FINAL REPORT

General Office Building

Greater Washington, DC

Brian S Rose

STRUCTURAL OPTION

Faculty Consultant:
Dr. Boothby

April 4, 2012


United Therapeutics: Phase 2B
Brian Rose | Structural Option
www.engr.psu.edu/ae/thesis/portfolios/2011/bsr5023
DESIGN TEAM:
Owner:
United Therapeutics
Architect & Engineer:
EWING COLE
General Contractor:
DPR
GENERAL INFORMATION:
Size: 120,000 SF
Number of Stories: 10
Cost: $40 million
Start of Construction: August 2010
Projected Completion: December 2011
SUSTAINABILITY:
•LEED Silver Rating
•Energy Recovery Wheels
•Extensive Green Roofs
•Roof PV Arrays
MECHANICAL:
•Boiler
•Chiller
•VAV Air System
•Forced Draft Cooling Tower
CONSTRUCTION:
•Design-Bid-Build
•Form, Shore and Pour Construction
STRUCTURE:
•Spread Footings
•Two-Way Flat Slab with Drop Panels
•Unitherium: Steel Beams, Columns and Metal Deck
OCCUPANCY:
•Offices
•Daycare
•Retail
•Parking
ELECTRICAL:
•460Y/265V Transformer
•3000 Amp Trip with 100% Rating Switchboard
Architecture:
•Street-Level Pedestrian Plaza
•Metal Panel and Curtain Wall Facade
•90 foot Long Bridge, Which Spans to the Existing Unither Research Building
•Three Story, Sun-Lit Artium
Lighting:
•Primarily Linear Fluorescent
•Luminaires
•Unitherium: Color-Changing LED Mesh

TABLE OF CONTENTS

ACKNOWLEDGEMENTS

I would like to sincerely tank the following people and companies for their help and support during the completion of this report.

EwingCole, for providing me the opportunity to work on the design of this project while interning at their firm, for allowing me to extensively use their work, and for providing me with all the necessary tools for this report. I would also like to specifically thank:

Brent Elleman

Peter Welsh

Aitor Sanchez-Prado

DPR Construction, for providing me with their cost and schedule information and for all their assistance with construction questions. I would also like to specifically thank:

Adam Ponder

Sarah Moreau

Randy Shumaker

Cannon Design, for introducing me to Progressive Collapse design and allowing me to thoroughly explore the analysis under helpful guidance.

Family and friends for their eternal support and help along the way

All my Penn State AE peers, who not only provided invaluable feed-back and suggestions, but who also made all this strenuous work enjoyable. I would also like to specifically thank:

Nathan McGraw

Thomas Kleinoshy

Rebecca Dick

1.) EXECUTIVE SUMMARY

The following report investigates the General Office Building and describes the solution to a proposed scenario. The goal for this report was to strengthen existing structural analysis skills by exploring a new and unique area of structural analysis. As part of this hypothetical situation, the new occupant required the building to resist progressive collapse and terrorist attack. This report describes the analysis conducted and the conclusions made.

The General Office Building is located in the greater Washington D.C. area. The primary use of the building is offices. The existing floor structure is comprised of two-way concrete flat slab with drop panels. Shear wall cores resist the lateral loads in the existing design.

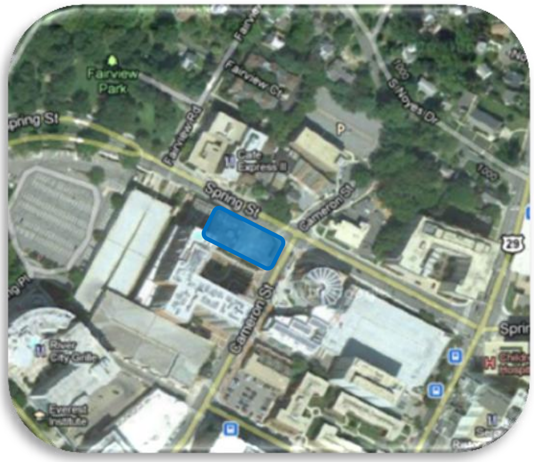
The existing structure was first redesigned using composite steel members and a moment frame lateral system. Computer programs and hand calculations were used to design this new system to standard code requirements, which include live, dead, wind, and seismic loads. Wind drift limitations were found to control the lateral system analysis. Modifications were made to the existing layout, when reasonable, which produced a more efficient design. This steel design was considered the base structural system, to which later redesigns were compared.

The Department of Defense's antiterrorism design guide, entitled Unified Facilities Criteria, was used as the basis for much of the second redesign. Three structural design methods were used to strengthen the base steel design against terrorist attacks. The Tie Force Method resulted in additional slab reinforcing. The Alternative Path Method was conducted at two locations and increased the exterior frame sizes. The computer analysis was verified using simplified non-linear hand calculations. The Enhanced Local Resistance Method reinforced the perimeter column against brittle failure. To ensure the moment connections were capable of the increased loading, a typical moment connection was designed. Masters level courses were used in this connection design and it was concluded that a sufficient connection could be constructed.

The architectural impacts of the structural alterations were investigated, along with the necessary site plan alterations. The south atrium, in particular, was investigated. Structural cables, which were designed to carry the blast loadings, were added to the space and investigated aesthetically and functionally. The existing site plan was also redesigned to accommodate a 100 foot standoff distance.

The cost and schedule of the proposed redesigns were investigated in a construction management breadth. Only the superstructure was examined. The base steel redesign was found to be more expensive, but faster to construction. When progressive collapse was added to the design requirements the cost and schedule both increased modestly.

2.) EXISTING CONDITIONS



This is the second and final phase of General Office Building (or GSB) headquarters expansion in the greater Washington, DC area. It can be seen from the Google Maps image to the right that GSB is located at the intersection of Cameron and Spring Streets. This intersection is at the edge of the city's business district and the surrounding suburban community. During the second of the headquarters expansion, an eight story circular building was erected on the east side of Cameron St. to house the laboratories and research operations of the corporation. The primary purpose of this phase is to house owner's offices. EwingCole took on this challenge and was both the architect and engineer of phase two.

The entire 120,000 sf. building rises 90 ft. above grade, with an additional penthouse above. To achieve this design DPR Construction Inc. submitted a winning bid of $40 million. DPR broke ground in August 2010 and expects substantial completion in December 2011.

Figure 2.1 – Aerial rendering looking across Spring St.: Courtesy of EwingCole

Figure 2.2 – Atrium rendering: Courtesy of EwingCole

GSB starts two stories below ground level. Located in this substructure are 25 parking spaces, boiler units and other mechanical equipment. As can be seen in Figure 2.1, a large arm of the building stretches westward and creates a landscaped pedestrian plaza at street level. In this arm is a state of the art conference auditorium that is used to showcase the owner's latest cardiovascular medicine.

Patrons enter the main building from this plaza into a large three-story atrium, as Figure 2.2 shows. From this curved atrium occupants may enter the two retail spaces on the ground floor or move to the upper floors.

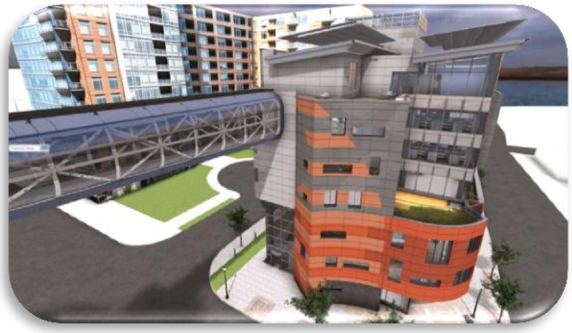


Figure 2.3 - Rendering looking from 6th floor of phase 2A building across Cameron St: Courtesy of EwingCole

Most of the upper stories house the offices, both open and private. The larger executive offices on the seventh floor offer extensive windows with views to the neighboring suburban community. A company daycare occupies the entire fourth floor. Large landscaped terraces at this level offer an outdoor play area for the young children. Figure 2.3 illustrates the 6th floor walking bridge that spans across Cameron St. to the first stage building. Steel tubes, which are bent into a helical shape, support this bridge. The overall architectural design focus was to present the owner as a modern research company that wishes to beautify and enrich the local cityscape.

Interlocking zinc metal panels comprise the majority of the building's enclosure. All metal panels are painted, mostly burnt orange or grey. An aluminum curtain wall system with 1 in. insulated glass is the primary window system of the GSB. This exterior is supported by 6 in. cold formed metal framing and insulated with 2.5 to 3.5 in. of rigid or semi-rigid insulation.

Two separate roofing systems are utilized, a polyvinyl chloride (PVC) membrane system on the main building and a green-roof system on the auditorium structure and some fourth floor terraces. The green roof is comprised of 8 in. of growth media separated from the structure by a vapor barrier.

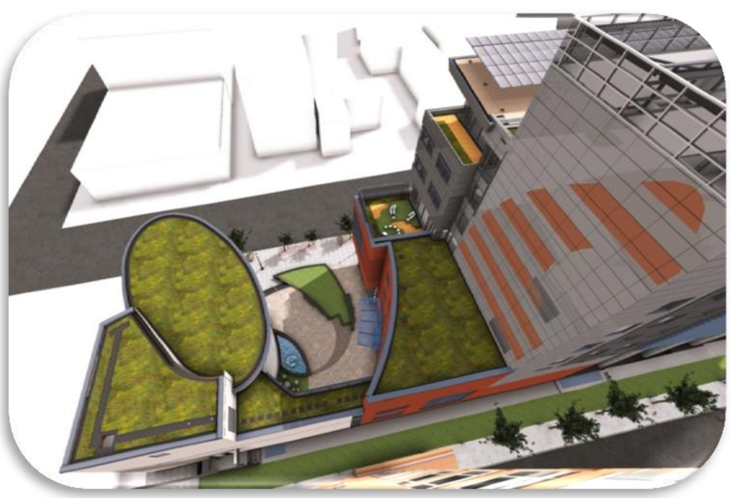


Figure 2.4 - Aerial rendering looking down towards Spring St.: Courtesy of EwingCole

The minimum design goal for this project is to be LEED Silver. To achieve this rating a photovoltaic (PV) array is stationed on the roof and was design for future PV array expansion. The PV arrays are elevated from the main roof to allow for solar shading in the summer. The green roof on the remaining portions of the building helps to collect and recycle rain water while regulating the temperature of the space below. Heat recovery systems help to increase the HVAC efficiency.

3.) STRUCTURAL OVERVIEW

The following is a description of the current building design. All comparisons proposed system redesigns were compared to this system. The two wings of the phase two expansion are composed of very different structural systems. The primary structural system of the main wing is a cast in place 8" two-way flat slab with 4" to 6" drop panels. The concrete slab on metal deck of the smaller auditorium wing is supported by composite steel beams. In both wings, 10" to 12" reinforced concrete shear walls resist lateral loads.

3.1) FOUNDATION

Schnabel Engineering, the geotechnical engineer for this project, conducted 4 soil test borings around the site. Analyzing these borings, Schnabel Engineering found that the fill soils and top soil extend down from 2.5' to 13.5' below grade; below this, a layer of sand extends down another 10' until a structural layer of disintegrated rock was reached at around 20' below grade. The geotechnical engineer originally recommended a maximum allowable stress of 10,000 psf for the sand layer, which is at the depth of the proposed foundations. This capacity was later cautiously raised to 15,000 psf after further investigation when higher than expected column loads developed in the design phase. Also during this study, groundwater was found as little as 7' below grade. These findings resulted in the entire foundation slab being designed for full drainage.

The foundation of the GSB consists of spread footings in combination with exterior strip footings, which support basement walls. All footings under the western four-story portion of the plan are supported by 8,000 psf bearing capacities, while all footings in the western, eight-story, portion had to be designed for a maximum 15,000 psf bearing capacity. The spread footings beneath individual columns range from 2 ft. to 3 ft. thick. Along the northern and western edges grade beams connect the interior foundations to the exterior strip footing. The perimeter strip footing's 2 ft. to 3 ft. thickness supports a 2 ft. thick foundation wall. Lateral soil pressures were not investigated in this report, but will be considered in future design. To resist overturning in the two shear wall cores have 4 ft. thick mat foundations that extend to adjacent column foundations.

3.2) FLOOR CONSTRUCTION

An expansion joint in the connecting walkway separates the two portions of the GSB; the auditorium is to the west and the main building is to the east. A composite steel beam system was chosen for the three-story auditorium for several reasons. First, this structure is located atop a concrete beam system above the parking levels; the light-weight steel frame allows for smaller concrete supporting beams. Second, the occupancy of the space (conference auditorium) requires large, uninterrupted spans with unsymmetrical bay sizes. Steel framing is best suited for this type of layout, which is why it was chosen for this wing.

The auditorium's W12 to W21 beams support a 4 ½" normal weight concrete on 2" composite metal deck floor system. Beams beneath the oval (conference) portion of the wing are spaced at 6'-0" on center and span up to 35'. For economy of materials, these beams have studs to create partially composite action with the floor slab. All of these beams either directly frame into the curved shear walls or frame into girders that span between shear wall ends. All beams and metal deck are sprayed with cementitous fireproofing to achieve a 2 hour fire rating.

Figure 3.1- Aerial construction camera looking at the Unitherium roof framing:
Courtesy of United Therapeutics and Oxblue.com

To create the complicated curved slab edges of the main wing, a concrete two-way flat slab was chosen. The predominance of concrete construction and cheaper labor in the Washington D.C. metro area, according to RS Means City Cost Index, was another major factor when selecting a concrete system. A majority of the building utilizes 8 in. slabs. Many a-typical areas have increased thickness of 12 in., and mechanical rooms have even larger 16 in. slabs. Drop panels provide the required punching shear and negative moment capacities at all floors. Typical drop panels measure 3'-8" by 3'-8" in plan and 4" thicker than the surrounding slab (for a total depth of 12"). Continuous drop panels run between several columns in the building. A typical bay measures 22' x 20'. All slabs start with a base reinforcing of #5 @ 10 on center, top and bottom. To take higher negative moments at columns, two to four additional #5 bars are added to the column strip where required. The 93' connector bridge on the sixth floor is supported by a huge 36" thick slab that redistributes the forces back into the building. This landmark bridge is formed from 7" round, twisting HSS tubes.

Figure 3.2 - Aerial construction camera looking at the third floor construction:
Courtesy of United Therapeutics and Oxblue.com

Concrete columns support all slabs and vary greatly in size. Concrete 24"x24" columns below the first floor support 6" to 8" steel round and square HSS tubes at the auditorium wing. In the main wing, columns vary greatly in size. Any column that was able to be architecturally hidden is square or rectangular. This saves on forming costs. All exposed columns, to be aesthetically pleasing, are circular in section. A typical column size shrinks from 28"x28" in the basement to 16" diameter at the seventh floor.

3.3) LATERAL SYSTEM

The GSB facility resists lateral loads through a series of cast in place reinforced concrete shear walls and continuous drop panels, which act as moment frames. As seen in the floor plans in Appendix A, each wing of the building contains two sets of shear wall cores to resist both direct and torsional shears. In the auditorium, three 12" curved shear walls are concealed in the conference room walls and three 10" walls surround the stair tower. The main wing utilizes similar layout; four 12" shear walls encase the elevator shaft and four 12" shear walls encase the stair tower. These two cast-in-place cores extend from foundations to the penthouse slab. Coupling beams, 24" deep, keep continuity around the large elevator door openings. Wall reinforcing ranges between #8 @ 12" o.c. and #5 @ 12" o.c.

The continuous drop panels shown in Appendix A are located on the fifth floor and represent the typical layout, but the size and location of these systems vary between floors. A typical continuous drop panel has a total depth of 14 to 20 inches and 6'-4" wide. This system of shallow beams was chosen because of structural depth limitations, especially around the mechanical rooms where large ducts take a majority of the plenum space. Moment frames were most likely added to help control wind drifts because the majority run in the North-South direction, which and the larger wind loads and shorter shear walls.

3.4) ROOF SYSTEMS

Two main roofing systems cover the GSB: a green roof and photo-voltaic array. As seen in earlier renderings, green roof covers both the conference room and terraces. The 8" of growing media rests on 2" metal roofing deck, which spans 6' across non-composite wide flange beams in the auditorium. On the main wing, steel channel mounting systems hold sloped PV arrays. Three feet down the HSS posts that support the PV mounting, 2" metal roof deck supports insulation and a waterproofing membrane. To carry the metal deck and roofing, W16 beams frame into tops of the building's concrete columns.

3.5) DESIGN CODES

The entire building project was design with the following general building codes:

- 2009 International Building Code (IBC 2009) with Montgomery County amendments
- 2009 International Energy Conservation Code (IECC 2009) with Montgomery County amendments
- 2009 International Fuel Gas Code (IFGC 2009) with Montgomery County amendments
- 2007 National Electric Code (NEC 2007)
- 2009 Montgomery County MD: Fire Safety Code- NFPA 1 and NFPA 101 (LSC - 2009)
- 2007 Uniform Fire Code (NFPA 2007)

The structural design codes used in the building analysis contained in this paper were the same as the codes used for the original design of the building, and are as follows:

- Minimum Design Loads for Buildings and Other Structures (ASCE 7 - 2005)
- Steel Construction Manual (AISC 360 – 13th Edition)
- Building Code Requirements for Structural Concrete (ACI 318 - 2008)
- Building Code & Specification for Masonry Structures (ACI 530 - 2008)

3.6) MATERIALS USED

Below are lists of the most common structural materials within the GSB. All information was derived from SG.1.

Concrete		
Usage	**Weight**	**Strength, f'c (psi)**
Slab on Grade	Normal	4500
Footings	Normal	4500
Foundation Walls	Normal	4500
Columns	Normal	5000
Transfer Girders	Normal	5000
Beams	Normal	5000
Suspended Slabs	Normal	5000
Shear Walls	Normal	6000
Concrete Slab on Mtl Deck	Normal	4500

Note: Strength measured with ASTM C873 28 day compressive strength test

Masonry		
Material	**Weight**	**Strength, f'm (psi)**
4" CMU	Normal	2,800
8" CMU	Normal	2,800
Mortar	Normal	2,000
Fine Grout	Normal	3,000

Note: Strength measured with ASTM C1314 – 11 compressive test
f'm is a measure of assembly strength

Reinforcing		
Material	**Standard**	**Yield Strength, fy (ksi)**
Reinforcing Bars	ASTM A615	60
Welded Masonry Reinforcing	ASTM A706	60
Welded Wire Fabric	ASTM A185	65
Horizontal Masonry Reinforcing	ASTM A82	65

Steel		
Material	**Standard**	**Yield Strength, fy (ksi)**
Wide Flanges	ASTM A992 Gr. 50	50
Base Plates	ASTM A597 Gr. 50	50
Moment Plates	ASTM A597 Gr. 50	50
Splice Plates	ASTM A597 Gr. 50	50
Other Plates	ASTM A36	36
Round HSS	ASTM A500 Gr. B	42
Rectangular HSS	ASTM A500 Gr. B	46
Angles	ASTM A36	36
Channels	ASTM A36	36
Metal Roof Deck	SDI	33
Non-Composite Metal Deck	SDI	50

Table 3.3: Material Properties

4.) BASE STEEL REDESIGN

The first step to this progressive collapse redesign was to create a base structural steel model, which was not only used to compare the effects of adding progressive collapse requirements, but will also in the Enhanced Local Resistance analysis. A major goal of this redesign was to have a minimal impact on the existing architecture. As part of the hypothetical scenario, the occupant was changed to the Department of Defense, but this owner required the same architectural program needs. In this section, the changes to the existing architectural will be described with respect to the structure. Later in this report, the architectural impacts of these changes will be discussed.

4.1) DESIGN ASSUMPTIONS

To develop a more accurate comparison between a standard building and a building with progressive collapse requirements, the alterations from the existing concrete layout were made with progressive collapse implications in mind. The efficiency benefits for the base steel design were weighted against the efficiency benefits for the progressive collapse design. The goal for this thesis was to keep the same layout for all the steel designs and only change the member sizes to satisfy progressive collapse requirements.

The first issued faced in the steel redesign was the relatively small bay size. Larger bay sizes of approximately 30’-to-35’ offer cost savings because of the reduced number of connections in the entire building. A study conducted by John Ruddy, P.E., of Structural Affiliates International concluded that a typical bay size with a length-to-width ratio between 1.25 and 1.5 resulted in the most efficient member sizes (Ricker, 2). The existing typical base measured 22x20 feet, which equates to a 1.1 length-to-width ratio. Although this ratio falls below the optimum range, altering the column spacing would drastically alter the terraces. Figure 4.1 shows the exterior step-backs that follow the existing column lines. From the second to fourth floor, the slab edge greatly varies to create the open-to-below spaces and terraces. Figure 4.2 illustrates the curved slabs that look into the atrium. Changing the column locations at the upper floors would place obstructions in these voids and create expensive cantilevers at certain locations. For these reasons, the existing 22’ bay sizes were kept.

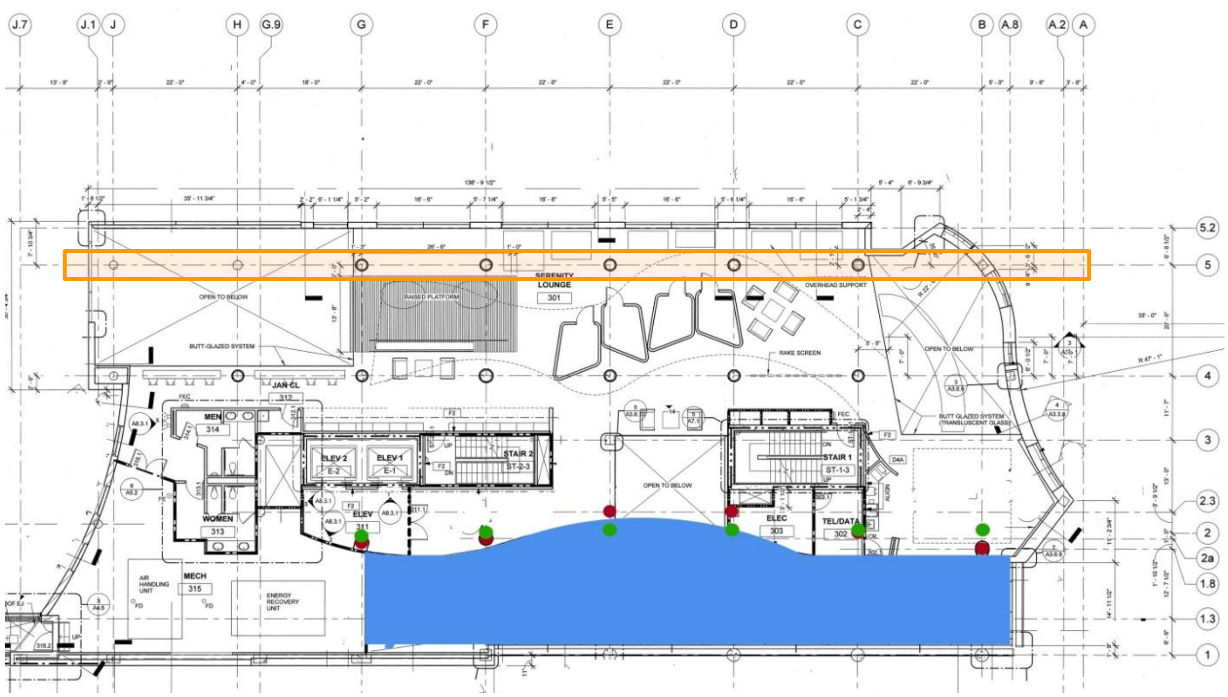
Figure 4.1 – Street View of North-West corner showing the terraces.

Also, this relatively small bay size offers increased redundancy and strength, which is vital for progressive collapse design. The smaller column tributary area, which is offered by this bay size, helps to redistribute a removed column load during alternative path analysis.

In the existing layout, a cantilever reaches out 6'-8" from the northern most column line to support the exterior façade. The location of this column line may be found in Figure 4.2. This cantilever was relatively inexpensive with concrete construction as compared to steel construction, which requires costly moment connections. In the redesign, column line 5 was pushed to the exterior to eliminate this cantilever. The architectural impacts of this change will be discussed in Section 10 of this report. With the columns running directly behind the exterior, the façade load was supported by a line of girders. This change was also beneficial for the progressive collapse analysis since these exterior frames will be designed to span a missing column, eliminating the cantilever reduced the tributary area of the critical exterior columns and thereby reduced the size of the columns.

The existing columns fall on a regular grid layout, except for the first interior row on the Southern façade. The red columns in Figure 4.2 represent the original location of these columns. These columns are offset a maximum of 3'-9 ½" from column line 2. The existing two-way flat slab could easily accommodate these offsets because it is only 19 percent of the 20' bay width. The continuous and distributive nature of two-way slabs makes this layout possible with little harm to its cost. This staggered column in a steel system, on the other hand, would be more problematic because of the required angled connections and uniqueness of each member. Standardized steel members and connections lead to more efficient fabrication and faster erection,

Figure 4.2 – Existing 2nd Floor Plan
Open Atrium
Existing Column Location
Redesigned Column Location
Original Cantilevered Col. Line



which saves the owner money. For these reasons, the columns were all shifted to align with column line 2. This shift placed columns closer to the open atrium, which my by architecturally unwanted. Most of these shifted columns, though, could be hidden with partition alterations.

Due to the complicated loading process for progressive collapse, a uniform live load was used for all interior loads. Progressive collapse load combinations will be discussed in Section 7 and reasons for this simplification will be given. All interior members were designed for 100 psf live load - typical assumption for general office buildings, where the specific partition layout is not known. Using this loading gave the new occupant the most layout flexibility. This produced a conservative design for the majority of the building, as the existing drawings prescribed 80 psf (office occupancy) live load. The mechanical rooms, which were originally designed for 150 psf, had non-conservative member sizes because the live load was reduced by 50 psf for this exercise. The penthouse was designed for 250 psf, as prescribed in the existing drawings. Refer to Table 4.1 for the live loads used in the redesign. If this analysis was continued in more depth, more precise live loads should be used.

Table 4.1: Live Loads

Type of Space	IBC 2006 Minimum	Specified in Drawings	Redesigned Value
Parking	40 psf	40 psf	N/A
Roof	20 psf	30 psf Snow	30 psf
Preschool Daycare	40 psf + 15 psf	80 psf	100 psf
Office, Shared Work Space	50 psf + 15 psf	80 psf	100 psf
Atrium, Reception and Plaza	100 psf	100 psf	100 psf
Retail Space	100 psf	100 psf	100 psf
Stairs and Stair Lobbies	100 psf	100 psf	100 psf
Corridor	100 psf	100 psf	100 psf
Conference Auditorium	100 psf	100 psf	100 psf
Terrace	100 psf	100 psf	100 psf
Computer Server Room	NOTE	250 psf	100 psf
Mechanical and Electrical Rooms	NOTE	150 psf	100 psf
Penthouse Mechanical	NOTE	250 psf	250 psf

All framing members were designed using an ETABS 3D computer model and typical members were checked using hand calculations. The design preferences were developed using standard steel design practices and code requirements. The 13th edition of AISC's Steel Construction Manual, ASCE 7- 2005, and IBC 2006 were the main codes used for this design. Economic data for optimizing members was taken from RS Means 2010 and was used to assign a price of $3300 / ton for structural steel and $3.10 / shear stud. A *Structure Magazine* article entitled "Cambering Rules of Thumb" was used to develop a cost for cambering beams and girders. According to the article, to account for cambering fabrication costs an approximate addition of 5 lbs/ft should be applied to the linear beam weight. The author also cites fabrication complications result when cambering beams with lengths less than 24', web thicknesses less than or equal to ¼", and nominal depths less than 14". These limitations were taken into mind when selecting member sizes. As is typical, the live load deflection was limited to L/360 and the total load deflection was limited to L/240.

4.2) DESIGN RESULTS

The standard 4 ½" normal weight concrete on 22 gauge, 2" VLI deck was used as the flooring system because it achieves the minimum two hour fire rating without requiring spray fireproofing and spans the typical beam spacing efficiently. Hand calculations of a typical gravity bay can be found in Appendix D. The ETABS analysis indicated that a W14x22 with (9) studs and 1" of camber would be the most efficient gravity beam size. Due to member damage, discussed above, a W14 was deemed too shallow to camber, therefore these members were upsized. Figure 4.3, below, represents the most typical bay. Typical infill beams were found to be W16x31 and require (16) ¾" diameter shear studs. The typical girder was found to be W18x40 and required (17) ¾" diameter shear studs.

Infill beams were kept to a maximum of W16 in most areas to limit the overall structural floor depth. To eliminate the need for bottom flange coping, the girders were limited to a minimum of W18. As compared to the original 8" two way slab, this 24 ½" deep floor system (18" beam and 6 ½" slab on deck) was 16 ½" deeper. To allow for spray fireproofing and other size increases, each floor plenum was increased by 18". The existing 9' floor to ceiling height was kept constant. This resulted in an overall building height increase of 10'-6". This is a substantial increase and will result in increased costs due to more façade area, larger mechanical risers, longer electrical runs, and other material increases. The 90' building height limitation, which was imposed by the local city ordinance, was also violated by this alteration. Due to site plan issues, discussed in Section 10.3, the building would have to be relocated outside of an urban area; therefore this height limitation would be void.

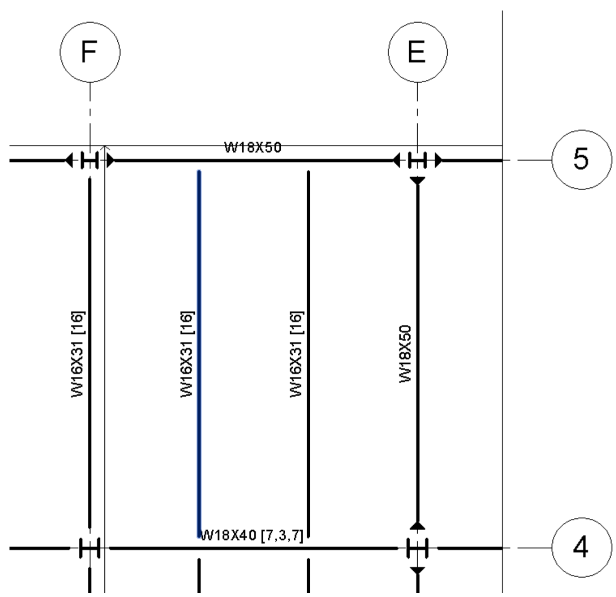


Figure 4.3 – Typical Bay Member Designs

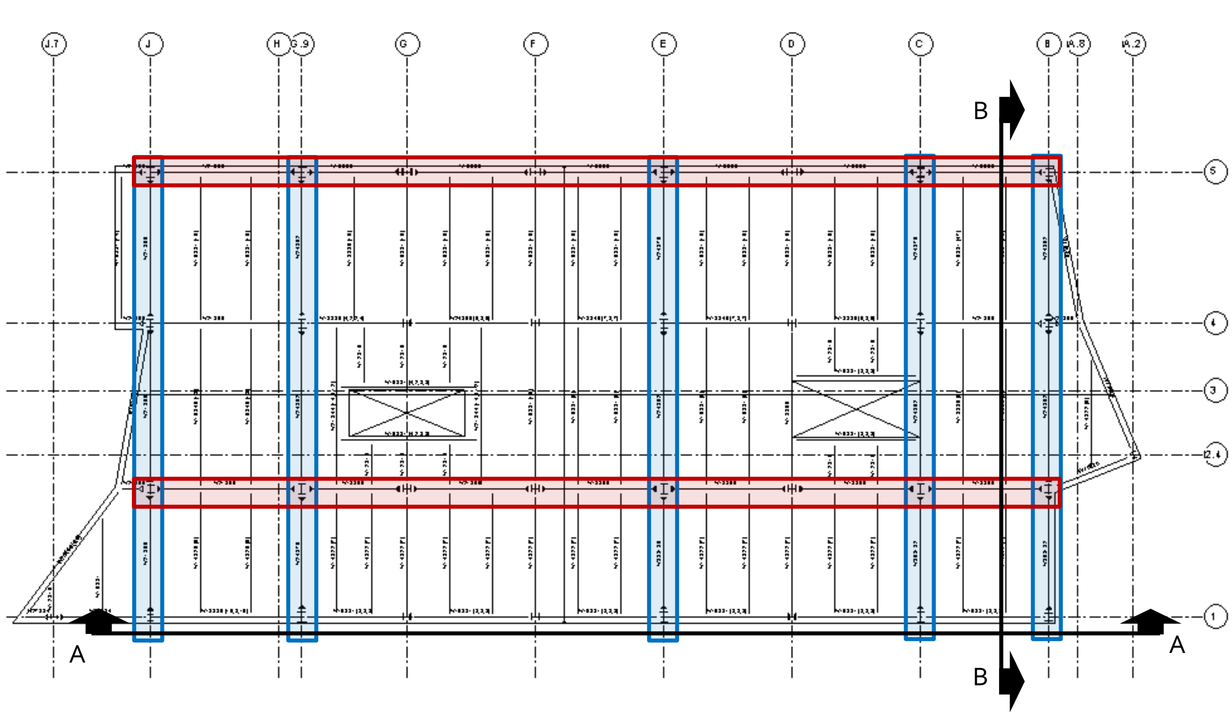
In areas where the existing slab was increased from the 8" typical depth, the infill beams were allowed a corresponding depth increase. For instance, the slab at the eastern portion of the second floor had a depth of 12", which is an increase of 4" as compared to the typical. Infill beams in this area were limited to 20" depth.

When reasonable, perimeter beams were kept as the same depth. This allowed for the same slab edge detail along the perimeter, resulting in simpler construction. Simplifying the construction details helped to prevent errors (both from contractors and designers), allowed cost savings when materials were bought in bulk, and faster construction with increased repetition.

The typical gravity columns, one of which was located at the intersection of column line F and 4, were calculated to be W14x132 at the lowest story. The columns were spliced 4 ft. above the fourth floor. This allows for 42.5' lower columns and 53' upper columns. Four story column splices also allowed for construction efficiency because OSHA required safety netting to be installed when erection crews work more than 2 stories or 30' from a decked floor. Four story columns allow two floors to be constructed at the same time without safety netting. The 53' upper column length may cause problems because the standard flatbed truck has a shipping length of 53'. To reduce the upper column length, the roof framing could be posed off of the penthouse level. This should be investigated further in the next phase of analysis.

The lateral forces in the base steel redesign were designed to be resisted entirely by moment frames. Moment frames were preferred for progressive collapse analysis because they are redundant and efficiently redistribute forces. Figure 4.4 shows the location of the moment frames. Ideally, column line 1 would have been used as a moment frame because it is on the exterior, which creates the largest moment arm for torsional resistance. Figure 4.5 illustrates

Figure 4.4 – Moment Frame Layout
North-South Frames
East- West Frame



that these columns run through the atrium.

Most of the columns are not connected by beams at the second and third stories. This made column line 1 relatively week, especially at the lower stories where the highest drifts occur in pinned moment frames. For these reasons, column line 2 was chosen as the second frame line in this direction. Similarly, column line B ran through several atriums and beam-free areas. Figure 4.6 shows the irregular and often interrupted configuration of column line B. For these reasons, a moment frame was added along column line C, which is more regular. Due to the larger forces resisted by the North-South direction frames, which were due to the larger wind surface area, another frame line was added at column line E. Column line G.9 was also selected as a frame line because it runs continuously from base to roof, whereas column line J terminates at below the fourth level.

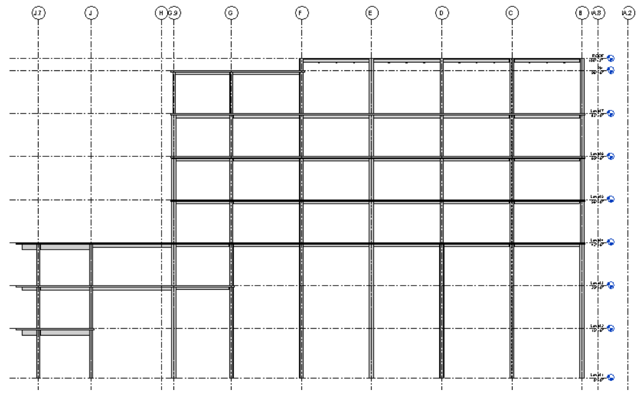
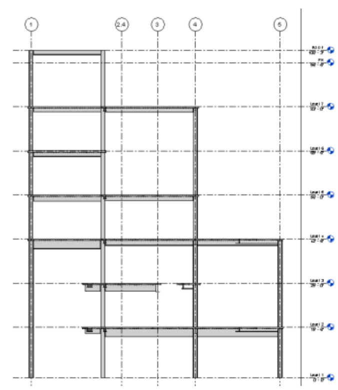


Figure 4.5 – Building Section A-A. Refer to Figure 4.4

Figure 4.6 – Building Section B-B. Refer to Figure 4.4



To better take advantage of the bending moment strength, the columns in the North-South frames (CL B,E, G.9, J) were oriented to resist the North-South forces by bending about their strong axis. Normally the columns would be oriented so their strong axis resists loads applied from girders, but in these frames the larger bending forces come from the lateral loads resisted by the frames. The shorter North-South frames must resist a larger load (due to the larger wind surface area), therefore the columns in these frames must resist larger individual moment. As expected, these columns were controlled by wind drift, which was limited to an inter-story drift of 1/400. When orienting the columns in such a manner their size was reduced from W14x283 to W14x233.

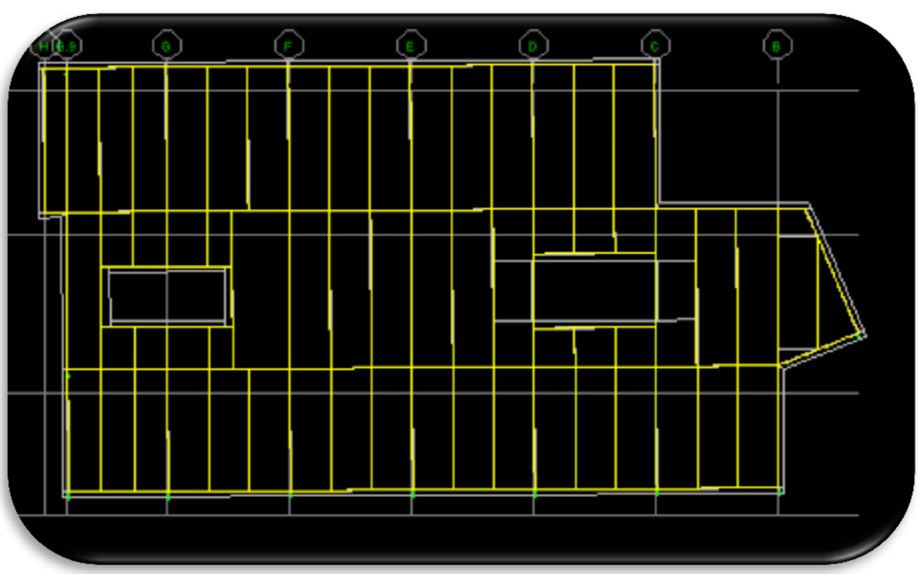
Table 4.2, below, defines the column sizes in the various areas of the building. The beams on the lower floors were found to be much larger than the rest of the building because the inter-story wind drifts at these stories were much larger, which is expected for a pin-based moment frame. ETABS increased these members because of their high energy to volume ratio. Changing the columns to fixed connected at the base should be investigated further. Due to time constraints and the progressive collapse focus of this thesis, fixing the column bases was not investigated.

Figure 4.5 is the deflected shape for the controlling wind load. The building experienced slight torsion, which is due to the variation in moment frame stiffnesses and locations. The stronger, uninterrupted, Western moment frames resisted more of the lateral load. The first story inter-story drift was calculated to be 0.464. The limit was set at 0.465, or L/400. This lateral drift was very close to the limit and was the controlling factor. This caused the lower story frame beams to be quite large. The first period was determined to be 1.93 seconds, and deflected along the length of the building. The second mode was found to be 1.45 seconds and deflected parallel to the short direction. If more time was available for further analysis, this lateral system would be investigated further. The base steel design was determined to be a success because the final structure behaved as expected and reasonable members were determined.

Table 4.2: Frame Sizes

	Typical Lower Column	Typical Upper Column	Typical 2^{nd} Floor Beam	Typical 3^{rd} Floor Beam	Typical 4^{th}+ Floor Beam
North-South Frames (C, E, & G.9)	W14x233	W14x233	W36x182	W30x108	W24x76
North-South Frames (B & J)	W14x211	W14x145	W36x182	W30x108	W24x76
East West Frames (2&5)	W14x176	W14x176	W18x50	W18x50	W18x50
Gravity Columns	W14x132	W14x82	---	---	---

Figure 4.6 – Deflected shape of the 5^{th} Floor when the direct wind load was applied to the South Facade



5.) PROGRESSIVE COLLAPSE INTRODUCTION

The Unified Facilities Criteria (UFC), written by the Department of Defense (DoD), was used as the basis for the progressive collapse redesign. This design guide is broken into several individual parts. The primary guide used was UFC 023-03, "Design of Buildings to Resist Progressive Collapse", which was originally published in 2003. The 2010 update was used for this report. The referencing, or parent, code was the UFC 010-01, "DoD Minimum Antiterrorism Standards for Buildings". The UFC 010 directs designers to the UFC 023 and the other design guides, several of which were used for the Architectural Breadth, described below.

Per UFC 010, the minimum allowed standoff distance is 18' and standoff distance for standard construction is 151'. Refer to Appendix K for the materials used to come to this conclusion. The standoff distance for this project fell between these values, which resulted in the need for direct progressive collapse analysis. Further discussion of standoff distances can be found in Section 10. As part of the proposed hypothetical redesign scenario, the new DoD occupant required an occupancy rating of IV. This was the most mission critical category and was only assigned to building which have a high priority to the department or have extremely high risk of a potential attack occurring. These categories are largely based on the IBC's occupancy categories. For instance, a hospital is falls under category IV in both. The DoD did make some additions, such as "Mission Critical Facility" that is largely defined by the occupant.

Figure 5.1, which was taken from UFC 023, outlines the required progressive collapse procedures for this occupancy category. It can be seen that Occupancy Category IV requires three types of procedure: Tie Force Method, Alternative Path Method, and Enhanced Local Resistance. The overall goal of Tie Force Analysis is to provide sufficient tensile strength in the floor so that damaged area can be spanned over and redundancy is developed in the floor. Alternative Path Analysis involved a more direct inspection of the structure. In this method, individual columns are removed and the structure must be able to redistribute the column's load without disproportionately collapsing. Enhanced Local Resistance design is only required for high priority occupancies and strives to strengthen the exterior column so that failure do not occur.

UFC 4-023-03
14 July 2009

Table 2-2. Occupancy Categories and Design Requirements

Occupancy Category	Design Requirement
I	No specific requirements
II	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. **OR** Option 2: Alternate Path for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

Figure 5.1 – Progressive Collapse Design Requirements for Each Occupancy

6.) TIE FORCE METHOD

The first step taken in progressive collapse design was the Tie-Force Method, which is outlined in section 3 of UFC 023. The overarching goal for the Tie-Force Method is to create enough tensile strength in the floor so that the floor can span over any damaged areas by using catenary action. This method is very similar to a standard British provision, which was developed after the Ronan Point collapse in 1968. Due to its simplicity and proven history, this method is required for most DoD occupancy categories.

The UFC outlined two different methods that could be used for this anlaysis. The first is to prove the structural framing (beams and girders) were capable of taking a required tensile force while undergoing large chord rotations. The second was to cast reinforcing within the floor that can resist the required tensile forces. For this thesis, the second approach was taken. A 4 ½" concrete slab on 2" VLI deck was selected as the flooring, as discussed earlier. The base steel design used welded wire fabric, but this floor system provided enough strength to place rebar within the concrete.

The load combination prescribed in the UFC 023 for this method is 1.2D + 0.5L. The tensile force needed to be carried by the slab reinforcing was determined from Equation 3-3, which is listed below in Figure 6.1. The term, L1, is determined from the largest column to column span in the floor plan. A uniform bay layout will create a more efficient system because the largest spacing will control the forces for the entire floor. For the GOB, the largest span was between column line C and D, which was 27'. Using this equation, the required tie strength was determined to be 12.1 k/ft in the East-West direction and 12.7 k/ft in the North-South direction. This resulted in #4 rebar spaced at 13 in running in the East-West direction and #4 rebar spaced at 12 in running in the North-South Direction. This reinforcing was also found to be sufficient for the minimum reinforcing required. Refer to Appendix E for all Tie-Force calculations.

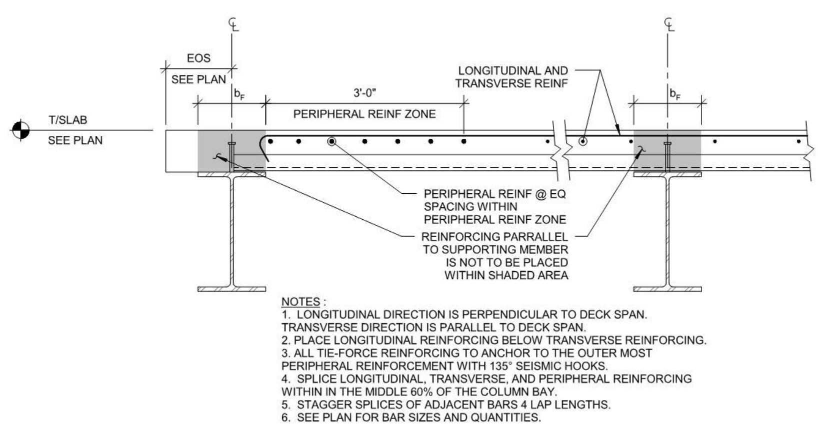
$$F_i = 3\, w_F\, L_1 \qquad \text{Equation (3-3)}$$

Where w_F = Floor load, determined per Section 3-1.2, in (lb/ft^2 or kN/m^2)

L_1 = Greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration (ft or m)

Figure 6.1 – Required Tie Strength. UFC 023

Figure 6.2 – Typical Placement of Ties



The UFC requires stronger, peripheral, ties to be located at the perimeter of the floor slab. These ties provide a location for the interior ties to anchor to and they take the larger loads experienced by the discontinuity of the edge. The peripherals were designed to have twice the capacity that the interior ties had and were distributed across the outer three feet of the floor. The UFC requires that if the supporting beams were not designed as the ties, the ties could not be placed directly above the beams, as illustrated in Figure 6.2. To strongly anchor into the peripheral ties, seismic hooks were attached to the end of all interior ties, as prescribed by the UFC. At any area where the interior ties were interrupted, such as elevator shafts and slab elevation changes, peripheral tie were required. For this reason the peripheral ties were designed to both support the exterior wall, as prescribed by the UFC, and to only support the internal area loads. As illustrated in Figure 6.3, (4) #6 rebar were required for peripheral zones next to exterior walls and (3) #6 were required for interior peripheral zones. The larger #6 rebar was specified in this area to allow for easy inspection and to emphasize the different reinforcing zones.

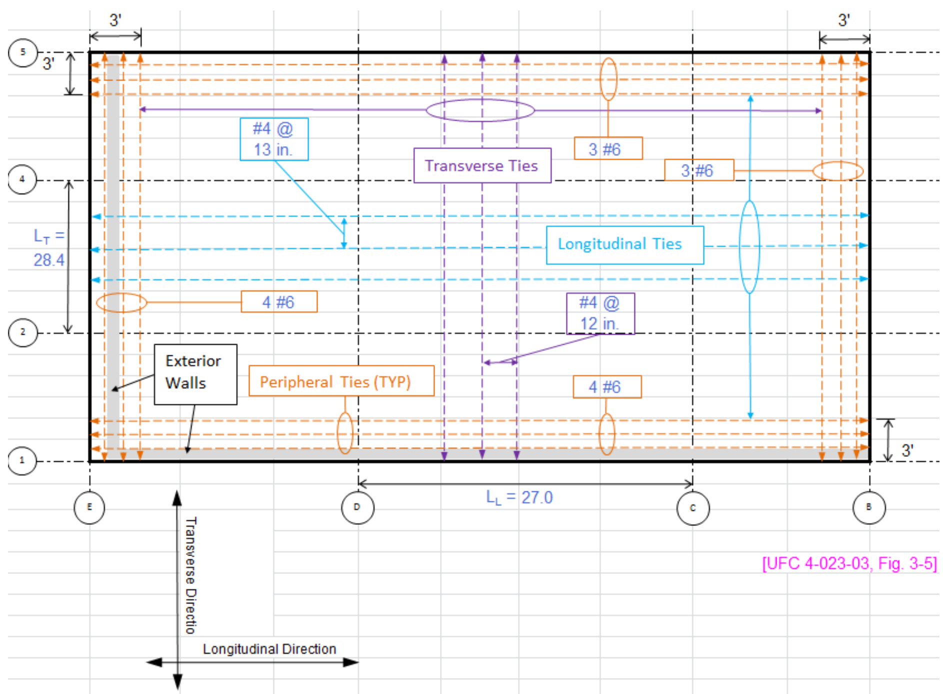


Figure 6.3 – Tie Placement and Quantity for Floors 2-7

The Tie-Force Method was required for all floors, including the roof. This required the roof to be the same 4 ½" concrete on deck as the rest of the building, instead of the normal bare roof deck. This alteration to the roof was included in all subsequent progressive collapse calculations. The full calculations and required reinforcing for all floors can be found in Appendix E.

The Tie-Force Analysis resulted in increased slab reinforcing. Typically, #4 bars, spaced at 12"-to-13" each directions were required for the interior and (4) #6 bars for the perimeter. This analysis provided the first level of protection against progressive collapse.

7.) ALTERNATIVE PATH METHOD

The second analysis method required by Occupancy Category 4 is the Alternative Path Method, which requires designing the building to continue to stand after a column is removed. This method ensures that the building has sufficient strength and redundancy to resist a disproportionate amount of damage once a single member is compromised. Three analysis procedures are described in UFC 023: linear static, nonlinear static and nonlinear dynamic. For this thesis, the linear static approach was investigated thoroughly and a simplified nonlinear static analysis was conducted.

The occupant was changed to the Department of Defense in the proposed scenario. This caused an occupancy category of IV to be assigned to the project because of the new occupants' high security work. Per section 3.2 of UFC 023, this occupancy category required the exterior column removal and investigation at the following plan locations:

1. Near the middle of the short side;
2. Near the middle of the long side;
3. At the corner of the building;
4. Locations of significant geometry changes; and
5. Locations of structural changes.

In addition, at these plan location the column was to be removed at the following elevation locations:

1. First story above grade;
2. Story directly below roof;
3. Story at mid-height; and
4. Story above the location of a column splice or change in column size.

Due to time constraints, only two columns were investigated: one typical column near the middle of the long side and one troublesome column along the East façade. If time would have permitted, the next area investigated would have been the Northwest corner column. If a complete analysis of this project were to be conducted, several other areas should be investigated, such as: the three-story atrium columns, the area that supports the connector bridge, and the connection to the auditorium.

Section B-2.1 of UFC 04-10 exempted all one and two story buildings from progressive collapse requirements. The DoD deemed that a two story collapse was within acceptable risk limits because only a relatively few amount of people would be at risk. This exemption applies to the entire auditorium structure. The expansion joint between the two wings ensures that if the auditorium is attacked, no forces will be transferred into the main wing. The 90' pedestrian bridge is also excluded because of the transient nature of the occupancy. Statistically, few people will be on the bridge during an attack, therefore only few losses will occur if the bridge collapses. It would be good engineering practice to continue some of the safety measures designed for the main wing into these other spaces. The Tie-Force requirements, for instance, would have little impact on the auditorium's cost and schedule, but would add a level of safety to the structure.

To calculate the large amount of repetitive equations, a spreadsheet was created. The design example in the commentary of the UFC 023 was used to verify and troubleshoot the spreadsheet. This spreadsheet was used in tandem with a

RISA 3D structural model. The member forces, member capacities, and joint deflections were taken from the RISA 3D output and copied into the spreadsheet, which compared these values to acceptable criteria outlined in the UFC. A brief explanation of this process is described below. The full spreadsheet is available upon request.

The initial sizes for all elements were taken from the base model, which was designed to the standard ASCE 7 live, dead, wind, and earthquake loading. This model was discussed in Section 4. The progressive collapse scenarios were analyzed and any member found to have insufficient capacity was upsized until all criteria was met. By designing in this manner, the building would only become stiffer and stronger, as compared to the base model, and would therefore still pass the standard ASCE 7 criteria. Although only a few locations were analyzed, any exterior column could be targeted; therefore any member size increase was applied to all similar members. Moment connections were added to column line 1 because these exterior columns could have been removed and analyzed, which would require moment connections to redistribute the forces.

It was assumed that the interior columns were not at risk. The basement parking structure would normally warrant the analysis of interior column removal, but this area was assumed to be secure. Section 10.3 includes a discussion of the parking garage entrance. If unauthorized vehicles could access the basement garage additional member increases would occur and several more moment connections would have been added to each floor. The UFC discourages underground parking, for this reason.

A large amount of progressive collapse methodology is based on seismic design because both events are extreme loadings that utilize plastic capacities. ASCE 41, "Seismic Rehabilitation of Existing Buildings," is directly referenced by UFC 023 in several sections. As part of the Linear Static Analysis, m-factors were calculated for each member and connection. ASCE 41 defined m-factors as "non-linear deformation capacities" and m-factors were used in linear analyses to account for non-linear behavior. Deformation and force controlled actions can be found in Figure 7.1. A larger m-factor was achieved by a member that has a high plastic to elastic strength ratio. Ductility was preferred in all elements because plastic deformations absorb large amount of energy. The applied deflection action was divided by the m-factor, and this quotient was compared to the member's elastic capacity for that particular action. These m-factors are only used when analyzing deflection controlled actions. The force controlled actions were compared directly to elastic capacities.

Component	Deformation-Controlled Action	Force- Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M[2]	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.
2. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

Figure 7.1 – Examples of Deformation-Controlled and Force Controlled Actions from ASCE 41

In addition to m-factors, overstrength factors were applied to all member capacities. Overstrength factors were used to transform the lower-bound material properties into expected strengths. In a real building nearly all material properties are higher than the properties used in design. The overtrength factor, Ω, was applied to these properties to increase the properties to the average value expected for all members. This loading scenario was considered extreme, so conservative reductions in strength were not applied. ASCE 41 was used to determine the overstrength factors. The yield strength of an A992 steel member, for example, was increased by an overstrength factor of 1.1.

When both force and deformation controlled actions were imposed on a member simultaneously, the deformation controlled force was divided by the m-factor and the force controlled action was not. The equation below illustrates a typical column interaction check. This equation originated from the AISC Specification, Section H for member with axial load greater that 20% of capacity. It can be seen that the m-factor was applied to the moment side of the equation. The result from this equation was compared to unity. Values less than 1.0 were deemed adequate. Further discussion of member acceptance criteria and m-factor calculations can be found in Sections 7.1, 7.2, and 7.3.

$$\frac{Pr}{\Omega * Pc} + \frac{\frac{8}{9}\left[\frac{Mrx}{\Omega * Mcx} + \frac{Mry}{\Omega * Mcy}\right]}{m - factor}$$

When the column was removed the beam to beam joint above the removed column was kept, per section 3.9 of UFC 023. Figure 7.11 depicts a typical deflected shape when a column was removed in the structure. As expected, large deflections occurred in the bays immediately adjacent to the removed column. Due to the continuity of the moment connections, the frame beams above the removed column acted essentially like double span beams.

Separate loading conditions were used for areas near the removed column and for areas away from the removed column, per section 3-2.12.4.1 of UFC 023. Increased loading was applied to members near the removed element to help account for the dynamic nature of the loading and increased forces from the blast pressures. Furthermore, separate loading conditions were used when investigating force controlled actions and when investigating deformation control actions. In every condition, the same lateral load of 0.002ΣP was applied. This load is .2% of the building weight and ensures lateral stability. All conditions used the same base load combination:

$$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.002\Sigma P$$

This represents four possible load combinations, but the controlling combination in most cases was found to be: 0.9D + 0.5L. These load factors were less than the typical load factors (1.2 and 1.6, respectively) because progressive collapse is an extreme loading. Using expected dead and live load helped to eliminate over conservatism.

The load cases, listed above, in floor bays not directly supported by the removed column ("far" bays) were multiplied by a factor of 1.0. Figure 7.3 and 7.4 illustrate these far loads. The load cases, listed above, in floor bays directly supported by the removed column ("near" bays) were multiplied by the load increase factor, Ω_L, corresponding to the action under consideration. Figure 7.5 and 7.6 illustrate these near loads. It can be seen that the exterior column near the middle was considered the near column for the loading depicted because the bays around that column have larger loading. When force controlled actions were investigated the Ω_{LF}, was applied. When force controlled actions were investigated the Ω_{LD}, was applied. Figure 7.2, defines this increase factor for various structural materials. It can be seen that the deformation load increase factor depended upon the m_{LIF}, or smallest m-factor in the near bays. As discussed in later sections, this was typically controlled by the moment connection's m-factor. Typically, the Ω_{LD} factor was found to be 3.4 for the final members used.

Table 3-4. Load Increase Factors for Linear Static Analysis

Material	Structure Type	Ω_{LD}, Deformation-controlled	Ω_{LF}, Force-controlled
Steel	Framed	0.9 m_{LIF} + 1.1	2.0
Reinforced Concrete	Framed[A]	1.2 m_{LIF} + 0.80	2.0
	Load-bearing Wall	2.0 m_{LIF}	2.0
Masonry	Load-bearing Wall	2.0 m_{LIF}	2.0
Wood	Load-bearing Wall	2.0 m_{LIF}	2.0
Cold-formed Steel	Load-bearing Wall	2.0 m_{LIF}	2.0

Figure 7.2 – Examples of Deformation-Controlled and Force Controlled Actions from ASCE 41

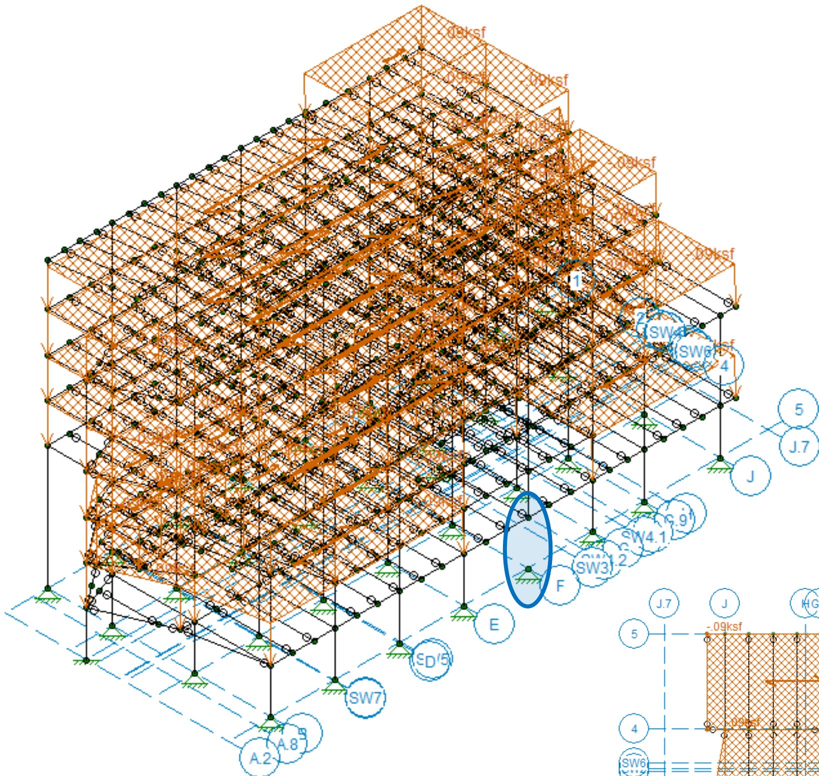
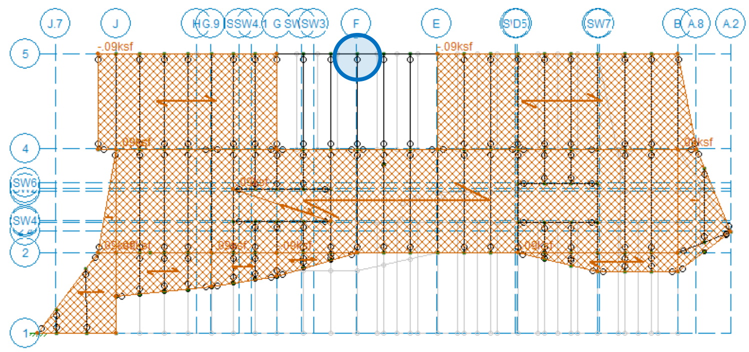


Figure 7.3 – Example of Far Loading. Column F5, which is highlighted, was removed. Isometric View

Figure 7.4 – Example of Far Loading. Column F5, which is highlighted, was removed. Plan View



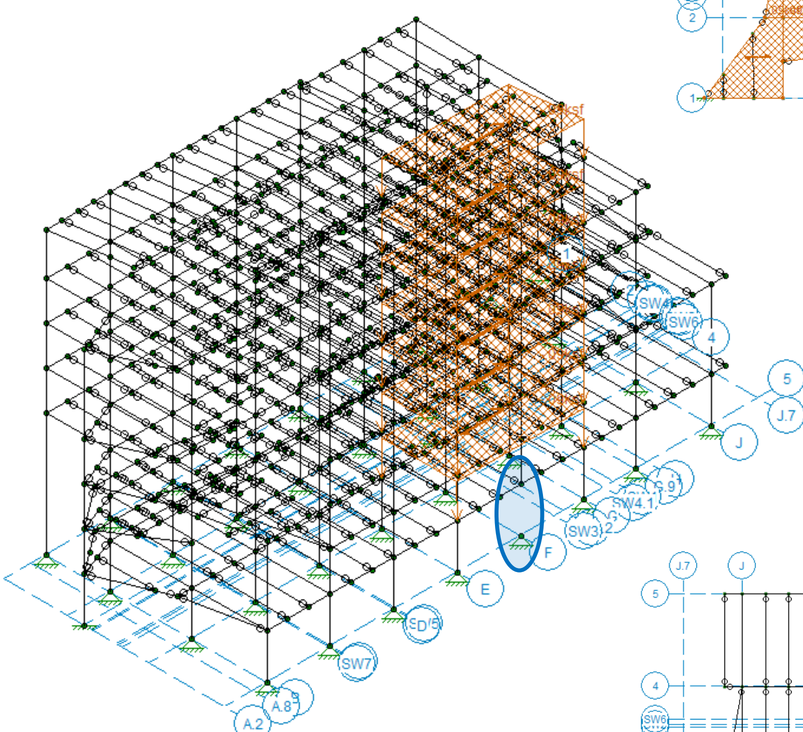


Figure 7.5 – Example of Near Loading. Column F5, which is highlighted, was removed. Isometric View

Figure 7.6 – Example of Near Loading. Column F5, which is highlighted, was removed. Plan View

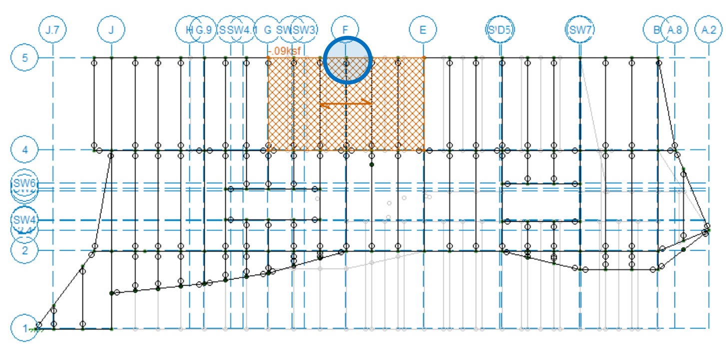


Figure 7.7 illustrates the different loadings in the progressive collapse model. These loads were broken into near and far load cases. The load combinations, shown in Figure 7.4, combined these load cases with the appropriate factor. The magnitude of the near and far loads were the same, they were only separated so that the different load factors could be applied. For instance, 100 psf distributed loads were used in both the "Far Typ Live" and Near Typ Live" load cases.

Figure 7.9 illustrates how these load cases were combined in the load combinations and what the values listed in Figure 7.8 represent. First the basic load cases were combined into categories, such as "Near Dead". This combined such things as exterior wall loads with superimposed dead load. Load factors of 1.0 were used for this step because it was only meant to group individual loads into load types. Near and far loads were still kept separate. Figure 7.9 illustrated that these load categories were then combined and the appropriate load factors were applied. This is where the load increase factors, discussed previously, came into play. The omega factor for deflection controlled actions was noted directly in the table because it varied with member sizes.

BLC Descrip...	C...	X..	Y..	Z..	J..	P..	Distri...	Area(...
Far Self Weight	DL			-1				
Far Dead Load	DL							37
Far Roof Dead	DL							2
Far Exterior Wall	DL						106	
Far Atrium Wall	DL						15	
Far Typ Live	LL							28
Far Mechanical L	LL							2
Far Snow	SL							2
	Non							
Near Self Weigh	DL			-1				
Near Dead Loac	DL							5
Near Roof Dead	DL							
Near Exterior Wa	DL						10	
Near Atrium Wal	DL						3	
Near Typ Live	LL							5
Near Mechanica	LL							
Near Snow	SL							
	Non							
	Non							
	Non							
	Non							
Notional Load X	NLX				7			
Notional Load Y	NLY				7			
	Non							

Figure 7.7 – All load cases.

Figure 7.8 – The load combinations.

FORCE CONTROLLED LC	☐																
	☐																
FC: (1.8D + 1.0L)near + (.9D + .5L)far + Lx	☐	Y		L8	1.8	L9	1			L11	.9	L12	.5			26	1
FC: (1.8D +1.0L)near + (.9D + .5L)far - Lx	☐	Y		L8	1.8	L9	1			L11	.9	L12	.5			26	-1
FC: (1.8D + 1.0L)near + (.9D + .5L)far + Ly	☐	Y		L8	1.8	L9	1			L11	.9	L12	.5			27	1
FC: (1.8D + 1.0L)near + (.9D + .5L)far - Ly	☐	Y		L8	1.8	L9	1			L11	.9	L12	.5			27	-1
FC: (2.4D + 1.0L)near + (1.2D + .5L)far + Lx	☐	Y		L8	2.4	L9	1			L11	1.2	L12	.5			26	1
FC: (2.4D + 1.0L)near + (1.2D + .5L)far - Lx	☐	Y		L8	2.4	L9	1			L11	1.2	L12	.5			26	-1
FC: (2.4D + 1.0L)near + (1.2D + .5L)far + Ly	☐	Y		L8	2.4	L9	1			L11	1.2	L12	.5			27	1
FC: (2.4D + 1.0L)near + (1.2D + .5L)far - Ly	☐	Y		L8	2.4	L9	1			L11	1.2	L12	.5			27	-1
FC: (1.8D + 0.4S)near + (.9D + .2S)far + Lx	☐	Y		L8	1.8	L10	.4			L11	.9	L13	.2			26	1
FC: (1.8D + 0.4S)near + (.9D + .2S)far - Lx	☐	Y		L8	1.8	L10	.4			L11	.9	L13	.2			26	-1
FC: (1.8D + 0.4S)near + (.9D + .2S)far + Ly	☐	Y		L8	1.8	L10	.4			L11	.9	L13	.2			27	1
FC: (1.8D + 0.4S)near + (.9D + .2S)far - Ly	☐	Y		L8	1.8	L10	.4			L11	.9	L13	.2			27	-1
FC: (2.4D + 0.4S)near + (1.2D + .2S)far + Lx	☐	Y		L8	2.4	L10	.4			L11	1.2	L13	.2			26	1
FC: (2.4D + 0.4S)near + (1.2D + .2S)far - Lx	☐	Y		L8	2.4	L10	.4			L11	1.2	L13	.2			26	-1
FC: (2.4D + 0.4S)near + (1.2D + .2S)far + Ly	☐	Y		L8	2.4	L10	.4			L11	1.2	L13	.2			27	1
FC: (2.4D + 0.4S)near + (1.2D + .2S)far - Ly	☐	Y		L8	2.4	L10	.4			L11	1.2	L13	.2			27	-1
	☐																
	☐																
DEFLECTION CONTROLED LC	☐																
	☐																
DC: Omega(.9D + .5L)near + (.9D + .5L)far + Lx	☑	Y		L8	3.06	L9	1.7			L11	.9	L12	.5			26	1
DC: Omega(.9D + .5L)near + (.9D + .5L)far - Lx	☑	Y		L8	3.06	L9	1.7			L11	.9	L12	.5			26	-1
DC: Omega(.9D + .5L)near + (.9D + .5L)far + Ly	☑	Y		L8	3.06	L9	1.7			L11	.9	L12	.5			27	1
DC: Omega(.9D + .5L)near + (.9D + .5L)far - Ly	☑	Y		L8	3.06	L9	1.7			L11	.9	L12	.5			27	-1
DC: Omega(1.2D + .5L)near + (1.2D + .5L)far + Lx	☑	Y		L8	4.08	L9	1.7			L11	1.2	L12	.5			26	1
DC: Omega(1.2D + .5L)near + (1.2D + .5L)far - Lx	☑	Y		L8	4.08	L9	1.7			L11	1.2	L12	.5			26	-1
DC: Omega(1.2D + .5L)near + (1.2D + .5L)far + Ly	☑	Y		L8	4.08	L9	1.7			L11	1.2	L12	.5			27	1
DC: Omega(1.2D + .5L)near + (1.2D + .5L)far - Ly	☑	Y		L8	4.08	L9	1.7			L11	1.2	L12	.5			27	-1
DC: Omega(.9D + .2S)near + (.9D + .2S)far + Lx	☑	Y		L8	3.06	L10	1.7			L11	.9	L13	.2			26	1
DC: Omega(.9D + .2S)near + (.9D + .2S)far - Lx	☑	Y		L8	3.06	L10	.68			L11	.9	L13	.2			26	-1
DC: Omega(.9D + .2S)near + (.9D + .2S)far + Ly	☑	Y		L8	3.06	L10	.68			L11	.9	L13	.2			27	1
DC: Omega(.9D + .2S)near + (.9D + .2S)far - Ly	☑	Y		L8	3.06	L10	.68			L11	.9	L13	.2			27	-1
DC: Omega(1.2D + .2S)near + (1.2D + .2S)far + Lx	☑	Y		L8	4.08	L10	.68			L11	1.2	L13	.2			26	1
DC: Omega(1.2D + .2S)near + (1.2D + .2S)far - Lx	☑	Y		L8	4.08	L10	.68			L11	1.2	L13	.2			26	-1
DC: Omega(1.2D + .2S)near + (1.2D + .2S)far + Ly	☑	Y		L8	4.08	L10	.68			L11	1.2	L13	.2			27	1
DC: Omega(1.2D + .2S)near + (1.2D + .2S)far - Ly	☑	Y		L8	4.08	L10	.68			L11	1.2	L13	.2			27	-1
Omega = 3.4	☐																

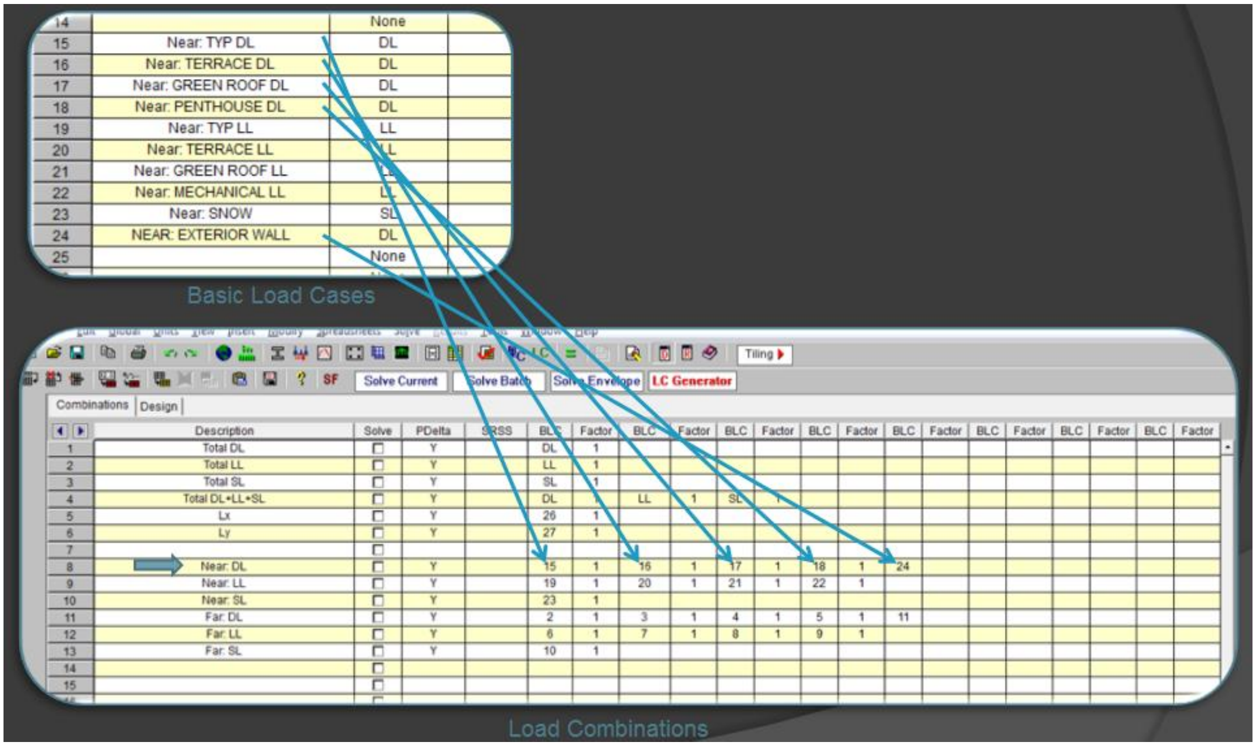


Figure 7.9 – Combination of Load Categories into Load Combinations

These load combinations were created in the RISA model and can be seen in Figure 7.8. The "Force Controlled Cases" were run when that action was investigated and the "Deformation Controlled Cases" were run when that action was investigated. The two categories were never run simultaneously. From this load case envelope the largest forces were determined automatically. The user was required to copy the correct force output into the spreadsheet. Once the correct forces were determined and recorded for one category, the other cases were run. Axial and shear forces were taken from the force controlled combinations. Moments and deflections were taken from the deflection controlled combinations. With the correct forces in hand, the member interaction and acceptance was determined.

The first area investigated was the removal of column F5 at the bottom story (between floors 1 and 2), as illustrated in Figure 7.10 and 7.11. This column was chosen because it is a typical column along column line 5. It also does not have frames connected to it in the North-South direction, only in the East-West direction. Having only two rigidly connected beams, instead of three, makes column F5 weaker than column E5, for instance, because the North-South frame will help to redistribute the forces and support the removed column.

7.1) PRIMARY ELEMENTS

All members were designated primary or secondary based on the definition given in UFC 023. Primary elements were defined in UFC 023 as "elements and components that provide the capacity of the structure to resist collapse due to removal of a vertical load-bearing element". For this system, of moment frames and beams, the columns, moment-connected beams, and pin-connected girders were designated as primary elements. Girders were included in this category because if they failed the infill beams, which connect to them, would lose their support and thereby fail. This would result in the failure of an entire bay or more, as opposed to the relatively small area of failure that occurs with an infill beam's failure.

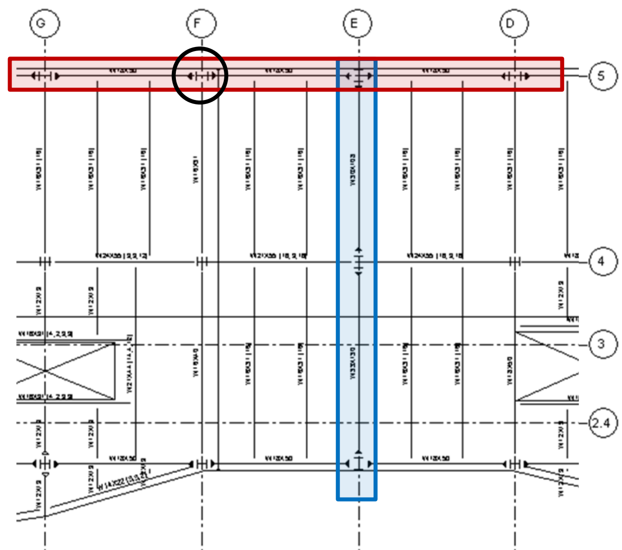
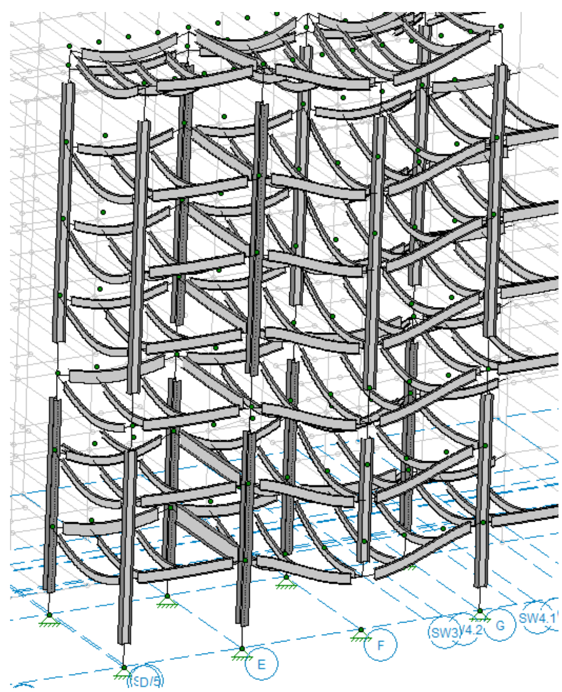


Figure 7.10 – Moment Frame Layout
North-South Frames
East- West Frame
Removed Column

Removing Column F5 required only the elements around the removed column to be upsized. The controlling m factor, m_{lif}, was for the W18x50 moment connection and was 2.81. This resulted in a deflection controlled load increase factor, Ω_{LD}, of 3.63. As expected the controlling member were the frames immediately above and connected to the removed column. The analysis output is illustrated in Figure 7.12 below. Refer to Figure 7.14 for the location of the aforementioned members. The applied moment, Muz, of these two moment frame beams exceeded their capacity, ΦMnz, by over 10. For these members, ASCE 41 Table 5-5 specifics a m-factor of 8. In other words, the applied deflection controlled forces, moment, may exceed the member's capacity by a factor of 8. When the moment capacity is divided by the m-factor the resulting interaction values is 1.15, as illustrated in Figure 7.12. This value still exceeds unity; therefore this member had to be upsized. After a few iterations, a final member size of W21x68 was found to pass, as depicted in Figure 7.14. Changing the member sizes also changed the associated m-factor and m_{lif}. This consequently changed the loading factor, Ω_{LD}, to 3.4. The change in loading was taken into account during each iteration. The final, acceptable, interaction value for the frames was found to be 0.92 and can be seen in Figure 7.14. These results were assumed to be similar to the results expected from removing any other exterior column. Consequently, all exterior frames were upsized to W21x68.

Figure 7.11 --Deflected Shape When Column F5 was removed at the 1^{st} story. For Clarity, only adjacent members are shown



REMOVAL OF COLUMN F5 at 1st Story - PRIMARY ELEMENTS										
Element Name	Next to Removed	Size	Connection Type	Connection m-factor	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	$M_{uy}/\Phi M_{CLy}$	Element m-factor (7)	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
FRAMES										
F2,G.9-J5	N	W21X62	WUF	2.56	0.00	0.66	0.00	8.00	0.08	0.19
F2,G5-G.95	N	W18X50	WUF	2.81	0.00	0.45	0.00	8.00	0.05	0.17
F2,F5-G5	Y	W18X50	WUF	2.81	0.00	10.11	0.00	8.00	1.15	0.69
F2,E5-F5	Y	W18X50	WUF	2.81	0.00	10.23	0.00	8.00	1.16	0.69
F2,D5-E5	N	W18X50	WUF	2.81	0.00	0.65	0.00	8.00	0.07	0.19
F2,C5-D5	N	W18X50	WUF	2.81	0.00	0.42	0.00	8.00	0.05	0.17
F2,B5-C5	N	W18X50	WUF	2.81	0.00	0.49	0.00	8.00	0.06	0.19

Figure 7.12 – Initial frame results when column F5 was removed at the lower story

Figure 7.13— Final frame sizes that were deemed adequate for the removal of column F5 and the first story.

REMOVAL OF COLUMN F5 at 1st Story - PRIMARY ELEMENTS										
Element Name	Next to Removed	Size	Connection Type	Connection m-factor	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	$M_{uy}/\Phi M_{CLy}$	Element m-factor (7)	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
FRAMES										
F2,G.9-J5	N	W21X68	WUF	2.55	0.00	0.58	0.00	8.00	0.07	0.18
F2,G5-G.95	N	W21X68	WUF	2.55	0.00	0.42	0.00	8.00	0.05	0.13
F2,F5-G5	Y	W21X68	WUF	2.55	0.00	3.24	0.00	8.00	0.89	0.52
F2,E5-F5	Y	W21X68	WUF	2.55	0.00	3.14	0.00	8.00	0.92	0.52
F2,D5-E5	N	W21X68	WUF	2.55	0.00	0.61	0.00	8.00	0.07	0.15
F2,C5-D5	N	W21X68	WUF	2.55	0.00	0.30	0.00	8.00	0.03	0.12

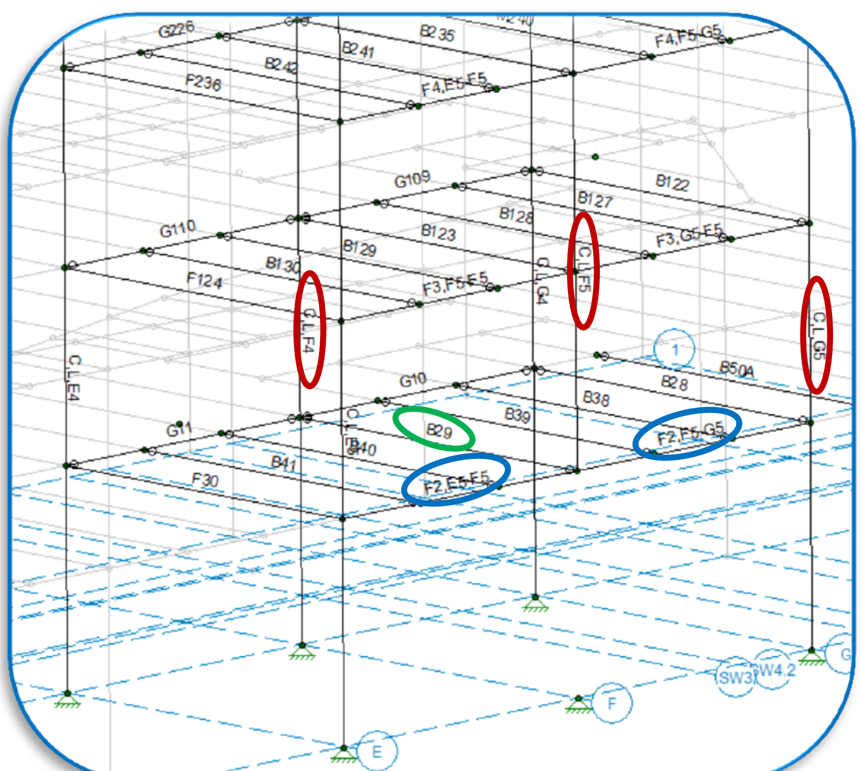


Figure 7.14 – Member Labels Around Column F5
Columns Discussed
Primary Beams Discussed
Secondary Beams Discussed

Also, as expected the controlling columns were located around the removed column. Columns G5, illustrated in Figure 7.14 and 7.15, had the highest interaction check value. Several gravity columns along column line 4 also fail because of the increased loading around the removed column. These results were expected because these are the columns that receive the highest percentage of column F5's load. These surrounding columns received both increased axial and bending load due to the large end moments resulting from the beams. Column C,U,G5 (column located at the intersection of grids G and 5 and located above the splice at the fourth story) received substantial moment because of its location and relatively small size. Several of the columns were axially loaded over 50% of their capacity. This places the columns into completely force controlled elements, therefore the m-factor no longer applied, per section 5.5 of UFC 023. The UFC code required all axially heavily loaded columns to be check using elastic capacities. Without the m-factor reduction on the moment capacity check, these members greatly exceeded their allowable limit.

REMOVAL OF COLUMN F5 at 1st Story - PRIMARY ELEMENTS

Element Name	Next to Removed	Size	Connection Type	Connection m-factor	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	$M_{uy}/\Phi M_{CLy}$	Element m-factor (7)	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
C,L,E5	Y	W14X500	Fixed Base	---	0.66	0.03	0.32	Col Mom Force Cont (8)	0.97	0.02
C,L,F5	Y	W14X193	Fixed Base	---	0.04	0.02	0.00	6.00	0.02	0.01
C,L,G5	Y	W14X193	Fixed Base	---	1.54	0.55	0.00	Col Mom Force Cont (8)	2.04	0.19
C,L,G.95	N	W14X500	Fixed Base	---	0.25	0.01	0.03	5.58	0.26	0.00
C,L,J5	N	W14X211	Fixed Base	---	0.24	0.06	0.12	5.73	0.27	0.01
C,U,C5	N	W14X500	Fixed Base	---	0.29	0.03	0.05	5.14	0.30	0.01
C,U,D5	N	W14X120	Fixed Base	---	1.49	0.03	0.00	Col Mom Force Cont (8)	1.52	0.01
C,U,E5	Y	W14X500	Fixed Base	---	0.72	0.03	0.40	Col Mom Force Cont (8)	1.11	0.02
C,U,F5	Y	W14X120	Fixed Base	---	0.92	0.09	0.00	Col Mom Force Cont (8)	1.00	0.02
C,U,G5	Y	W14X120	Fixed Base	---	3.11	0.99	0.00	Col Mom Force Cont (8)	3.99	0.22

Figure 7.15 – Initial column results when column F5 was removed at the lower story

Figure 7.16— Final column sizes that were deemed adequate for the removal of column F5 and the first story.

REMOVAL OF COLUMN F5 at 1st Story - PRIMARY ELEMENTS

Element Name	Next to Removed	Size	Connection Type	Connection m-factor	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	$M_{uy}/\Phi M_{CLy}$	Element m-factor (7)	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
C,L,G4	Y	W14X370	Pinned Base	---	0.69	0.01	0.00	Col Mom Force Cont (8)	0.70	0.00
C,L,F4	Y	W14X370	Pinned Base	---	0.89	0.00	0.01	Col Mom Force Cont (8)	0.90	0.00
C,L,E4	Y	W14X500	Fixed Base	---	0.66	0.01	0.00	Col Mom Force Cont (8)	0.67	0.00
C,L,D4	N	W14X370	Pinned Base	---	0.73	0.00	0.01	Col Mom Force Cont (8)	0.74	0.00
C,L,C4	N	W14X500	Fixed Base	---	0.48	0.02	0.04	2.50	0.50	0.01
C,L,B4	N	W14X370	Fixed Base	---	0.40	0.01	0.07	3.58	0.42	0.00
C,U,G.94	N	W14X500	Fixed Base	---	0.37	0.03	0.07	3.96	0.39	0.01
C,U,G4	Y	W14X370	Fixed Base	---	0.71	0.03	0.00	Col Mom Force Cont (8)	0.74	0.02
C,U,F4	Y	W14X370	Fixed Base	---	0.66	0.00	0.00	Col Mom Force Cont (8)	0.67	0.01
C,U,E4	Y	W14X500	Fixed Base	---	0.66	0.02	0.00	Col Mom Force Cont (8)	0.68	0.01
C,U,D4	N	W14X370	Fixed Base	---	0.75	0.00	0.00	Col Mom Force Cont (8)	0.76	0.01
C,U,C4	N	W14X500	Fixed Base	---	0.45	0.03	0.04	2.92	0.47	0.02
C,U,B4	N	W14X370	Fixed Base	---	0.06	0.04	0.09	6.00	0.05	0.01
C,L,B5	N	W14X370	Fixed Base	---	0.12	0.04	0.08	6.00	0.08	0.01
C,L,C5	N	W14X500	Fixed Base	---	0.30	0.02	0.03	4.99	0.31	0.01
C,L,D5	N	W14X370	Fixed Base	---	0.44	0.01	0.00	3.13	0.44	0.00
C,L,E5	Y	W14X500	Fixed Base	---	0.52	0.03	0.41	1.98	0.70	0.03
C,L,F5	Y	W14X370	Fixed Base	---	0.01	0.01	0.00	6.00	0.01	0.00
C,L,G5	Y	W14X370	Fixed Base	---	0.70	0.32	0.00	Col Mom Force Cont (8)	0.99	0.10
C,L,G.95	N	W14X500	Fixed Base	---	0.26	0.01	0.03	5.52	0.26	0.00
C,L,J5	N	W14X370	Fixed Base	---	0.15	0.04	0.08	6.00	0.09	0.01

All columns that did not pass the interaction pass were upsized. Several considerations were taken into mind when upsizing. Columns that were force controlled, above 50% axial capacity, were upsized using the RISA's suggested shapes, because these elements were checked using elastic capacities. Typical sections were used to simplify the process and because removing different columns would make other member critical. The results of removing column F5 were used to predict the results of removing several other columns. Table 7.1, below, summarizes the results of this analysis. The same column size was used for all columns for simplicity and uniformity because any column could be removed. Very heavy columns were required for the elements to pass UFC criteria. The typical exterior column was found to be W14x370.

Table 7.1: Existing Column Results

Location	Size	Pu/ΦPn	Mu/ΦMn	m-factor	Interaction
Exterior Moment Frame	W14x193	1.54	0.55	Force Controlled: 1.0	2.03
Interior Gravity Column	W14x120	2.10	0.00	Force Controlled: 1.0	2.10

Column F5 was also removed at the 4th story and investigated because this is where the columns were spliced. The column splice was a critical area because this is the location that will cause the greatest axial force in the smaller columns above the splice. The same loading described above and seen in Figures 7.3 - 7.6 was used for this investigation because the same column was investigated. Figure 7.17 illustrates the deflected shape of the exterior frame when this column was removed. As expected, the upper column at grid intersection G5 was the controlling element. Since the sizes from the previous progressive collapse scenario were used when analyzing the removal of this column, no members failed acceptable criteria.

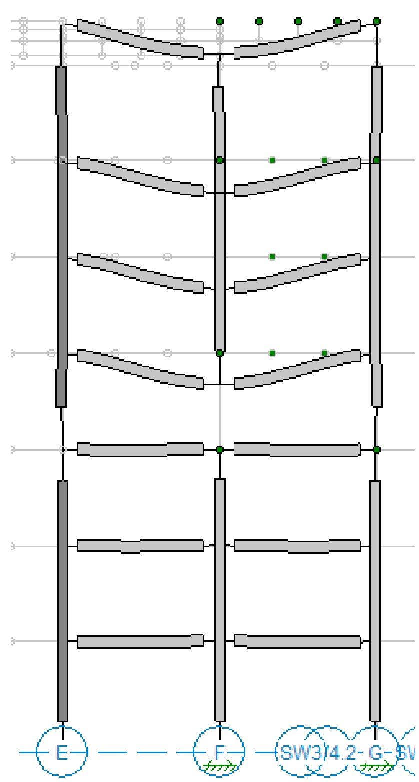


Figure 7.17 – Deflected shape of a portion of the frame that ran along column line when column F5 was removed at the 4th story.

Column F5 was also removed just below the roof level, at the 7th story. This location was selected for investigation because this area does not have a column above to act as a hanger and redistribute some force. The roof beams have to span across the missing column completely on their own. Figure 7.19 shows the roof beam that connects to the removed column.

The roof beams, which run North-South, were investigated for this column removal. Due to the large deflections at this level, several of the roof beams did not pass. Figure 7.21 shows a few of these members that were located around the removed column deflected substantially. Member B763 connected directly into the top of the removed column, and therefore saw the most displacement. Figure 7.21 shows the sloped roof. This sloped layout caused the pinned beams to receive axial forces because they want to rotate and push out on the East-West frame. Due to this axial force these beams fell into the column member category (Pu/Pn>0.1), which in turn caused the moment to become force controlled and forfeit the m-factor reduction.

These initial results can be seen in Figure 7.18 and the final results can be found in figure 7.20. These roof beams were upsized from W16x26 to W16x40 after a few iterations. Beam weight was increased over depth because the m-factor is inversely proportional to the beam depth and the m-factor is directly proportional to the beam's moment capacity.

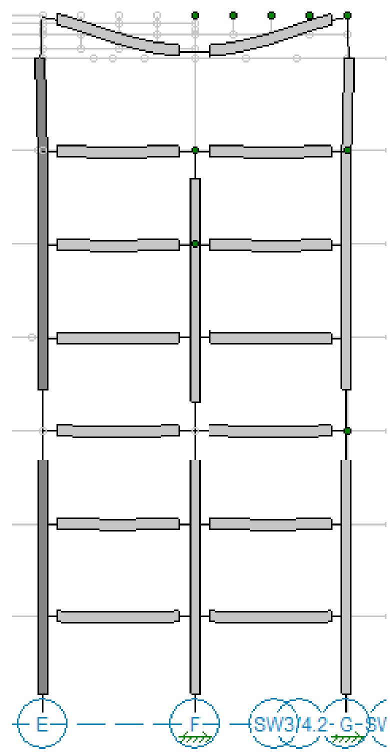


Figure 7.19 – Deflected shape of a portion of the frame that ran along column line when column F5 was removed at the 7th story.

REMOVAL OF COLUMN F5 at 7th Story - SECONDARY ELEMENTS								
Element Name	Size	Conn Type	Conn m-factor	Element m-factor (7)	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
B741	W16X31	shear Tab	7.41	12.00	0.00	2.35	0.18	0.20
B757	W14X26	shear Tab	7.41	12.00	0.00	1.52	0.12	0.05
B763	W16X26	shear Tab	7.41	mn Member	0.30	2.70	2.70	0.09
B765	W16X26	shear Tab	7.41	7.49	0.03	1.26	0.17	0.05
B769	W16X26	shear Tab	7.41	7.49	0.10	2.32	0.33	0.08
B770	W16X26	shear Tab	7.41	7.49	0.07	2.70	0.36	0.09
B771	W16X26	shear Tab	7.41	7.49	0.08	2.32	0.32	0.08
B772	W16X26	shear Tab	7.41	7.49	0.07	2.32	0.32	0.08
B773	W16X26	shear Tab	7.41	7.49	0.09	2.70	0.37	0.09
B774	W16X26	shear Tab	7.41	mn Member	0.12	2.32	2.37	0.08
B775	W16X26	shear Tab	7.41	7.49	0.08	1.13	0.18	0.05
B776	W16X26	shear Tab	7.41	7.49	0.10	1.26	0.20	0.05
B777	W16X26	shear Tab	7.41	7.49	0.08	1.13	0.17	0.05
B778	W16X26	shear Tab	7.41	7.49	0.07	1.13	0.17	0.05
B779	W16X26	shear Tab	7.41	7.49	0.09	1.26	0.20	0.05
B780	W16X26	shear Tab	7.41	7.49	0.07	1.13	0.17	0.05
B781	W16X26	shear Tab	7.41	7.49	0.08	1.90	0.27	0.05

Figure 7.18 – Removal of Column F5 at the 7th story: Initial frame results.

Figure 7.20— Removal of Column F5 at the 7th story: Passing frame results.

REMOVAL OF COLUMN F5 at 7th Story - SECONDARY ELEMENTS								
Element Name	Size	Conn Type	Conn m-factor	Element m-factor (7)	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
B740	W16X31	shear Tab	7.41	12.00	0.00	0.64	0.05	0.20
B741	W16X31	shear Tab	7.41	12.00	0.00	0.64	0.05	0.20
B757	W16X40	shear Tab	7.09	12.00	0.01	0.20	0.02	0.04
B763	W16X40	shear Tab	7.09	mn Member	0.36	0.32	0.65	0.07
B765	W16X40	shear Tab	7.09	12.00	0.04	0.17	0.03	0.04
B769	W16X40	shear Tab	7.09	mn Member	0.11	0.28	0.34	0.06
B770	W16X40	shear Tab	7.09	12.00	0.09	0.32	0.07	0.07
B771	W16X40	shear Tab	7.09	12.00	0.09	0.28	0.07	0.07
B772	W16X40	shear Tab	7.09	12.00	0.08	0.28	0.06	0.07
B773	W16X40	shear Tab	7.09	mn Member	0.10	0.32	0.37	0.07
B774	W16X40	shear Tab	7.09	mn Member	0.14	0.28	0.35	0.06
B775	W16X40	shear Tab	7.09	12.00	0.09	0.16	0.06	0.04
B776	W16X40	shear Tab	7.09	mn Member	0.11	0.17	0.22	0.04
B777	W16X40	shear Tab	7.09	12.00	0.08	0.16	0.05	0.04
B778	W16X40	shear Tab	7.09	12.00	0.08	0.16	0.05	0.04
B779	W16X40	shear Tab	7.09	12.00	0.10	0.17	0.06	0.04
B780	W16X40	shear Tab	7.09	12.00	0.08	0.16	0.05	0.04
B781	W16X40	shear Tab	7.09	12.00	0.09	0.21	0.06	0.05

Figure 7.21—Isometric view of Figure 7.18

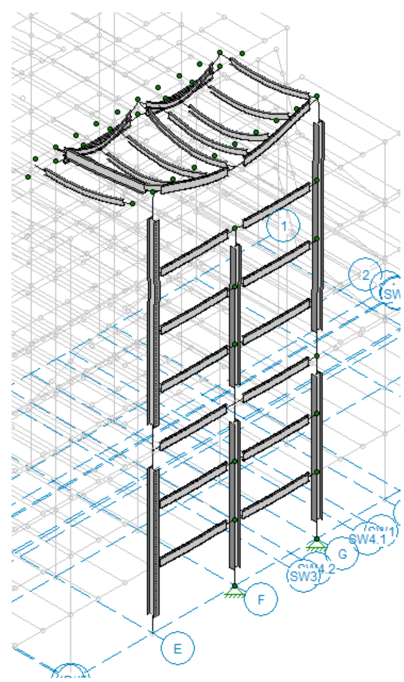


Table 7.2 summarizes the frame sizes that were found to pass this analysis. The majority of the beams were not changed from the base steel design because they were located on the interior of the building. The most noticeable increase occurred in the exterior column sizes, and this increase was due to both the increase axial and moment force applied when column F5 was removed at the first story.

Table 7.2: Frame Sizes That Pass Removal of Column F5 Scenario					
	Typical Lower Column	Typical Upper Column	Typical 2nd Floor Beam	Typical 3rd Floor Beam	Typical 4th+ Floor Beam
North-South Frames (C, E, & G.9)	W14x500	W14x500	W36x182	W30x108	W24x76
North-South Frames (B & J)	W14x370	W14x370	W36x182	W30x108	W24x76
East West Frames (1&5)	W14x370	W14x370	W21x68	W21x68	W21x68
East West Frames (2)	W14x370	W14x370	W18x50	W18x50	W18x50
Gravity Columns	W14x132	W14x82	---	---	---

7.2) SECONDARY ELEMENTS

For each column removal, the secondary beams were also analyzed. As stated earlier, only the infill beams were defined as secondary members. All secondary members passed the UFC criteria for each area investigated. Due to their place in the load path, if a secondary beam fails only the decking directly above the member (a relatively small area) would be adversely effected. As a result, secondary member have larger m-factors than primary members and therefore more capacity. The typical secondary member m-factor was 12.

Each secondary element was analyzed using the same method as the primary elements, including loading and acceptance. Just like primary elements, the moment was defined as deflection controlled, and thereby divided by the m-factor, and the axial was defined as force controlled. When column F5 was removed at the 1st story the critical member was, as expected, beam B29. As illustrated in Figure 7.14, this member spans between column F4 and the removed F5 at the second level. Figure 7.21 illustrates that the increased loading around the removed column caused this beam to have applied mid-span moment that was 2.3 times its capacity. When the plastic strength was taken into account, via the m-factor, this member passed UFC criteria. When the 4th story column was removed similar, acceptable results were achieved.

Figure 7.21— Secondary member analysis for the removal of column F5 and the first story.

REMOVAL OF COLUMN F5 at 1st Story - SECONDARY ELEMENTS								
Element Name	Size	Conn Type	Conn m-factor	Element m-factor (7)	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
B28	W16X31	Shear Tab	7.41	12.00	0.00	1.50	0.11	0.31
B29	W16X31	Shear Tab	7.41	12.00	0.00	2.31	0.17	0.44
B31	W16X31	Shear Tab	7.41	12.00	0.00	0.83	0.06	0.22
B33	W16X31	Shear Tab	7.41	12.00	0.00	0.53	0.04	0.14
B38	W16X31	Shear Tab	7.41	12.00	0.00	2.03	0.15	0.40
B39	W16X31	Shear Tab	7.41	12.00	0.00	2.03	0.15	0.41
B40	W16X31	Shear Tab	7.41	12.00	0.00	2.03	0.15	0.41

The third column removal location was just below the roof level. Due to the large deflections at this level, several of the roof beams did not pass. This was expected because there was no column above that was able to act as a hanger and give some support to the beams. Essentially, these roof beams were forced to span a distance twice what they were originally designed for. Member B763 connected directly into the top of the removed column, and therefore saw the most displacement. Figure 7.21 shows the sloped roof. This sloped layout caused the pinned beams to receive axial forces because they want to rotate and push out on the East-West frame. ASCE 41 defines a member with axial load to capacity ratio greater than 10% as completely force controlled. Due to this axial force these beams fell into this category (Pu/Pn>0.1), which in turn caused them to forfeit the m-factor reduction. Figure 7.22 illustrates that both beams B763 and B774 failed because of this axial load.

REMOVAL OF COLUMN F5 at 7th Story - SECONDARY ELEMENTS								
Element Name	Size	Conn Type	Conn m-factor	Element m-factor (7)	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
B741	W16X31	shear Tab	7.41	12.00	0.00	2.35	0.18	0.20
B757	W14X26	shear Tab	7.41	12.00	0.00	1.52	0.12	0.05
B763	W16X26	shear Tab	7.41	mn Member	0.30	2.70	2.70	0.09
B765	W16X26	shear Tab	7.41	7.49	0.03	1.26	0.17	0.05
B769	W16X26	shear Tab	7.41	7.49	0.10	2.32	0.33	0.08
B770	W16X26	shear Tab	7.41	7.49	0.07	2.70	0.36	0.09
B771	W16X26	shear Tab	7.41	7.49	0.08	2.32	0.32	0.08
B772	W16X26	shear Tab	7.41	7.49	0.07	2.32	0.32	0.08
B773	W16X26	shear Tab	7.41	7.49	0.09	2.70	0.37	0.09
B774	W16X26	shear Tab	7.41	mn Member	0.12	2.32	2.37	0.08
B775	W16X26	shear Tab	7.41	7.49	0.08	1.13	0.18	0.05

Figure 7.22— Secondary member analysis for the removal of column F5 and the seventh story.

The roof beams were upsized from their original W16x26 to w16x31 and finally w16x40. Although the original interaction was far above acceptability, 2.7 times, the members only had to be upsized a relatively small amount because of the 10% axial cutoff. Once the beams had sufficient axial capacity, the moment portion of the interaction equation decreased drastically due to the m-factor.

7.3) CONNECTIONS- MAE MATERIAL

From the progressive collapse analysis described above the end moments and shears were determined. The removal of column F5 at the first story was found to produce the largest forces. These end moments were used to determine the minimum connection moment capacity. A standard flange welded, web bolted moment connection was used. The UFC and ASCE 41 both refer to this connection as a WUF. UFC 023 Table 5-1 prescribed its own m-factors and these super succeeded the m-factors proscribed in ASCE 41. Table 5-1 can be found in Appendix F. The connection m-factors are dependent on the beam depth; therefore the connections were only designed once the final frame sizes were determined. The connection designed was at column F5 and was between a W21x68 beam and a W14x370 column. Figure 7.24 shows the beam framing into the removed column was the controlling connection because this location produced the largest end moments. The listed ΦMclz is the minimum connection moment capacity. Similar to the analysis described above, the applied moment to moment capacity ratio is compared to the connection m-factor. Through iterations, the minimum WUF moment capacity was determined to be 465 k-ft.

To design the moment connection, skills developed in master's classes were heavily relied upon. AE 534, Steel Connections, was used as the basis for this analysis. First, the beam side limit states were investigated. The progressive collapse spreadsheet verified that the beam had sufficient capacity to resist the applied end moments. A 1" fillet weld was required to connect the beam flange to the column flange. This was deemed too large for constructability, therefor full penetration welds were used instead, which is common practice. Next, the column side limit states were investigated, which included local flange bending, local web yielding, local web crippling, web buckling, and panel zone shear. Local web yielding was found to control, but still fall far within acceptable criteria. The extremely large column size also provided sufficient panel zone shear capacity; therefore no stiffeners or doubler plates were required. Standard shear tab connections were used to transfer the shear into the column. AISC 360's Table 10-9 was conservatively used for

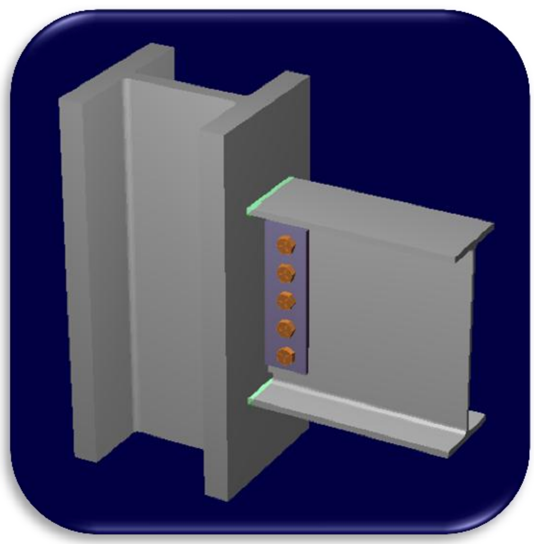


Figure 7.23 –Isometric of Moment Connection Developed in RAM Connection

Figure 7.24 – Spreadsheet connection output. Controlling connections are highlighted

REMOVAL OF COLUMN F5 at 1st Story - BEAM TO COLUMN CONNECTION CHECK																
Beam							Connection									
Element Name	Size	Primary or Secondary	Beam Depth, d	Beam Length (ft)	$\delta_{y,I}$ (in)	$\delta_{y,J}$ (in)	Connection Type (1)	Bolt Group Depth, d_{bg}	$Vu/\Phi V_{CL} \le 1.0$	Max End M_{uz}	Fixed Conn. ΦM_{CLz} (16)	Max End $M_{uz}/\Phi M_{CEz}$	m-Factor (2)	Pinned Conn. $M_u/\Phi M_{CE}$ (18)	Connection m-factor-1 (9)
F2,G.9-J5	W21X68	Primary	21.13	26.00	-0.014	-0.028	WUF	N/A	0.00	165	465	0.32	2.55	N/A	N/A
F2,G5-G.95	W21X68	Primary	21.13	18.00	-0.028	-0.112	WUF	N/A	0.00	167	465	0.33	2.55	N/A	N/A
F2,F5-G5	W21X68	Primary	21.13	22.00	-0.112	-4.87	WUF	N/A	0.00	1,295	465	2.53	2.55	N/A	N/A
F2,E5-F5	W21X68	Primary	21.13	22.00	-4.87	-0.094	WUF	N/A	0.00	1,254	465	2.45	2.55	N/A	N/A
F2,D5-E5	W21X68	Primary	21.13	22.00	-0.094	-0.042	WUF	N/A	0.00	244	465	0.48	2.55	N/A	N/A
F2,C5-D5	W21X68	Primary	21.13	22.00	-0.042	-0.033	WUF	N/A	0.00	119	465	0.23	2.55	N/A	N/A
F2,B5-C5	W21X68	Primary	21.13	22.00	-0.033	-0.012	WUF	N/A	0.00	119	465	0.23	2.55	N/A	N/A
G8CANT	W21X62	Primary	20.99	26.00	-0.028	-0.048	Cantilever	12.0	0.00	363	75	4.39	4.52	0.01	3.52
G9	W18X35	Primary	17.7	18.00	-0.048	-0.063	Shear Tab	10.0	0.00	0	N/A	N/A	4.73	0.01	3.73

this analysis. The desired bolt size was ¾" A325N, but these bolts were found to have insufficient capacity. The final design used (5) 1" diameter A325N bolts with a 9/16" A36 plate and can be seen in Figure 7.23. RAM Connection was used to verify these hand calculations. The shear connection resulted in a unity check of 0.99 and was controlled by bolt shear. The full results can be seen in Appendix G along with a dimensioned diagram.

The shear tab connections were also inspected using the prescribed UFC criteria. To protect against failure, the UFC requires pinned connections to be able to withstand the relatively large chord rotations experience during a progressive collapse. The member capacity is dependent on the connection bolt group depth; deeper connections are less flexible and therefore have smaller capacities. A typical design practice is to make shear tabs half as deep as the connected member for stability reasons. All shear tabs were assumed to follow this and have bolt group depths equal to half of the beam depth (rounded up to the nearest even number). All shear tabs passed this check and therefore all shear tabs should be specified to be less than or equal to half the beam depth. Beam B29, which is located in the removed column's bay, was the controlling shear tab connection, but was still only 64% of capacity. The results can be seen in Figure 7.25, below.

REMOVAL OF COLUMN F5 at 1st Story - BEAM TO COLUMN CONNECTION CHECK															
Beam							Connection								
Element Name	Size	Primary or Secondary	Beam Depth, d	Beam Length (ft)	$\delta_{y,i}$ (in)	$\delta_{y,j}$ (in)	Connection Type (1)	Bolt Group Depth, d_{bg}	$V_u/\Phi V_{CL}$ ≤ 1.0	Max End M_{uz}	Fixed Conn. ΦM_{CLz} (16)	Max End $M_{uz}/\Phi M_{CEz}$	m-Factor (2)	Pinned Conn. $M_u/\Phi M_{CE}$ (18)	Connection m-factor-1 (9)
F2,J1-J.71	W21X68	Primary	21.13	16.50	-0.027	-0.016	WUF	N/A	0.00	42	325	0.12	2.55	N/A	N/A
F2,J4-J5	W36X182	Primary	36.33	26.00	-0.028	-0.014	WUF	N/A	0.00	105	325	0.29	1.28	N/A	N/A
F27	W36X182	Primary	36.33	26.00	-0.048	-0.028	WUF	N/A	0.00	98	325	0.27	1.28	N/A	N/A
B28	W16X31	Secondary	15.88	26.00	-0.063	-0.112	Shear Tab	8.0	0.00	0	0	N/A	7.41	0.03	6.41
B29	W16X31	Secondary	15.88	26.00	-0.101	-4.87	Shear Tab	8.0	0.00	0	0	N/A	7.41	3.06	6.41
F30	W36X182	Primary	36.33	26.00	-0.078	-0.094	WUF	N/A	0.00	225	325	0.63	1.28	N/A	N/A
B31	W16X31	Secondary	15.88	26.00	-0.055	-0.042	Shear Tab	8.0	0.00	0	0	N/A	7.41	0.01	6.41
F32	W36X182	Primary	36.33	26.00	-0.05	-0.033	WUF	N/A	0.00	132	325	0.37	1.28	N/A	N/A
B33	W16X31	Secondary	15.88	26.00	-0.076	-0.012	Shear Tab	8.0	0.00	0	0	N/A	7.41	0.04	6.41

Figure 7.25— Secondary member connection analysis.

7.4) NON-LINEAR ANALYSIS

To verify the alternative path computer analysis, hand calculations were conducted at the same column location. This allowed for a more precise comparison between analysis methods. A simplified plastic hinge approach was used for these hand calculations. The results of this analysis were found to be similar to the computer analysis. The full calculations can be found in Appendix I.

The loading prescribed by the UFC was used for this analysis. The same procedure that was described in Section 7 was used to find the loading. A frame size of W21x62 was assumed, which lead to a m_{lif} of 2.56. This value was controlled, similar to before, by the moment connection m-factor. The load increase factor was found to be 3.4. Only deflection controlled loading was used in this analysis because only the flexural capacity of the beams was under examination. The final, factored, distributed load on the typical frame was found to be 7.33 klf. The roof, because it held no exterior wall and a lighter live load, was found to only carry 71.2% of this distributed load.

Figure 7.26 illustrates the locations were plastic hinges were assumed to form. The structure was assumed to be unstable once all the hinges depicted in Figure 7.26 formed. Other hinge locations were investigated, such as column hinges and beam hinges in adjacent bays, but only the configuration depicted was deemed feasible. The number of hinges required was greater in all other scenarios or hinges in the stronger columns were needed to establish instability, therefore this was the only configuration analyzed.

The beams were assumed to have fixed end conditions due to the continuous framing on either side of this bay. Using equations for this assumption, the required plastic section modulus was calculated. Expected yield strength was used in these calculations. The most economical beam size was found to be a W21x62.

These results align well with the computer results. The size determined by the UFC, Alternative Path analysis was W21x68, which is only one size greater than the size found in this calculation. It was expected that the 'simplified' linear analysis would be slightly more conservative because several assumption are required for that analysis.

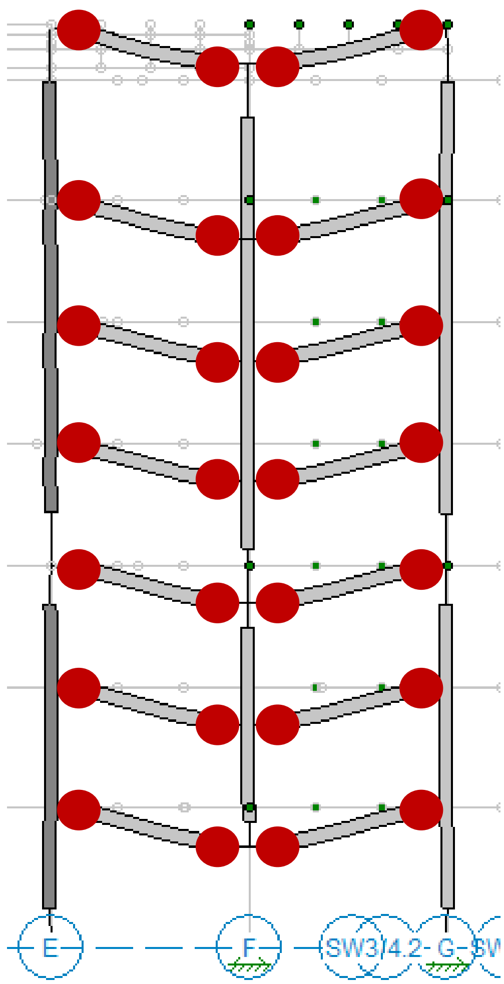


Figure 7.26 – Plan Location of Column A.2-2.4
Plastic hinge

7.4) WEST FAÇADE COLUMN REMOVAL

The second area investigated in the building was column A.8-2.4, which is located at the far West portion of the building and can be seen in Figure 7.26. This area was chosen for investigation because it lies outside of the moment frame line that runs along column line B. The irregular shape of the façade at this location made it difficult to support Column A.8-2.4. The commentary in both UFC 023 and UFC 010 discourages these types offset columns because of this very problem. Had progressive collapse been a requirement from the start of design, the exterior façade would most likely have been regularized. The first story of the column was removed because it redistributed the most force into the adjacent columns and left the most beams unsupported.

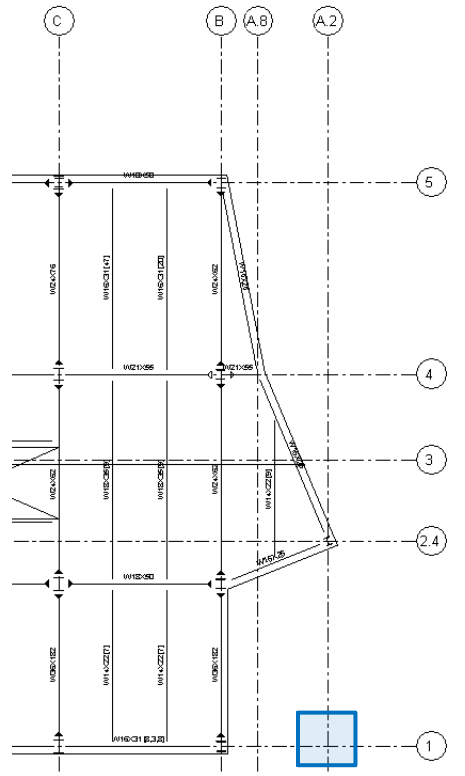
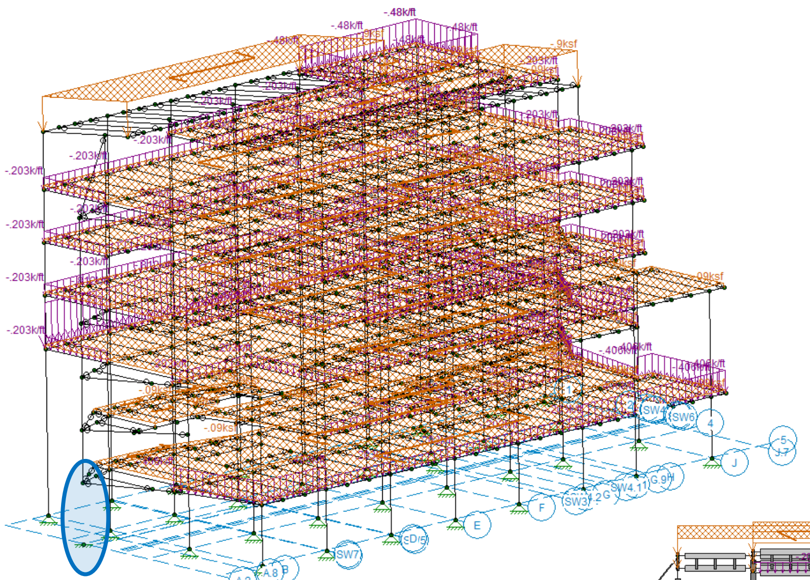


Figure 7.26 – Plan Location of Column A.2-2.4

To study this column, the loading was changed to represent the new column removal location (this process is discussed in Section 7). The near loads were placed in the bays supported by column A.8-2.4 and an example loading can be seen in Figures 7.28 to 7.31. The first floor column was removed in the RISA model and was run using the framing layout shown in Figure 7.26. The model was unstable because only pin-connected beams attach to the column and therefore the vertical forces could not be supported. The girders which connect to the column were changed to fixed ends and upsized to W21x68 (to match the other exterior moment frames), in hopes that this would take the vertical forces. Figure 7.32 illustrates the second framing option attempted. The model was run and the results were evaluated using the same spreadsheet discussed earlier. This solution is not optimum because not only does it use more total beam weight, but it also adds 24 moment frame connections, which cost money and erection time. Member G23, which is highlighted in Figure 7.32, had and interaction of 2.54 that far exceeds acceptable limits. This was expected because this member was forced to frame into a cantilever and have an inadequate backspan, which wasn't in line with member G23. The frame applied significant torsion into the adjacent beams

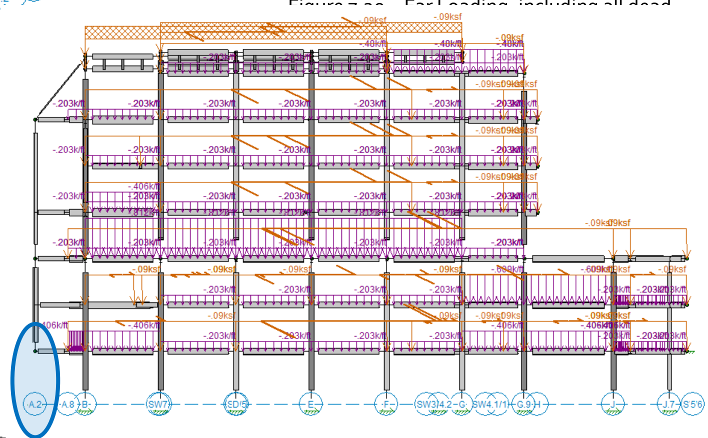
REMOVAL OF COLUMN A8-2.4 at 1st Story - PRIMARY ELEMENTS										
Element Name	Next to Removed	Size	Connection Type	Connection m-factor	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	$M_{uy}/\Phi M_{CLy}$	Element m-factor (7)	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
FPH,G.91-G.92	N	W24X76	WUF	2.31	0.00	1.30	0.00	3.00	0.01	0.09
G735	N	W18X60	shear Tab	4.73	0.00	2.48	0.00	3.00	0.01	0.17
FPH,F1-G1	N	W21X68	WUF	2.55	0.00	2.55	0.00	3.00	0.01	0.15
FPH,G1-G.91	N	W21X68	WUF	2.55	0.00	1.56	0.00	3.00	0.01	0.12
FR,B1-C1	N	W21X68	WUF	2.55	0.00	18.04	0.01	3.00	0.10	1.26
FR,C1-D1	N	W21X68	WUF	2.55	0.00	17.66	0.01	3.00	0.09	1.25
FR,D1-E1	N	W21X68	WUF	2.55	0.00	17.51	0.01	3.00	0.09	1.26



and columns, due to the skewed connections. Also several beams failed in shear, as illustrated in Figure 7.27. For these reasons other options were explored.

Figure 7.28 – Far Loading, including all dead loads. Column A.2-2.4, which is highlighted, was removed. Isometric View

Figure 7.29 – Far Loading, including all dead



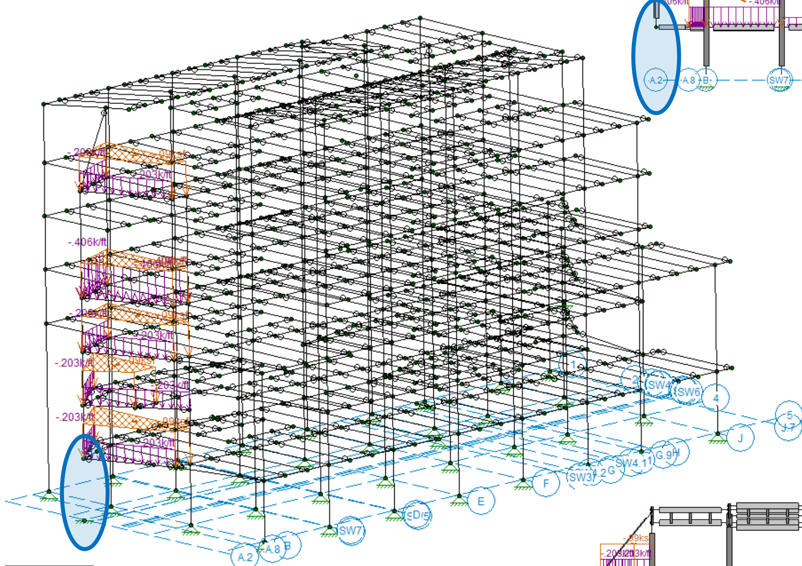
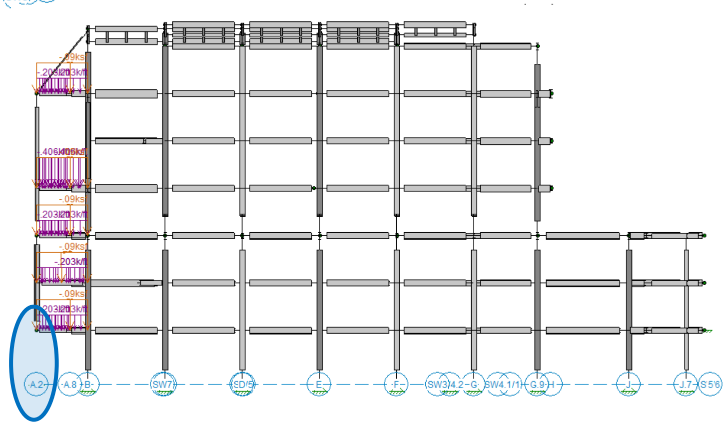


Figure 7.30 – Near Loading, including all dead loads. Column A.2-2.4, which is highlighted, was removed. Isometric View

Figure 7.31 – Near Loading, including all dead loads. Column A.2-2.4, which is highlighted, was



The option that was finally deems the most efficient was to support the column from above with a truss. Figures 7.33 to 7.35 illustrate the configuration of the simple truss. The seventh floor steps back at this location, so a diagonal member could be connected from column A.2-2.4 at the 7th to column B2 at the penthouse. A tensile member attached the top of column A.2-2.4 to the frame running along column line 2. Two member sections were investigated, a cable and a steel tube. A standard 250 ksi steel wire was used for the analysis. Wire properties were taken from EriggindSupply.com, which is an engineering testing supplier. A vertical restraint was added to the top of column A.2-2.4 and the Force load case was run. The vertical reaction at the top of the column was found to be 238 kips. Tensile member were then designed by hand to resist this load. The calculations can be seen in Appendix F.

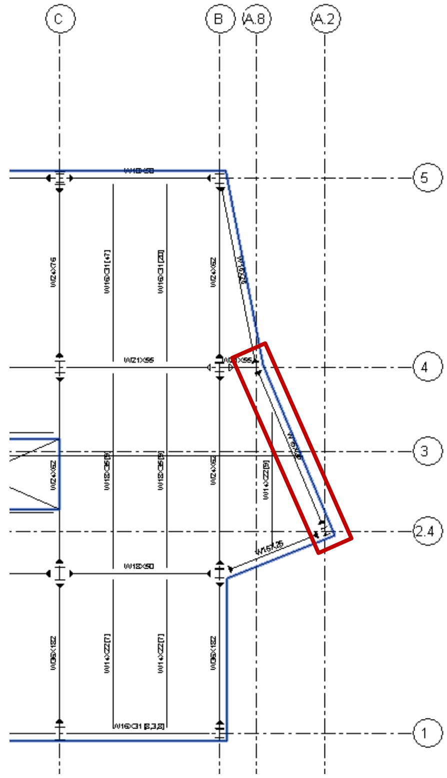


Figure 7.32 – Plan Location of Column A.2-2.4. Moment connections added.

Figure 7.33—Isometric view of roof area around column A.2-2.4 with added tension member

A 1" diameter wire rope was found to have a breaking strength of 103,400 pounds (Erggingsupply.com). Using this data it was determined that (3) 1" diameter cables would be required to support the removed column. Alternatively, an HSS3.00x0.216 tube would have the sufficient tensile strength. Through discussions with the AE faculty, it was determined that the HSS would prove to be the best section. Since this member is exposed, as seen in Figure 7.35, waterproofing would be an issue. A tube has less corrosion and waterproofing complications. The tube also would have simpler connections and these connections could be constructed by a standard steel erection crew. The tube could also fit with the steel tube walk way, which is located approximately 20 feet from this location. For these reasons the HSS was selected as the support.

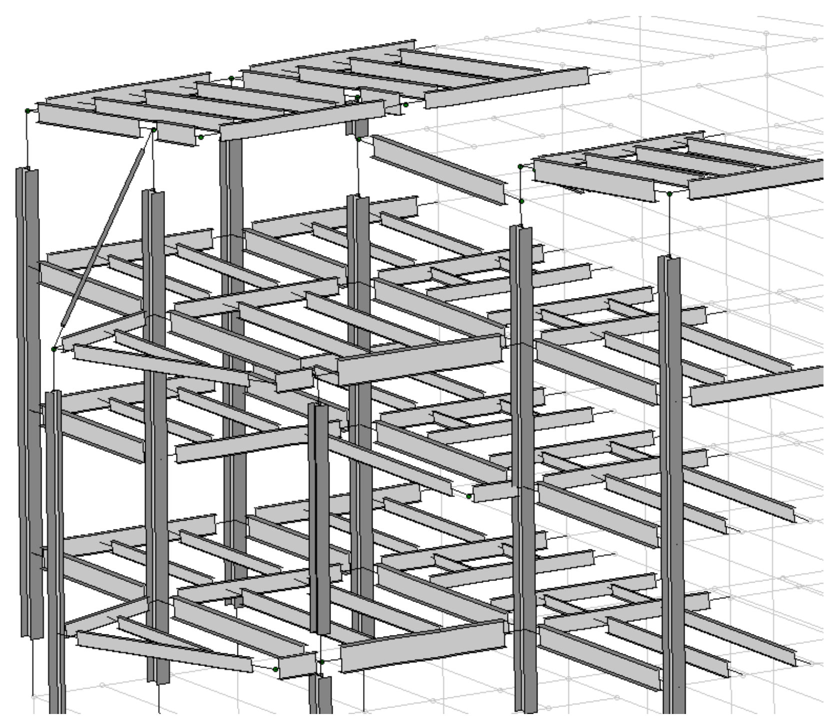
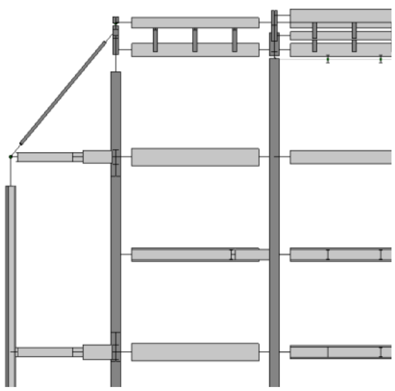


Figure 7.34—Elevation view of roof area around column A.2-2.4 with added tension member



The alternative path spreadsheet was used to evaluate the compression member and frame 2. The compression strut connected to the top of the column had to be upsized to a W21x83.The results can be found in Figure 7.36. Column B2 was found to fail, due to the increased axial load, therefore that column was upsized to a W14x426. With the cable member supporting from above, the girders framing into the removed column did not need to be upsized. All secondary elements passed UFC criteria as well.

Figure 7.35 – Architectural Model Showing Proposed HSS Diagonal Spanning Across the Terrace

Figure 7.35 illustrates the largest downside to this solution. The penthouse steps back to allow for an exterior terrace. The executives can walk out from their offices onto this terrace and look out upon the city. Adding a diagonal will inhibit the circulation at this terrace, which is a major downside. Occupants would still be able to walk under the diagonal, but only within a narrow region along the wall. Due to time constraints this architectural issue was not investigated further. Do to these constraints, the diagonal option may be deemed inadequate.

REMOVAL OF COLUMN A8-2.4 at 1st Story - PRIMARY ELEMENTS

Element Name	Next to Removed	Size	Connection Type	Connection m-factor	$P_u/\Phi P_{CL}$ (15)	$M_{uz}/\Phi M_{CLz}$	$M_{uy}/\Phi M_{CLy}$	Element m-factor (7)	Interaction ≤ 1.0	$V_u/\Phi V_{CL}$ ≤ 1.0
C,U,C2	N	W14X500	Fixed Base	---	0.67	0.15	0.08	Col Mom Force Cont (8)	0.87	0.01
C,U,B2	N	W14X370	Fixed Base	---	1.06	0.39	0.09	Col Mom Force Cont (8)	1.48	0.04
C,L,J4	N	W14X370	Fixed Base	---	0.29	0.06	0.31	5.08	0.35	0.02
C,L,G.94	N	W14X500	Fixed Base	---	0.45	0.05	0.01	2.86	0.47	0.01
C,L,G4	Y	W14X370	Pinned Base	---	0.65	0.02	0.01	Col Mom Force Cont (8)	0.68	0.00

Figure 7.36 – Alternative Path analysis for the removal of column A.2-2.4 with the added tension member support.

8.) ENHANCED LOCAL RESISTANCE

The third and final progressive collapse design procedure conducted was the Enhanced Local Resistance Method (ELR). The purpose of this step in the design was to strengthen the columns to the point where they would be able to withstand an attack and if the column would fail they would do so in a ductile manner. Terrorist attacks are very unpredictable events and researchers have little history to use as a basis for requirements. For these reasons, ELR analysis uses very general guidelines: ensure the flexural capacity of the columns is twice the base capacity and the shear capacity is greater than the flexural capacity.

UFC 023 defines the flexural capacity as “the magnitude of a uniform load acting over the height of the… column which causes flexural failure”. The flexural capacity of the column size required by AP analysis, or “existing” size, was compared to the flexural capacity of a column that was sized purely for gravity loads, or “baseline” size. For occupancy IV, the final column design was required have flexural capacity greater than the existing column design and twice the baseline column design. This requirement is meant to ensure that the column will be significantly stronger than a typical, non-progressive collapse, column. The second ELR provision, which is the same for all occupancy categories, requires that the column have greater shear capacity than flexural capacity. In other words, the columns must have enough shear strength to allow the column to yield in flexure and enter the plastic domain, which absorbs substantial energy. It can be seen in Figure 5.1 that occupancy category IV requires these ELR checks at the lower two stories, which is the most stringent ELR requirement.

For this analysis, a simplified approach was taken because of the vagueness of the design guide and the simplicity of the design check. Each column investigated was modeled independently, as can be seen in Figure 8.1 and Figure 8.2, in RISA 3D. A distributed load was applied normal to the façade. Through an iterative process, the load was increased until the column failed in bending. This load was recorded as the column’s flexural capacity and used to determine the adequacy of the design. Both the baseline and larger, existing, column sizes were modeled in the same manner. Plastic capacities and expected material strengths were not used in this analysis because they would add approximately the same amount of strength to both the baseline and existing columns’ flexural capacity. Comparing the two capacities canceled out these strength increases. Per section 3.32 of UFC 023, “…in no case shall the flexural resistance be less than that of the column or wall with zero axial load acting.” Column axial loads cause p-delta effects, which typically reduce the flexural capacity of the column, therefore the axial loads were not included in the ELR analysis.

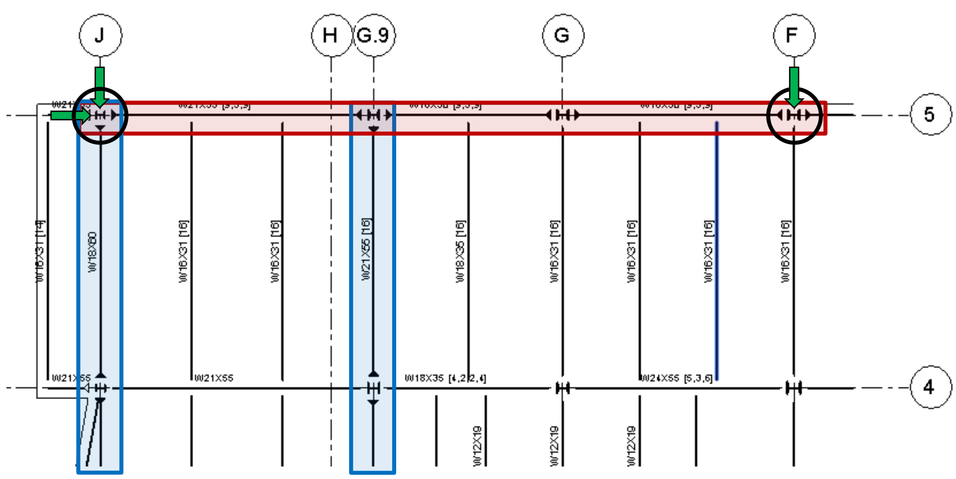


Figure 8.1 – Col. Designed For ELR
E-W Frames
N-S Frames
Col Invest.
Applied Loads

The first column analyzed was at the intersection of column lines F and 5, which was a typical column located in the middle of the Northern façade. The baseline column size, determined by a gravity-only analysis, was W12x72. A distributed load was applied to this column perpendicular to the building façade, a plan view can be found in Figure 8.1 and an elevation of the RISA model can be seen in Figure 8.2. Through an iterative process, the load was increased until the column failed in bending. The design output is illustrated in Figure 8.3. A load of 5.4 k/ft was found to be the flexural capacity.

This same process was conducted using the existing column size of W14x370, which was found in the alternative path analysis in Section 7.1. The failing load for this column was found to be 40 k/ft, as seen in Figure 8.2. Section 3-3.3.3.1 of UFC 023 defines acceptable flexural resistance as "the larger of the existing flexural resistance or 2.0 times the baseline flexural resistance". Therefore, the column size determined from alternative path analysis was found to be adequate for flexural strength. The UFC also prescribes that the "application of the uniform load that defines the… flexural resistance must not fail the column… in shear." Figure 8.4 shows that the W14x370 was only at 23.4% of its shear capacity when it failed in flexure. This means that the W14x370 passes all Enhanced Local Resistance criteria.

A corner column was also investigated using the same procedure described above. Column J5, which can be seen in Figure 8.1, was selected for analysis because it was most typical. Corner columns are unique because the lateral load could be applied perpendicular to each of the two facades. This meant that the column could bend in either strong or weak axis flexure. For this analysis the load that caused strong axis bending was applied because it was different than the previous scenario. The flexural resistance

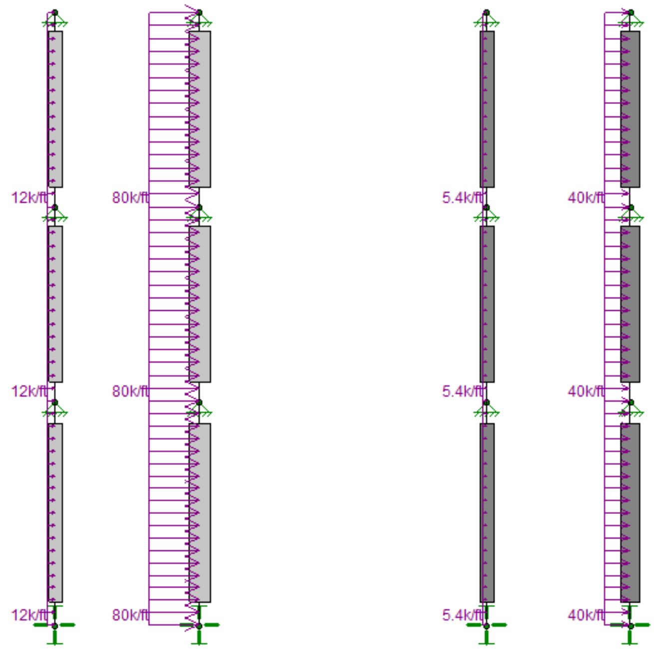


Figure 8.2 —Elevation view all four columns investigate in the ELR analysis

Figure 8.3 —W12x72, base, design analysis output.

AISC 13th(360-05): ASD Code Check
Direct Analysis Method

Max Bending Check	0.996	Max Shear Check	0.172 (z)
Location	15.5 ft	Location	15.5 ft
Equation	H1-1b	Max Defl Ratio	L/216
Bending Flange	Compact	Compression Flange	Non-Slender
Bending Web	Compact	Compression Web	Non-Slender

			y-y	z-z
Fy	50 ksi			
Pnc/om	480.468 k	Lb	15.5 ft	15.5 ft
Pnt/om	631.737 k	KL/r	61.184	34.968
Mny/om	122.754 k-ft	Sway	No	No
Mnz/om	251.68 k-ft	L Comp Flange	15.5 ft	
Vny/om	105.78 k	Torque Length	42.5 ft	
Vnz/om	288.862 k	Tau_b	1	
Cb	1			

Figure 8.4 —W14x370, progressive collapse, design analysis output.

AISC 13th(360-05): ASD Code Check
Direct Analysis Method

Max Bending Check	0.980	Max Shear Check	0.234 (z)
Location	15.5 ft	Location	15.5 ft
Equation	H1-1b	Max Defl Ratio	L/298
Bending Flange	Compact	Compression Flange	Non-Slender
Bending Web	Compact	Compression Web	Non-Slender

			y-y	z-z
Fy	50 ksi			
Pnc/om	2841.232 k	Lb	15.5 ft	15.5 ft
Pnt/om	3263.473 k	KL/r	43.531	26.329
Mny/om	923.154 k-ft	Sway	No	No
Mnz/om	1833.946 k-ft	L Comp Flange	15.5 ft	
Vny/om	594.28 k	Torque Length	42.5 ft	
Vnz/om	1576.886 k	Tau_b	1	
Cb	1			

of the baseline W12x72 was found to be 12 k/ft. The flexural resistance of the existing W14x370 was found to be 80 k/ft, therefore these column pass the flexural strength requirements.

Figure 8.5 outlines a complication with this column size. The distributed load causes shear failure just below the second floor and therefore does not pass the ELR requirements fully. A shear failure is undesirable because it is a sudden, brittle failure that cannot absorb large amounts of energy. Bending forces have a large plastic range, which allows for large deformations and energy absorption. This failure was expected because in this direction the column has little shear strength and a large amount of flexural strength. When the pressures are applied to the column flange the main shear resistance comes from the smaller column web. As opposed to when the load is applied to the web and the thick flanges resist the shear.

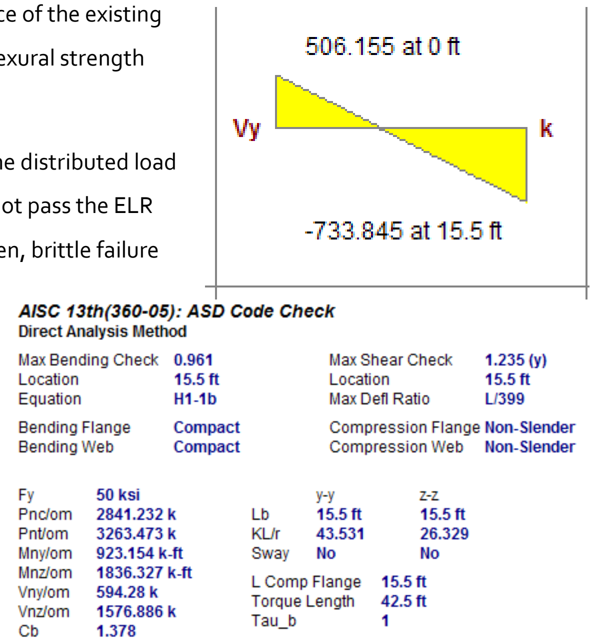
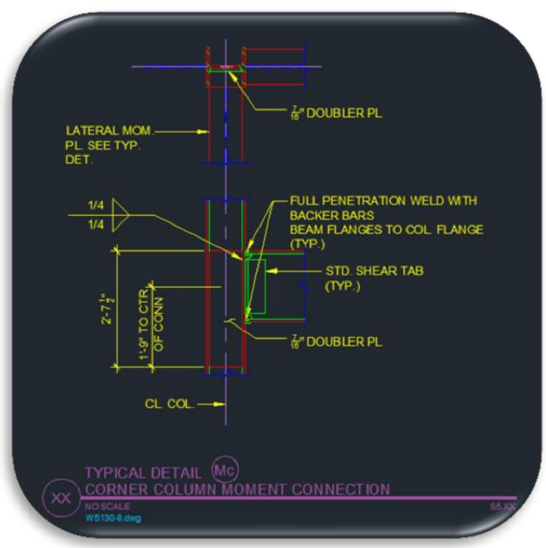


Figure 8.5 —Corner W14x370, progressive collapse, design analysis output and shear diagram.

Hand calculations, which can be found in Appendix I, were performed to determine the required doubler plate size. A 7/16 in thick A36 plate was found to have enough shear capacity to allow the column to fail in bending first. Quarter inch fillet welds were found to have the required capacity to secure the plate. Using the shear diagram found in Figure 8.5, the location where the column had sufficient shear capacity was calculated. The double plate was ended at this location. The doubler plate was required to extend 1'9" below connection center. For safety, the plate was extended from the top of the beam to the required 1'9" location, for a total length of 2'-7 ½". The connection detail can be seen in Figure 8.7, refer to Section 7.3 for the moment connection design. This plate was assumed to be required at every exterior column that had its web parallel to the façade.

Figure 8.6 – Corner Column Moment Connection With Double Plate



The final results from the ELR analysis can be seen in Table 8.1. The large, W13x370 size required be the Alternative Path analysis provided sufficient flexural capacity. Only the corner columns were determined to require double plates along a portion of their height.

Table 8.1: Enhanced Local Resistance Design Results

Column Location	Size	Baseline Flexural Capacity (k/ft)	Existing Flexural Capacity (k/ft)	Column Shear Capacity (k/ft)	Doubler Plate Thickness (in)
Exterior	W14x370	5.4	80	342	None
Corner	W14x370	12	80		7/16

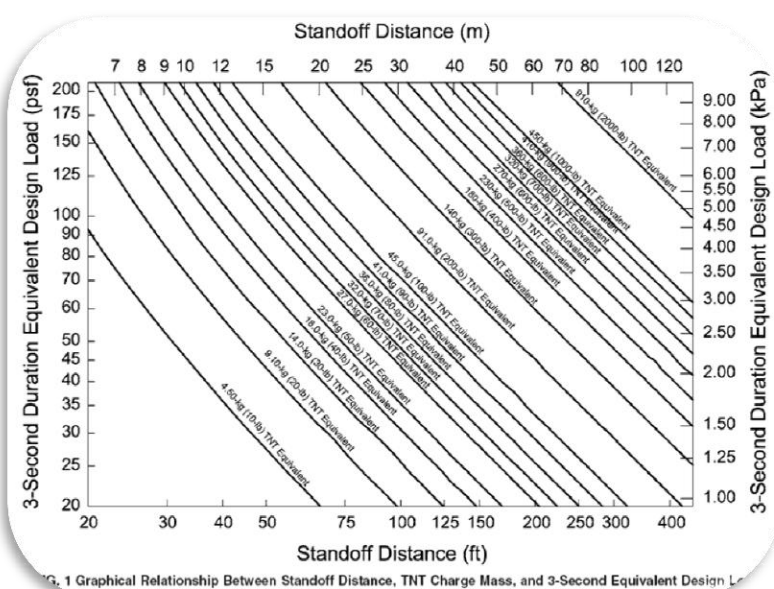
9.) ATRIUM WALL SUPPORT

The final area investigated for antiterrorism measures was the southwest atrium. As can be seen in Figure 9.1, this glass atrium runs along the majority of the southern façade and rises almost a full three stories from the ground level. This atrium is not interrupted by horizontal beams, so the columns span approximately 40' without bracing. Large glass curtain walls usually fare poorly when subjected to blast loading because, due to architectural constraints, the supporting structure is usually held to minimal size. If the glazing fails, the blast pressures carry the glass shards and debris into the occupied space, which poses a significant hazard. A supporting structure was design to resist the equivalent blast loads with these considerations in mind.

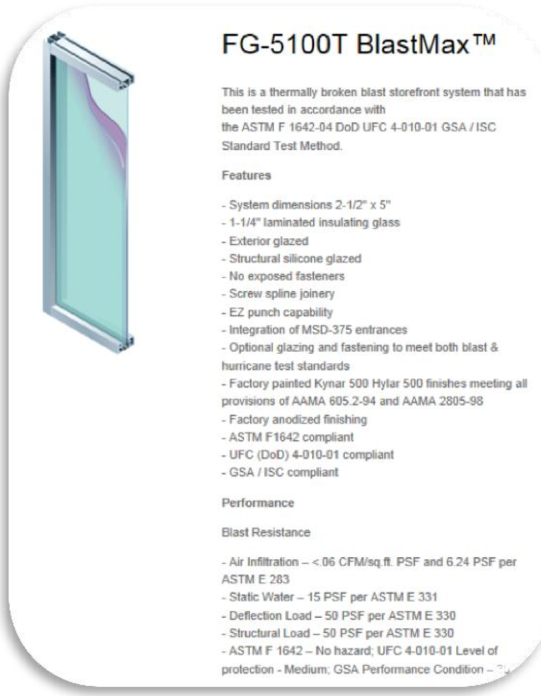
Level of protections, which were used to determine equivalent blast pressures were determined from UFC 010. It was assumed that the standoff distance was 100', see the "Site Redesign Section", and the explosive equivalent weight was 200 lb of TNT. Using Figure 1 of ASTM F2248, "Standard Practice for Specifying an Equivalent 3-Second Duration Design Load", the equivalent lateral load was found to be 100 psf. The graph used for this calculation can be seen below in Figure 9.2. The façade framing was designed to resist this lateral load.

Figure 9.1 – Rendering of Atrium, Courtesy of EwingCole

Figure 9.2 – Determination of Equivalent Blast Pressure, Credit to ASTM F2248,



ASTM F2248 also requires that the mullions limit glazing edge deflections to L/60 under a load of 2.03 times the pressure capacity of the glazing. This, along with other requirements, ensures that the glazing should fail prior to the supporting structure, thereby limiting the failure area and debris size. Published product data from Old Castle Glazing, a façade manufacturer, was selected for this analysis. Old Castle Glazing's Blast Mitigation Glass, which can be seen in Figure 9.3, was selected as the glazing. FG-5100T BlastMax mullion system, which can be seen in Figure 9.4, was selected as the supporting structure. Both systems were rated for Medium level of protection, according to UFC criteria.



The existing mullion dimensions were taken from the architectural plans. All mullions were spaced at 4.5' on center. To transfer these forces into the columns, horizontal cables were sized and configured. Each mullion was assumed to act as individual rectangular elements, therefore at each support the mullion was connected by pins. Various horizontal lengths were investigated, ranging from 3'-8" x 4'-6" panels to 5'-6"x 4'-6" panels. Various cable sizes were used to determine the minimum cable sag required to carry the load.

The atrium façade could be moved to the exterior of the columns, thereby creating 3'-6" of plan space for these cable supports. The architectural impacts of these alternations are discussed in Section 10.1.

The hand calculations can be found in Appendix J. Wire strengths were, again, taken from EriggindSupply.com. Three eights, one half, and five eights inch diameter cables were investigated. The 3/8" cable was found to be the most cost effective, at $17 for a single cable, but this size would require 36" of cable sag. This means that the cable would come close to entering the atrium and create large viewing obstructions. The 1/2" cable was the next most cost effective and this only required 20" of cable sag. It was found that adding more bracing points along the cable length had little effect on the cable efficiency and only increased the amount of mullion to cable connections. If the BlastMax mullion system is able to function in a 4'-6" x 5'-6" panel size, this spacing would be preferred. For corrosion issues, stainless steel cables were also investigated. This material slightly reduced the tensile strength and greatly increased the cost. The final cable selected was a ½" diameter, 1x19 stainless steel cable that was attached at third point and required 20" of mid span sag. Figure 10.6 illustrates this configuration. Vertical cables were run from the first floor to the ceiling of the atrium to prevent lateral torsional buckling. Steel compression rods, which measured ½" in diameter, were used to connect the cable system to mullions.

The cable forces determined from the analysis above were then applied to the atrium columns, along with the gravity loads determined from the base steel model. Figure 10.6 shows this atrium column model. It was found that the columns had sufficient moment and axial capacity to resist the thrust from the cables.

10.) ARCHITECTURAL BREADTH

10.1) ATRIUM FAÇADE REDESIGN

The architectural effects of altering the atrium façade were investigated using the existing Revit Architecture model. To accommodate the new cable structures the existing façade was moved to align with the rest of the south elevation. The cables were then added to the model and renderings were developed to illustrate the new architecture.

The existing first floor plan can be seen in Figure 10.3. It can be seen that the large atrium being investigated is located at the plan south east of the building. The atrium was approximately 100' long and 20' wide. Figure 10.2 is a building section through the atrium which illustrates large void created by the atrium. It can be seen from Figure 10.3 that the curves second and third floor slab edges face out into the space. The existing atrium façade rose from ground level up to the third story and measured 28' tall. The curtain wall stepped back from the rest of the building at this atrium. This placed the columns on the exterior of the space. The columns also rose freely in this space and were not interrupted by horizontal beams.

Figure 10.1 – Interior Rendering of Atrium. View from SW Entrance. Credit to EwingCole

The existing vertical mullions were spaced 11' on center and the existing horizontal mullions were spaced at 4'-6" on center. The vertical mullions were originally 2 ½" wide and 11 ¼" deep. The horizontal mullions were originally 2 ½" wide and 5 ¾" deep.

Figure 10.1 is a rendering of the existing façade and illustrated the sunlight, open feeling of the space. The rendering is viewing the space from just inside the revolving door. The atrium also creates an informal gathering area for the first three floors. The blue stone flooring and "living wall" vegetation helped create a sustainable green aura, which is prominent throughout the project. The atrium runes the entire length of the southern façade and connects the first floor retail spaces.

To design the curtain wall to resist the blast loading cable structures were proposed, as discussed in the Atrium Façade Redesign section above. ½" diameter cables were selected and then modeled to see the architectural impacts. The cable layout can be seen in Figure 10.6. Two additional vertical mullions were added to the curtain wall and the existing 4'-6" horizontal mullion spacing were used. The mullions were changed to match the BlastMax system from OldCastle Building Envelope. The new horizontal mullions were 5 ½" deep.

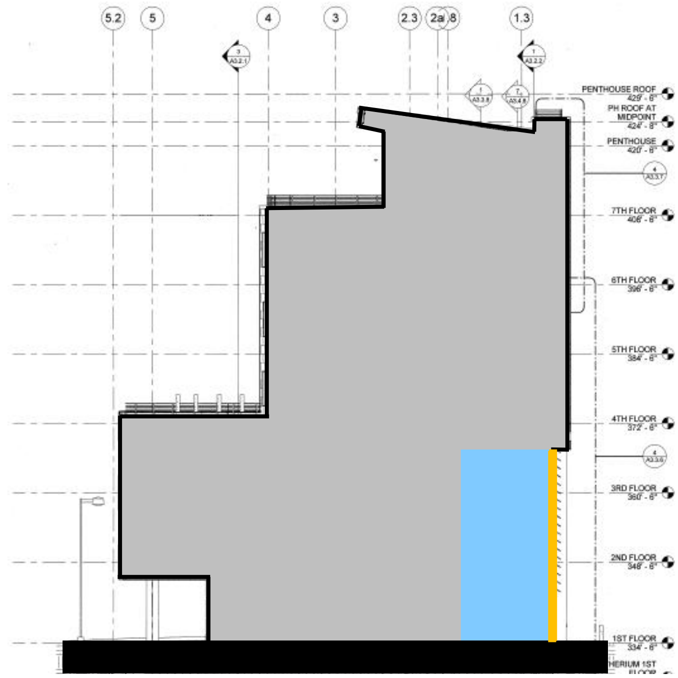


Figure 10.2 – Building Section Near CL F

Atrium
Occupied Space
Atrium Curtain Wall

Figure 10.5 shows the original atrium, as viewed from inside the atrium. This image was taken from the original Revit Façade model. The model was courtesy of EwingCole, who developed it for the project documents. Figure 10.6 shows the redesigned atrium and was taken from approximately the same location in the building. Similarly Figure 10.7, the existing atrium,

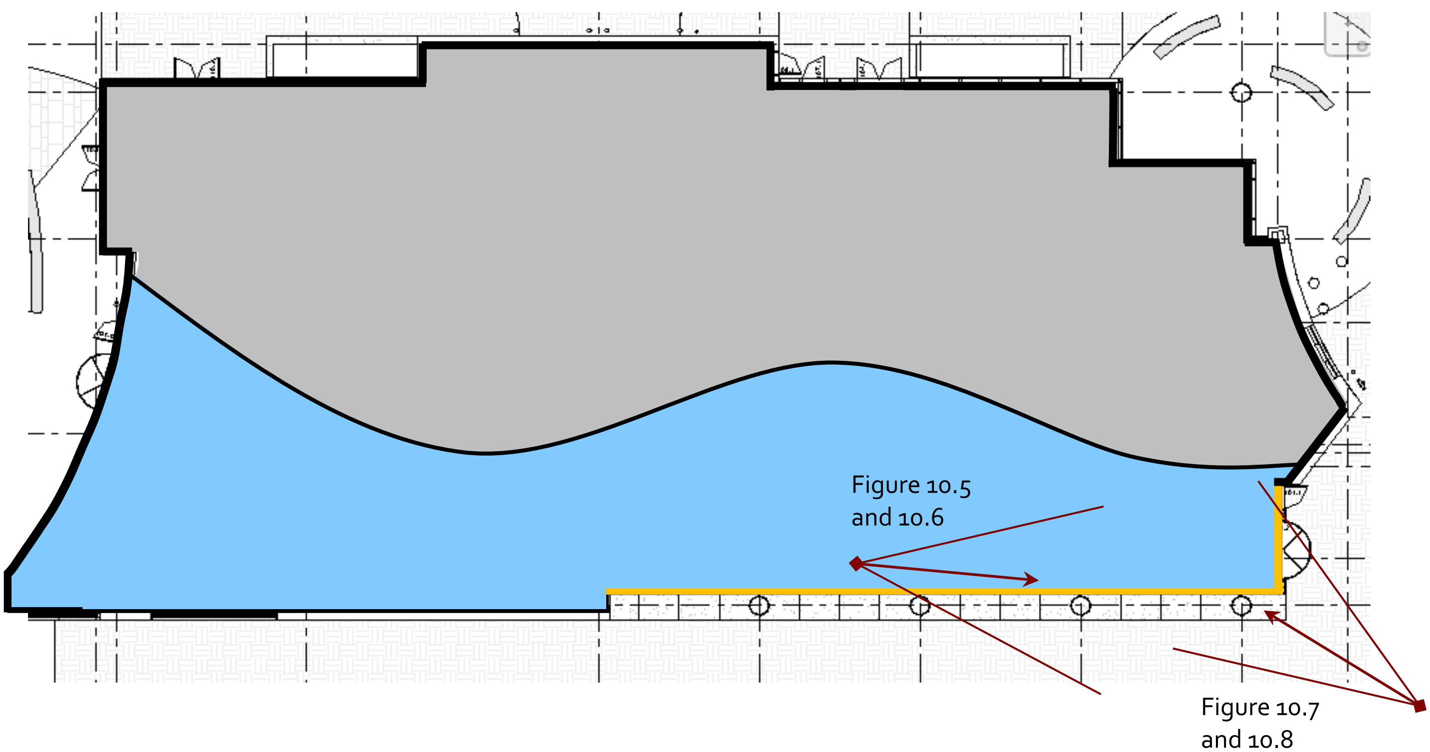


Figure 10.3 – Existing First Floor Plan

and Figure 10.8, redesigned atrium, show the same exterior portion of the building. Figure 10.3 illustrates the plan location of these rendering.

By moving the curtain wall 3'-6" out, the cable structure did not intrude upon the existing space. The program square footage for the atrium was also kept approximately the same. Moving the curtain wall to outside of the columns did, however, create a more cluttered space. The original design had a clean, simple line of glass, as illustrated in Figure 10.4. The columns penetrated this void created by the straight curtain wall. This was deemed a negative of the redesign because it altered the architectural aesthetic in a negative manner. When the cable structures were added this clutter was only increased.

One may argue, however, that the lines created by the cables created a new implied space. The atrium then had an intermediate volume. The occupants walked in the open void of the atrium. The cables then occupied the transition area between the columns and next to the curtain wall. The parabola plan of the cables created a dynamic surface to this intermediate volume. The curved slab edge on the opposite side of the atrium works well with this undulating surface. The architect (EwingCole) in a presentation to the owner during schematic design envisioned this circulation space as a flowing stream. The plan of the atrium space and materials used certainly embrace this goal. The addition of a dynamic surface at the curtain wall may help to add to this by acting like boulders and ripples in the stream.

Overall the redesign was deems acceptable. Although the new curtain wall system had some negative consequences, the proposed system effectively resolved the high blast loadings with few negative impacts to the existing aesthetics.

Figure 10.4 – Interior Rendering of Atrium, Looking West from 2nd Floor. Credit to EwingCole

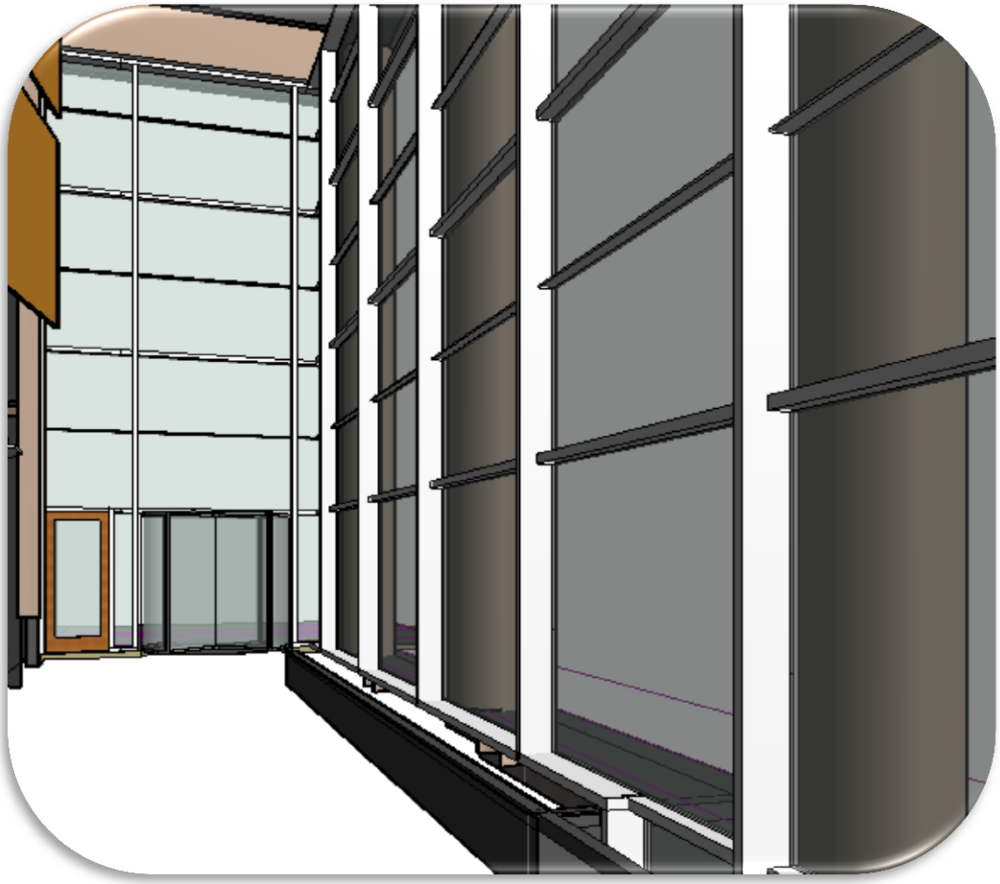


Figure 10.5 – Interior image of existing atrium. Refer to Figure 10.3 for image orientation

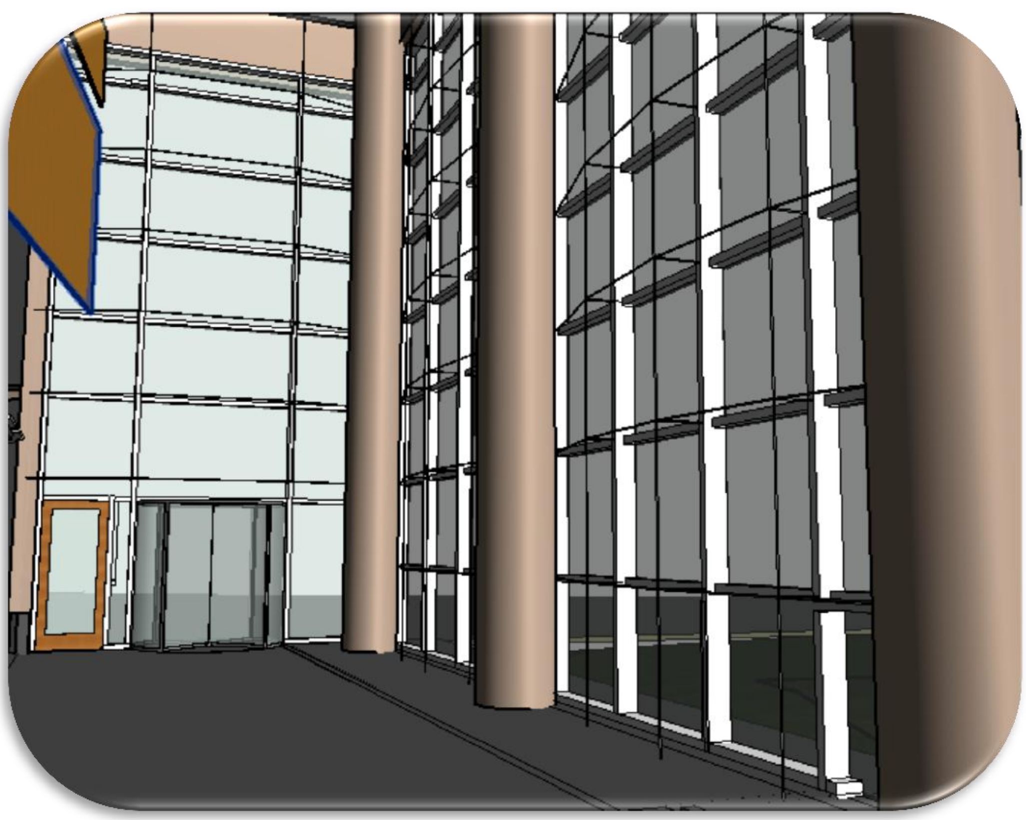


Figure 10.6 – Interior image of proposed atrium redesign. Refer to Figure 10.3 for Image orientation

Figure 10.7 – Exterior image of existing atrium. Refer to Figure 10.3 for image orientation

Figure 10.8 – Exterior image of proposed atrium redesign. Refer to Figure 10.3 For image orientation

10.2) PLAN RECONFIGURATION

As part of the proposed steel redesign, the exterior column line was shifted 6'-8" to the exterior of the building to eliminate troublesome cantilevers. The structural discussion of this topic can be found in the Section 4. The architectural impacts of this shift are discussed below.

At the first floor the façade steps back into the building to create an entrance space for the retail spaces. Moving column line 5 closer to the north will place columns in this entrance area, which may not be aesthetically pleasing, but is still functional. As can be seen in Figure 10.9, below two retail space entrances are located on either side of the parking garage entrance. Pedestrians could easily distinguish these entrances because of the recessed building cove in these areas.

As part of the site redesign, discussed in the next section, these doorways were eliminated for security reasons. Without entrances at these locations, the first floor step back has little functional use. It was not eliminated, however, because it would have considerable impact on the front façade. This façade is broken up into four sections: the recessed base, the orange metal panel middle, the grey metal panel middle, and the curtain wall cornice. If the first floor curtain wall would have been pushed out to align with the rest of the façade, this segmentation would have been compromised. Also, altering the curtain wall in these areas would have significantly increased the plan space for these areas. The stated goal for this report was to not alter the existing program, and as such the first floor walls remained recessed.

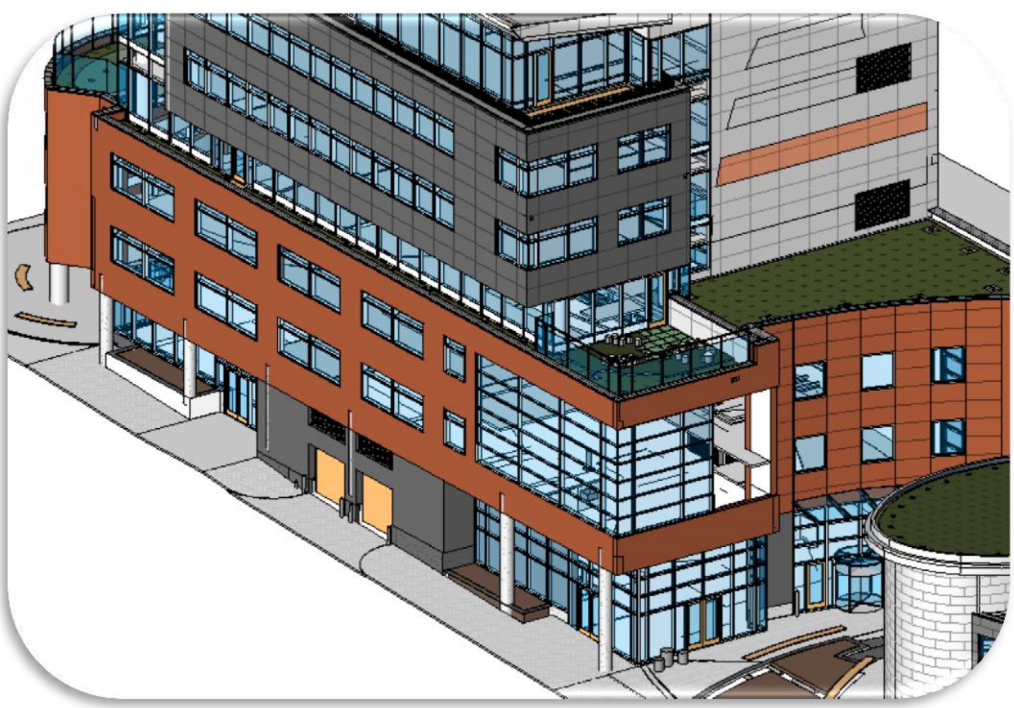


Figure 10.9 – Exterior Image of Existing North Facade. Credit to EwingCole

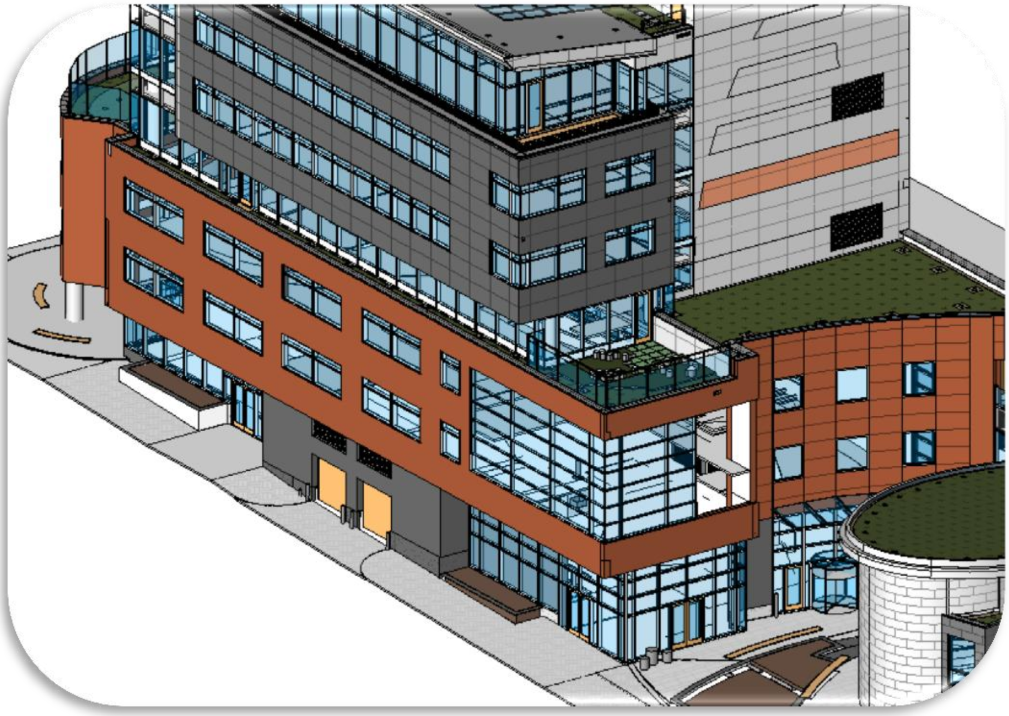


Figure 10.10 – Exterior Image of Existing North Facade. Credit to EwingCole

10.3) SITE PLAN REDESIGN

The final architectural area investigated was the site plan. As discussed previously, a key component to antiterrorism design is site security and standoff distance. The existing site was redesigned to meet UFC 010 criteria, which included standoff distance and unauthorized access. The existing project location was found to be inadequate, therefore a new location was proposed.

Figure 10.11 is an aerial view of the existing building location. It can be seen that the GOB shares property lines with several other existing buildings. As stated previously, the project investigated in this report is the second phase of a two phase headquarters expansion. The first phase, 2a, was located across the street from the new phase 2b building. The entire headquarters campus, phase 2a and 2b, was investigated in the site redesign. The first phase was assumed to have the same hypothetical security requirements.

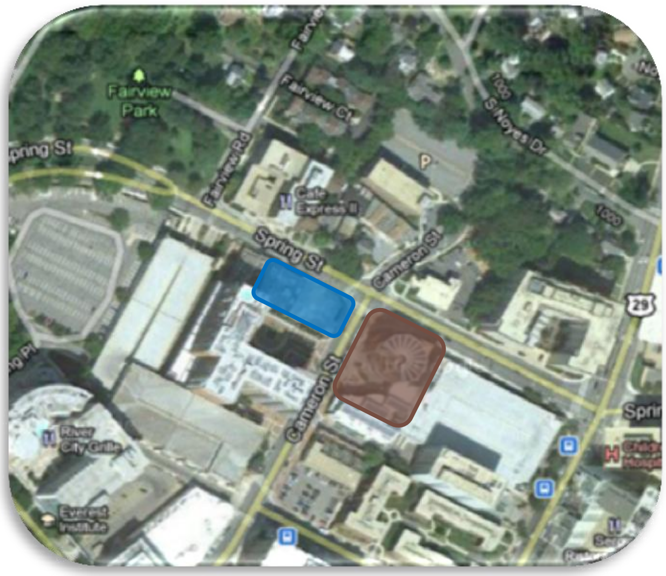


Figure 10.11 – Project Location
GOB, Phase 2b, Site
Existing, Phase 2a, Site

Figure 10.16 is the existing site plan, taken from the landscape and architectural drawings. The proposed site redesign can be seen in Figure 10.17. The landscaping and layout was kept largely the same in the redesign. The thin red line in Figure 10.17 represents the 100' standoff distance.

UFC 010 was used to determine the minimum standoff distance. As part of the design scenario, the proposed new owner mandated that the building be designed to medium level of security. This may be warranted by mission critical type of work performed by the new occupant. This level of protection requires a minimum standoff distance of 18', as can be seen in Appendix K, which is table 5.1 and 5.2 of UFC 010. This standoff distance was defined as the minimum distance from any point on the building to a controlled perimeter. If the controlled perimeter was located farther than 18' from the building, no special blast analysis needed to be conducted. The conventional construction standoff distance was found to be 151'. If the controlled perimeter was located outside of this range conventional building methods could be used and the building would not need to have been designed for progressive collapse. A 100' standoff distance was chosen because it fell within these two ranges and provided enough distance for the blast pressures to dissipate. As discussed in Section 9, the atrium curtain wall was designed to resist blast loading from this distance.

To create the outer most defenses a concrete barrier wall and trapezoidal ditch were located just outside of the 100' standoff distance. The red line in Figure 10.17 illustrates the 100' standoff from the buildings' exterior. Both the existing Phase 2a and new Phase 2b building were included in the analysis, as stated earlier. The concrete wall restricts unauthorized pedestrian access. It also interrupts line of site to the entrances, which inhibits small arms attacks on the occupants. The trapezoidal ditch, which can be seen in Figure 10.12 from FEMA 430, was added to hinder vehicular attacks on the site wall. This ditch runs the entire length of the wall.

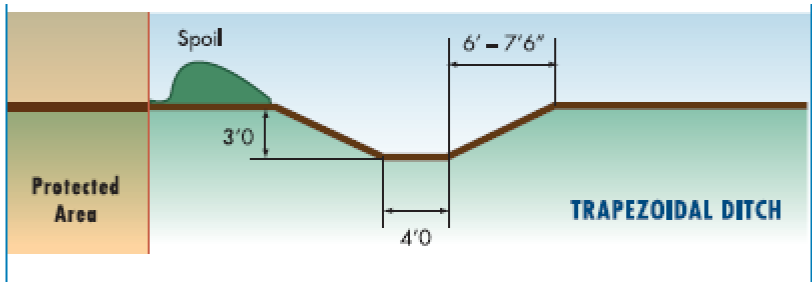


Figure 10.12 – Trapezoidal Ditch Configuration. Credit to FEMA 430

UFC 010 defines the Type I vehicular charge as the type of explosive that can easily be spotted at a car check point. The specific explosive weight is classified. This type of explosive can be equated to a car filled with homemade explosives. Type II vehicular explosives are smaller and therefore pose less of a threat. To limit the possibility of the larger Type I explosives being detonated close to the building, an access control point was located at the only vehicular entrance to the site. Per UFC 010's recommendations the number of access points was kept to a minimum because it both saved money and limited the number of vulnerable areas of the site. At this access control point both a guard house and a vehicular barrier were positioned along the line of traffic. The guard house gave the security an area to be located and enter if threatened. A rising wedge barrier system was chosen to bar unauthorized entry to the site. An image of this barrier can be seen in Figure 10.13. FEMA 430, "Site and Urban Design for Security", was used for the site redesign and the barrier images below are credited to that document.

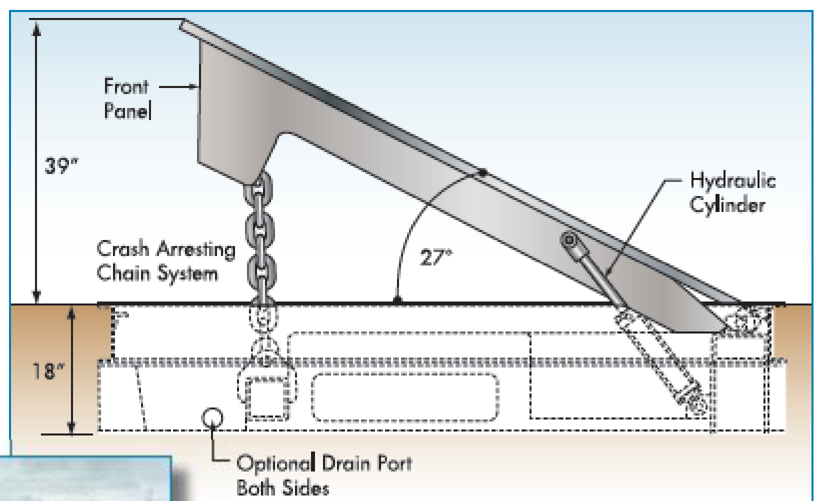


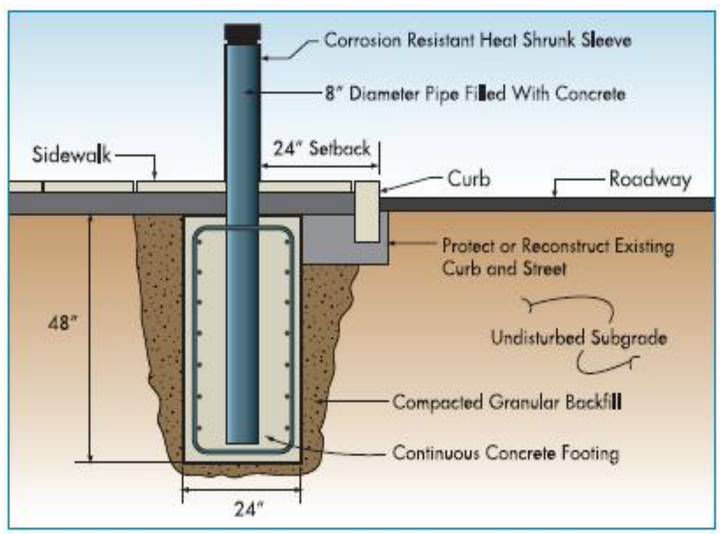
Figure 10.13 – Rising Wedge Barriers. Credited to FEMA 430

This same barrier system was selected for the entrance to the basement parking garage. A second check point was placed at this location to allow for additional inspection of vehicles that enter the garage. Eliminating the garage completely would be a preferred option because the structure was not designed for the removal of interior columns. Additional progressive collapse analysis would have to be conducted if the parking garage was kept in the project program. The ground parking was added to the site as an alternative to garage parking. Further analysis of the impact of removing the parking garage would have to be performed.

Bollards were also placed along the front façade of the building. An example of the design can be seen in Figures 10.14 and 10.15. The trees of the existing site plan were kept and used with in combination with the proposed bollards. This system is mean as a last defense against any vehicular attacks that make it through the outer defenses. It can be seen in Figure 10.15 that the bollards are supported by a 48" deep concrete foundation that allows the bollards to resist large lateral loads. As with the rising wedge barrier, this particular schematic is capable of restraining a 4,500 lb truck traveling at 50mph.

Figure 10.14 – Bollard and Tree Site Barrier. Credit to FEMA 430

Figure 10.15 – Bollard Construction Diagram. Credit to FEMA 430



The first floor retail entrances were eliminated for security reasons. The main front entrance and atrium entrance were kept. The doors eliminated may still be sued for fire exits, but they would have to be exit only. Security guards were also required at these two entrances to check in all occupants.

An acceptable site redesign was found for this scenario. The project was moved outside of the current urban location to a development park or suburban area for standoff reasons.

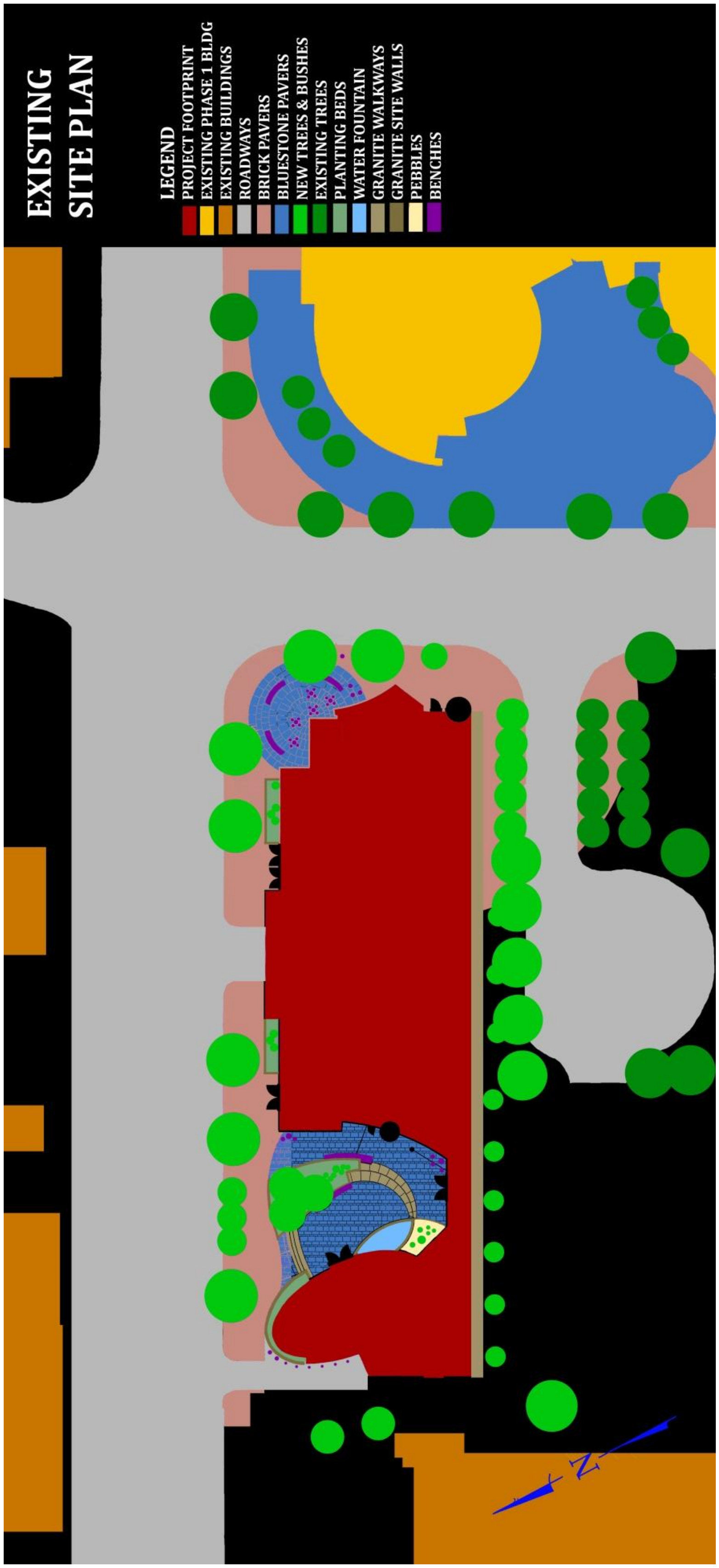


Figure 10.16 – Existing Site Plan

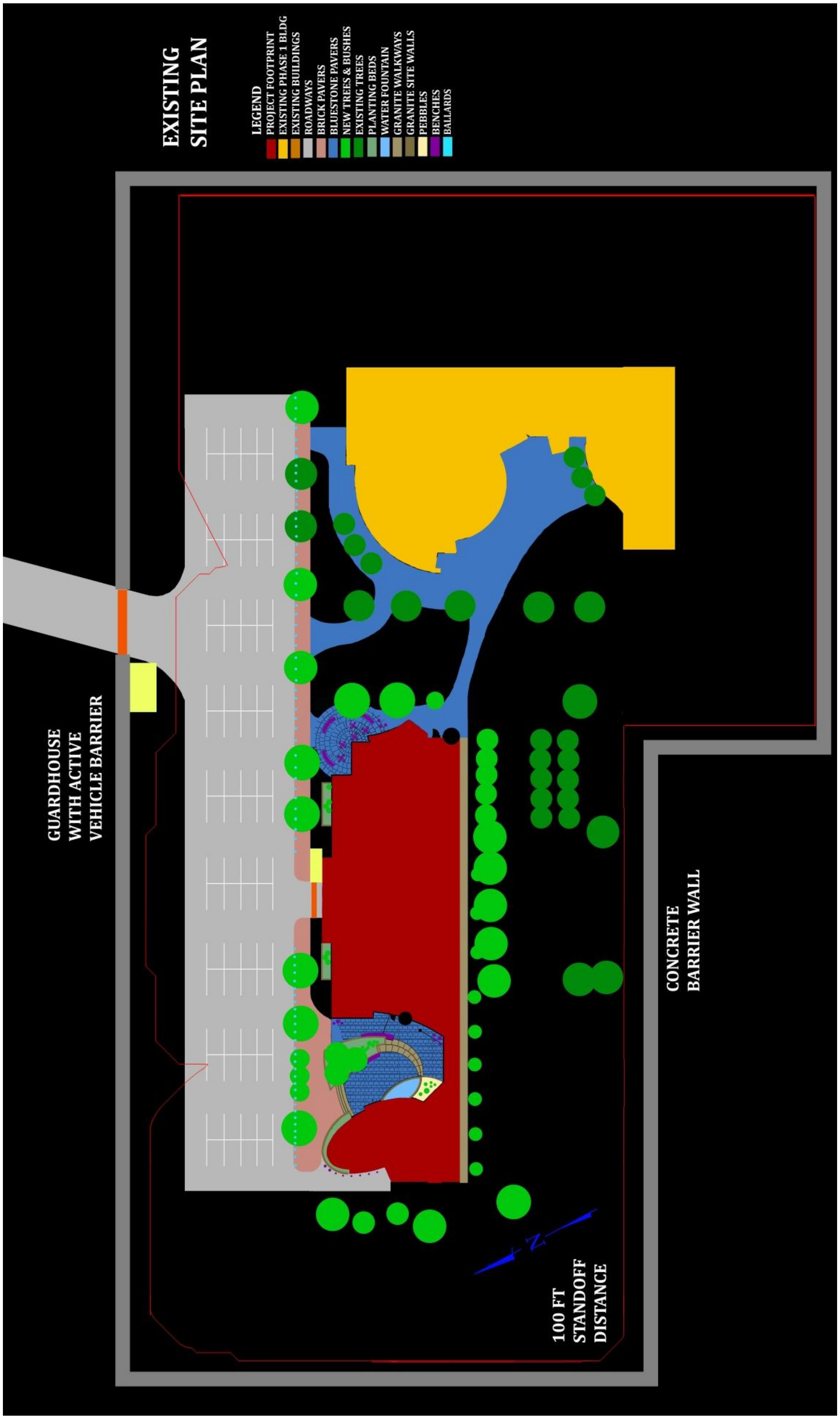


Figure 10.17 – Redesigned Site Plan

11.) CONSTRUCTION MANAGEMENT BREADTH

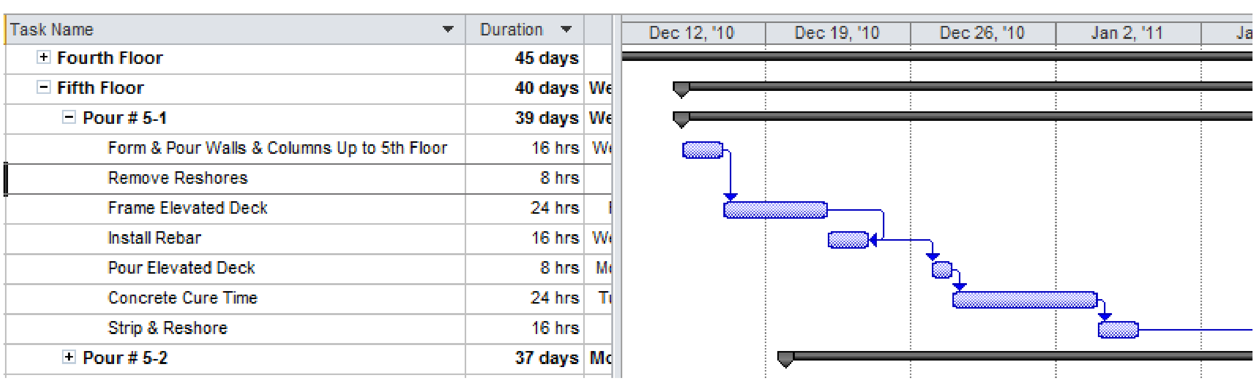
Both the alternative base steel redesign and steel progressive collapse redesign would have a significant impact on the overall cost of the project. To quantify this impact, a detailed cost estimate was constructed for the structural elements in both the base steel design and the progressive collapse design. These two quantities were then compared to the estimated existing structural cost. In addition a simplified construction schedule was developed for both redesigns and compared to the existing schedule. As a result of this study, a more in depth comparison can be drawn toward the effects of designing for progressive collapse.

11.1) EXISTING CONCRETE COST AND SCHEDULE

The existing structural costs were unavailable during the completion of this report. As such a rough estimation of the structural cost was made using square foot estimation and RS Means cost data. The existing two-way flat slab superstructure was found to be $10.50 per square foot. This estimate includes the location modifier for the Washington D.C. area. If the national cost averages were used, this estimate increases to $11.80 per square foot. A more detailed breakdown of this cost estimate can be found in Appendix L.

The existing construction schedule was obtained, courtesy of Randy Shumaker and DPR Construction. The schedule found in this report represented the original bid schedule, not the completed schedule. The schedule was simplified and can be found in Appendix M. The total superstructure was projected to take a total of 70 days, or 14 weeks. Each floor was formed and poured in two sections. Each section took a total of 11 days from stripping previous forms to pouring the slab. This resulted in the contractor being able to construct an entire floor approximately every 3 weeks. Figure 11.1 below shows the typical construction process for a single floor section.

Figure 11.1 – Portion of existing construction schedule



11.2) REDESIGN COST

A detailed cost estimate was conducted on both the base steel redesign and progressive collapse steel redesign to study the effects that progressive collapse requirements had on the building's cost. RS Means 2012 cost data was used for all calculations. The specific cost data used is available upon request. Material take-offs were only conducted using both RISA 3D and a created Revit Structure model.

A major factor that contributes to the cost and schedule of a project is welding. Moment frames were the proposed lateral system for both redesigns. Also, a large difference between the base and progressive collapse design was the number of moment connections. The progressive collapse design required considerably more moment frames, mostly along column line 1. To account for this, a typical moment connection was estimated using the results from Section 7.3. RS Means assumes standard pin connections on all beam members, therefore only the full penetration welds were labeled as additional costs. Welds were required on both top and bottom flanges, therefore difficult positional welding was taken into account. RS Means lists a labor increase of between 20% and 300% to be applied for positional welding. An increase of 50% on labor costs was used because, although the weld was overhead, it was not in a difficult location or shape. Table 11.1, below, illustrates the cost and time required to complete a typical moment connection on the base design. This exercise was performed for both the base design and progressive collapse design because the frame sizes increased, therefore the length of required weld increased.

Typical Moment Connection												
Connection Analysis			RS Means Cost Data								Results	
Size of Weld	Unit	Total Length of Weld in	Crew	Daily Output	Material Cost	Labor Cost	Positional Welding Increase	Equip Cost	O&P Cost	Total Cost	Number of Connections Per	Total Connection Cost
3/4"	L.F.	1.25	E-14	12	2.74	34	50%	10.15	28.11	92	9.6	$115.00

Table 11.1

The framing members and structural floor were also included in this estimate. The progressive collapse analysis required much larger column sizes. This resulted in a greater expense, but the schedule was not greatly impacted because this was just an increase in material. Another major difference between the two redesigns was the floor reinforcing. Due to Tie-Force requirements, the progressive collapse redesign required rebar in the slab, as opposed to welded wire fabric, which is much cheaper and faster to install.

Only superstructure was included in this estimate. It is expected that the foundations would decrease in size when the existing concrete building was changed to steel. This is because the steel structure weighed significantly less than the concrete structure. The existing structure used spread footings. The steel design would use the same system because it is the most inexpensive, therefore the cost savings would not be great. When designed for progressive collapse, the footing sizes are expected to increase in size and expense as compared to the base steel design. The increase is expected to be proportional to the increase in the column size because, like the columns, the foundations were largely force controlled and are primary elements.

Table 11.2, below, is an example of a typical floor estimate. The full cost estimate, for both base and progressive collapse redesigns, can be found in Appendix L. These tables were constructed using the national average costs. Only the final costs were multiplied by the location factor. It can be seen that the total cost was $125 thousand for the framing, $5.5 thousand for the rigid connections, moment frame and cantilever, and $50 thousand for the concrete on metal deck floor. When totaled, the base steel design was found to cost $2.15 million and the progressive collapse was found to cost $2.31 million. This results in an estimate square foot cost of $17.52/S.F. and $18.8/S.F., respectively. When adjusted for the greater Washington D.C. area, these estimates were $15.28/S.F. and $16.39/S.F, respectively.

Base Design Cost and Schedule

Floor	System	Material Take-Offs					RS Means Cost Data					Total Estimate	
		Member	Count	Unit	Total	# Studs	Crew	Daily Output	Cost (Incl O&P)	Studs Daily Output	Stud Cost (Incl O&P)	Total Cost	Total Schedule (Days)
Fifth	Beams	W12X19	12	L.F.	120.25	0	E-2	880	35.75	910	2.82	$4,299	0.14
		W14X22	20	L.F.	447.5	157	E-2	990	41.5	910	2.82	$19,014	0.62
		W16x26	2	L.F.	40.2	0	E-2	1000	46	910	2.82	$1,849	0.04
		W16X31	17	L.F.	414	290	E-2	900	54	910	2.82	$23,174	0.78
		W16x40	5	L.F.	141.85	45	E-2	800	68.5	910	2.82	$9,844	0.23
		W18x35	5	L.F.	126.75	63	E-2	960	62	910	2.82	$8,036	0.20
		W18x40	2	L.F.	44	34	W-2	960	69.5	910	2.82	$3,154	0.08
		W18x50	10	L.F.	221.85	0	E-2	912	85	910	2.82	$18,857	0.24
		W21x44	2	L.F.	56.75	56	E-2	1064	68	910	2.82	$4,017	0.11
		W21x55	4	L.F.	107.1	0	E-2	1064	92.75	910	2.82	$9,934	0.10
		W24x62	8	L.F.	194.3	0	E-2	1110	92	910	2.82	$17,876	0.18
		W24x76	2	L.F.	22	8	E-2	1110	111	910	2.82	$2,465	0.03
		W27x84	3	L.F.	26	0	E-2	1190	121	910	2.82	$3,146	0.02
		Total A992 Steel	92		1962.6	653						**$125,663**	**2.8**
	Connections	Moment Connections	44	Ea.			E-14	9.6	115			$5,060	4.6
		Cantilever Connections	4	Ea.			E-14	9.6	115			$460	0.4
		Total Rigid Connections										**$5,520**	**5.0**
	Deck	22 Ga 2"VLI		S.F.	9637.7		E-4	3560	2.24			$21,588	2.71
		6x6- W2.1xW2.1		C.S.F.	9.6377		2 Rodm	31	61			$588	0.31
		4500 psi Concrete		C.Y.	193.3		C-20		116			$22,423	
		Pump Placing, Elev. Slab		C.Y.	193.3		C-20	140	28			$5,412	1.38
		Total Floor System										**$50,012**	**4.4**

Table 11.2

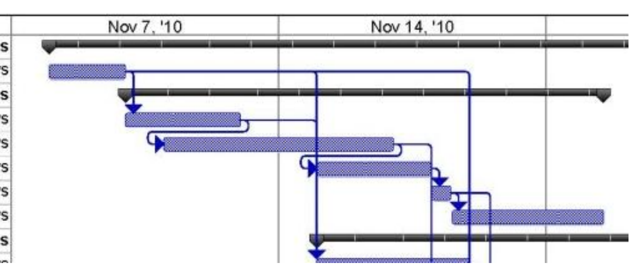
11.3) REDESIGN SCHEDULE

New construction schedules were developed using labor rates from RS Means. The schedule for each task was developed from the material takeoff and can be viewed in Table 11.2, above. Only the structural steel, rigid connections, and flooring were included in the schedule. Miscellaneous metals and other structural materials were not included. Refer to Appendix M for the complete schedules.

An abbreviated version of the base steel design's construction schedule can be seen in Figure 11.2. The full schedule can be seen in Appendix M. The base steel design was expected to take a total of 33 days. It was projected that the crews, if properly scheduled, could erect a typical floor in a week and a half. The larger, lower floors would require slightly longer time, but still approximately 2 weeks per floor. The entire structure was projected to take 5 weeks, which is a reduction of 5 weeks from the original, concrete, schedule.

Figure 11.2 – Portion of base steel redesign construction schedule

ID	Task Name	Duration
1	**Office Tower Structure**	**33 days**
2	Base Column Erection	2 days
3	**Second Floor**	**8.5 days**
4	Steel Erection	3 days
5	Rigid Connections	4 days
6	Place Deck	3 days
7	Place Reinforcing	0.5 days
8	Pour Concrete	2 days
9	**Third Floor**	**10 days**
10	Steel Erection	4 days



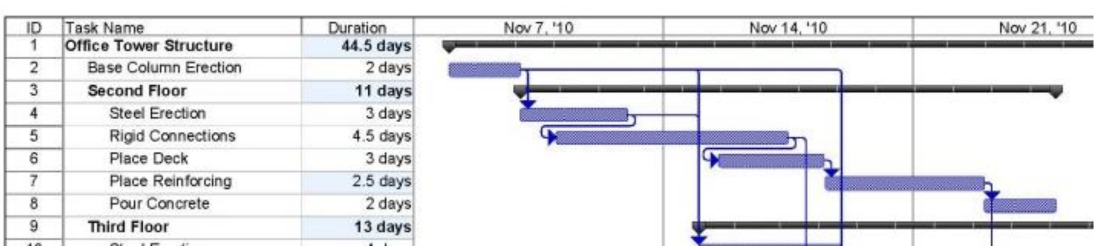
It was assumed that the moment connection welders would work half a day behind the steel erection crew. Having multiple welding crews for the moment connection should be strongly considered because the erection crews worked faster than the welding crews. One erection crew was expected to take only 3 days to complete an entire typical (5^{th}) floor. Whereas, the moment connections required 6 days to be completed by a single welding crew. The decking crews were also held up by this single welding crew. It was expected to take only 3 days to place the deck and reinforcing. The deck could not be placed until the connections were finished, therefore the decking crews would have been idle for between 3 and 4 days. Placing a second welding crew would cut the required time in half and would result in a streamlined construction crew with all trades working in unison.

The progressive collapse redesign was also scheduled using RS Means production data. This schedule can be seen in its entirety in Appendix M. An abbreviated schedule can be seen in Figure 11.3. The base steel and progressive collapse steel redesigns were very similar. The progressive collapse design was found to take a slightly longer period to construct. This is due to the larger members, the increased number of moment connections, and the slab rebar placement. The entire progressive collapse superstructure was predicted to take 6.5 weeks to complete. The crews could be expected to erect a typical floor in slightly more than two weeks. This is a 30% increase in time, as compared to the base steel design. With more trade coordination, especially at the penthouse and roof levels, this increase can be greatly reduced.

The base steel design used welded wire fabric for the slab on metal deck reinforcing, which is common practice. The tie force analysis required larger reinforcement, so #4 bars were specified instead of the W.W.F. It was found that the rebar was much more labor intensive and therefore the slab construction increased in duration. The W.W.F. required less than half a day to place for a typical floor. The #4's required slightly more than 2 days to place.

It was found that both the base steel and the progressive collapse redesign could be constructed in less time than the existing concrete structure. The base steel could have been completed in 5 weeks, which was 50% less time than the existing schedule. The progressive collapse redesign could have been completed in 6.5 weeks, which was 35% less time than the existing schedule. It should be noted that the redesigns' schedule do not include several things like miscellaneous metals and edge of slab conditions, which will increase the construction time. This decrease in schedule would result in cost savings for several areas, such as equipment rental and overhead.

Figure 11.3 – Portion of progressive collapse steel redesign construction schedule



12.) CONCLUSIONS

To explore progressive collapse and strength fundamental structural engineering design skills, a hypothetical redesign scenario was created for the General Office Building. In this scenario the new occupant required strict antiterrorism design, which followed the Department of Defense's Unified Facilities Criteria. These goals were met and all analyses met resulted in adequate results.

The existing General Office Building was located in the greater Washington, D.C. area. The structure was originally a two-way flat slab system with drop panels. Shear wall lateral systems were located in the interior of the building. No antiterrorism measures were directly considered in the original design. This paper attempted to keep as much of the existing architecture and program as possible, but some compromises had to be made.

The existing structure was redesigned using composite steel floor systems and steel moment frame later systems because this area of analysis was preferred. The base structural redesign was held to typical structural engineering codes and standards. ASCE 7-05 was used to determine all loading and criteria for the first redesign. The gravity members were designed with height limitations in mind, but the existing building height could not accommodate the change in material. As expected for the Washington, D.C. area, wind drift controlled the lateral system. The final design resulted in a high, but reasonable primary mode of 1.93 seconds.

The structure was then subjected to progressive collapse requirements, as defined by UFC 023. The Tie-Force analysis resulted in and increases slab reinforcing of #4's at approximately 12" on center in each direction. A large amount of time and analysis was given to Alternative Path Analysis. Linear static analysis procedures were conducted on various locations of the building. Additional exterior moment frames and roof structure were required for this analysis to meet the criteria set forth. Non-linear hand calculations were performed, which verifies the linear static computer output. A typical WUF moment connection was design using MAE course material. A ½" diameter cable support system was designed to resist equivalent blast pressures at the large atrium curtain wall. All exterior columns, except for the corner columns were found to meet Enhanced Local Resistance criteria. A 7/16" thick doubler plate was welded to the corner columns to ensure a ductile failure under lateral blast loads.

Both the architectural and construction management breadths focused on comparing the impacts of progressive collapse requirements to the existing and base steel designs. The cable façade supports were deemed acceptable architecturally, but the project location was deemed inadequate for standoff distance requirements. Both the base steel and progressive collapse steel redesigns were found to be more expensive and require less construction time, as compared to the existing structure. The cost of the superstructure was found to increase by 7.3% when progressive collapse requirement were added to the design criteria.

13.) BIBLIOGRAPHY

Arnold, Christopher, and Mary Ann. Lasch. *Site and Urban Design for Security: Guidance against Potential Terrorist Attacks : Providing Protection to People and Buildings.* [Washington, D.C.]: FEMA, 2007. Print.

Committee, 318. *Building Code Requirements for Structural Concrete*. [S.l.]: Curran Associates, 2008. Print.

Criste, Erin. "Beam Cambering Methods and Costs." *Structure* (2009). Print.

Horne, M. R. *Plastic Theory of Structures*. Oxford: Pergamon, 1979. Print.

Krishna, Prem. *Cable- Suspended Roofs.* New York [u.a.: McGraw-Hill, 1978. Print.

Minimum Design Loads for Buildings And Other Structures Sei/asce 7-05. Amer Society of Civil Engineers, 2005. Print.

Ricker, David T. "Value Engineering for Steel Construction." *American Society of Steel Construction* (2000). Print.

Specification for Structural Steel Buildings: March 9, 2005. 13th ed. Chicago, IL: Institute, 2005. Print.

Unified Facilities Criteria (UFC): DoD Minimum Antiterrorism Standards for Buildings. [Washington, D.C.]: Dept. of Defense, 2002. Print.