Concrete Redesign

Before deciding on using a steel redesign for the new structure, the existing concrete structure was moved to the new location and analyzed to see what changes would need to be made so that the structural integrity wouldn't be jeopardized under the new lateral loads. Using the 3D model created for technical report three using RAM Structural System, the seismic conditions were updated and a new analysis was run. The existing columns were not sufficient to carry the new loadings. The existing structure had 24"x24" columns on the ground floor and 22"x22" columns above. Since weight controls the seismic loads, it was favorable to keep the columns sizes as small as possible. The new columns were set at 30"x30" for each story however the new columns needed to be reinforced greatly with #11 bars in order to maintain that updated size.

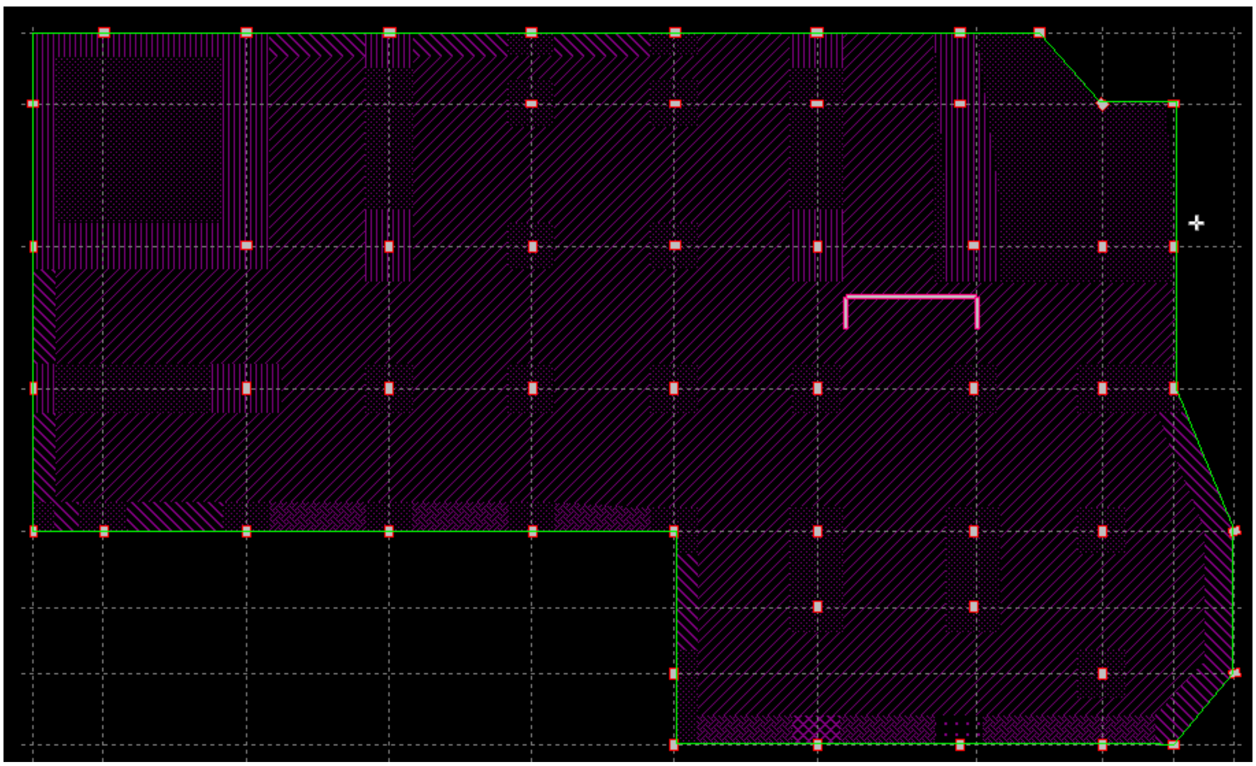


Fig. 16 Framing Plan for Updated Concrete Structure: the different hatches shown indicate different slab thicknesses

Next, a look at how the slab reinforcement would be impacted was necessary. In order to analyze this, the RAM Structural System model was imported into RAM Concept, a program specifically designed to analyze concrete floor systems of all types. Before importing the model into Concept, the slab openings were added to the Structural System model.

The most complicated aspect of analyzing this structure using RAM Concept was detailing and modeling the post tensioning. Column and middle strips were generated so that the model would be treated as a two way slab and tendons were added manually to match the existing design. The loads from RAM Structural System were imported with the model and the reinforcement for the slab could then be designed by the program.

The first significant difference found after Concept completed its analysis was a large increase in bottom reinforcement bars. The bottom bars were designed to be continuous over the column locations, it is assumed this was done so that if a shear failure at the column to slab connection were to occur, the continuous bottom bars would serve to hold the slab in place and prevent a progressive collapse. Without this increase in bottom bars a single slab-column connection could fail and the loads intended to be carried by that connection would disperse through the slab to nearby connections making it more difficult to be properly resisted.

Another noticeable difference found in the reinforcement plan for the redesigned concrete structure compared to the original design for Arlington Virginia was an increase in edge reinforcement. The edges of the floor slabs in 1776 Wilson Boulevard have continuous drops connecting each of the exterior columns. These drops are either 5 ½ or 3 ½ inches thick. A complicated detailing process would need to occur in order to properly distribute the reinforcement over the highest stressed regions near the slab edges.

After updating the concrete structure and lateral loads, it was found that increasing sizes of columns and reinforcement numbers would need to occur in order to create a structural system able to successfully resist the new loads. The increase in member sizes will increase the weight of the structure thus further increasing the seismic loads. This coupled with the increased reinforcement quantities resulted in the decision to redesign the structure in steel which includes the design of a new lateral force resisting system.

Composite Steel Pro/Con

Before moving on to the steel redesign, it is important to note that a switch to a composite steel system was analyzed for a previous technical report. The goal of the analysis was to determine if the composite steel system would be a possible alternative for the building. It was concluded that the composite system would not be a viable option for the building located in Arlington Virginia. However, after the relocation to Oakland California, the biggest advantage that the composite steel system had over its post tensioned concrete counterpart made it a viable option. This main advantage is lower weight. Since the weight controls the seismic loads and Oakland is located in a high hazard seismic region, the lower weight makes a significant difference in seismic loads and overturning moments. Another advantage composite steel has over post tensioned concrete is constructability. It is expected that this system would take less time to construct.

Although this system has the distinct advantage of offering a light structure, there are still some cons that would need to be addressed. An in depth look at these cons is not included in the scope of work for this project but they will still be discussed briefly.

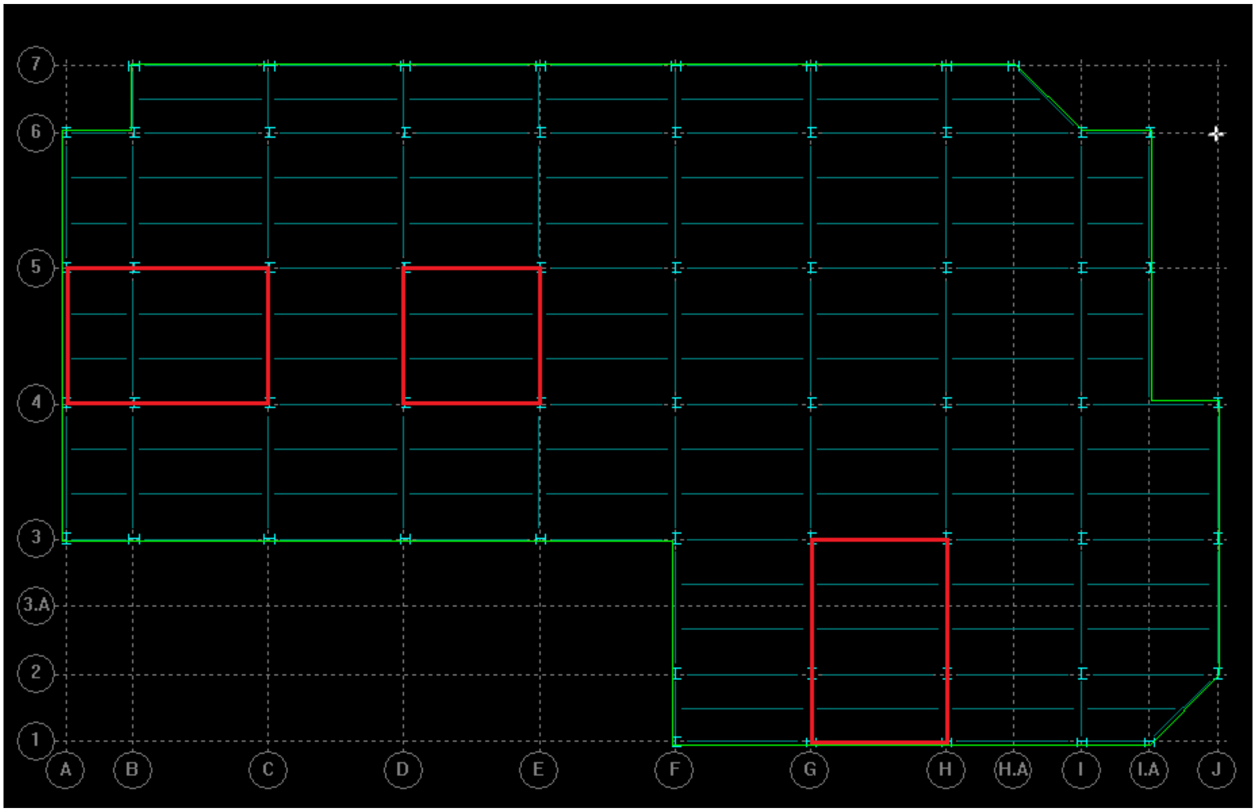
A concrete structure would need no additional fireproofing to achieve a required rating. A steel system on the other hand will need to be fireproofed which increases the cost. Fireproofing was considered as part of the construction management breadth topic found later in this report.

A light weight composite steel structure could also have potential issues with vibrations. It is important to deal with these issues during the design process because it can become very expensive and impractical to try and fix them after the building is already constructed. The most prominent vibration issues that would need to be designed for are vibrations due to people and due to seismic events. Not just initial deflections but the vibrations that can continue afterwards as well. Composite steel structures are able to minimize the continued vibrations inherently because the composite beams combine with the deck to act as a single unit. Normal weight concrete will also be used for the composite decks in order to increase the weight. While this isn't favorable for seismic reasons, the heavier the slab the more force it will take to cause vibrations. RAM Structural System also has options to check vibration issues as you design in the program, this will be utilized throughout the process.

One final con related to 1776 Wilson is that composite steel isn't the best option for parking structures and this project includes below grade parking. An updated concrete foundation similar to the one utilized in the structure at Arlington could be used for the relocated project to cover any concerns about foundation design.

Steel Gravity Redesign

After it was decided a concrete redesign would not be a suitable choice for the new site location, a steel redesign began. First, the system was redesigned only for gravity loads. A new column layout was chosen, resulting in typical 30'x30' bays, and then trial sizes for the beams and columns were chosen based on hand calculations of typical bays to resist the gravity loads. Figure **** highlights the typical bays that were used for this portion of the redesign. Some bays needed to be looked at on different floors due to variations in live loads. For example, both the lower and upper roof of the building had added live loads due to the terrace and mechanical penthouse. After all gravity members were assigned trial sizes, the new gravity system could be modeled in RAM Structural System.



lations

Maintaining the 45'x30' bays that are found in the original concrete design led to large trial sizes for the gravity members when designed as steel. Since the lateral loads were not yet incorporated at this time and they would increase the size of the steel members, it was decided to not use any 45'x30' bays and instead utilize 30'x30' bays as the largest found in the layout.

Fig. 18 Hypothetical Floor Plan

The above figure uses a hypothetical floor plan provided by Skanska and adds red boxes to represent where new steel columns would go, on top of replacing the concrete columns with steel ones. This shows that only minor alterations to the interior architectural features need to be made.

Once the layout was finalized and the trial members were assigned their sizes in RAM Structural System, the RAM Steel Beam and Column modules were used to verify the gravity redesign. The following table details the criteria used for the gravity checks performed on the beams and columns using the RAM modules.

Beam Design and Check Criteria	Column Design and Check Criteria
AISC 360-05 LRFD	AISC 360-05 LRFD for columns and base plates
Deck perpendicular to composite beam braces the flange	Trial groups of W14, W12, and W10 used
Camber included in design if necessary	Deck braces the column
Max stud spacing follows code Stud Placement: e mid-ht. < 2”	

Table 8 Design and Check Criteria for Beams and Columns

Beam Deflection Criteria	
Composite Unshored	Live Load = L/360 Superimposed = L/240 Total = L/240
Composite Shored	Live Load = L/360 Total = L/240
Noncomposite	Live Load = L/360 Total = L/240

Table 9 Beam Deflection Criteria

Since RAM Structural System will design members based simply on being able to resist the loads and the criteria defined by the user, all members were assigned the sizes calculated by the performed hand calculations so that there was consistency to the model, as well as to be economical. Trial groups are assigned to the columns in RAM, one trial group based on hand calculations and two automatically chosen by the program. The results again needed to be looked over and columns were resized to create consistency within the model.

Moment Frame Layouts

After the gravity system was designed and analyzed, it was time to choose moment frame layouts to resist the lateral loads. This process was iterative; numerous layouts were designed and analyzed. There were three main layouts that then gave way to minor alterations. The following table summarizes the criteria used to analyze the different layouts.

Load Case Analysis Criteria	Steel Standard Provisions
Rigid end zone effects are ignored	AISC 360-05 LRFD
P-Delta effects considered	B1 and B2 factors not applied due to the consideration of P-Delta effects
All diaphragms rigid	Top flange of beams continuously braced while the deck braces the column
Redundancy factors from IBC 2000/2003	KL/r < 200 for compression members L/r < 300 for tension members
Seismic Provisions	
AISC 341-05 LRFD	
Top flange of beams continuously braced while the deck braces the column	
Braced frames assigned as Special Moment Resisting Frames	

Table 10 RAM Frame Module Criteria

One of the goals when deciding on a layout for the moment frames was to limit the eccentricity so that torsional moments would be minimized. Placing the frames so that the center of rigidity and the center of mass were located within close proximity to one another guided the first layout which incorporated all unbraced frames. These frames were located around the elevator core where the shear walls were located in the concrete structure as well as around the perimeter of the building. This layout, however, would not provide a stable structure under the applied seismic loads. Braced frames were then incorporated into the layout.

The braced frames were located around the elevator core. Much like shear walls, the braced frames are highly rigid compared to their unbraced counterparts. This created large eccentricities which would in turn lead to significant torsion issues. To counteract this, the perimeter frames were also braced. This layout, shown in Figure 19, was satisfactory in terms of locating the center of rigidity close to the center of mass.

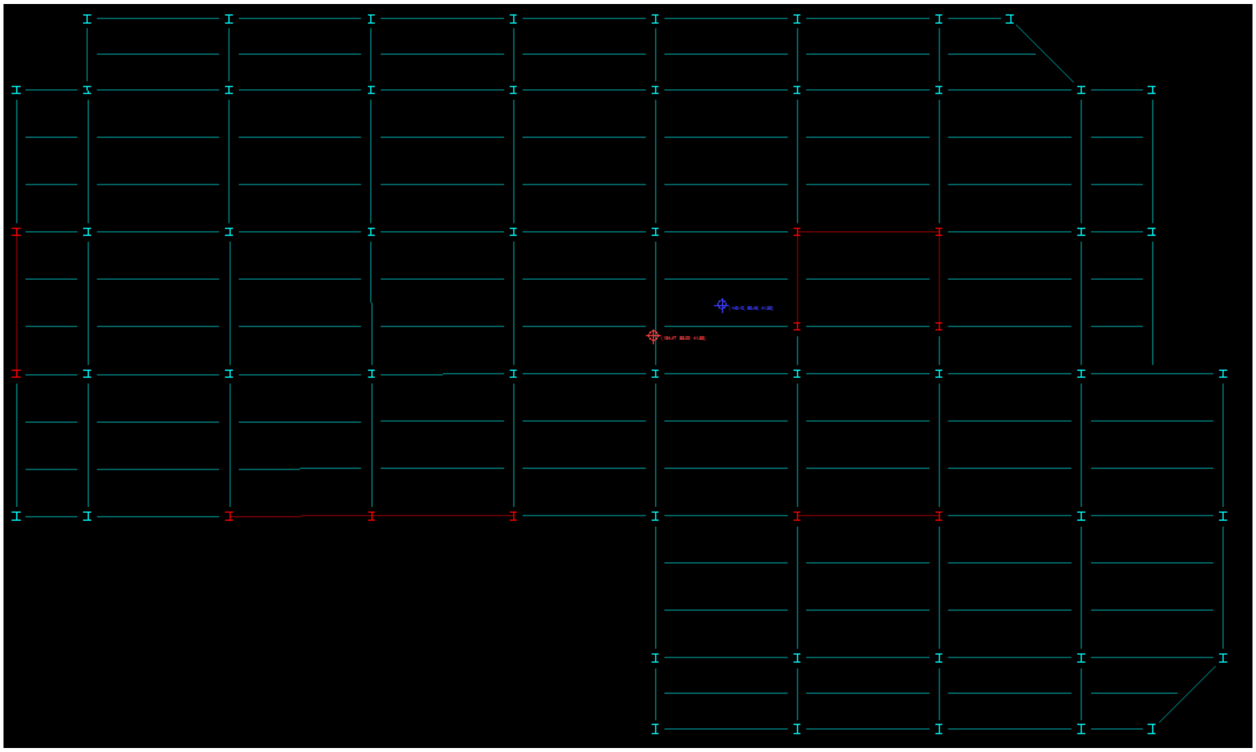


Fig. 19 Moment Frame Layout

While satisfactory in one of the initial goals, this layout was still flawed. It was favorable to place the braced perimeter frames on the South and East faces of the building but there was limited space to work with. Because the corner columns don't see much dead load, there would be potential uplift issues if they were included in a braced frame. This limited the South perimeter braced frame to only being one bay, three stories high. An interior unbraced frame also needed to be included in the Northern half of the building (the right side of figure 19) so that the center of rigidity and center of mass could be balanced on the 4th and 5th floors where the braced frames were discontinued, much like they were in the concrete structure.

The braced frames also came with significant architectural issues. The perimeter frames were designed using the special brace option within RAM Structural System. This allowed the frames to be modeled in a way such that the percentage of windows wouldn't have to be decreased as significantly as they would have been with standard braces. The ground floor, however, is all glazed around the perimeter of the building so there would be no work around to that aspect of the architecture. The braced frames would simply need to be visible through the windows. Potentially, if this layout was chosen, a faux bracing system could be incorporated around the ground floor to create the illusion that the braces are being used as part of the architectural design and are present in each exterior frame of the retail spaces. Despite these workarounds, the braced perimeter frames would compromise too much of the architectural design and it was decided another layout should be designed and analyzed. Figure 20 shows an elevation of the specially braced frame model that was under consideration.

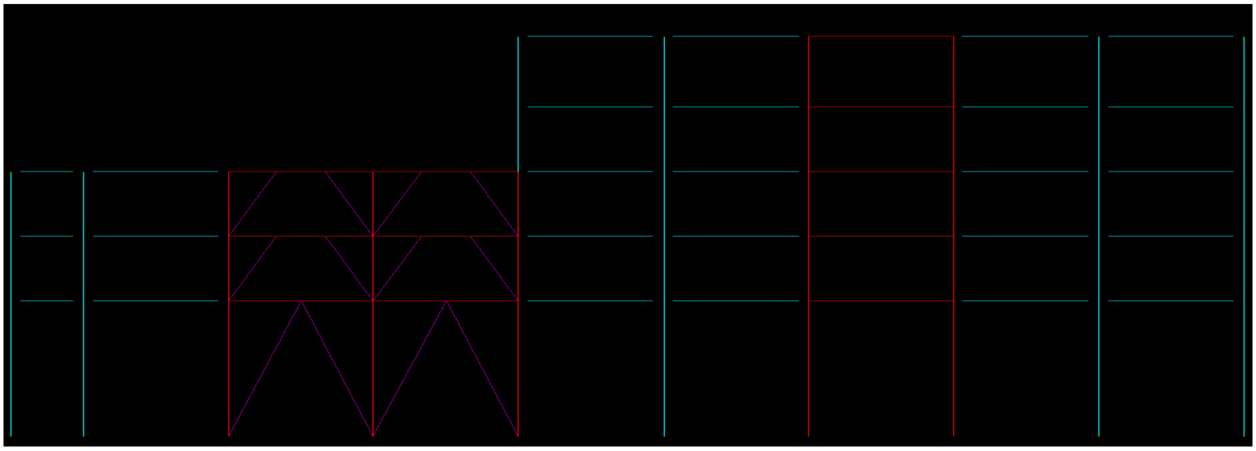


Fig. 20 Moment Frame Elevation: Specially modeled frames seen to the left

Due to architectural issues with the previous moment frame layout, it was decided to keep the braced frames on the interior of the building in locations that created minimal architectural impact. There are two stair wells in 1776 Wilson along with the elevator core. For the next layout, it was decided to place braced frames around the stair wells as well as keeping the frames around the elevator core. This layout resulted in a sufficient lateral force resisting system. The center of mass and center of rigidity for each floor are located in close proximity and unbraced moment frames are not necessarily needed which helps reduce the cost and schedule of a system that utilized both braced and unbraced frames. Figures 21 through 23 show framing plans with the locations of the braced frames highlighted in red, the locations of the centers of mass and rigidity, as well as a 3D view of the redesigned steel structure.

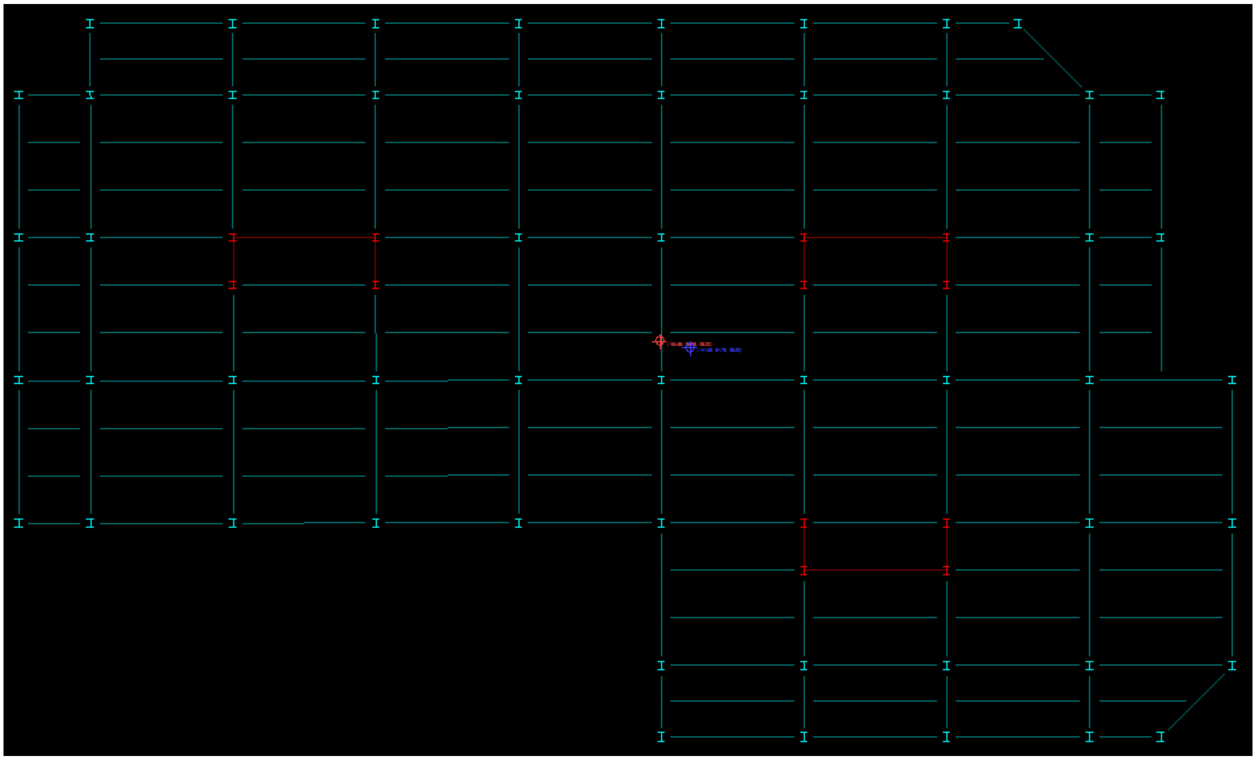


Fig. 21 2Nd Floor Framing Plan

The blue coordinates represent the center of rigidity and the red coordinates represent the center of mass. As shown, torsional effects are minimized in both the X and Y directions on the first 3 floors. Each braced moment frame was modeled as a special concentrically braced frame in accordance with seismic provisions for high seismic hazard regions.

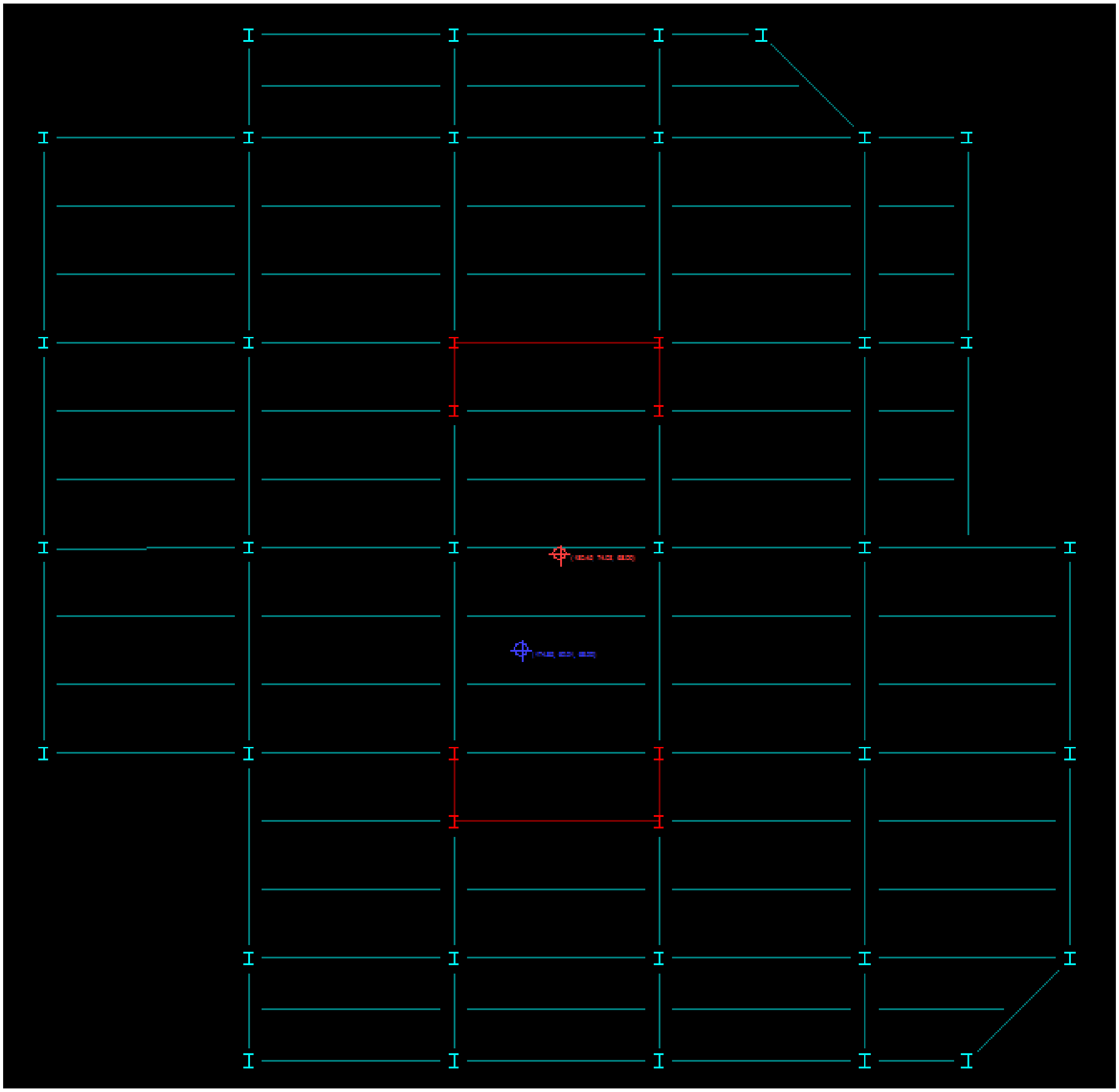


Fig. 22 5th Floor Framing Plan

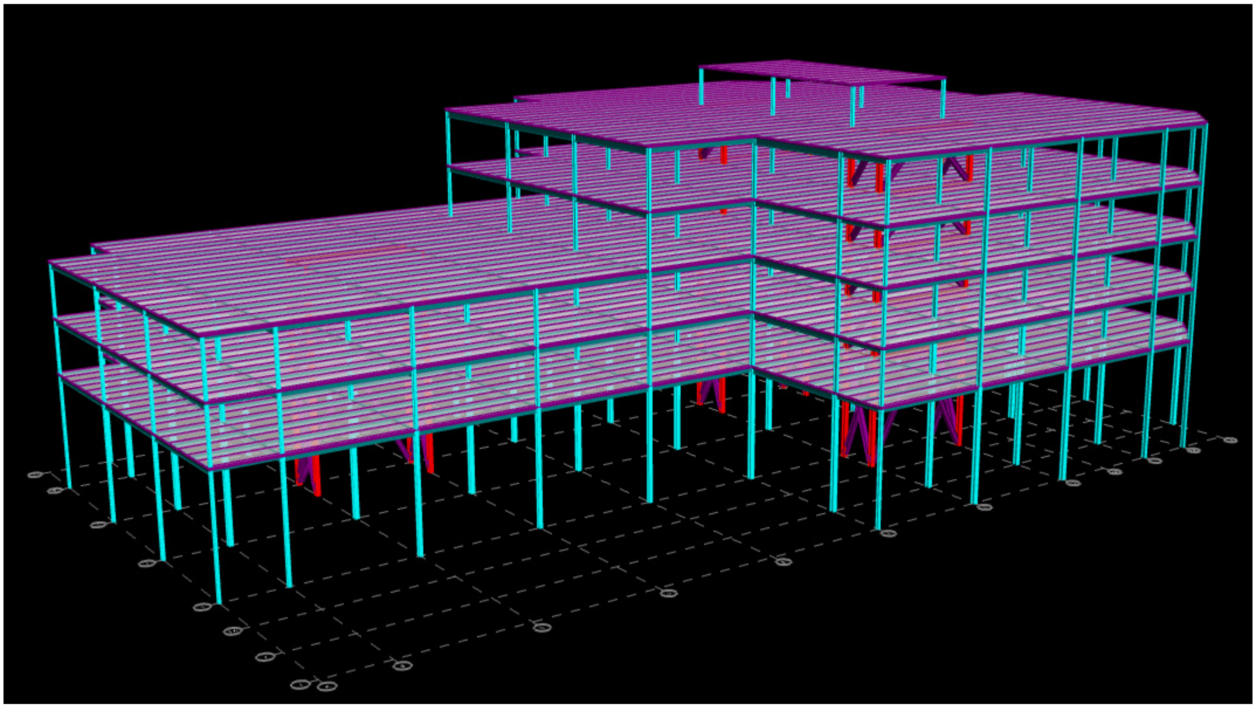


Fig. 23 3D View of Redesigned Steel System

The Northern half of the structure has a 4th and 5th floor, there are only two sets of braced frames in this area of the building. The upper braced frames around the elevator core and the lower braced frames around the stair well. The centers of mass and rigidity aren't balanced as well as on previous floors but the torsional effects due to this were taken into account in designing the members of the frames.

The final redesigned steel structure utilizes a composite system with 3VLI22 decking with 3" topping. Spray fireproofing is to be used to achieve the appropriate fire rating. This final layout removes braced frames from the perimeter of the building which gives it a distinct advantage in cost and schedule impacts because braced frames take less time to construct and cost less as well. All braced frames are designed to the seismic provisions found in AISC 341-05. Table 11 summarizes the member sizes found in the redesign.

Beam Sizes	Column Sizes	Brace Sizes
W10x22, W14x26, W16x26, W18x35, W21x44, W24x55, W24x76	W14x99, W14x109, W14x120, W14x132 W14x211	W14x120

Table 11 Summary of Member Sizes

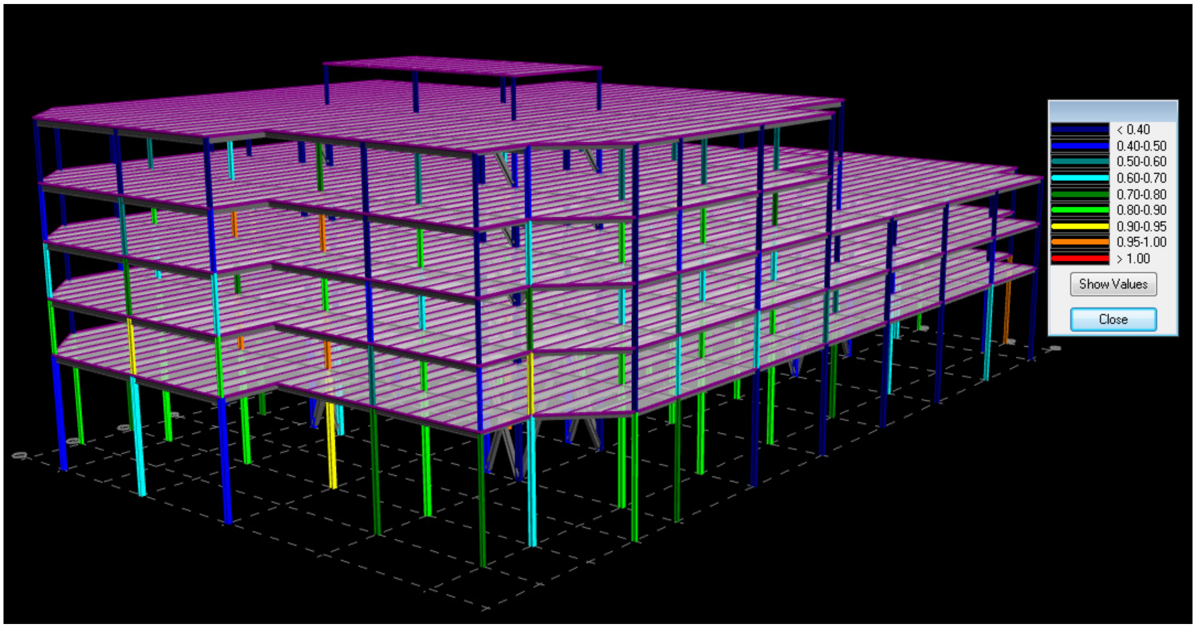


Fig. 24 3D View taken from RAM Steel Column module

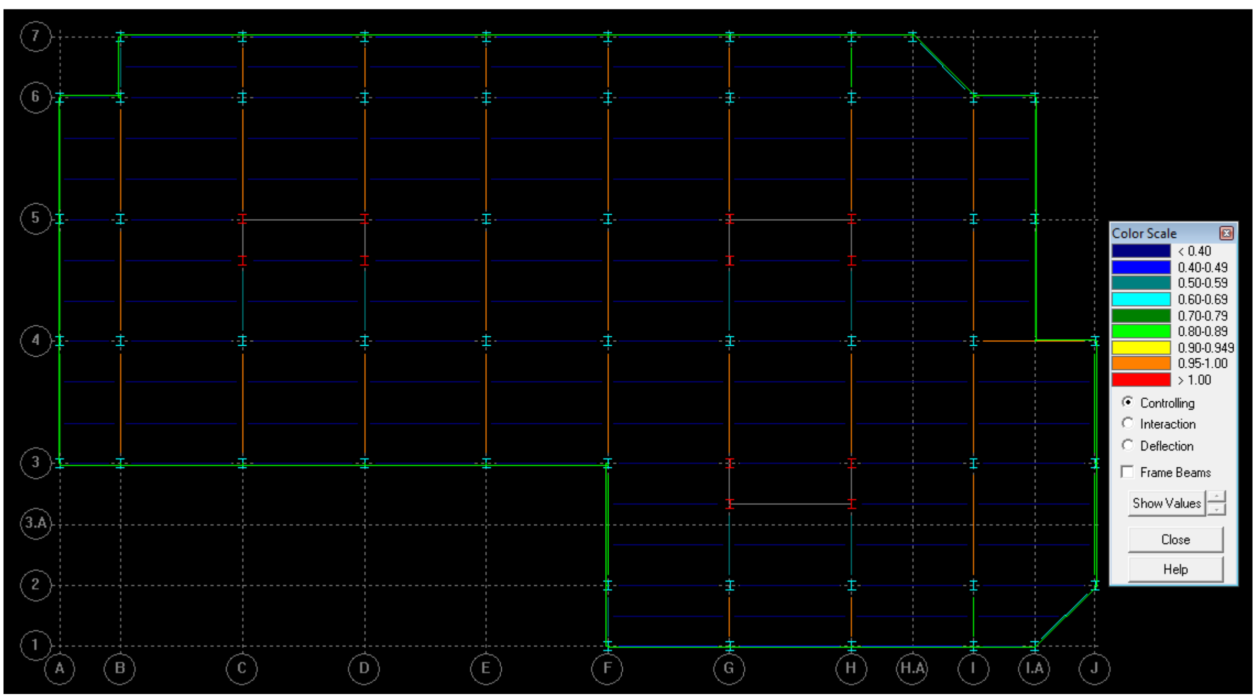


Fig. 25 Framing Plan taken from RAM Steel Beam module

Figures 24 and 25 show the RAM Structural System output for column designs as well as a framing plan with the output for the beam designs. RAM uses a simplified color coded system to express how well individual members handle the loads they are being designed for. The design results from the program agree with the trial member sizes selected through hand calculations verifying the accuracy of the model.

RAM uses trial groups for columns when designing and checking. Based on hand calculations, all W14's were used for columns but RAM analyzes three different sizes as the trial groups. Because of this some columns are designed through the program as W10's or W12's. All columns were manually updated so that consistency was created through the column size choices.

For the framing plan showing the beam designs, RAM uses interaction and deflection values to show how sufficient members are. The controlling value is used for the color coded results seen in Figure 25.

Breadth Topics

Construction Management

The purpose of the Construction Management breadth topic for this project was to determine what cost and schedule changes would occur because of the relocation of 1776 Wilson Boulevard to Oakland California. Any delay in the opening date of the building would cause a loss of money for the owners and for the tenants set to lease the office and residential spaces. This makes it important to ensure the redesigned schedule doesn't increase the length of construction. It is assumed that the building will hold true to the same construction plans as were set in place for the building in Arlington Virginia which is an August 2012 opening date.

The schedules will be done for the superstructures and critical paths will be determined through available floats for each of the tasks. The overall cost of the designs is dependent on the durations of each task so the schedules must be completed before the cost analysis can begin.

First, the existing schedule for the concrete structure in Arlington was acquired and a simpler version was made in MS Project to use as a base for the redesigned system's schedule. It was decided that the best approach for structural changes was to take note of volume changes in materials, in cubic yards for concrete, and use RS Means for duration and cost data based on the change in volume. Table 12 shows a sample calculation in terms of volume of concrete used for columns.

Design	1st-2nd	2nd-3rd	3rd-4th	4th-5th	5th-Roof	Roof-PH	Total
Existing	211.6	81.6	81.6	59.7	65.7	9.96	510.16
New	330.6	157.4	157.4	125.6	138.3	20.9	930.2
RS Means	Existing	New			Duration	Cost	Increase
Daily Output	17.71	22.84		Existing	28.8 Days	$675,962	
Total Incl. O&P	1325	1425		New	41 Days	$1,325,535	$649,573

Table 12 Column Cost Comparison

The above table includes forms, reinforcing steel, concrete, placement, and finishing. Units for quantities are in cubic yards. Daily output is in cubic yards per day and total cost (including overhead and profit) is dollars per cubic yards.

After compiling the updated information for the redesigned concrete structure, a schedule was made in MS Project and Excel. The Excel schedules were created to provide more depth and information to each individual task. Each task is given a box that notes the duration, early and late start/finishes, as well as available floats. An example of this can be seen in Figure 26. Full Excel and MS Project schedules can be found in Appendix F.

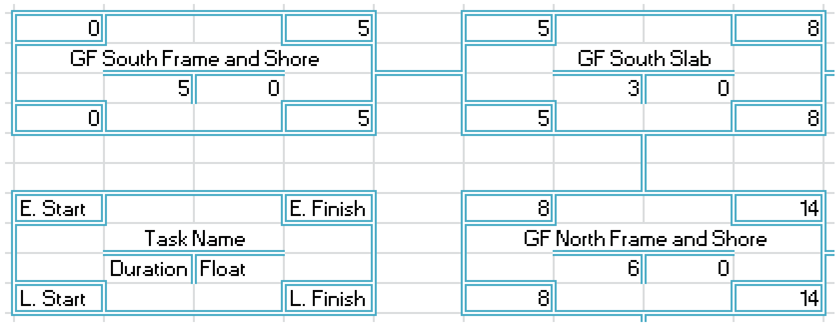


Fig. 26 Excel Schedule Excerpt

After completing a forward pass of the Excel schedule to determine the total duration of the work, a backward pass was performed to determine the available floats for each task. The critical path is defined by the tasks that have zero float which indicates any delay in that particular task would delay the final completion date for the project. A similar Excel schedule was created for the steel redesign as well.

Once the concrete schedule was completed, the steel schedule was created. Similarly to the previous work, RS Means was used to help calculate the durations and costs of each task. Since this part of the project was a complete redesign, there was no existing information to use a basis for the work done on this part of the project. Complete take offs for all structural elements would need to be performed.

Using the created framing plans, take offs were done by hand and taken from RAM Structural System where possible. Take offs performed by hand were done to verify the data found in RAM. After all quantities were found, durations and costs were determined in a similar fashion to the concrete process. RAM Connections was used to design typical connections so that bolt and plate quantities would also be taken into consideration.

All of this data was compiled and the appropriate costs and schedules were documented. Lateral members take longer to install and cost more; these members were included in the appropriate sections (lateral columns were combined with gravity columns, braces were combined with columns, etc.).

System	Base Cost	Location Factor	Updated Cost	Duration
Existing Concrete			$3,992,527	73 Days
Redesigned Concrete	$4,642,100	1.234	$5,728,351	77 Days
Redesigned Steel	$3,743,084	1.234	$4,618,966	67 Days

Table 13 Results Summary

As indicated by the summary of results seen in Table 13, both redesigned systems will cost more than the existing concrete design in Arlington. Due to higher seismic loads and more detailed designs, reinforcement in the concrete system and connections in the steel system, this is not a surprising result. The redesigned steel structure would offer significant cost savings compared to the updated concrete structure and the construction time of the superstructure would also be shorter.

The steel numbers for cost include fireproofing. The concrete structure would not need additional fireproofing like the steel structure. It was assumed that fireproofing would not be a critical part of the schedule, as the superstructure is constructed fireproofing can begin on lower floors that are complete while the upper floors are erected. It is also assumed that fireproofing that continues past the final superstructure date will be concurrent with other aspects of the schedule so it will still remain not critical. If the fireproofing was deemed critical, it is important to note that the steel structure would have the longest duration.

Sustainability

One of the biggest selling points of 1776 Wilson Boulevard is its LEED Platinum rating. The building boasts many sustainable features including a photovoltaic solar panel array and a 17,000 SF green roof. Upon changing locations to Oakland California, it is essential that the LEED rating is maintained. The various green aspects of the project, found via a LEED scorecard provided by Skanska, were broken down into a priority list to see which features would be most important to 1776 Wilson and should be investigated for this breadth topic.

Upon inspection of the LEED scorecard, it was found that 1776 Wilson, while LEED Platinum rated, might not actually provide the energy savings you might expect from a LEED Platinum building. Some major aspects of the score would not change, or be changed only slightly, with a location change. These were not investigated or covered in the scope of work. Site related features turned out to be very important to the LEED rating. A redeveloped site, local amenities, and cutting back on transportation were among location and site related goals that 1776 Wilson achieved. These aspects were chosen as the highest priority features and played a role in the new location choice.

Priority	Category	Tasks
1	Sustainable Sites	Research new location choices Site type Local amenities Minimize traffic impact Heat island effects
2	Energy and Atmosphere	Green roof performance PV Solar Panel performance
3	Indoor Environmental Quality	Heating and cooling loads

Table 14 Priority List

As mentioned throughout this report, the new site location was chosen to be Oakland California. A major green feature of the current site in Arlington is that it is a redeveloped brownfield site. The specific site chosen for the relocation, the Central District of the metropolitan San Francisco/Oakland area, provides brownfield redevelopment opportunities. Oakland has a strong history of obtaining funds to study and redevelop brownfield sites. Because of this, it became a strong favorite in being chosen as the final relocation site.

The Central District specifically is undergoing a redevelopment plan in order to strengthen its role as an important office and entertainment center with strong community connectivity. This is similar to a redevelopment plan that is going on, and has been going on, in Arlington. Because many locations can be found with similar goals and of varying amounts of success, this wasn't a crucial aspect of choosing a new site location. However, there are still some benefits that go along with this that help with LEED ratings.

40 local AC transit bus lines connect the Central District with nearby locations around Oakland. Restaurants and retail outlets can be found within walking distance as well. These greatly help reduce the traffic impact the building will have on the area. The included below grade parking will prevent tenants from needing to find parking nearby and will also negate any impact on public parking space the building would have had if the below grade parking was not included.

A new site will also come with a different heat island. The city of San Francisco/Oakland area is much denser than Arlington and will therefore have a greater urban heat island effect. Because of this, research was performed to see how effective the existing green roof would be after the relocation was made. To do this, a green roof calculator created in large part due to the efforts of Dr. David Sailor of Portland State University was used to gather data on green roof performance.

The same green roof was modeled for both locations. An extensive green roof was used with 6 inches of growing media depth and a typical leaf area index value of 3 was used. The green roof located on the lower roof covers 67% of the roof surface, sharing space with the 3600 SF terrace. The green roof on the upper roof covers approximately 41% of the surface and shares space with the solar panel array. The closest location to Arlington that the program had included was Richmond, so that became the site to analyze the green roof at. After running the calculator, the location was switched to San Francisco California. Table 15 summarizes the data found through this calculator.

	Richmond Virginia	**San Francisco California**
Energy Savings Compared to White Roof	$863	-$160
Energy Savings Compared to Dark Roof	$1409	$957
Summer Peak Daily Average Sensible Heat Flux	-53.3 W/m^2	132.4 W/m^2
Summer Peak Daily Average Latent Heat Flux	124.4 W/m^2	0.2 W/m^2

Table 15 Green Roof Calculation Summary

Full results can be found in Appendix G.

As depicted by the calculator's results, the green roof as modeled in San Francisco doesn't exhibit significant energy changes or aid to the urban heat island effect. The sensible heat flux is heat given off by the roof's surface that causes a temperature change in the nearby environment. This is counteracted by the conversion of solar energy to latent heat energy. The latent heat is a result of evapotranspiration and helps cool the roof's surface as opposed to warming it through sensible heat. The existing green roof is much more efficient for the Richmond area and offers some cost savings as well. When compared to a white roof, with an albedo value of 0.65, the San Francisco green roof will actually cost money instead of providing savings to the owner.

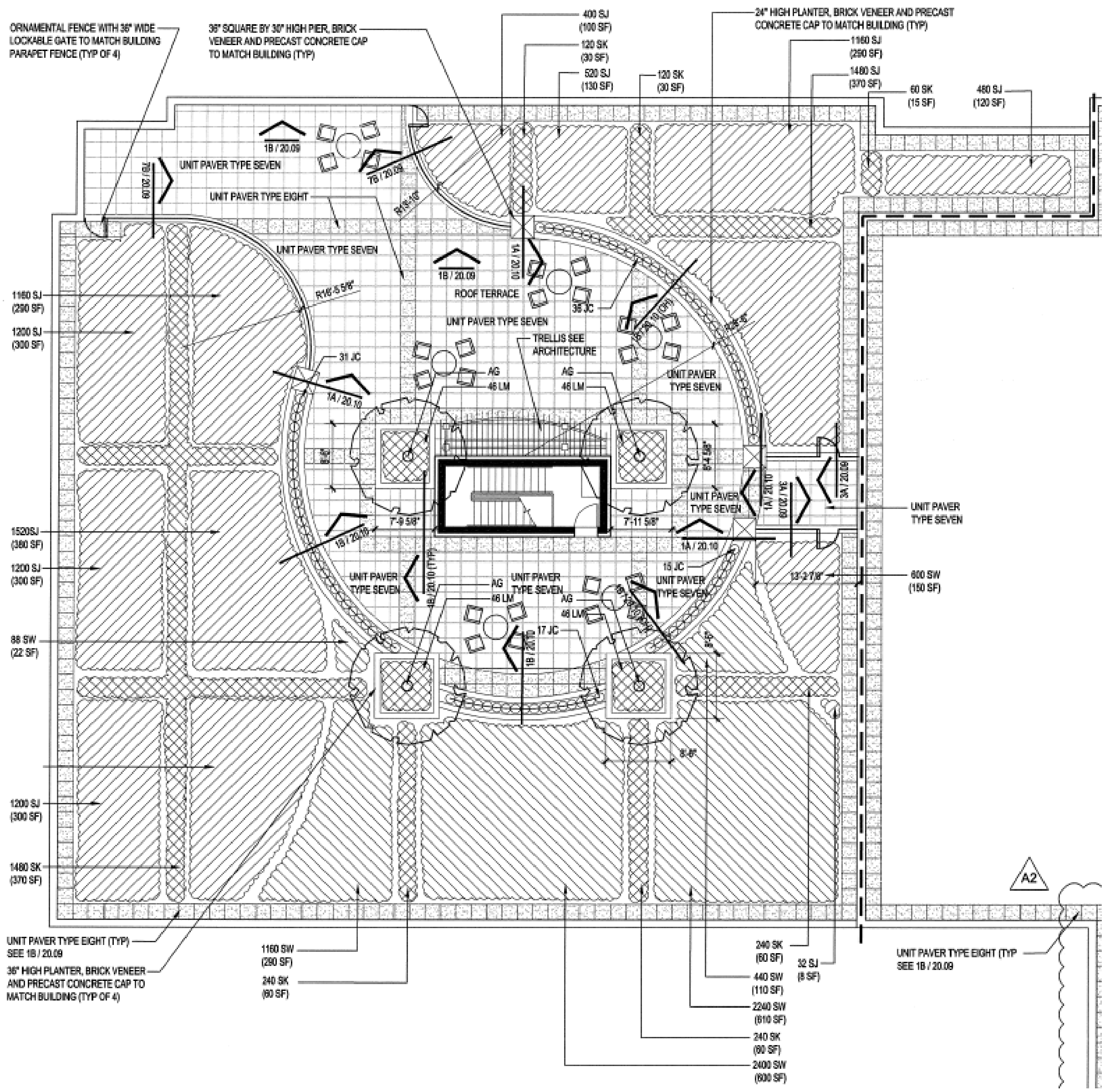


Fig. 27 Lower Green Roof Plan

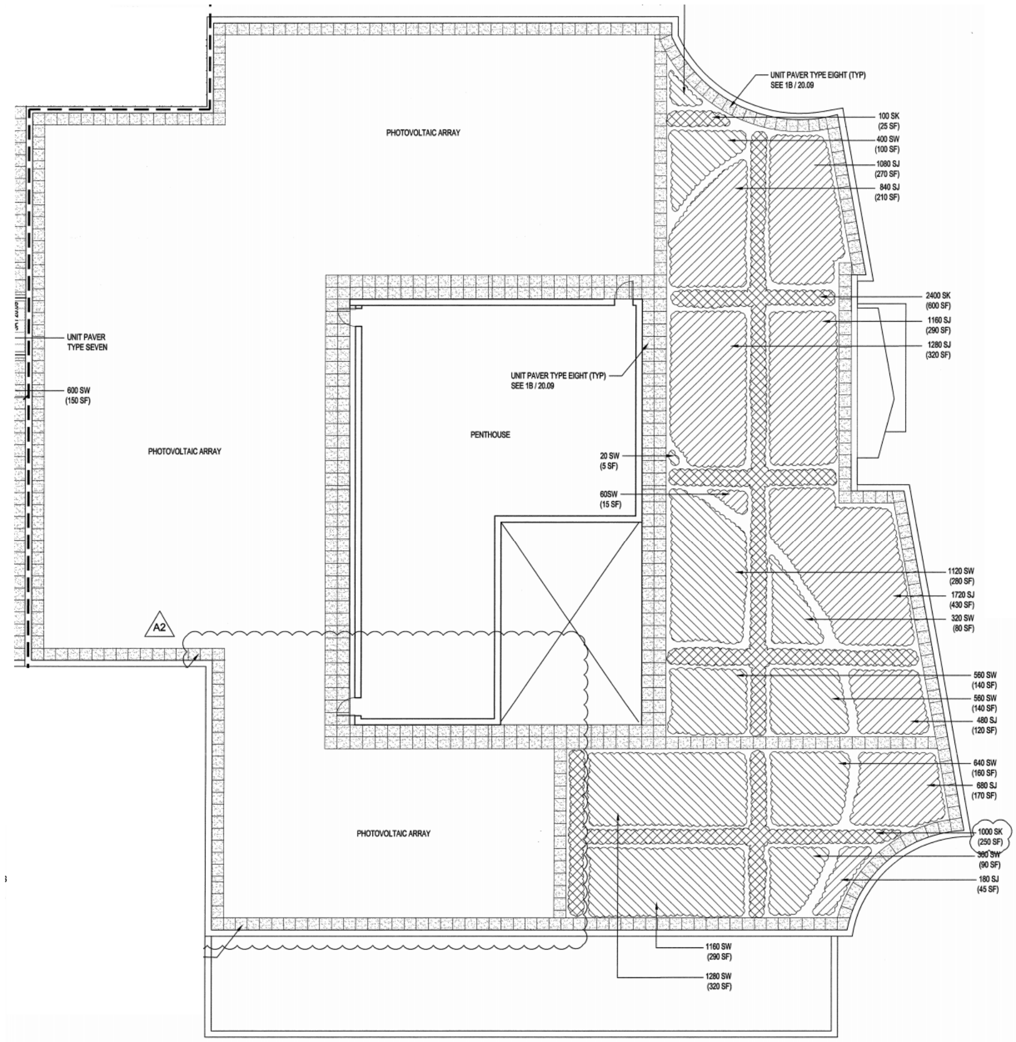


Fig. 28 Upper Green Roof Plan

The albedo value reduces the daytime energy input as it increases. White roofs typically have an albedo value of 0.65. In this instance, based on energy performance and impact on the urban heat island effect, the white roof would be a more economical choice. However, the green roof helps maintain the LEED Platinum rating which could be seen as a much better selling point and could be well worth the costs, which is a difference of $160 annually.

The next step in analyzing the sustainable features of 1776 Wilson Boulevard was a solar photovoltaic panel analysis and comparison. This resulted in much more favorable results for the new site location.

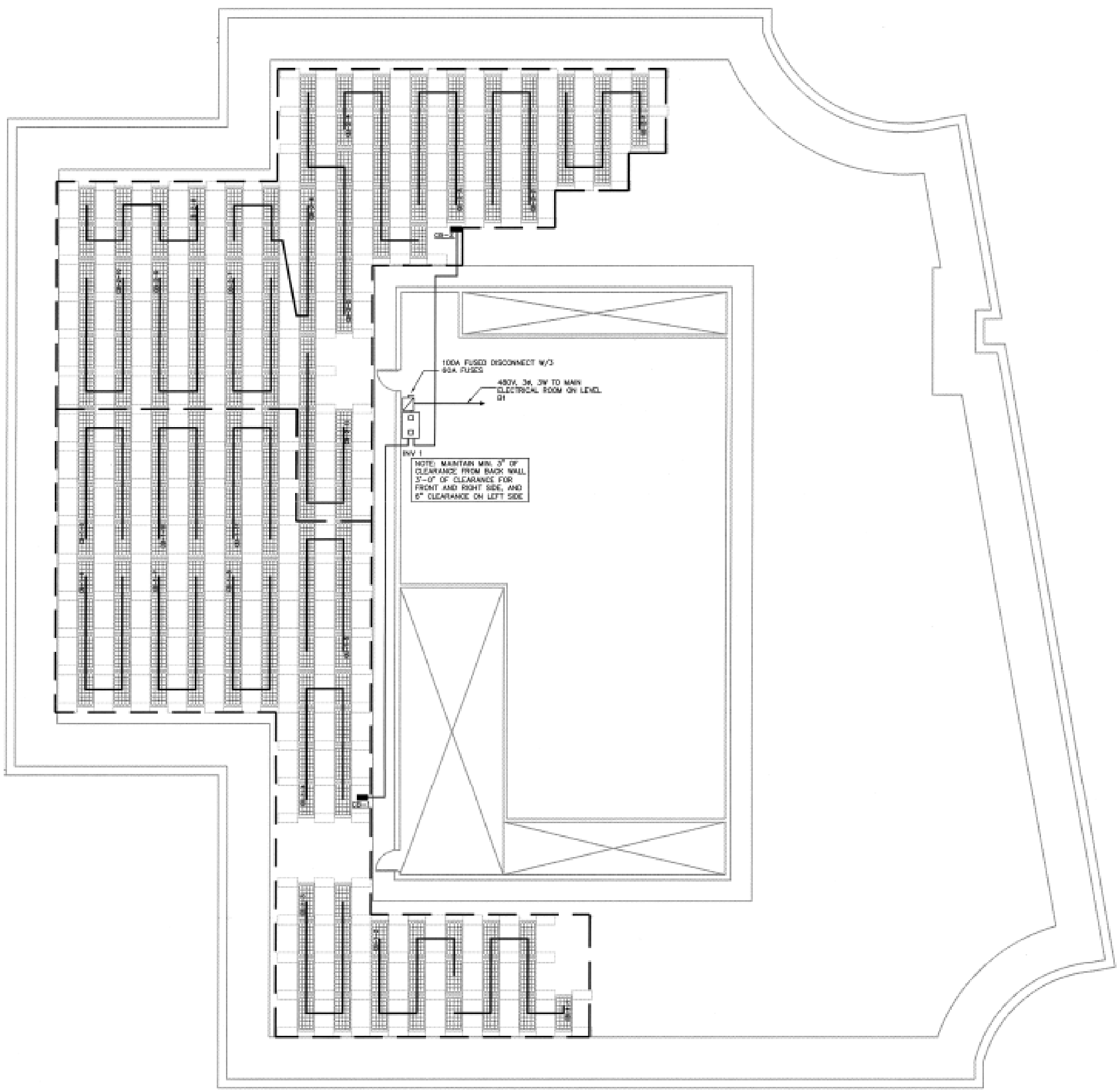


Fig. 29 Solar Panel Layout

1776 Wilson uses an array of SANYO HIT Power 220A solar pv panels, the product sheet can be found in appendix G. The array, pictured in Figure 29, utilizes two combiner boxes with 11 strings per box. Each string connects 8 PV panels for a total of 176 panels and a total system size of 38.72kW.

"Photovoltaic Systems" by James Dunlop (2010) was used to determine peak sun hours for both Arlington and Oakland. These hours were broken down into summer, fall, winter, and spring peak values that were then used to calculate annual kW-hr values. It was assumed that the conversion from DC to AC power resulted in 20% losses so the calculated DC values were multiplied by 0.80 to arrive at the power generated by the system in both locations.

The web based computer program PV Watts was used to get values that could be compared to, and subsequently, verify the hand calculations performed. Table 16 summarizes these results and differences. The closest locations available to Oakland and Arlington were used.

Location	Hand Calc Value (kW-hr/year)	PV Watts Result (kW-hr/year)	% Difference
San Francisco	54,429	53,180	2.29%
Sterling	44,876	44,954	0.17%

Table 16 Comparison of Hand Calculations and PV Watts Results

Full details on this portion of work can be found in Appendix G.

Using the utility rates from PV Watts as well as researched annual rate increases, an excel spreadsheet was created to determine the payback period of the solar panel array so that it could be compared with the payback period of the system in Arlington. Due to a higher cost of electricity and higher AC energy output, it seemed likely large savings would be found in the relocated system and that the payback period would be significantly shortened.

The US Treasury offers an ITC grant for solar panel systems under the American Recovery and Reinvestment Act of 2009. This grant covers just over $71,000 of the cost to install the system. This leaves a net installation cost and initial investment of $166,690. Data was provided from the current project that detailed tax assets and liabilities, however due to limited information it was decided to look at the pay back periods without incorporating the taxes. In the end, the initial one-time tax asset was greater than the sum of the tax liabilities creating additional profit.

There are additional maintenance costs that needed to be incorporated in the study. Every 5 years there would need to be a meter replacement valued at $2500 each time. Every 10 years, the inverter will need to be replaced adding an additional $7000 on top of the meter replacement cost for that year. Operation and maintenance costs will also increase each year along with a decrease in production each year. It is assumed the panels will have a decreased production of approximately 0.7% per year and the O&M costs will increase 2% per year.

Using the data provided, a payback period neglecting tax assets and liabilities of 30 years was found. Using Excel, a payback period for the relocated system was found next. This new payback period was 18 years, a significant 12 year decrease. The big reason for such a huge difference in payback periods, and a big reason on why solar panels are gaining popularity in California, is because the utility rates have been increasing 6-8% per year and it is projected that these percentages will hold true for at least another decade. California already pays above the national average for electricity and with this trend; citizens of the state will continue to pay more and more. This creates a great opportunity for solar energy. As electricity prices continue to rise, the amount of energy you gain from solar panels will be worth more. Even though the production of the system will decrease over time, it will be offset by the fact that the value of the energy you're saving is increasing dramatically.

Due to these significant savings, research was done to find possible uses for this saved money and if it could be used in any way to increase energy efficiency of 1776 Wilson. One idea investigated was to modify the green roof so that it would be more efficient for the new location. Green roof prices range from $15-$30 dollars per square foot depending on the type of green roof so unless significant differences could be made, it wasn't a viable option. Through various iterations using Portland State University's Green Roof Building Calculator, it was confirmed that this wouldn't be the best place to spend more for sustainability.

Another idea investigated, something that has gained popularity in Europe and is starting to catch on in the United States, is incorporating a green roof and solar panel array into one system as opposed to using separate areas of the roof for both. Heat is the biggest threat to the efficiency of a solar panel. The green roof serves to cool the roof so by placing panels over the vegetation, you will be cooling the solar panels as well which increases efficiency.

Fig. 30 Solar Panels On Green Roof

Another advantage found in this system is the fact that the racking system can be integrated into the green roof in a way such that the roof layers act as a ballast. This could prevent the need for penetrations or concrete pavers.

The biggest issue with trying to determine how useful this system integration could be is the lack of options. Since it hasn't been used extensively yet in the United States, it is troublesome to model and analyze. The energy analysis software available to the author for this report didn't have options to place solar panels over a green roof and analyze the effectiveness. Through research, it was found that efficiency of panels could be increased up to 6%. It was decided to use the mean value, 3%, as the efficiency increase the solar panels on 1776 Wilson Boulevard would see.

Using the same excel spreadsheet as before, the energy benefits of the more efficient panel system replaced the previous values and a new payback period was determined. The plan was to place a green roof system underneath the existing PV array on the upper roof. This would require 9,900SF of green roof. Since the system is required to be extensive, the pricing was on the low end. A total value of $148,500 dollars would need to be added to the total cost of the project nearly doubling the installation cost of the PV system. This value was included in the installation cost to see when the solar panels would not only reach their payback point but to also see when they would make up for the cost of the green roof as well.

Solar panel efficiency is based on W/m^2. A 100% efficient collector would absorb 1000 watts per square meter of collector area. The area of the panels used on this project is 1.26 m which means 1260 W/m^2 is 100% efficient. Using the data given on the product sheet the efficiency was calculated at 17.5% which agrees with the value given by the manufacturer. The new efficiency was then set at 20.5% and the total system size increased to 45.5 kW. In the end, a 21 year payback period was found which includes the cost of the green roof addition.

Investing nearly $150,000 extra dollars on the solar panel and green roof system will increase the payback period by three years however, even at just 3% more efficient significant increases in energy value are found. Once the system pays for itself and the green roof addition, 1776 Wilson will be seeing a huge decrease in annual electricity costs which makes this investment worthwhile.

It is important to note that as this system is put to more use in the United States, more precise data will be known on how efficiency is affected and a more accurate analysis could be done.

The last step of this sustainability breadth topic is to calculate sample heating and cooling loads for a south facing office to see if the current HVAC system is oversized or undersized for the new loads. An Excel spreadsheet was used to help simplify the arduous calculation procedure that is outlined in the ASHRAE Fundamentals Handbook. Appendix G shows detailed information on this portion of the project.

Information acquired from the mechanical engineers on the project helped fill in bits of information that were needed to complete this process. Research into the climate of Oakland was also performed to try and gather accurate information that could be used for the calculations. Sample hand calculations were performed following an example listed in ASHRAE, however calculations need to be performed for a 24 hour profile so using a spreadsheet was a necessity. Cooling loads were calculated first since they are inherently more complex.

The first step was to determine the room characteristics and the design conditions. Table **** outlines this data.

Room Characteristics	
Area	150 ft^2
Wall Area	100 ft^2
Glass Area	63 ft^2
Design Conditions	
Heating	75 Degrees F
Cooling	77 Degrees F 50% RH
Internal Loads	
Lights	614 BTU/hr
People	450 BTU/hr
Misc.	1,408 BTU/hr

Table 17 Room Characteristics and Design Conditions

The Radiant Time Series method was used for the cooling load calculations. It was assumed that the internal loads would remain the same despite the location change. These loads include those due to people and lighting, along with miscellaneous loadings.

ASHRAE details different wall types in which you choose the closest that matches your building in order to determine certain factors that need to be used; this is mostly used to determine the Conduction Time Factors.

After working through the entire process outlined in ASHRAE, cooling loads were determined that could be compared to the cooling loads needed in Arlington. The heating loads are determined in a much simpler fashion. Several assumptions are made that lead to the simplification, as listed below.

- Single outside temperature
- No heat gain from solar or internal sources
- Steady state conditions

Tables 18 and 19 show the results of these calculations and the comparisons to the loads in Arlington.

Cooling Loads							
Oakland	Walls	Windows	Lights	People	Misc.		Total
	127	1242.44	614	450	1408		3841
Arlington							
							5916

Table 18 Cooling Loads

Heating Loads	
Oakland	Total
	4384
Arlington	
	4706

Table 19 Heating Loads

As indicated by these results, Oakland is in an area where heating is the dominate design concern and energy consumption is relatively moderate for both heating and cooling. Oakland is located in California's Climate Zone 3 which verifies these calculation results in terms of heating being the dominate design concern.

Since both heating and cooling loads are lower than their Arlington counterparts, the designed HVAC system would be overdesigned after the relocation was made. Resizing equipment was not a part of the scope of work for this project but it is concluded that the cost savings found after downsizing the equipment can be put towards more energy efficient equipment.

Conclusion

1776 Wilson Boulevard was moved from one extreme to another in terms of seismic loads when it was relocated from Arlington Virginia to Oakland California. After moving the building to a high seismic hazard region, the post tensioned concrete system needed an increase in member sizes and significant reinforcement increases, particularly in bottom bars over column locations and near slab edges. Due to this it was decided that a steel alternative system should be designed.

The result was a composite steel structure with special braced concentric frames around the elevator core and stair wells. These frames provided enough resistance to the increased lateral loads and minimized effects due to torsion. This layout also keeps all braced frames in the interior of the building which means there are no impacts on the exterior architecture of the building. Interior impacts on architecture were found to be minimal.

For the construction management breadth, the superstructure costs and schedules were analyzed. The redesigned steel structure is able to be constructed 10 working days faster than the redesigned concrete structure and it would cost approximately $1.1 million dollars less as well. Because of inherent cons that a composite steel system has in comparison with a post tensioned concrete building, especially an office building that calls for open space floor plans, it was crucial that the steel redesign had an advantage in time and cost. The new structure can also be constructed quicker than the current design for Arlington, however it would be more expensive. This is important to note because a delay in the opening date of the building will cost the owner's money and the tenants who are leasing the retail and office spaces will be delayed in opening at their new location which could also impact their profits and productivity.

1776 Wilson will be able to maintain its LEED Platinum rating after being relocated. The new site area offers brownfield redevelopment opportunities and has plenty of bus transit and nearby amenities to cut back on transportation. On-site parking also aids the LEED rating in this area. It was found that a white roof would be more economical than a green roof in Oakland, but in order to get LEED credit for reducing the urban heat island effect, the green roof will still be implemented. The photovoltaic solar panels provide a much greater energy benefit after being relocated, cutting the payback period for the system down by a significant 12 years (not including tax assets and liabilities).

Because of those savings, a study was done on using a green roof under the PV solar panel array to help cool the panels and increase their efficiency. The more efficient solar panels would take 21 years to pay back the cost of installation and the additional cost for adding more to the green roof.

Lastly, heating and cooling loads were calculated for a south facing office to see if the existing HVAC design would be oversized or undersized in the new climate. The cooling load for Oakland was calculated to be 3841 BTU/hr and the heating load was found to be 4384 BTU/hr. Both of these loads are lower than those found in the Arlington building which means the HVAC equipment would be oversized after relocation. Downsizing the equipment can create cost savings that can be used towards more energy efficient HVAC equipment.