# FDA OC/ORA Office Building

Silver Spring, MD

Final Report

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Structural Option

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**Architecture:**

- Mixed use office building that forms the final phase of the consolidation efforts of the FDA.
- The 500,000 S.F. Office Building was laid out to morrior the existing buildings on the site. The building is broken up into 4 wings, A through D.

**Structural:**

- Spread Footings were used for the building's foundation. Where 95% compaction could not be met; Geopiers were used under the footings.
- The structure of the building was designed to prevent progressive collapse. The exterior beams of each floor are the primary elements in the progressive collapse design.

**Building Statistics:**

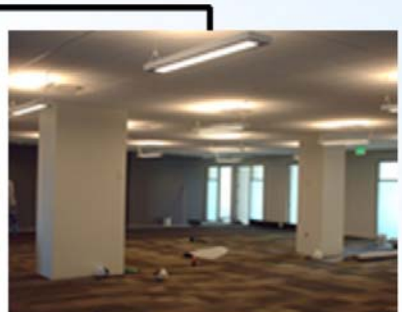
Size: 500,000 S.F.
Construction Dates: 6/07 to 12/10
Final Contract Cost: $110 Million
Delivery Method: Lump Sum Project

**Mechanical Systems:**

- 4 AHU's are provied on the roof with VAV boxes throughout the building.
- The AHU range rom 20,000 to 30,000 L/s.

**Project Team:**

Owner:
GSA
Occupant:
FDA
General Contractor:
Tompkins Builders
Construction Manager:
Heery-Tishman
Architectural Engineer:
KlingStubbins in association with RTKL
Geotechnical Engineer:
Schnabel Engineering Association



**Lighting / Electrical Systems:**

- Daylighting controls are provided with the florescent lighting system.
- 13.8 kV is supplied from the Central Utility Plant.
- A Unit Substation in Building 31 supplies the swtchgear and then supplies 208Y/120 V to each wing.

# Table of Contents

# Executive Summary

A building design is very integrated; every change to a building has some effect on other functions of the building. A change to the structure can affect cost, schedule, serviceability, MEP Coordination, and Architecture. It is the purpose of this thesis to investigate some of the affects of a change to the building, while exploring more advanced structure design topics. It is proposed to change the superstructure of the FDA OC/ORA Office Building Wing B from concrete to steel.

Using the existing grid and column lay out with a few minor changes, the steel framing was determined, and the gravity system was designed. Braced frames around the core were designed to take the primary lateral load and to limit the deflection of the wing. Exterior moment frames were designed to aid in the resistance of progressive collapse and reduce the eccentric effects of the wing.

To show the constructability of the new structure typical connections were designed for both the gravity and lateral system. A typical beam to girder connection and a girder to column connection were designed for the gravity system. Typical lateral frame connections were designed as a flange bolted moment connection, and a heavy braced connection.

As the threat for unforeseeable events increase, the design of structures to resist progressive collapse is becoming more important. The original structure was designed to resist progressive collapse, and part of this thesis was devoted to the research of the design of structures to resist progressive collapse. Two methods exist for the design to resist collapse, and both methods are presented in this thesis. To further understand the design progressive collapse a method will be chosen, and the design of removal of a column will be implemented.

Impacts on the cost and schedule were studied and compared to the original design and the new structure design. Both cost and schedules were created using RS Means Building Construction Data 2007.

The mechanical system was studied in respects to the ceiling to floor space where the mechanical ductwork passed through, the new structure design limited the space for the mechanical ducts and at a critical location, and two of the ducts will be resized. The ducts were resized using the existing airflow through the system, and were sized to fit in the new adjusted space.

# Introduction

Starting the fifth phase of the consolidation efforts by the FDA, the OC/ ORA Office building plans to move the Office of Commissioner (OC), Office of Regulatory Affairs (ORA) Office building to the White Oak Campus. On the site of the former US Navy facility at the Federal Research Center- Naval Ordnance Laboratory, the OC/ ORA Office Building sits on the southern end, and forms its shape around the existing buildings.

Forming an S shaped building, the 500,000 S.F. office building was laid out and designed to mirror the existing buildings on the site and to form a unique face of the campus from the main drive off of New Hampshire Ave.  Broken up into two buildings with four wings, Building 31 is comprised of Wing A, and Building 32 is comprised of wings B through D (Figure 1).

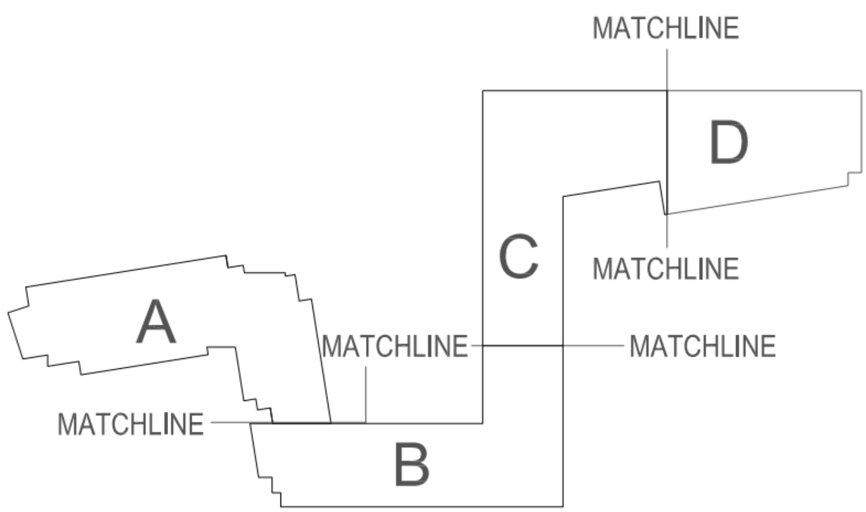



Figure 1: Key Plan

# Structural System

## Foundation:

The foundation of the building is separated into two categories. Spread footings that bear on undisturbed soil or spread footings that sit on a number of Geopiers. Schnabel Engineering conducted soil test to determine the bearing capacities of the soils. Where 95% compaction could not be met the use of Geopiers or vibropiers was recommended.

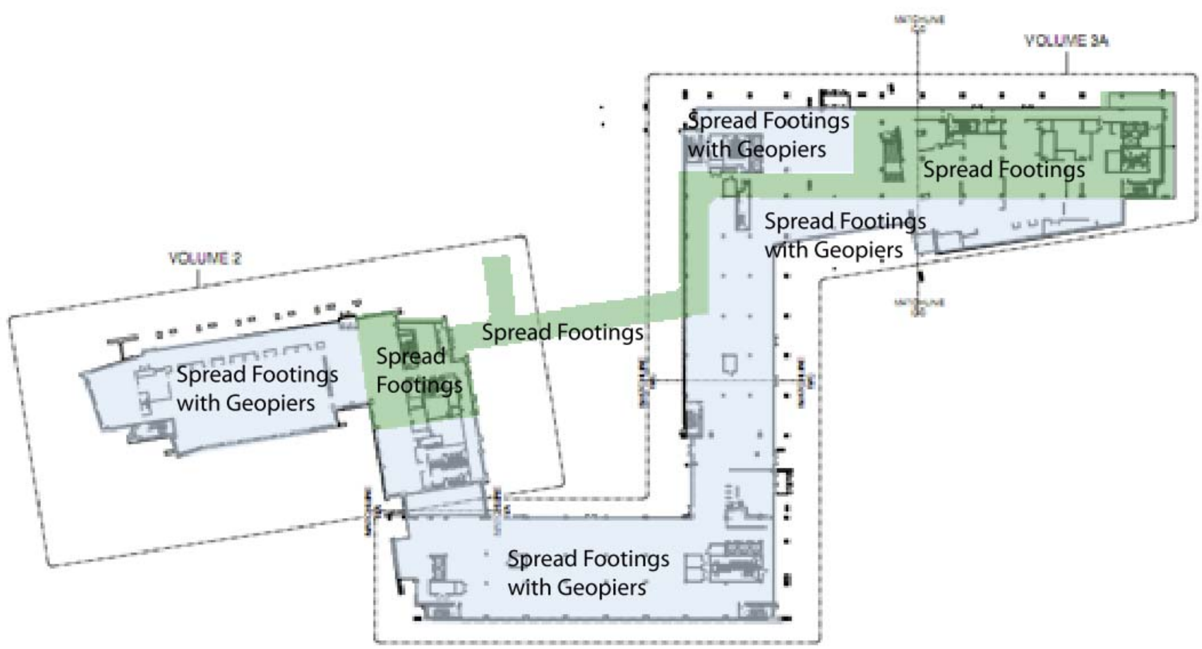



Figure 2: Foundation Key

For non-basement areas of Building 31 (Wing A), the western and central wings (Wings B and C) of Building 32, and the non-basement areas of Wing D, deep existing fill is expected within the majority of the buildings footprint. Geopiers are to be used in these areas to provide adequate bearing capacity (Figure 2). Geopiers use the concept of over consolidation to increase the soils bearing capacity. The 30 inch diameter Geopeirs should reach a depth of at least 10 feet. A detail of the typical spread footing with Geopiers is shown in Figure 3.

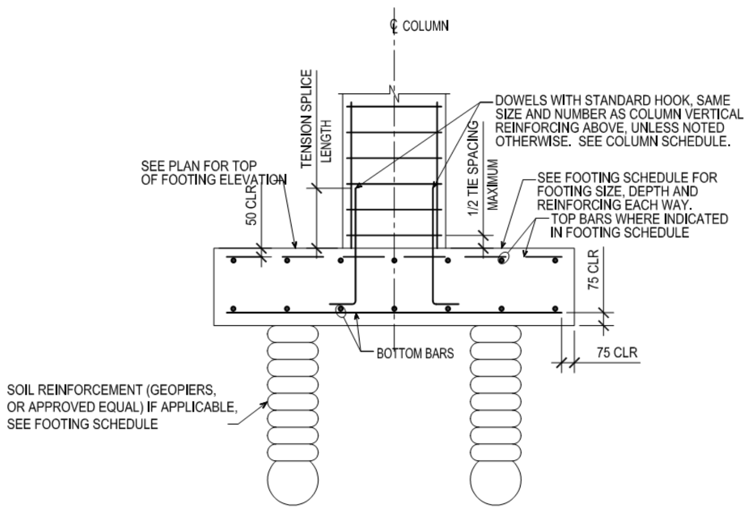



Figure 3: Typical Geopier Foundation Detail

For the basement level of Building 31 (Wing A), the basement level of Wing D of Building 32, and the underground tunnels, the foundations reach a sufficient depth where the bearing capacities on the spread footings are adequate (Figure 2).

Normal weight concrete was designed to be used with all the spread footings of the foundations. With a unit weight of 2350 kg/$m^3$ (147 pcf), the concrete has a 28 day strength of 28 MPa (4061 psi) concrete. A water to cement ratio of .48 is specified along with only 1% maximum chloride content.

Schnabel Engineering recommended the use minimum safe bearing capacities at the different locations of the foundation system. Where spread footings bear on undisturbed soil a bearing capacity of 192 kPa (4010 psf) was estimated. Beneath the spread footings of Wing A, where Geopiers were used, the estimated bearing capacity is 192 kPa (4010 psf). In the sections of Building 32 where Geopiers were used, a bearing capacity of 287 kPa (5994 psf) was estimated.

# Floor System:

## Building 31:

Building 31 utilizes a one way slab floor system for the majority of the buildings layout. The typical one way slab construction is an 8.07 inch thick slab with 5.91 inch drop panels, unless noted differently on the drawings. On the first three floors of Wing A there is a large open assembly space, and prevents any typical bay spacing. However, on the fourth floor the typical bay spacing is 21.85' x 26.74' to 19.685' x 19.685'.

Resistance to progressive collapse was designed into the exterior reinforced beams of building 31. Typical progressive collapse beam sizes range from 23.62" x 42.32" to 18.11" x 35.43". The interior beams on Building 31 are reinforced concrete beams with typical sizes of 18.11" x 35.43" to 18.11" x 23.62".

A large assembly pace on the first floor of Wing A is open up through the third floor. On the fourth floor framing level, post tension transfer girders were designed to support the column loads above the fourth floor and transfer the load to the foundation (Figure 4). The post tension transfer girders are 35.43" x 70.89" and have a post tension strand force of 4540 kN.

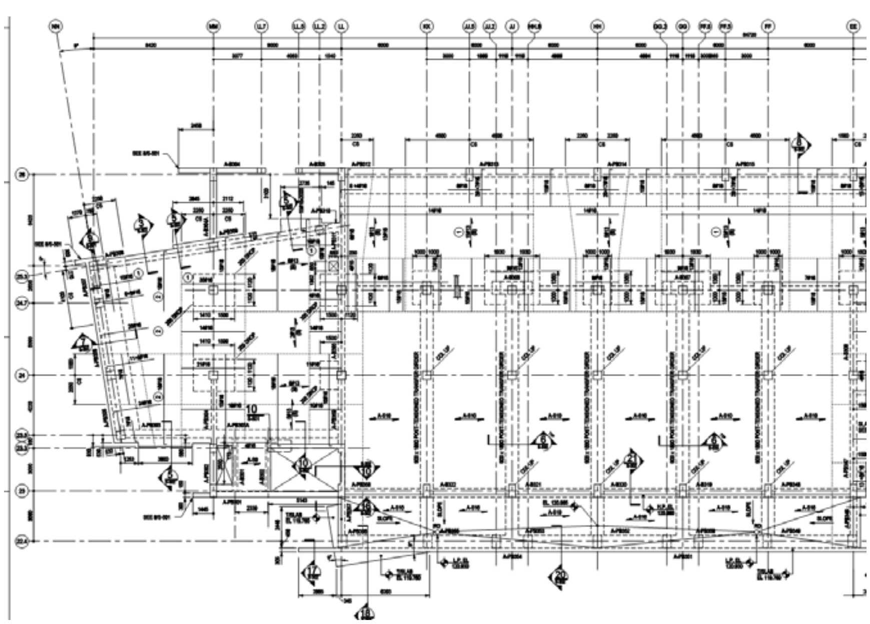


Figure 4: Framing Flan for Post Tension Transfer Girders

An atrium is provided between Wing A and Wing B that is primarily a steel superstructure with lightweight concrete on metal deck (Figure 5). The walkways over the atrium connecting the two wings are cast in place lightweight concrete on steel metal deck. The rib height on the metal deck is 50 mm with an additional 83 mm of concrete above. Supporting the walkway is W360 x 32.9 steel beams that frame into W360 x 32.9 girders with a shear connection. On the Wing A side of the atrium the girders site on an L152x152x9.5 that is attached to the concrete beam in Wing A. On the Wing B side on the atrium, an expansion joint is place, so the girders rest on a sliding connection that is connected to a beam in Wing B (Figure 6 and 7).

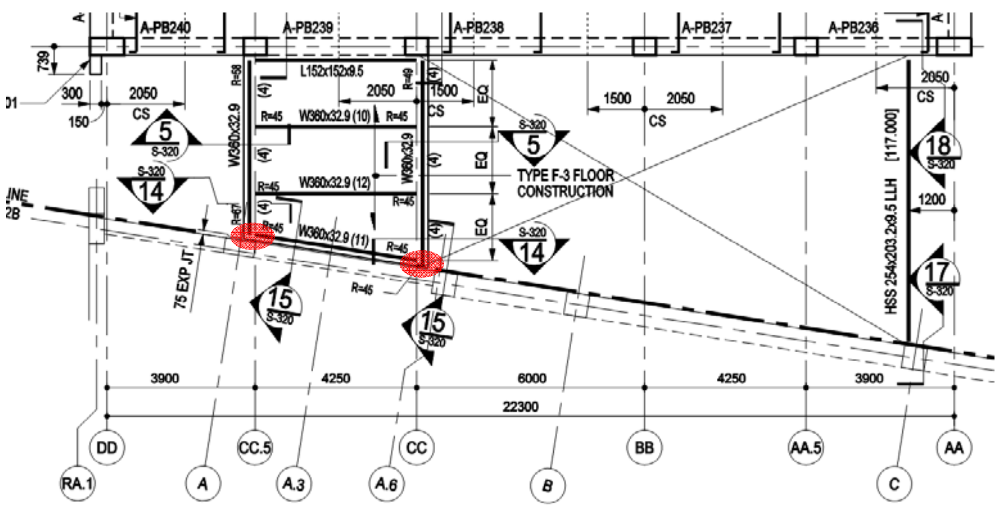



Figure 5: Wing A Atrium

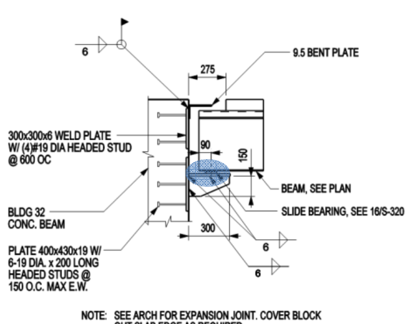



Figure 6: Expansion Joint Detail (Red)

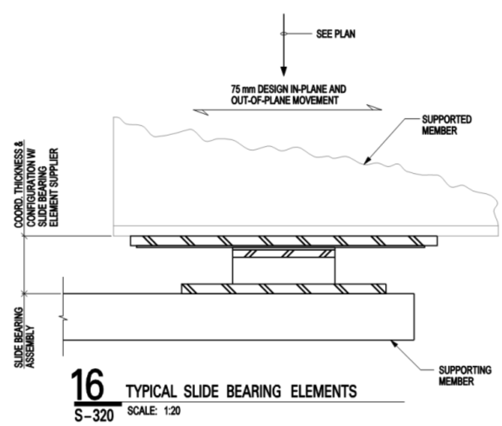



Figure 7: Expansion Joint Detail (Red)

## Building 32:

Building 32 utilizes a two way flat slab system for the majority of the building's floor system. A 5.91 inch thick slab on grade is provided for the ground level and the basement levels of the building. The two-way flat slab is typically 9.449" thick with a 7.09" thick drop panel, unless noted differently on the structural drawings. The typical interior bay spacing for Building 32 is 29.528' x 19.685', and the typical exterior bay spacing of 27.559' x 29.528', figure 8 shows the typical layout of the bays.

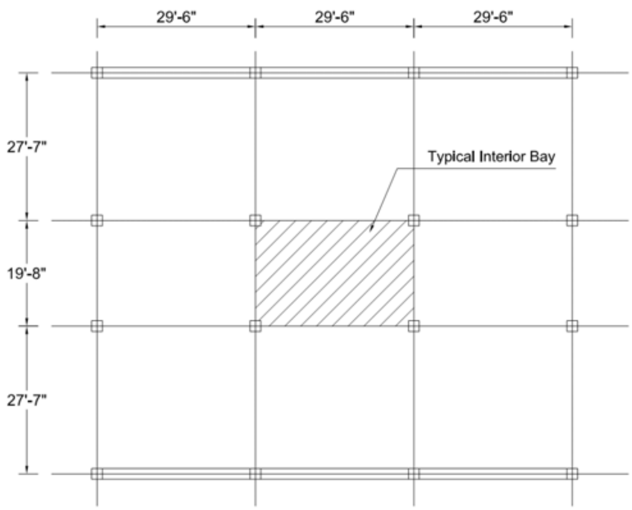



Figure 8: Building 32 Wing B Typical Bay Layout

Resistance to progressive collapse was designed into the exterior reinforced concrete beams of building 32. Typical progressive collapse beam sizes ranging from 23.62" x 40.95" to 15.75" x 40.95".

Atriums are provided between Wings B and C, and between wings C and D. The floor system for the atriums is a cast in place lightweight concrete on metal deck. The rib height on the metal deck is 1.97" with an additional 2.52" of concrete above. Supporting the walkways are W150 x 30 steel beams that frame into W610 x 217 girders with a shear connections. Expansion joints at the Intersections of the wings are provided and sliding connections are required at the atrium walkways.

## Columns

Typical reinforced concrete columns were designed for the FDA OC/ ORA Office Building. Designed as the primary gravity system, the typical sizes of the columns are 600mm x 600mm, 900mm x 600mm, and 600 mm diameter. Various types of columns are provided ranging from square columns, rectangular columns and circular columns (Figure 9). The concrete for the columns is a normal weight concrete with 28 day strength of 28 MPa (4061 psi). The slab and the beams are monolithic with the columns forming a continuous system.

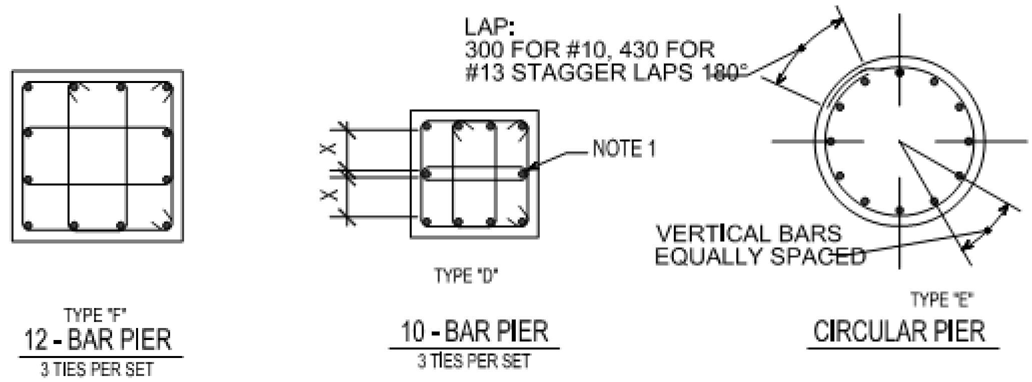



Figure 9: Typical Column Details

## Lateral System

Ordinary reinforced concrete shear walls were design for the primary lateral resisting system. The typical shear wall has #16 at 300mm (#5 at 11.82 inches) for both vertical and horizontal reinforcement with 13 #16 (13 #5) for the end zone reinforcement and #13 ties at 300mm (#5 ties at 11.81 inches) for the vertical reinforcement (Figure 10 and 11).

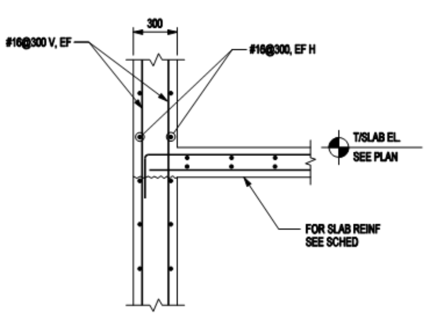



Figure 10: Shear Wall Detail

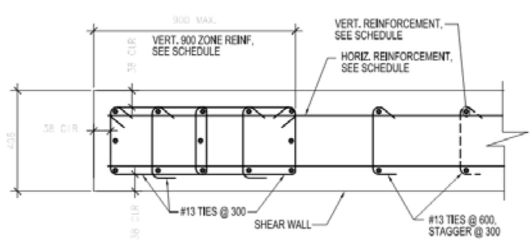



Figure 11: Shear Wall End Zone

Shear walls are provided around each elevator core and the stair shaft of Wing A. Wings B through D provide shear walls around each elevator core; Figures 16 through 19 shows the location of the shears walls in each wing, shown in red. At the intersection of each wing, in the atriums, slide bearing connections are provided at the expansion joints, shown in blue. These connections allow each wing's lateral systems to act independently of the other wing.

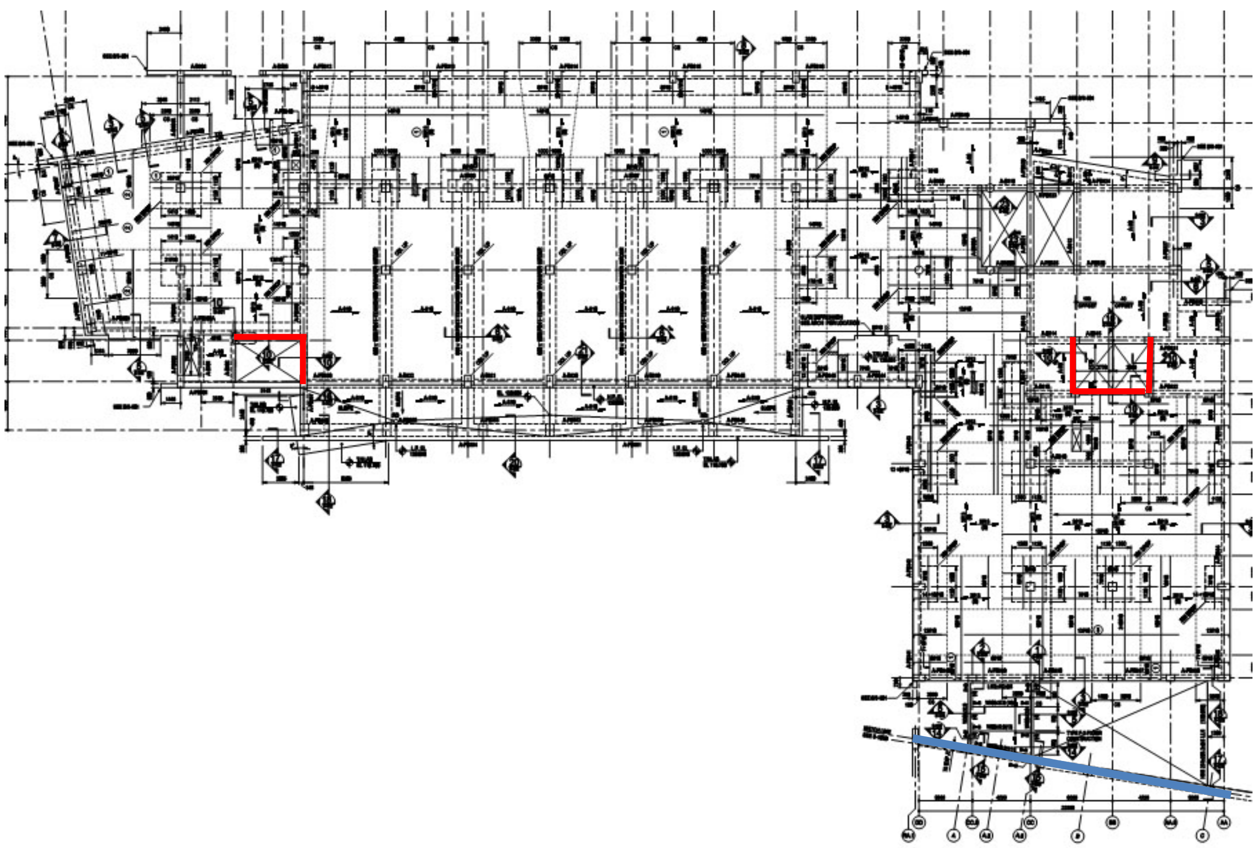


Figure 12: Shears Walls of Wing A

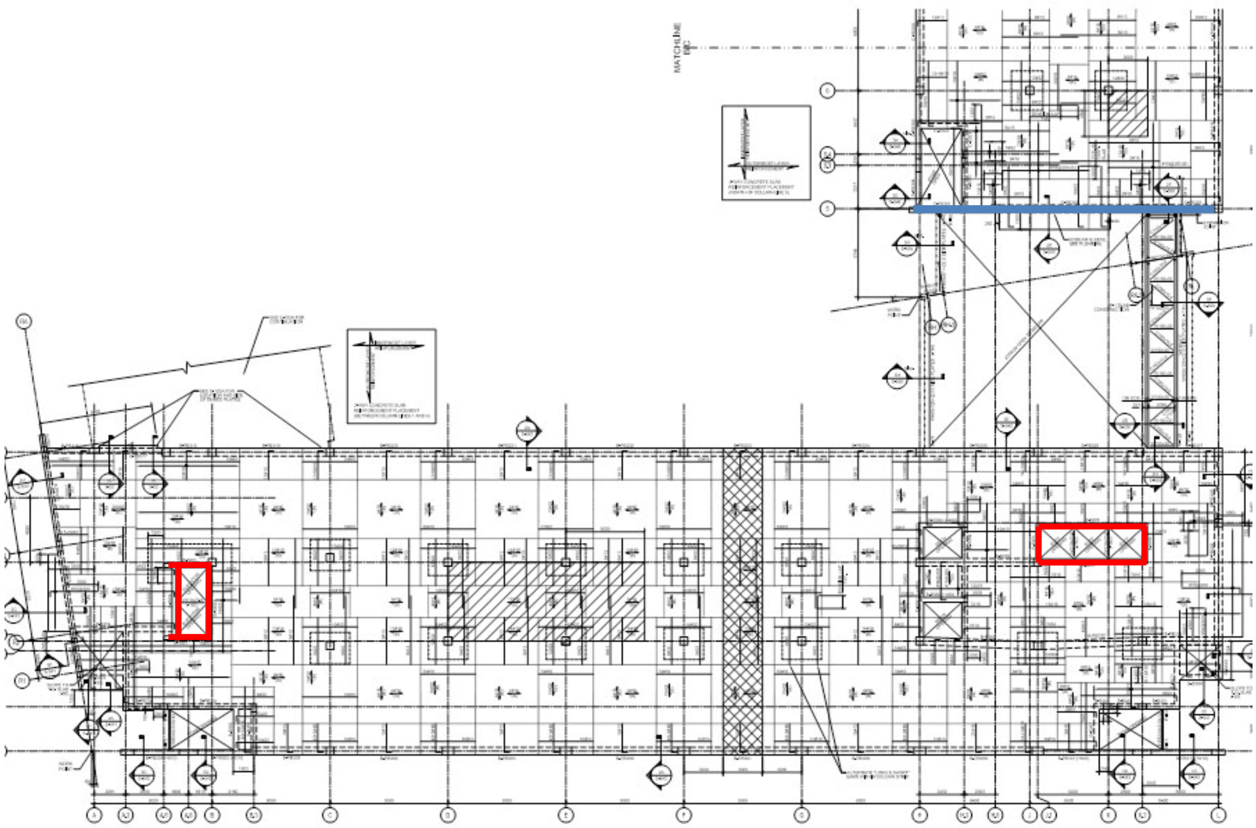


Figure 13: Shear Walls of Wing B

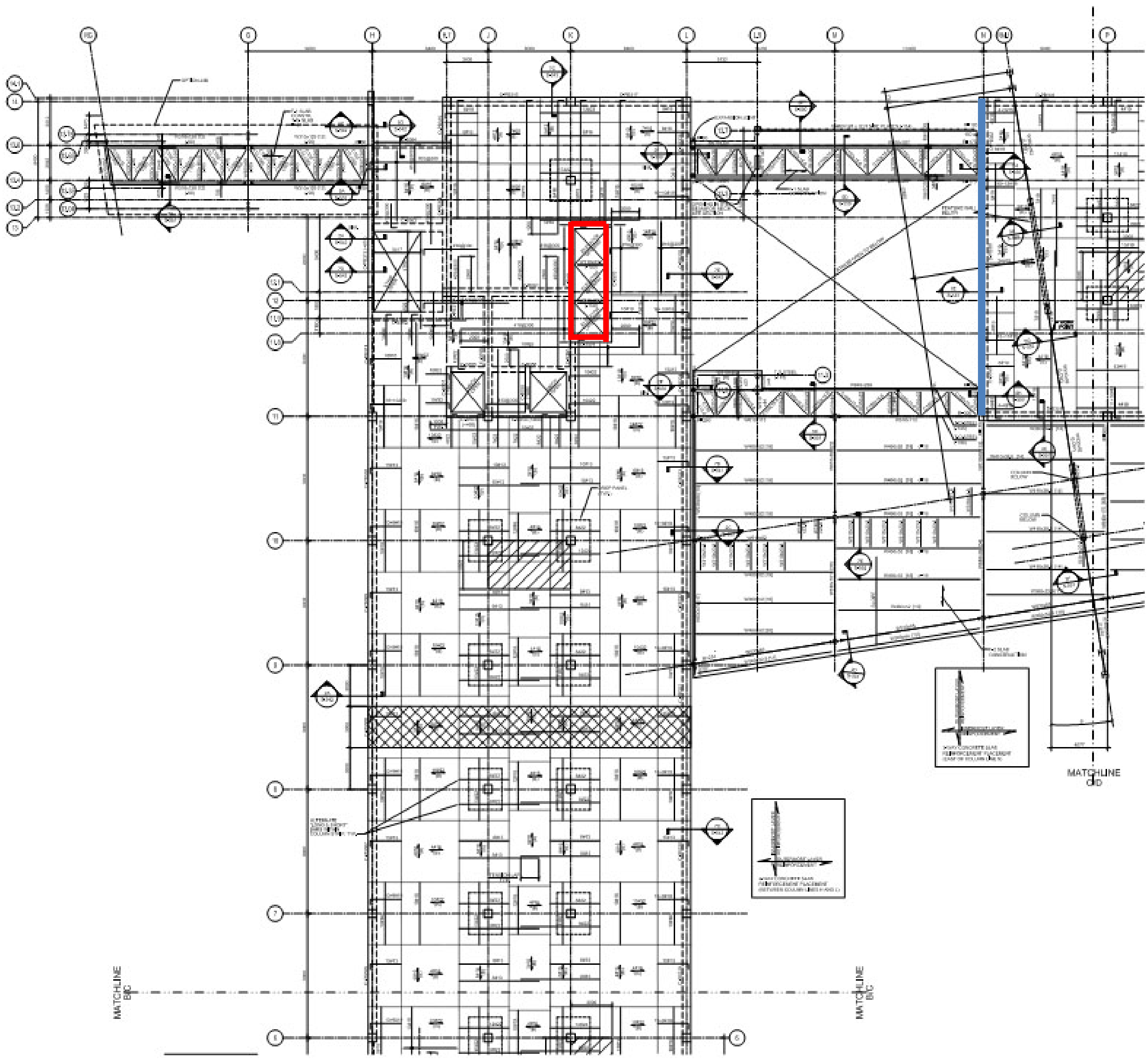


Figure 14: Shear Walls of Wing C

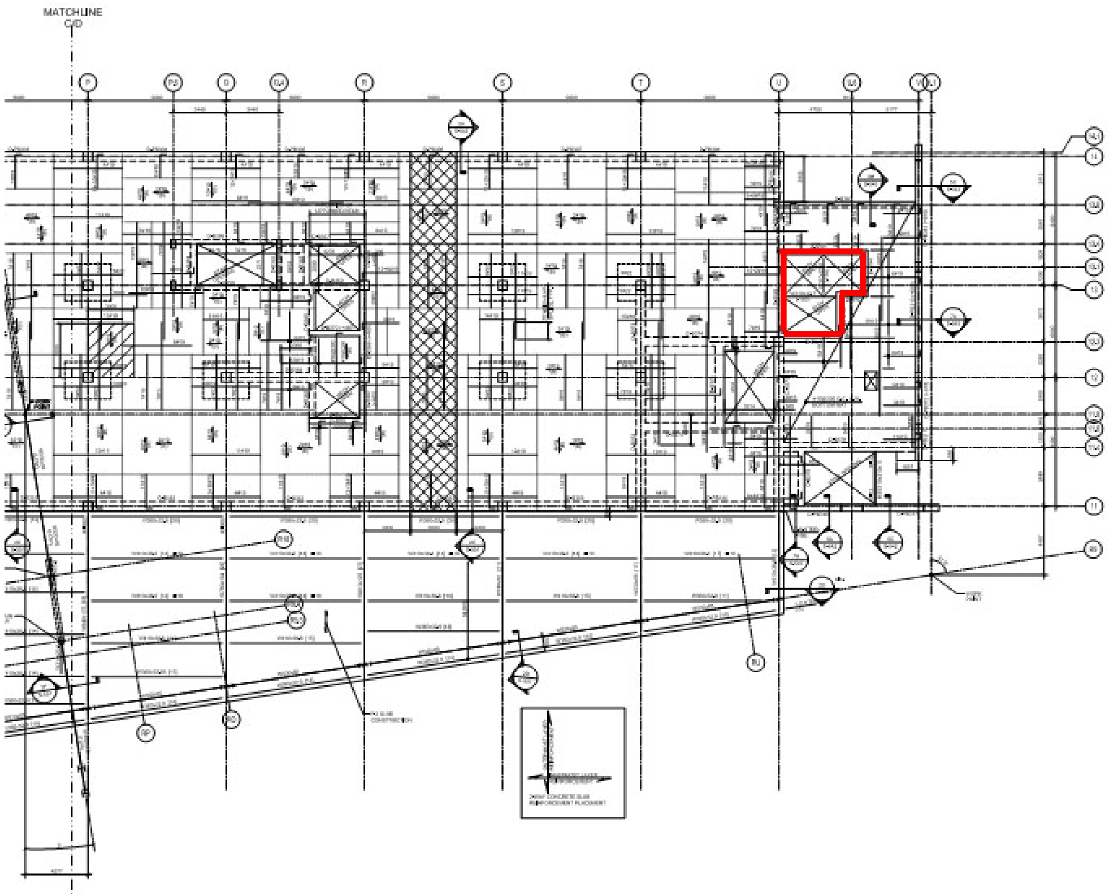


Figure 15: Shear Walls of Wing D

## Load Paths

Gravity Load Resisting System:

Reinforced Concrete columns make up the primary gravity load resisting system. The live load, self weight and superimposed dead load that sits on the floor system is transferred to the reinforced concrete beams. Reinforced concrete columns pick up the loads from the beams and the load is transferred to the buildings foundations. In Wing A reinforced concrete columns bear on a post tension transfer girder. Figure 16, shows a diagram of the post tension transfer girder that transfers the gravity load to the exterior columns. Surrounding columns that the transfer girders bear on transfer the load from the girders into the columns. Columns then transfer the load into the foundation of the building.

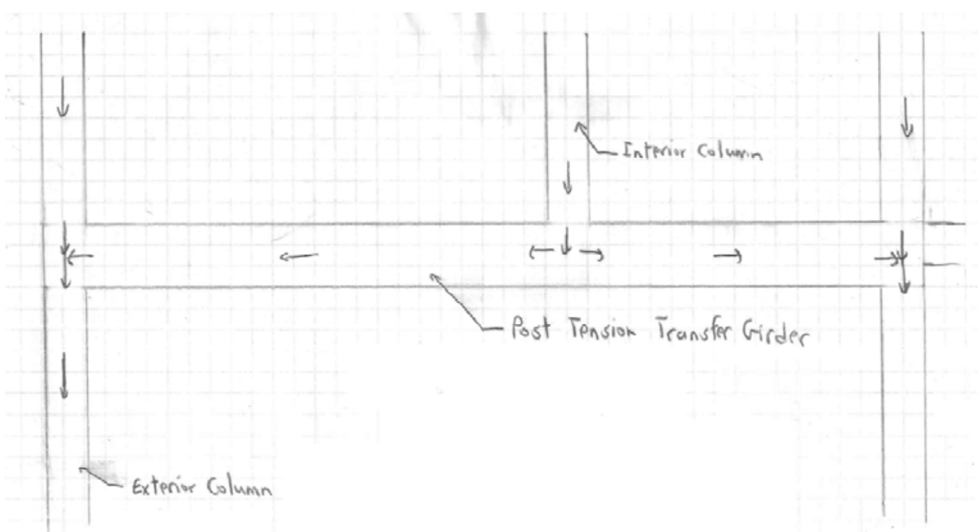



Figure 16: Transfer of Gravity Load

Resistance to progressive collapse has been designed for the office building. Design considerations that are involved with this design are removing an exterior column, and the floor system above and the adjacent columns are designed to carry the additional load.

## Lateral Load Resisting System:

Reinforced concrete shear walls are the primary lateral load resisting system. Lateral force due to wind is transmitted against the curtain wall of the building. Rigid floor system picks up each story shear at each level and transmits the lateral force to the shear walls located around each elevator core. Shear walls are design to resist the moment from the lateral load. The resisting moment forces are transmitted through the shear walls onto large spread footings.

Each wing acts independently with respect to the others wings. This is primarily due to the large expansion joints provided between each wing, along with the slide bearing connections design at the atriums connections.

This report specifically looks at the lateral system in Wing B. There are eight shear walls that are provided around the two elevator cores that are provided in Wing B. In shear walls 4 and 8 coupling beams are provided between the elevator doors and shear wall piers. Figure 17 provides the layout and location of the shear walls in Wing B. Appendix C provided dimension and details of the shear walls that are provided in Wing B.

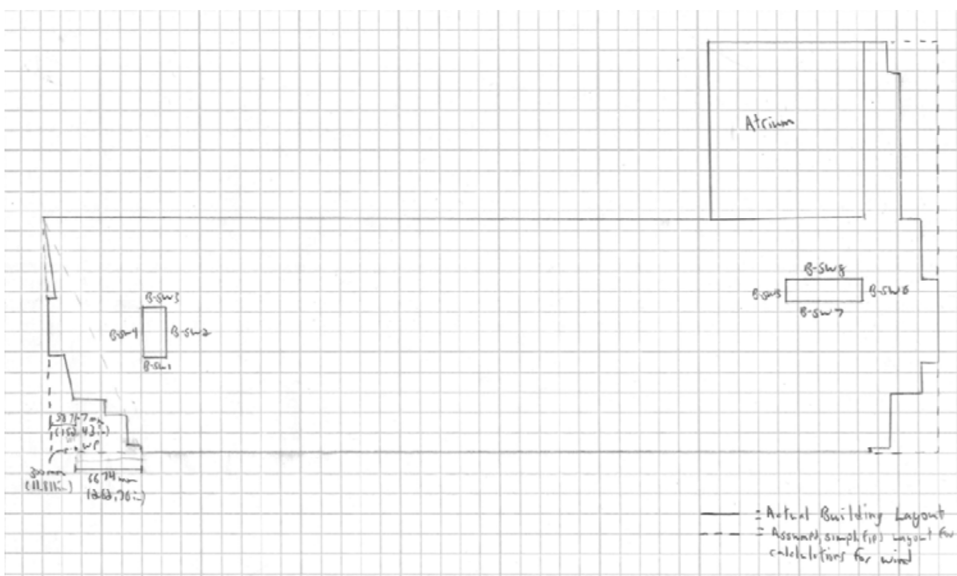



Figure 17: Wing B Shear Wall Layout

# Codes and References

## Design Codes:

National Model Code:

GSA Facilities Standards for the Public Building Service
International Building Code 2003

Structural Standards:

GSA Facilities Standards for the Public Building Service
ASCE 7-02, Minimum Design Loads for Buildings and other Structures

Design Codes:

AISC-ASD, Specifications for Structural Steel Buildings – Allowable Stress Design
ACE 318-02, Building code Requirements for Structural Concrete

### Design Codes (Used for this Thesis)

National Model Code:

GSA Facilities Standards for the Public Building Service – 2005
2006 International Building Code

Structural Standards

GSA Facilities Standards for the Public Building Service – 2005
ASCE 7-05, Minimum Design Loads for Buildings and other Structures
ASCE 41-06, Seismic Rehabilitation of Existing Buildings

Design Standards:

Steel Construction Manual 13th edition, American Institute of Steel Construction
ACI 318-05, Building Code Requirements for Structural Concrete, ACI
Design of Buildings to Resist Progressive Collapse 2005, Unified Facilities Criteria
Progressive Collapse Analysis and Design Guidelines June 2003, GSA

# Gravity Loads

The primary design guide lines for the FDA OC/ORA Office Building are the GSA Facilities Standards for the Public Service-2005, and the ASCE 7-02. The GSA outlines general requirements for the required live load for office interiors and the telecom room. The GSA Facilities Standards for the Public Building Service requires the designer to implement progressive collapse design into the structural design.

The latest version of design codes is being used for the analysis of the buildings gravity and lateral systems. When comparing to the designed loads and the ASCE 7-05 required loads, only one major difference appeared. ASCE 7-05 requires a load of 100 psf for special purpose roofs, specifically green roofs. Comparing to the designed load of 31.33 psf, one possible reason for the significant difference is the dead load; the structural engineer added a green roof dead load.

| Live Loads | | | | | |
|---|---|---|---|---|---|
| | Design | | GSA 05 | ASCE 7-05 | |
| Location | kPa | psf | psf | psf | |
| Office | 3.8 | 79.36 | 80 | 50 | (Partitions) |
| Typical Roof | 1.5 | 31.33 | | 20 | |
| Public Lobbies | 4.8 | 100.25 | | 100 | |
| Mech Room | 7.3 | 152.46 | | 150 | (Assumed) |
| Telecom Room | 12 | 250.63 | 250 | 150 | |
| Pedestrian Bridge | 4.8 | 100.25 | | 60 | |
| Balconies | 4.8 | 100.25 | | 100 | |
| High Density Filing | 12 | 250.63 | | 250 | (Assumed) |
| Green Roof | 1.5 | 31.33 | | 100 | |

Figure 18: Live Loads

| Dead Loads | | |
|---|---|---|
| | psf | |
| Superimposed Dead Load (MEP, Ceiling) | 15 | (Assumed) |
| Roofing System | 40 | (Assumed) |
| Mechanical Unit | 150 | (Assumed) |
| Exteior Curtain Wall | 30 | (Assumed) |
| Atrium Cutrain Wall | 20 | (Assumed) |
| Mechanical Pentouse Walls | 20 | (Assumed) |

Figure 19: Dead Loads

| SNOW LOADS (S) | | | | | ASCE 7-05 Ref. |
|---|---|---|---|---|---|
| Ground Snow Load | $p_g$ = | 25 | psf | | Figure 7-1 |
| Exposure Factor | $C_e$ = | 1 | | Terrain Category B | Table 7-2 |
| Thermal Factor | $C_t$ = | 1 | | | Table 7-3 |
| Importance Factor | I = | 1 | | Occupance Category II | Table 7-3 |
| | $p_f$ = | 17.5 | psf | $p_f$ = .7*Ce*Ct*I*pg | Eq. 7-1 |
| | $p_{fmin}$ = | 20 | psf | $p_{fmin}$ = $p_g$*I | Section 7.3 |
| | **$p_f$ =** | **20** | **psf** | | |
| Snow Drift | | | | | |
| Snow Density | γ = | 30 | pcf | | Eq. 7-3 |
| | h = | 14.66 | ft | | |
| | $h_d$ = | 0.67 | ft | | |
| | $h_c$ = | 13.99 | ft | | |
| Snow Surcharge | **$S_d$ =** | **52.5** | **psf** | | Section 7.7.1 |

Figure 20: Snow Loads

# Lateral Loads

To simplify the lateral analysis of the office building, lateral loads were determined for only Wing B. This was allowed because the wings have different lateral systems that do not interact with the other wings. The structural engineer also provided large expansion joints in the atriums that connect each wing, along with slide bearing connections. The slide bearing connections allow the wings to move and react independently from the lateral forces.

## Wind Loads

The wind loads were determined using Method 2 of the ASCE 7-05 Chapter 6. The first assumption under the wind analysis was that the 5 story braced frames and exterior moment frames structure would act rigidly under lateral loads. Appendix A contains a summary of the results from the Wind Calculations. Detailed information on the calculation of the wind design variables can be provided upon request.

In the North to South direction the Base Shear was larger than the East to West direction; this is due to the large façade area in this direction. The wind forces are shown in Figures 21 and 22.

| Design Wind Loads in N-S Direction | | | | |
|---|---|---|---|---|
| | External Windward Load (kips) | External Leeward Loads (kips) | Base Shear (kips) | |
| | | | 1.0W | 1.6W |
| Level 1 | 0 | 0 | | |
| Level 2 | 31.771 | 16.907 | **48.678** | **77.884** |
| Level 3 | 33.480 | 15.398 | **48.878** | **78.205** |
| Level 4 | 36.700 | 15.398 | **52.098** | **83.356** |
| level 5 | 39.327 | 15.398 | **54.725** | **87.560** |
| Roof | 25.274 | 9.578 | **34.851** | **55.762** |
| Parapet | 5.010 | 1.879 | **6.889** | **11.022** |
| Base Shear | | | **246.119** | **393.790** |

Figure 21: N-S Wind Loads

| Design Wind Loads in E-W Direction | | | | |
|---|---|---|---|---|
| | External Windward Load (kips) | External Leeward Loads (kips) | Base Shear (kips) | |
| | | | 1.0W | 1.6W |
| Level 1 | 0 | 0 | | |
| Level 2 | 14.675 | 7.809 | **22.484** | **35.974** |
| Level 3 | 15.464 | 7.112 | **22.576** | **36.122** |
| Level 4 | 16.951 | 7.112 | **24.064** | **38.502** |
| level 5 | 18.165 | 7.112 | **25.277** | **40.443** |
| Roof | 11.674 | 4.424 | **16.098** | **25.756** |
| Parapet | 2.314 | 0.868 | **3.182** | **5.091** |
| Base Shear | | | **113.680** | **181.888** |

Figure 22: E-W Wind Loads

## Seismic Loads

Seismic Loads for the FDA OC/ ORA Office Building were calculated using ASCE 7-05 Chapter 11 and 12. Initially the self weight of each floor needed to be estimated for the seismic calculations. This was done by assuming the framing systems for each floor were close enough to be approximated as the equal. The slab, beams and columns were all measured and their self weights were added up is Microsoft Excel. The exterior wall weight was assumed to be 30 psf because of the cmu backup behind the brick veneer curtain wall.

The Seismic Design Category was calculated using Table 11.6-1 and 11.6-2 in the ASCE 7-05. A SDC of A was determined for the Wing B of the office building; Appendix B contains the summary of the results from the seismic calculations, more detailed calculations can be provided upon request. The SDC calculated is different than the SDC of B that was designed by the structural engineer. A possible reason for this difference is the use of the USGS Ground Motion Parameter gave a much lower mapped acceleration. The story lateral forces and story shear forces were calculated with the equivalent lateral force procedure, using excel in Figure 35. Figure 23 shows a table of the story forces along with the calculated base shear of 95 k.

| Seismic Loads | | | | | |
|---|---|---|---|---|---|
| Level | Story Weight $w_x$ (kips) | Height $h_x$ (ft) | Lateral Force $F_x$ (Kips) | Base Shear (kips) | |
| 2 | 1711.82 | 15.82 | 17.12 | | |
| 3 | 1696.03 | 28.31 | 16.96 | | |
| 4 | 1696.03 | 41.2 | 16.96 | | |
| 5 | 1696.03 | 54.09 | 16.96 | | |
| Roof | 2680.3 | 66.98 | 26.80 | | |
| | | | | | |
| | | | **$\Sigma F_x = V_x =$** | **95** | **kips** |

Figure 23: Seismic Loads

# Load Combinations

Load Combinations provided by ASCE 7-05 for strength design are listed below.

- 1.4(D + F)
- 1.2(D + F + T) + 1.6(L + H) + .5($L_r$ or S or R)
- 1.2D + 1.6($L_r$ or S or R) + (L or .8W)
- 1.2D + 1.6W + L + .5($L_r$ or S or R)
- 1.2D + 1.0E + L +.2S
- .9D + 1.6W + 1.6H
- .9D + 1.0E +1.6H

Lateral load analysis was performed for this report and the load combinations that did not include lateral load forces were disregarded. It was also noted that the load combination including a factor of .9D are used to calculate the uplift forces for the lateral loads. For strength design the load combinations including 1.6W and 1.0E were the controlling load combinations. The combinations that were considered for this report are;

- 1.2D + 1.6W + L + .5($L_r$ or S or R)
- 1.2D + 1.0E + L +.2S
- .9D + 1.6W + 1.6H
- .9D + 1.0E +1.6H

# Proposed Thesis

Using the existing grid for Wing B, a steel framing layout for each floor will be determined and modeled using RAM Structural System. Typical steel on metal deck system will be designed for the gravity system, and later the impact on MEP will be considered. After the initial design is accomplished, the lateral loads will be determined and the lateral resisting system will be designed.

In the previous semester's work it was determined that the existing lateral system provided an eccentricity problem and deflection issues. It is planned to design braced frames around the two elevator cores of Wing B, and design exterior moment resisting frames. It is expected that the moment frames will help reduce the eccentricity problems observed in the previous semesters work.

Advanced computer modeling techniques will be included in the lateral design to optimize the lateral system design. A RAM Structural System Model will be created to initiate the design process and determine the required sizes for the lateral members. Using an ETABS model, torsional properties and distribution of lateral forces will be analyzed. An optimized layout will be determined to minimize eccentricity effects of the loading conditions

The design of structures to resist progressive collapses was designed in the original structure. It is intended to research the procedures required to design a steel structure to resist progressive collapse. Currently there are two primary methods for progressive collapse design. The first method is standards published by The General Service Administration, and the second is released by the Department of Defenses.

## Steel Connection Design

The connections for steel buildings have to be designed to not only meet the structural requirements, but also meet constructability requirements. For the master's integration, the typical connections for the steel building will be designed. The typical beam connection will include shear connections for the floor systems, and moment connections for the moment resisting frames. Also the braced frames will incorporate the design of a bracing connection.

## In-Depth Cost and Schedule Comparison

The first breadth study was chosen with its connection to the structural depth. The proposed changes to the lateral system with post tension design will have an impact on the scheduling on construction. The scheduling changes that would involve the additional construction time for the jacking of the post tension strands. A cost comparison of the existing structural system to the proposed changes will be made to the lateral system. Once the scheduling impact and the cost changes are considered, the feasibility of redesigning the progressive collapse beams as post tension beams will be evaluated.

## MEP Coordination Study

After the gravity system is design and the depth of the structural beams a MEP Coordination Study will be performed. Using the current ceiling height and re-adjusting the mechanical, electrical and plumbing that passes through a section of the building, the change in the allowable space without having interferences. The proposal is to re-design the MEP coordination to allow for the increased depth of the structural floor without impacting the architectural of the space.

# MAE Integrated Work

It was the purpose of this thesis to integrate the course work learned in the graduate course into the thesis. There are multiple courses that were used to aid in the integration of the graduate course work, and one topic which was not learned in any course was implemented into the thesis. Advance Computer Modeling: AE 597A, Steel Connection Design: AE 534 and Progressive Collapse Design.

Advance computer modeling techniques were implanted in the creation of two 3D Models to aid in the design of the steel structure. RAM Structural Analysis was initially used for the gravity system design and the lateral system design. Using lateral design concepts and mode and period analysis the design of the lateral system was chosen to minimize torisonal effects on the structure. An advanced 3D ETABS model was created to determine the lateral distribution of forces. Only the lateral system was modeled, and a 1000 kip load was applied to the structure, and the results were analyzed. This model was used to determine the validity of the RAM Model in the design checks of the lateral system.

Typical Steel Connections were designed for both the gravity and lateral system, using the information learned in the connection design course. Simple shear connections were design for the beam to girder connection and girder to column connections. The lateral connections were more advance and required more design work. A moment connection for the moment frame was designed, using a shear tab and bolted welded flange plates. A heavy braced connection was also designed using the uniform force method. The angle to gusset, gusset to column and beam to column connections were all design and checked for all the limit states.

The design of structures to resist progressive collapse was also looked at for the new structural design. Progressive Collapse Design is not covered in any course, and its advanced research in nature, put it in a higher level of work for this thesis. There are two methods of progressive collapse, both methods were studied and the procedures for each method will be reported on. Time permitting the application of this research will be implemented to the structure.

## Thesis Research

Course Materials:

| | |
|---|---|
| AE 597A | "Advance Computer Modeling" |
| AE 534 | "Steel Connection Design" |
| AE 403 | "Advanced Steel Design" |

Geschwinder, Louis F. Unified Design of Steel Structures. John Wiley & Sons Inc., 2008.

Structural Standards:

ASCE 7-05, Minimum Design Loads for Buildings and other Structures
ASCE 41-06, Seismic Rehabilitation of Existing Buildings

Design Standards:

Steel Construction Manual 13th edition, American Institute of Steel Construction, 2005.

Unified Facilities Criteria. Design of Buildings to Resist Progressive Collapse. Depart of Defense, July 2009.

General Services Administration. *Progress Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*. U.S. General Services Administration, June 2003.

# Structural Steel Framing

The existing grid for Wing B was used to determine the column layout for the new structural system, Figure 24. It was determined the column sizes would be smaller in steel, and impact on the architectural space would be positive and not be considered. A few locations the columns were aligned with the grids, where previous design was offset. The impact on the space was assumed minimal. The framing was separated into three sections; left, middle, and right. The middle framing layout was the most typical. Near the ends the framing is not as typical, and a general layout was used. Columns were designed to be two story columns and spliced every other floor.

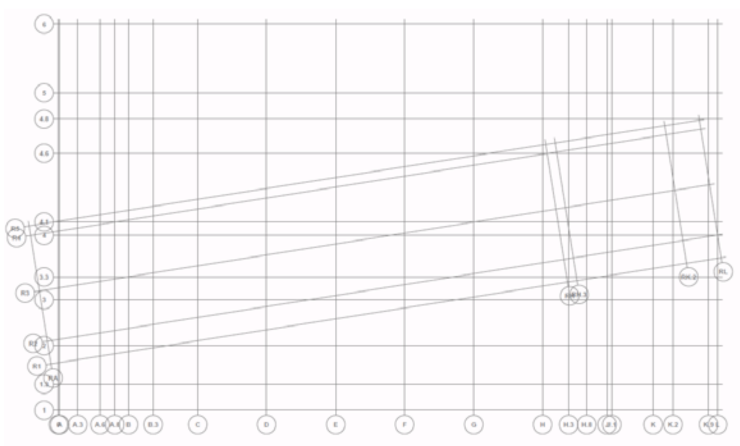


Figure 24: RAM Grid Layout

## Gravity System Design

The gravity system was designed using RAM Structural Systems using the gravity loads determined to act on the structure. The girders were designed to span across the short direction between columns in the vertical direction. Intermediate beams were laid in the opposite direction with intermediate beams intersecting the mid span of the girder in the interior bays, and intersecting the third points for the exterior bays. A 2” LOK Metal Steel Deck was pick from United Steel Decking to support the superimposed loads, and the designed beam spacing is less than the allowable unsupported width. The floor system is designed to be 4 ½” concrete which was more than adequate to support the loads. Live load reduction was used to design the beams and columns for the gravity system, and composite action was assumed and designed.

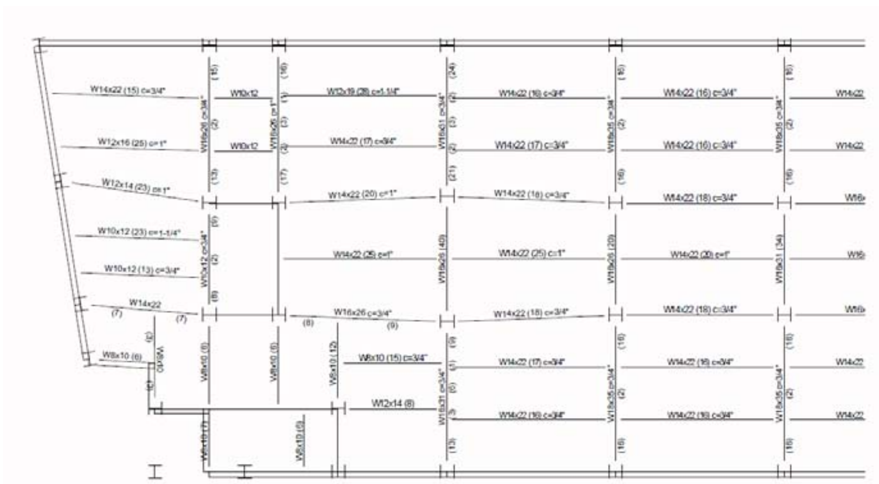


Figure 25: Framing Plan Part 1

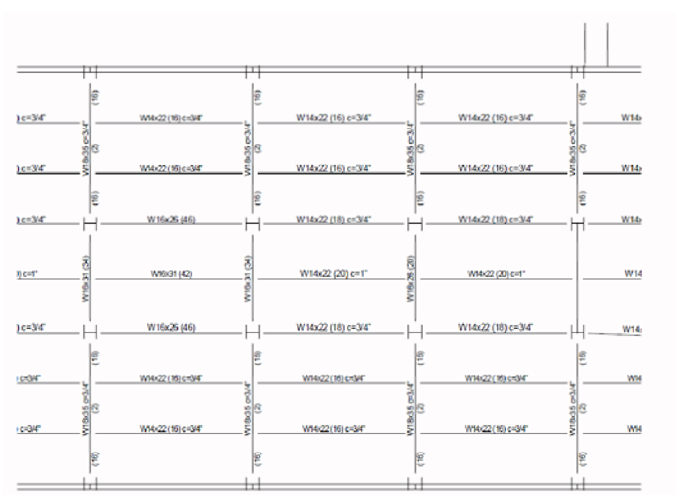


Figure 26: Framing Plan Part

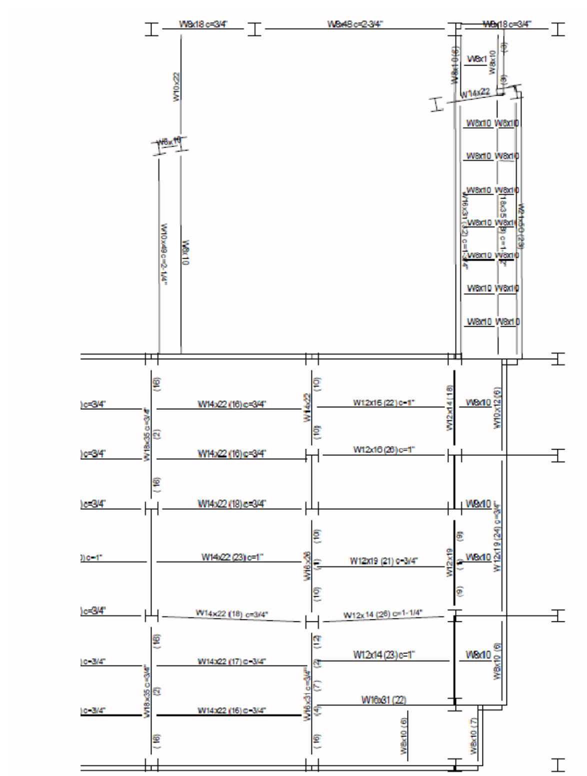


Figure 27: Framing Plan Part 3

Typical framing plans were created from RAM and can be found in Appendix C. Figures 25 through 27 show the second floor framing plan with the beam design in detail. The typical interior beam was designed as a W14 x 22 with a 1" camber. The beams frame into the girders that were designed as W16 x 31 for the Interior Bay and W18 x 35 for the exterior bays. The gravity columns were also designed, the design summary can be found in Appendix C, and Figure 28 shows an excerpt from the design summary. The interior columns along Grid D were designed as W10 x 54 for stories 2 and 3, W10 x 39 for stories 4 and 5, and a W10 x 33 for the column below the roof story. The gravity check for the column was designed as a W12 x 65; however a W10 x 77 would have been sufficient. The difference in the designs is attributed to live load reduction which was included in the RAM design, and not in the hand checks.

**Column Line D-2**

| Level | Pu | Mux | Muy | LC | Interaction Eq. | Angle | Fy | Size |
|---|---|---|---|---|---|---|---|---|
| Roof | 65.7 | 1.8 | 10.2 | 6 | 0.64 Eq (II1-1a) | 0.0 | 50 | W10X33 |
| Floor 5 | 134.8 | 1.6 | 4.6 | 3 | 0.72 Eq (H1-1a) | 0.0 | 50 | W10X39 |
| Floor 4 | 191.1 | 1.4 | 5.5 | 6 | 0.99 Eq (H1-1a) | 0.0 | 50 | W10X39 |
| Floor 3 | 258.4 | 1.5 | 5.2 | 3 | 0.78 Eq (H1-1a) | 0.0 | 50 | W10X54 |
| Floor 2 | 318.5 | 0.0 | 5.7 | 6 | 1.00 Eq (H1-1a) | 0.0 | 50 | W10X54 |

**Column Line D-3**

| Level | Pu | Mux | Muy | LC | Interaction Eq. | Angle | Fy | Size |
|---|---|---|---|---|---|---|---|---|
| Roof | 65.7 | 1.8 | 10.2 | 11 | 0.64 Eq (H1-1a) | 0.0 | 50 | W10X33 |
| Floor 5 | 134.8 | 1.6 | 4.6 | 4 | 0.72 Eq (H1-1a) | 0.0 | 50 | W10X39 |
| Floor 4 | 191.1 | 1.4 | 5.5 | 11 | 0.99 Eq (H1-1a) | 0.0 | 50 | W10X39 |
| Floor 3 | 258.4 | 1.5 | 5.2 | 4 | 0.78 Eq (H1-1a) | 0.0 | 50 | W10X54 |
| Floor 2 | 318.5 | 0.0 | 5.7 | 10 | 1.00 Eq (H1-1a) | 0.0 | 50 | W10X54 |

Figure 28: Column Design Summary

Hand calculations were performed for random gravity members to determine the validity of the RAM Model. Using the Thirteenth Edition of the AISC Steel Manual composite beams and girders were sized and checked against other limit states; refer to Appendix E for the calculations. It was determined that the gravity members sized in RAM matched the sizes determined by hand calculations.

There are two columns that were not designed with RAM due to the slenderness limits. The two columns are on the south side of the building, and are primary architectural features and take very little load. The columns were designed as composite columns with a W10x33 and 18" by 22" Concrete column surrounding the W shape. The effective length of the column and the slenderness effects reduced the load carrying capacity of the column. However, the column received very little load, and the column was still adequate to carry the loads.

## Foundation

The structural redesign to structural steel reduced the required load on the foundations. Spread footings were designed using the allowable bearing capacity determined from the soils report; 6 kips per square foot was used for the soil under Wing B. The spread footings were designed for the gravity load from the column, the column sits on a base plate that attached to a concrete pier that will transfer load to the footing.

The new spread footing was designed to be a square footing with a length and width of 10 feet. After the size of the footing was chosen, the depth and reinforcement was designed. A height of 24 inches was sufficient for the punching shear and flexure forces. Using (11) #7 reinforcing bars in both directions for the design of the reinforcement for the tensile flexure forces. The new design was compared to the original footing design, and the new footing decreased the height of the footing by 12 inches. Since the building got lighter with the new structural steel design, it was expected to see a smaller spread footing. The design of the footing can be found in Appendix G.

An overall stability check was performed on the building to determine if overturning would be a problem. Using the Wind in the North South direction and the dead load for the building, the resistance to overturning was check. It was determined that the uplift force was counteracted by the gravity load. However in a few instances the base plates to the foundation piers will have to be design to take individual uplift forces from the lateral resisting system.

## Lateral System Design

It was determined that braced frames around the core would be designed as the primary lateral system to control the drift of the building, and moment frames around the exterior would decrease the eccentricity of the lateral system. In the design of structures to resist progressive collapse, moment frames are the preferred method which also contributed in the reduction of the eccentric effects. Initially the gravity sizes of the beams and columns were used to size the lateral members. It was later determined that the beams were undersized and needed to be revaluated, Figure 29 shows the 3D model that was created in RAM.

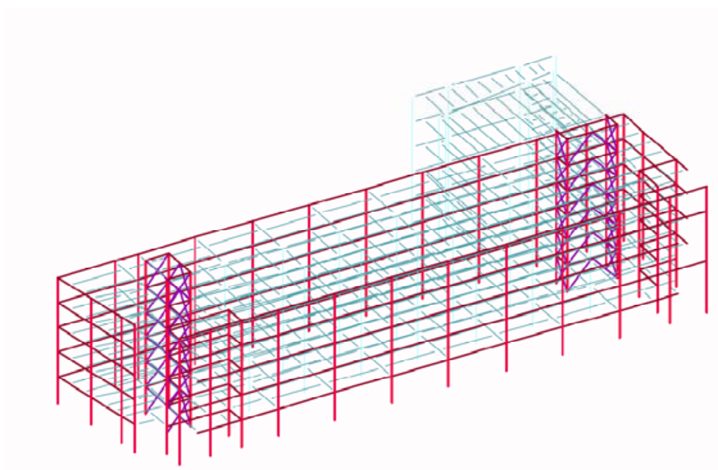


Figure 29: RAM Lateral Model

Using the AISC Thirteenth Edition of the Steel Manual, and equations for max moment for a fix ended beam, assuming gravity loads would control the design of the moment frame beams. A preliminary size for the moment frame beams were designed, assuming mid span bracing at the bottom flange of the beam and full lateral bracing on the top of the beam. The sizes were inputted into the RAM Model, and the analysis of the lateral system began.

Using RAM Analysis the periods of vibrations and Center of Rigidities were calculated. It was observed that the braced frames at the elevator cores and moment frames at the exterior did decrease the eccentricity from the shear wall design, but still produced an eccentric loading. A new design was created, and compared to the first braced frame design. A braced frame was added between the two elevator cores along grid H, shown in Figure 30.

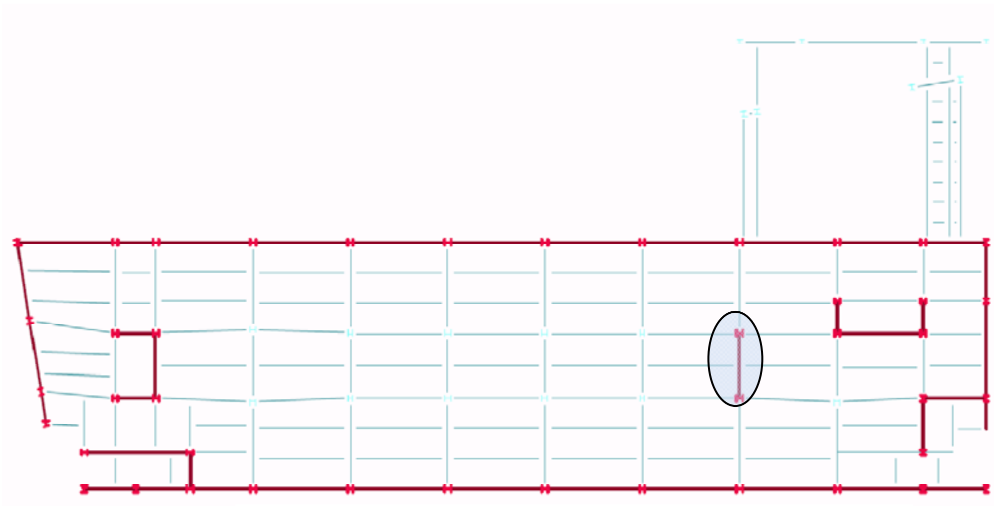


Figure 30: Lateral Framing

Lateral analysis of both models was performed and the center of rigidities were calculated and compared, Figure 31. It was determined that the second model that includes the added brace did decrease the eccentric effects of the lateral system. The new design is more efficient than the original braced frame design, and the final design steps were carried out. A 3D ETABS model was created to perform distribution analysis of the lateral system, the Modal periods of vibrations were calculated and compare, Figure 32. It was observed that the Mode 1 for all three models was the torisional mode, and the additional brace did decrease the period.

| Center of Rigidity Compariosn for RAM Models | | | | | | |
|---|---|---|---|---|---|---|
| | RAM Model 1 | | RAM Model 2 | | COM of Both Models | |
| | X (ft) | Y (ft) | X (ft) | Y (ft) | X (ft) | Y(ft) |
| Roof | 85.26 | 41.25 | 128.47 | 41.35 | 154.33 | 49.98 |
| Floor 5 | 91.23 | 41.02 | 131.95 | 41.09 | 142.63 | 41.12 |
| Floor 4 | 104.65 | 41.025 | 140.67 | 41.13 | 142.64 | 41.12 |
| Floor 3 | 126.14 | 40386 | 153.38 | 40.86 | 142.65 | 41.11 |
| Floor 2 | 158.01 | 40.42 | 171.24 | 40.32 | 138.75 | 40.98 |
| * RAM Model 2 adds Brace at Grid H | | | | | | |

Figure 31: Center of Rigidity Comparisons

| 3D Model Period Comaprisons | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| RAM Model 1 | | | | RAM Model 2 | | | | ETABS Model | | |
| | Direction | Period (s) | | | Direction | Period (s) | | | Direction | Period (s) |
| Mode 1 | 2.145 | Z | | Mode 1 | 1.4798 | Z | | Mode 1 | 1.1971 | Z |
| Mode 2 | 1.2959 | X | | Mode 2 | 1.3521 | X | | Mode 2 | 1.0513 | X |
| Mode 3 | 1.1244 | Z | | Mode 3 | 1.1173 | Z | | Mode 3 | 0.8627 | Z |
| *Braced Frames at Core | | | | *Additional Brace | | | | *Same as RAM Model 2 | | |
| *Exterior Moment Frames | | | | | | | | | | |

Figure 32: Modal Period Comparisons

The lateral system was analyzed using RAM Structural Analysis in the RAM Frame Module. Lateral members were sized and analyzed for different load combinations in RAM, the typical brace member was a double angle ranging from 2L 5"x 5" x ¾" to 2L 4" x 4" x ½". Two brace configurations were chosen for the lateral system. The first is a concentric chevron bracing used on the braced frames with a width of twenty feet or more, Figure 33. The second brace is concentric x bracing for the braced frames with a width of fifteen feet or less, Figure 34. These configurations will maximize the axial force in the braces to best optimize the braced frame system. Moment frames were designed around the exterior of the building to include lateral resisting elements and also aid in the design against progressive collapse. Figure 35 and 36 shows the layout used in the moment frame design along Grid 1 and Grid 4 respectfully. A full detail of each frame design can be found in appendix D.

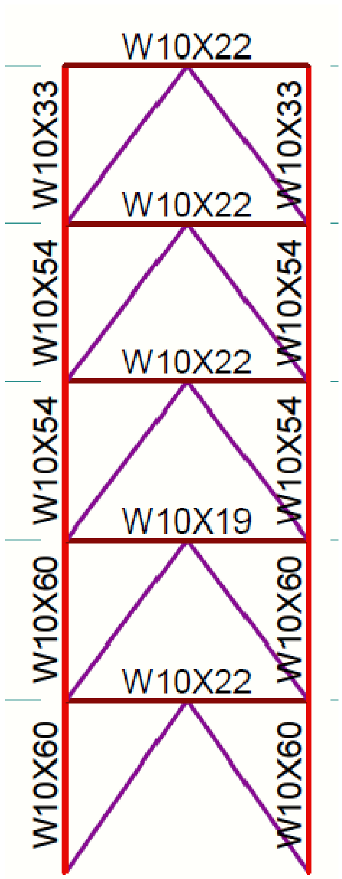



Figure 33: Chevron Brace Frame

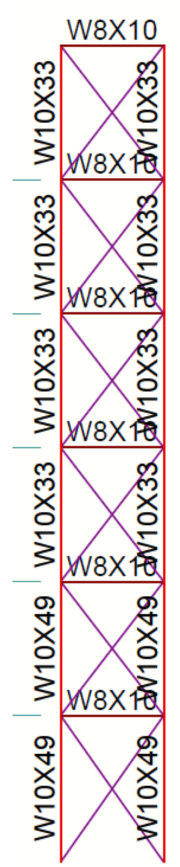



Figure 34: X Braced Frame

Figure 35: Moment Frame along Grid 1

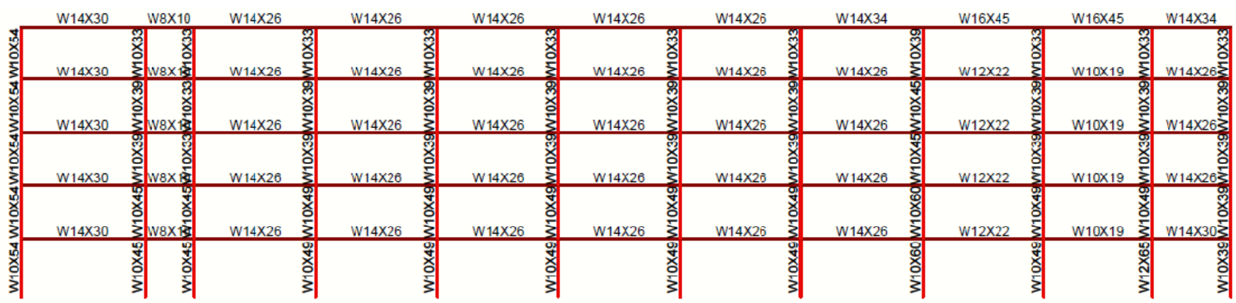


Figure 36: Moment Frame along Grid 4

After the lateral system was designed for strength requirements, the drift was analyzed using RAM Structural Systems. A drift limit of L/400 for story drift and story displacements was used to determine if the lateral system met the drift requirements. It was determined that the lateral system met the requirements and did not pose any drift problems. Figure 37 is the drift report summarized from the RAM Output.

| Drift Study from RAM | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Displacements | | | | Drift | | | |
| | hx | Displ X | Allow. | Displ Y | Allow. | Drift X | Allow. | Drift Y | Allow. |
| Roof | 12.89 | 0.3611 | 1.9716 | 1.1775 | 1.9716 | 0.0543 | 0.3867 | 0.2794 | 0.059148 |
| 5th | 12.89 | 0.3068 | 1.5849 | 0.8981 | 1.5849 | 0.0747 | 0.3867 | 0.29 | 0.047547 |
| 4th | 12.89 | 0.2321 | 1.1982 | 0.6081 | 1.1982 | 0.0878 | 0.3867 | 0.271 | 0.035946 |
| 3rd | 12.89 | 0.1443 | 0.8115 | 0.3371 | 0.8115 | 0.1443 | 0.3867 | 0.3371 | 0.024345 |
| 2nd | 14.16 | 0.0596 | 0.4248 | 0.135 | 0.4248 | 0.0596 | 0.4248 | 0.135 | 0.012744 |

Figure 37: Drift Calculations

## Computer Modeling

Advanced computer modeling techniques were used to generate a more accurate design along with variations in the design for different load conditions. RAM Structural Systems was used as the primary modeling system to design both the gravity systems and lateral systems. Assumptions for the RAM Structural Model are:

1. P- Delta Effects Included.
2. Center line modeling.
3. Panel Zone Shear is not included.
4. Braces assumed to be pinned at both ends.
5. Moment Beams assumed to be fixed at both ends.
6. The structure is assumed to be pinned at the base due to the spread footings.

Along with a RAM Structural Model, a 3D ETABS model was created to model the lateral system only. The design that was determined in RAM for the lateral system will be modeled in ETABS using the same assumptions. The ETABS model will be used to perform an analysis of the lateral systems to assist in the design process of lateral resisting elements. Figure 38 shows the summary of the analysis.

| ETABS Lateral Distribtion for Floor 2 | | |
|---|---|---|
| X Direction Loading | | |
| | V (kips) | % |
| Moment Frame 1 | -31.408 | 0.031 |
| Moment Frame 1.3 | -5.0398 | 0.005 |
| Moment Frame 2 | -2.694 | 0.003 |
| Moment Frame 4 | -48.5955 | 0.049 |
| | | |
| Brace Frame 2 | -210.508 | 0.210 |
| Brace Frame 3.1 | -282.217 | 0.282 |
| Brace Frame 3.2 | -421.105 | 0.420 |
| Total | **-1001.57** | |
| Y Direction Loading | | |
| Moment Frame RA | 0.594 | -0.001 |
| Moment Frame B.3 | -1.1589 | 0.001 |
| Moment Frame K.2 | -3.47 | 0.003 |
| Moment Frame L | -2.874 | 0.003 |
| | | |
| Brace Frame B | -366.71 | 0.367 |
| Brace Frame H | -358.85 | 0.359 |
| Brace Frame J.1 | -117.28 | 0.117 |
| Brace Frame K.2 | -149.767 | 0.150 |
| Total | **-1000.11** | |

Figure 38: ETABS Lateral Study

# Connection Design

Typical gravity and lateral connections were designed using the requirements and design guides prescribed by the Thirteenth Edition of the Steel Construction Manual. A typical beam to girder connection was designed as a simple shear tab connection shown in Figure 39. A shear tab connection was chosen for the constructability issues in lowering the beam down between the girders with little interferences. Limit states for bolt shear, bearing, tear out were and block shear were considered in the design of the shear tab connection using the Thirteenth Edition of the Steel Construction Manual.

A typical Girder to column web connection was designed as a seated connection, shown in Figure 40. The girder flange width is less than the width of the opening between flanges, so a simple seated connection was chosen for the connection. Limit states considered for the seated connection were beam web yielding and crippling, along with angle flexure and yielding. A stabilizing angle is welded to the top of the beam after erection to prevent the beam from rolling over, but it is not design to take any load.

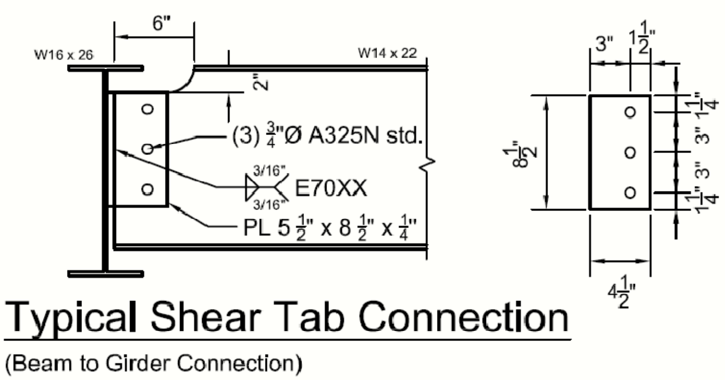



Figure 39: Shear Tab Connection

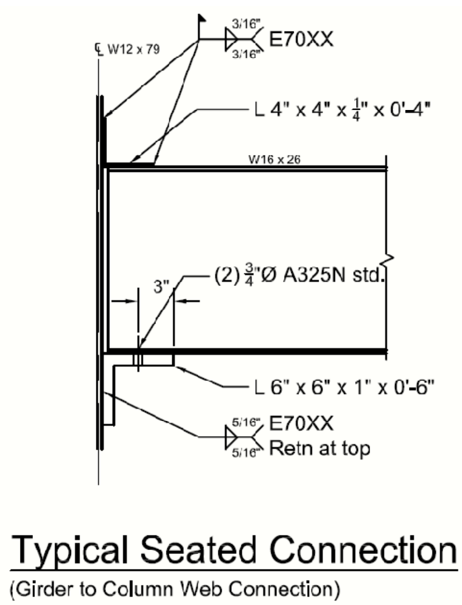



Figure 40: Seated Connection

Typical lateral connections were designed for the building, a moment connection and a heavy braced connection. The moment connection was designed in two parts, a shear tab connection to take the shear load, and a bolted flange plate welded to the column for the moment load. The plates bolted to the beam flange were checked for both the compressive and tensile loads from the moment, and the controlling design was used for all sides of the moment connection. It was determined the 2 rows of (3) ¾" bolts would be used to support the moment capacity. Figure 41 shows the detail of the moment connection.

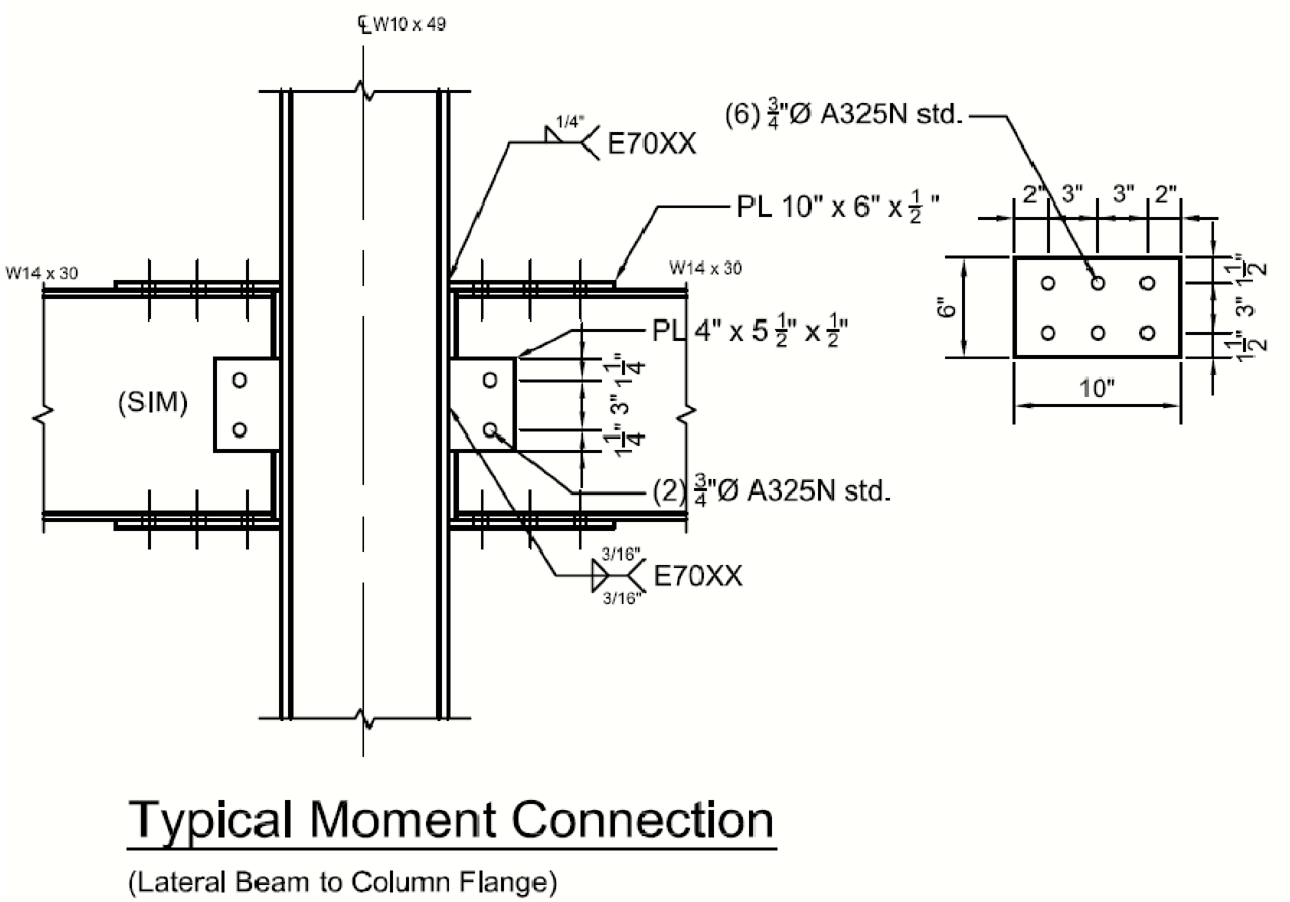



Figure 41: Moment Connection

A heavy braced connection was designed for the braced frame to adequately take lateral loads acting on the braced frames. A double angle was designed for the brace and it was designed as a bolted connection to the gusset plate. Using the Uniform Force method to size the gusset plate, dimension were chosen to reduce the moments at the connection interfaces. A bolted welded double angle was designed for the gusset to column flange connection. Prying of the angle was checked, and did not control the design of the connection. A simple shear tab connection was used for the beam to column connection. Compressive buckling of the plate did not control the design, the design of the shear resistance was sufficient. Figure 42 shows a detail of the braced connection.

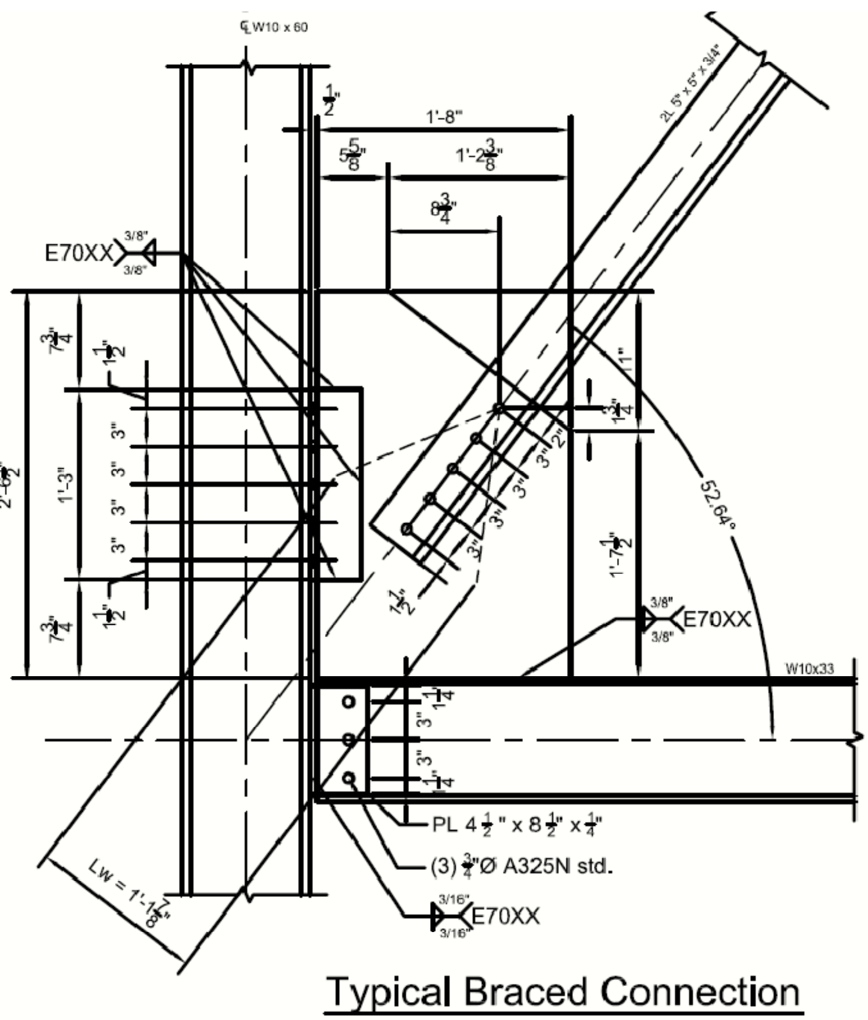



Figure 42: Heavy braced Connection

# Progressive Collapse

Progressive collapse or disproportionate collapse in respect to the original cause of the damage, is when a local element is damaged leading to a collapse of adjoining members. The concepts behind the design of structures to meet progressive collapse requirements are to design redundancy in the gravity system so that if a member is damaged the collapse will remain localized. There are currently two methods of designing structures to resist progressive collapse, the first method is proposed by the General Service Administration, and the second is set by the Department of Defense.

Both methods recommend that the structure implement passive defenses to mitigate the effects on the structure. These methods include but are not limited to stand of distances and blast resistance design. Design the exterior of the structure to be more robust to resist the damaging effects from the unforeseen event.

For the purpose of this thesis the Alternate Path Method is studied. This method assumes the loss of a primary structure element and the adjoining structure elements must be designed to support the additional load. Each beam will theoretically be designed to support double span conditions. Figure 43 shows the concepts behind the removal of the column and the start of the Alternate Path Method.

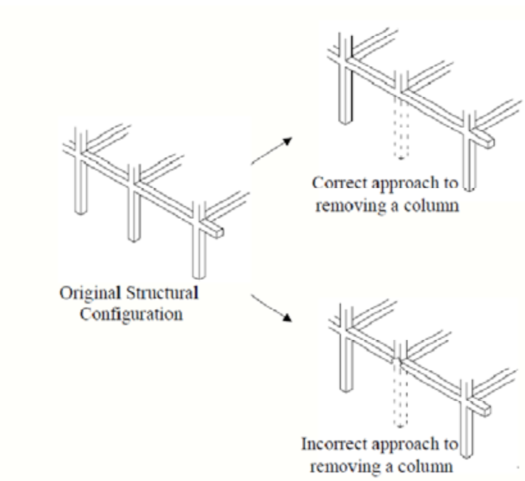



Figure 43: Progressive Collapse Concepts

# GSA Standards

## Design Guidance

This method places the imports on the floor girders and beams be designed to span two full spans, which requires beam to beam continuity over the missing column. The first requirement is to design connections that are capable of producing discrete beam-to-beam continuity. This states that the connection be able to provided a beam-to-beam continuity link that is able to redistribute gravity loads for a multiple span condition.

Connections should be designed to meet three criteria; resilience, redundancy, rotational capacity. Providing connection resilience is in the designing of the configuration of the weld geometry, to provide a ductile connection under instantaneous loss of a primary structural element. Connection redundancy is implemented so that there are multiple load paths to be used to distribute the gravity loads. Connection rotational capacity is provided to allow the connection to deform and rotate and form the formation of the plastic hinges in the beam of girder, while maintaining sufficient strength after the loss of the column. Figure 44, shows the response expected from a typical steel connection.

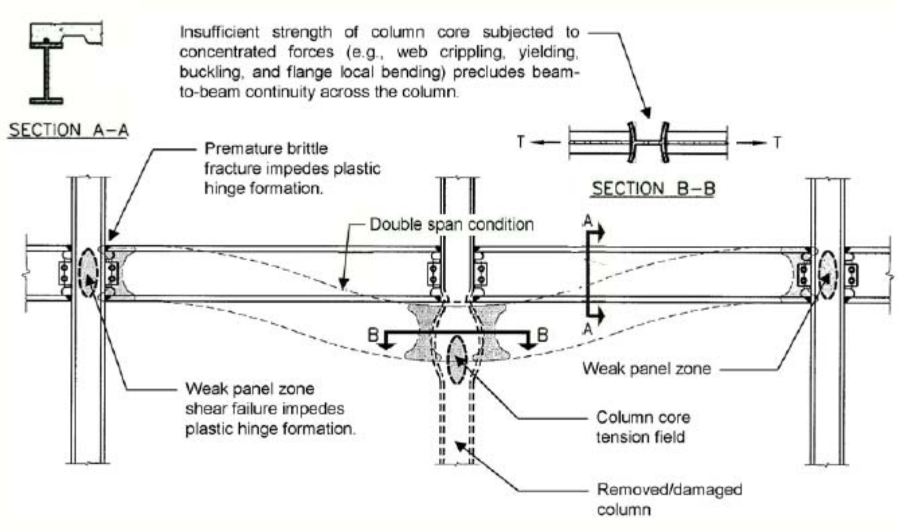



Figure 44: Response of the structure

## Analysis Procedure

Analysis procedures for this method use linear elastic, static analysis approach to look at the potential for progressive collapse. It is recommended that the use of 3-dimensional models be used to accurately account for the effects from secondary elements. 2-dinemensional models are accepted but are more conservative. A structure can be analyzed for progressive collapse by looking at components and connections for the loss in primary vertical support.

For typical structure configurations the analysis considerations should include all unique structural difference that would require a different analysis for the design to resist progressive collapse. Such difference are, beam to column connection change for simple to moment, or large change in span sizes on either side of the column, and a change in beam size.

Exterior considerations for the loss of a primary structural element include the removal of a column at critical locations. If any other location is determined to be critical in the design of progressive collapse it is required to consider that condition as well. Figure 45 shows a diagram that illustrates the key locations of column removal. The standard procedures are the analysis of the loss of a column for one floor above grade at the corner of the building, middle of the long side of the building, and the middle of the short side of the building. Interior considerations for progressive collapse shown in Figure 46, explain for interior columns unprotected by underground parking or uncontrolled public ground floor.

When determining the potential of a structure for progressive collapse the following vertical load shall be applied to the structure; Load = 2(DL + .25LL). For exterior column removals the maximum extent of collapse to be considered is the minimum of; the structural bays directly adjacent to the removed column, and 1,800 square feet at the floor level directly above the removed column. For interior considerations the maximum extent of collapse is to be the structural bays adjacent to the removed column up to 3,600 square feet at the floor level directly above the removed column.

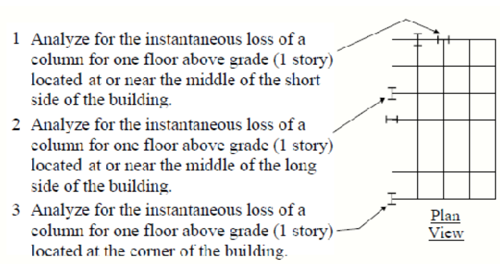



Figure 45: Exterior Considerations

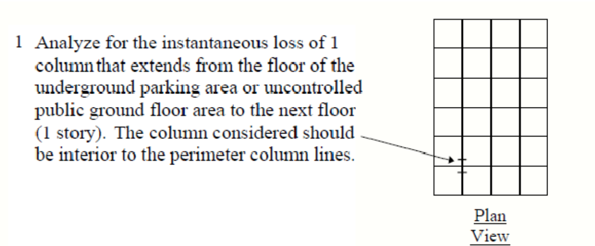



Figure 46: Interior Considerations

## Acceptance Criteria

Using linear elastic analysis results the forces and rotations are compared to the acceptance criteria to determine if the structure is adequate to resist progressive collapse. Once the primary structural element is removed the affected; beams, girders, columns, and connections are compared to maximum allowable demands. They are checked by Demand-Capacity Ratios (DCR). Members that exceed these limits will generate a plastic hinge and the redistribution of forces until the structure is adequate, Figure 47 shows the formation of the plastic hinge. Some members will have to be redesigned so they will meet these criteria, and a collapse mechanism is not initiated.

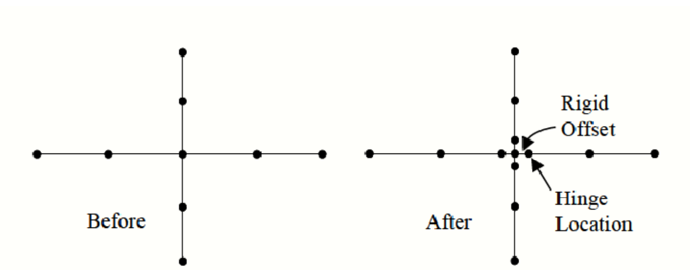



Figure 47: Plastic Hinge Formation

Primary and secondary components must be checked for the acceptance criteria from Equation 1. Where $Q_{UD}$ is the acting force demand determined by analysis, and $Q_{CE}$ is the expected ultimate, un-factored capacity of the component under analysis. The DCR is then compared to the allowable DCR requirements. For beams under flexure the DCR limit is either 2 or 3 depending on the beam dimensions.

$$DCR = {}^{Q_{UD}}/_{Q_{CE}} \qquad \text{Equation 1}$$

## Non Linear Analysis

Using the method set forth by the General Service Administration, progressive collapse analysis and redesign was implemented. Using the moment frame along Grid 1, a typical column along the middle of the frame was removed and the structure was analyzed to determine if it was adequate to resist progressive collapse. A non linear method was used to model the progressive collapse which is assumed to be a more conservative method. Using virtual work to determine the plastic hinge formation to create a collapse mechanism, the plastic moment required was determined.

Assuming that each level must support the summation of the gravity load above that floor, the beam must span a double span condition was well as the full load transfer from the column above. Using the load combination prescribed by the GSA the floor loads were determined and transferred to the beam under investigation. Using virtual work, the required plastic moment for the beam at level 2, supporting a 1434 kip load at the location of the removed column, is 7059.6 ft-kips. Using a DCR of 3, a W33 x 169 was adequate to take the applied load, to resist the formation of a collapse mechanism.

The applied moment was transferred to the column with half the gravity load to design the column for combined axial and flexure loads. The column used a DCR of 2, and the interaction diagram for the column was studied. A W14 x 500 was selected to support the large moment with the relatively small axial load. This column is considerable large, and assumed to be highly conservative.

This method of progressive collapse design does not account for the total structures resistance to progressive collapse instead it applies the load to the bay directly being analyzed and requires the beam to support the load, and prevent the formation of a plastic hinge. Realistic the structure will act as a truss, bridging over the removed column. To obtain this type of result the structure must be model in 3D modeling software. This method will accurately show how the structure responds under the loss of the column and how the structure works as one to resist the formation of a plastic hinge.

## Department of Defense Standards

The Department of Defense standards for progressive collapse design use three methods for design and analysis which are; The Tie Force Method, Alternate Path Method, and Enhanced Local Resistance. The Tie Force Method approach assumes the building is tied together as a continuous structure, and the loading conditions will be used to check the structural members for acceptance. The Alternate Path assumes the loss of a primary structural element and the structural is checked for double span conditions. The Enhanced Local Resistance checks the flexural and shear resistance of the perimeter columns to check for the design of progressive collapse. For the purpose of this thesis and the comparison to the GSA Standards, only the Alternate Path Method will be researched and studied.

### Design Guidance

For the Alternate Path Method, there are three analysis procedures that can be used to design for progressive collapse. The three methods are; Linear Static, Nonlinear Static, and Nonlinear Dynamic. Each method uses ASCE 7 and ASCE 41 for the load combinations and material strength estimates.

The structure is separated into two elements; primary and secondary components. Primary components are the structural members that will directly resist the potential collapse, or the members that are going to be designed in progressive collapse. Secondary components are any other structural elements that do not meet the requirements as primary components.

The designs are categorized into force controlled actions and deformation controlled actions. For moment frames the shear and axial loads are considered as force controlled actions and the moment is considered a deformation action. Progressive Collapse should be analyzed for the following stories; first story about grade, story directly below roof, story at mid height, and story above the location of a change in wall size. The removal of columns should be considered for the corner column, column at mid span along the long side, and the column at mid span along the short side, Figure 48.

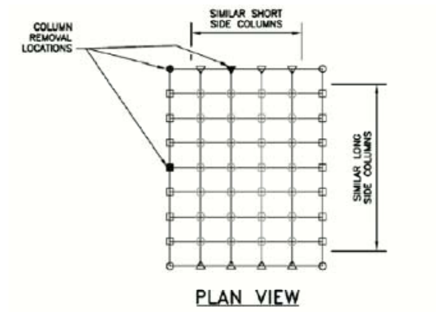



Figure 48: Column Removal

## Linear Static Procedure

The use of the linear static procedure is limited to certain structures that are considered to be structurally regular. If the structure is considered to be irregular the linear static procedure may be used if the DCR's are less than 2.0; Equation 1. Where $Q_{UD}$ is the acting force demand determined by analysis, and $Q_{CE}$ is the expected ultimate, un-factored capacity of the component under analysis. The DCR is then compared to the allowable DCR requirements. For beams under flexure the DCR limit is either 2 or 3 depending on the beam dimensions.

Two loading conditions shall be considered one for deformation controlled actions and another for force controlled actions. The increased load shall be applied to the bays immediately adjacent to the removed element and at all floors above the removed element, Equations 2 and 3. For the rest of the bays not adjacent to the removed column a load that is not modified for the force and deformation controlled actions, Equation 4. $\Omega_{Ld}$ and $\Omega_{LF}$ are load increased factors that are calculated from the table 3-4 provide in Appendix J. For lateral loads that will be applied to the structure, Equation 5 shall be applied in each side of the building in combination with each gravity combination.

$$G_{LD} = \Omega_{LD}[(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 2$$

$$G_{LF} = \Omega_{LF}[(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 3$$

$$G = [(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 4$$

$$L_{LAT} = .002\sum P, P = gravity\ load\ acting\ on\ a\ floor \quad Equation\ 5$$

For element acceptance criteria the components are analyzed using the linear procedures. For deformation controlled actions the deformation controlled action is compared to the expected strength of the component modified by a m-factor and strength reduction factor. The strength reduction factors are determined by the force that is being analyzed and the requirements set by the steel design standards. M-factors are determined using Table 5-1 supplied in Appendix J.

$$Deformation\ Controlled\ Actions:\ \Phi m Q_{CE} \geq\ Q_{UD}$$

$$Force\ Controlled\ Actions:\ \Phi m Q_{CL} \geq\ Q_{UF}$$

Nonlinear Static Procedure

The use of nonlinear static procedure is used for any structure; there are no DCR or geometric irregularity limitations. A 3-dimensional model must be used to model the primary and/or secondary structural elements. If secondary elements are modeled there stiffness's must be ignored.

Increase gravity loads shall be applied to the bays directly adjacent to the removed column, and every floor above the removed column, Equation 6. For the rest of the bays not adjacent to the removed column a load that is not modified for the force and deformation controlled actions, Equation 7. $\Omega_N$ is the load increased factor that are calculated from the table 3-5 provide in Appendix J. For lateral loads that will be applied to the structure, Equation 8 shall be applied in each side of the building in combination with each gravity combination.

$$G_N =\ \Omega_N[(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 6$$

$$G =\ [(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 7$$

$$L_{LAT} =\ .002 \sum P\ , P = gravity\ load\ acting\ on\ a\ floor \quad Equation\ 8$$

For element acceptance criteria the components are analyzed using the nonlinear procedures. Primary and secondary elements shall have deformation capacities greater the maximum calculated deformation demands. The strength reduction factors are determined by the force that is being analyzed and the requirements set by the steel design standards.

$$Force\ Controlled\ Actions:\ \Phi Q_{CL} \geq\ Q_{UF}$$

## Nonlinear Dynamic Procedure

The use of nonlinear dynamic procedure is used for any structure; there are no DCR or geometric irregularity limitations. A 3-dimensional model must be used to model the primary and/or secondary structural elements. If secondary elements are modeled there stiffness's must be ignored.

A gravity load combination is applied to the entire structure, Equation 9. For lateral loads that will be applied to the structure, Equation 10 shall be applied in each side of the building in combination with each gravity combination. For lateral loads that will be applied to the structure, Equation 10 all be applied in each side of the building in combination with each gravity combination. The loading procedure for the dynamic analysis is to start at zero load and proportionally increase the gravity and lateral loads to the entire model, the column has not been removed yet. Once the structure reaches equilibrium the column is removed instantaneously, the duration of the removal should be less than one tenth of the period. The analysis is continued until a maximum deflection is reached or one cycle of vertical motion occurs.

$$G_{ND} = \Omega_{ND}[(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 9$$

$$L_{LAT} = .002\sum P\ , P = gravity\ load\ acting\ on\ a\ floor \quad Equation\ 10$$

For element acceptance criteria the components are analyzed using the nonlinear procedures. Primary and secondary elements shall have deformation capacities greater the maximum calculated deformation demands. The strength reduction factors are determined by the force that is being analyzed and the requirements set by the steel design standards.

$$Force\ Controlled\ Actions\colon\ \Phi Q_{CL} \geq Q_{UF}$$

# Cost and Schedule Comparison

A cost and schedule comparison of the original concrete design to the new design of structural steel, was created using RS Means 2007. The final contract cost for the project is $110 Million; however the final structural cost is unknown. Takeoffs for both structural systems were performed to compare the change in cost and the change in schedule. Construction began in 2007, which is why RS Means 2007 was chosen to perform the cost and schedule for this thesis.

## Goals

1. Reduce the cost of the structure
2. Reduce the schedule of the structure
3. Compare results to actual market in DC

## Cost Analysis

Detailed structural takeoffs were performed for the entire building for both designs. Concrete takeoffs were taken from the construction documents, and the steel takeoff was taken from the 3D RAM Model. More detail takeoffs of the structure can be found in Appendix H. RS Means 2007 was used for the cost takeoff to accurately portray the cost difference RS Means shows for the two systems.

| Cost Summary | | |
|---|---|---|
| | Cost | Cost per SF |
| Steel Design | $2,848,700.43 | $22.93 |
| Concrete Design | $3,472,186.16 | $27.95 |

Figure 49: Cost Summary

A summary of the cost analysis is provided in Figure 49. IT was observed that the new steel design was cheaper than the concrete cost takeoff that I performed. For the new steel design it was determined the structural system would cost approximately $23 per square foot This value matched with the RS Means Assemblies value reported in previous thesis work, when the comparison of different floor systems. The Concrete design obtained a $28 per square foot cost. This value was increasingly higher than the value obtain from RS Means Assemblies, the difference can be attributed to the higher amount of formwork for the larger beams, because this cost report for an interior bay only and not an exterior bay.

## Schedule Analysis

From the quantity takeoff performed for the cost study, the schedule of tasks was created. Using the recommended crews and the crew output data from RS Means, a detailed schedule breakdown was created using Excel. This schedule assumes a linear progression of takes, display that the next task will start directly at the completion of the previous task. Appendix H is provided for a detailed takeoff of the schedule breakdown. Microsoft Project was used to create a more accurate schedule to show how tasks can over lap during the construction process. A summary of the schedule comparison is provided in Figure 50.

| Schedule Summary | | |
|---|---|---|
| | # Days * | # Days ** |
| Steel Design | 300 | 133 |
| Concrete Design | 1071 | 456 |
| * Assumes Linear Progression of Work | | |
| ** Assumse early start times on some taks | | |

Figure 50: Schedule Summary

It was observed that the new steel design could be constructed in a shorter duration than the original concrete design. In both schedules created the steel outperformed the concrete, but in the schedule using Microsoft Project it was observed the steel design could be complete in 133 days, and the concrete design took 456 days. The difference in schedule days is approximately 5 months and for a project that does not have a time constraint is not an issue.

It is important to note that local markets would vary from what RS Means shows as the cost and schedule breakdown. In the DC market the preferred method of construction is a two way flay slab. In this area the contractors are more efficient in erecting two way flat slab buildings. The cost and schedule for these systems will decrease with the increase experienced in the area. This effect can actually cause the concrete system to be more effective than a typical steel system. Also when there is a height restriction on the building, a two way flat slab system provides low floor to floor heights.

# Mechanical Coordination

The impact of the new floor system is considered on the mechanical system, and new design of the mechanical duct is proposed. Originally the floor system depth at the critical duct location is 9 ½ inches. The new steel design places a W18 x 35 across the corridor, with a new floor system depth of 22 1/4". Figure 51 shows the location of the critical on the framing plans.

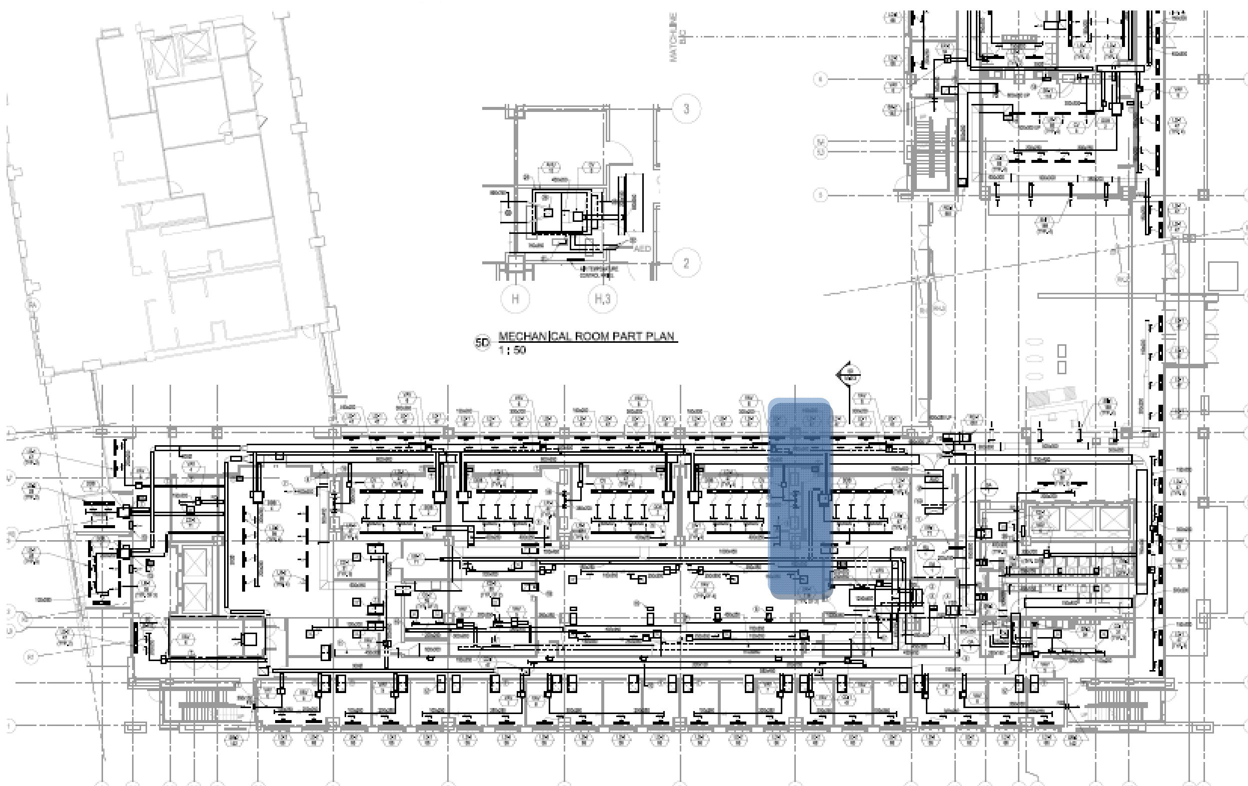



Figure 51: Critical Mechanical Location

## Goals

1. Evaluate the existing mechanical system
2. Determine new design for mechanical ducts
3. Determine if new design is adequate

It was determined that the original mechanical duct design would not be able to fit under the new structure, and the mechanical ducts would need to be redesigned. Using the equivalent velocity method, the area of each duct was determined. Using the existing airflow passing through the two supply ducts, the total airflow was approximated. Assume a duct velocity of 20 feet per second, which is standard for a main supply duct, the required area for the duct was calculated. Figure 52 shows a summary of the mechanical design.

| Mechanical Design | | | |
|---|---|---|---|
| | cfm | v (ft/s) | A (ft^2) |
| Duct 1 | 3385.97 | 20 | 2.82 |
| Duct 2 | 2101.93 | 20 | 1.75 |

Figure 52: Design Summary

After looking at the allowable space between the bottom of the structural steel, to the top of the lighting system, the mechanical ducts were sized. Using the area found from the equal velocity method, the allowable height for each duct was determined, and the width was sized to match the allowable area, Figure 53 shows a summer of the design. Figure 54 is providing to show the cross section of the new design. Although the duct could be redesigned to fit in the new space, the friction changes for the new duct size were not taken into account. A deeper mechanical study including friction loss could be performed. Details of the redesign can be found in Appendix I.

| Mechanical Design Summary | | | |
|---|---|---|---|
| | | Width | Heigth |
| Original Design | Duct 1 | 29.53" | 15.75" |
| | Duct 2 | 25.59" | 11.81" |
| New Design | Duct 1 | 35.5" | 12" |
| | Duct 2 | 24" | 11" |

Figure 53: Mechanical Design Table

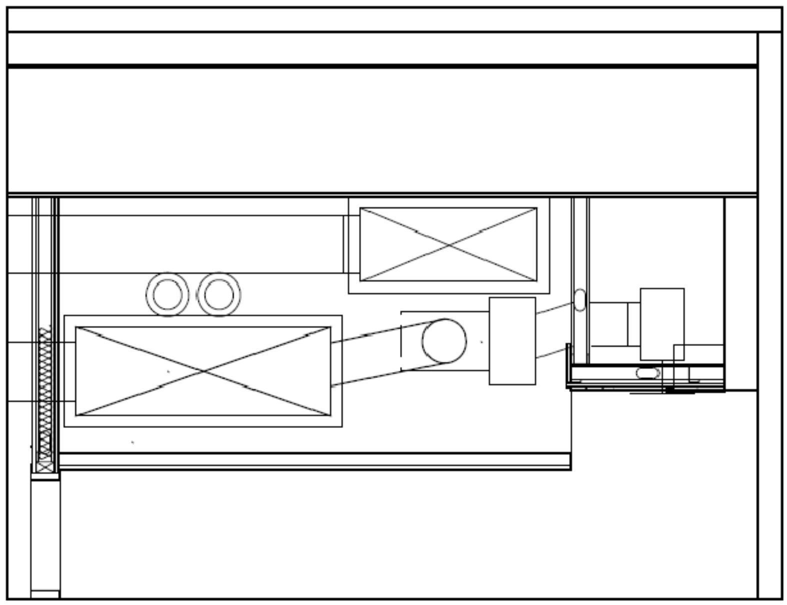


Figure 54: New Mechanical Design

# Conclusion

The goal of this thesis is to study the effects of an entire structural redesign from impacts on the structure to cost and schedule. The structural system was redesigned using typical steel on metal deck floor system. The gravity system was designed using the ASCE 7-05 and AISC Steel Manual 13th Edition. IT was determined that the steel was more effective at spanning longer distances; however the depth of the floor system did conflict with the mechanical system.

The new lateral system design was more involved and required an iterative approach. Once the initial design was chosen it was analyzed and a better optimized. Braced frames around the core were designed to take the primary lateral load and to limit the deflection of the wing. Exterior moment frames were designed to aid in the resistance of progressive collapse and reduce the eccentric effects of the wing. Through the lateral analysis, a braced frame along Grid H was added to reduce the eccentric effects on the wing.

In structural steel design, the connections are very important in the transfer of the loads. Typical gravity connections were designed as a simple shear tab connection and an un-stiffened seated connection. Lateral connections were design as a moment connection for the moment frame, and a heavy braced connection for the braced frame.

As the threat for unforeseeable events increase, the design of structures to resist progressive collapse is becoming more important. The original structure was designed to resist progressive collapse, and part of this thesis was devoted to the research of the design of structures to resist progressive collapse. Two methods exist for the design to resist collapse, and both methods are presented in this thesis. The GSA method for progressive collapse was implemented for the moment frame along Grid 1.

The design procedure used was non-linear static because the instantaneous loss of column is non linear in nature. Using virtual work, and plastic hinge formations to form the collapse mechanism, the beams were sized. For the beam at level 2, the beam required to meet the acceptance criteria is W33 x 169. This size was exceedingly larger than the original design. It is apparent that the GSA Method and non linear method form a highly conservative method for progressive collapse.

Impacts on the cost and schedule were studied and compared to the original design and the new structure design. Both cost and schedules were created using RS Means Building Construction Data 2007. It was determined that the structural steel cost was $23 per square foot, and the original cost was $28 per square foot. It is important to note that the local market in DC favors two way flat slab constructions and the cost and schedule will be favored towards the original design.

The mechanical system was looked in respects to the ceiling to floor space where the mechanical ductwork passed through, the new structure design limited the space for the mechanical ducts and at a critical location, two of the ducts need to be resized. The first floor corridor was the most critical location for redesign because it had two ducts passing through the W18x35. The ducts were resized using the existing airflow through the system, and were sized to fit in the new adjusted space.

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