Final Report

Army National Guard Readiness Center
Arlington, Virginia

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April 7, 2010

Army National Guard Readiness Center

Arlington, VA

PROJECT TEAM

Location: Arlington Hall Station, Arlington, VA
Occupancy: Primarily Offices & Assembly
Contract Value: $100,000,000
Size: 251,000 sq ft
Levels: 8 Stories; 5 Above Grade
Construction Dates: Dec 2008-Jan 2011
Delivery Method: Design-Bid-Build

PROJECT TEAM

Owner: Army National Guard
General Contractor: Tompkins Builders, Inc.
A/E: DMJM H & N | AECOM
Geotech: CH2M Hill

ARCHITECTURE

The Army National Guard Readiness Center Addition is an 8 story Joint Headquarter Administrative building. The building is comprised of a 3 levels underground and a 5 story triangular tower. The facility will house special administrative areas within sensitive compartmented information facilities (SCIFs) as well as an auditorium, fitness center, general office space, conference rooms and a one story bridge connecting to the existing building. Physical security features have also been incorporated into the design including maximum standoff distances, internal bracing to prevent progressive collapse, blast walls, berms, and bollards. This facility is expected to achieve a LEED Silver certification rating.

STRUCURAL SYSTEM

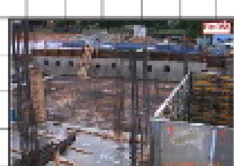
~Two-Way Concrete Slab on Metal Deck
~Reinforced Concrete Columns
~Spread Footing Foundation
~Shear Wall Lateral System

MEP SYSTEM

~Forced Air System using chilled and heated water
~AHUs on each floor supplied with 100% outdoor air
~Power via 34.5kV underground concrete ductbank
~Backup Generators and UPS
~Mainly fluorescent lighting with programmable controls

BUILDING ENVELOPE

~Battered & Ribbed Precast Panels
~Glazed Aluminum Curtain Wall System
~Single Ply Roofing for Tower
~Intensive Green Roofing System at Plaza



Amanda C. Farace | Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2010/acf5033

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Executive Summary

The Army National Guard Readiness Center addition is an eight story joint headquarters building located in Arlington, Virginia. The structure consists of a 43” mat foundation, flat slab concrete floor system with column strips and edge beams, ordinary reinforced concrete shear walls and various reinforced concrete columns. Typical interior columns are 22” by 22” with variations in reinforcement. Due to deviations in footprint between the sub grade levels and the tower levels, a 2” expansion joint is located in the 9” floor slabs on the sub grade levels. The building features telecommunication centers, joint operations center, general officer suites, an auditorium, conference rooms, gym area, and training rooms. The Army National Guard Readiness Center has unique triangular shape, which is emphasized with a glass-enclosed staircase in the northern corner that is topped with a distinctive steel tricorn.

For this thesis report, the goal was to investigate and discuss the affects of redesigning the structural system of the Army National Guard Readiness Center from cast-in-place concrete to steel framing. It was necessary to keep unique architecture and layout of the building relatively unchanged. A progressive collapse analysis was also necessary due to the national significance of the building. The Army National Guard Readiness Center was redesigned from the existing two-way concrete slab and ordinary reinforced shear walls to a reliable and efficient steel structural system. It was determined that composite steel beams and metal decking would be a viable alternative to the current concrete structure. Preliminary framing elements were sized using the AISC 13th Edition Steel Construction Manual and Vulcraft’s *Steel Roof and Deck Catalog.* A RAM model was then generated to optimize the structural system. Several lateral force resisting systems were also considered and after much research it was determined that a moment frames would be the most effective lateral system for this building.

Two breadth studies were also conducted for this report to determine how the structural redesign affects other aspects of the building. The first breadth topic was an acoustical study to analyze the transmission loss of the steel structure from the mechanical penthouse to the office areas on the 5T level. An area below two cooling towers was chosen as the focus for this study. Once the sound pressure created by the cooling towers was determined, the required transmission loss could be calculated. A new roof detail was designed and studied. It was concluded from this analysis that the steel deck and concrete thickness provided was adequate in providing the necessary transmission loss so it is anticipated that there will not be any acoustical issues in the spaces at the 5T level.

The second breadth study was a construction management analysis that performed to investigate and compare the cost and schedule of both the existing concrete structure and the proposed steel structure. Details takeoffs were used for both systems to determine a cost break down for the material, equipment, and labor costs using *R.S. Means Construction Costs Data.* Estimated schedules were generated using time acquired from labor crews and unit amounts. From this study it was concluded that the concrete structure could be constructed for less than the steel structure, however it was the steel structure that could be erected quicker.

ACKNOWLEDGMENTS

The author wishes to recognize the following individuals for their understanding, patience, and assistance in the completion of this thesis report. The completion of this report would not have been possible without their support.

Tompkins Builders, Inc

Samuel Gillespe
Chris Kulik
Cory Poet
George VanSanford
Everyone working on the Army National Guard Readiness Center Addition

Army National Guard

LTC Rodney Graham

The Pennsylvania State University

Thesis Consultants
- Dr. Thomas Boothby
- Professor Kevin M. Parfitt
- Professor Robert Holland

The entire Architectural Engineering Department and Professors at Penn State for their support and guidance over the years and especially throughout the completion of this report.

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A special thank you to family, friends, and peers for the much needed support and help they have provided. It would have been a difficult process without their love and understanding throughout this process.

INTRODUCTION

The ArNG headquarters addition is sited to the south of the existing facility, where the storm water retention pond had been located. Due to the loss of the retention pond, the project also includes the installation of storm water detention tanks. The new building is 82 feet above grade and approximately 251,000 square feet. The contract value was $100 million and is a Design-Bid-Build with Tompkins Builders, Inc., the general contractor, holding lump sum contracts with all subcontractors. The eight-story facility is comprised of 3 underground levels (Referred to as Levels 3P, 2P and 1P) and a 5 level tower component (Levels referred to as 1T – 5T) as well as a mechanical penthouse. The three underground levels account for the majority of the building's square footage, with a much larger footprint than the above ground floors. The underground encompasses approximately 150,000 square feet and the five-story tower encompasses 100,000 square feet. This design was developed to increase the amount of green space since a large portion of the underground levels will be topped with an intensive green roof system.

The addition is designed to meet Department of Defense Anti-Terrorism and Force Protection Requirements. This required that physical security measures, such as internal bracing to prevent progressive collapse, blast walls, berms, bollards and heavy landscape, to have been integrated into the design of the building. The facility is also expected to achieve LEED Silver Certification. LEED points are anticipated through the green roof system, offering bicycle storage and changing rooms, low-emitting and fuel efficient vehicles, reduction of water usage, water efficient landscaping, use of low-emitting as well as recycled and regional materials, and creating office space that can be 75% daylight. The building will incorporate open office spaces, general office suites, conference rooms, specialized compartmented information facilities, a fitness center, small library, and an auditorium.

As a result of the location and the existing facilities that are on site, several other entities have been incorporated into the project. This includes the installation of the storm water detention tanks, the relocation of an existing radio tower, relocation of existing gate, a one story bridge connecting to the new facility with the existing headquarters, construction of a new mailroom, and a construction of a new multi-story parking facility. This report will focus on the new Army National Guard Readiness Center Addition and none of the other project entities will be discussed of analyzed.

BACKGROUND

The Army National Guard (ArNG) Readiness Center is located at 111 South George Mason Drive in Arlington County, Virginia. The site is bordered on the east by the U.S. Department of State, National Foreign Affairs Training Center, on the north by Arlington Boulevard, on the west by George Mason Drive, and on the south by a residential community. The fifteen-acre site is comprised of a 248,000 square foot headquarters facility, two 3-story parking garages and several small outbuildings.

The Army National Guard Readiness Center houses administrative and resource functions that provide support and liaison to the National Guard in all 50 States and requisite territories and to the Pentagon. Currently there is about 1,300 staff based at this facility. The 2005 Base Realignment and Closure Act (BRAC) actions required the realignment of Jefferson Plaza 1 in Crystal City by relocating National Guard Bureau Headquarters and Air Force Headquarters to the Army National Guard Readiness Center in Arlington and to Andrews Air Force Base, in Maryland. This means the relocation of more than 1,200 National Guard Bureau Joint staff and Army National Guard staff to relocate to the Readiness Center. This relocation has created a great need for a Readiness Center Addition. Due to the BRAC Requirements the 1,200 personnel must be relocated before 2011. This makes the construction schedule particularly crucial.

Figure 1: West Perspective

GENERAL BUILDING DATA

Building Location:
111 S. George Mason Drive
Arlington Hall Station
Arlington, Va 22204

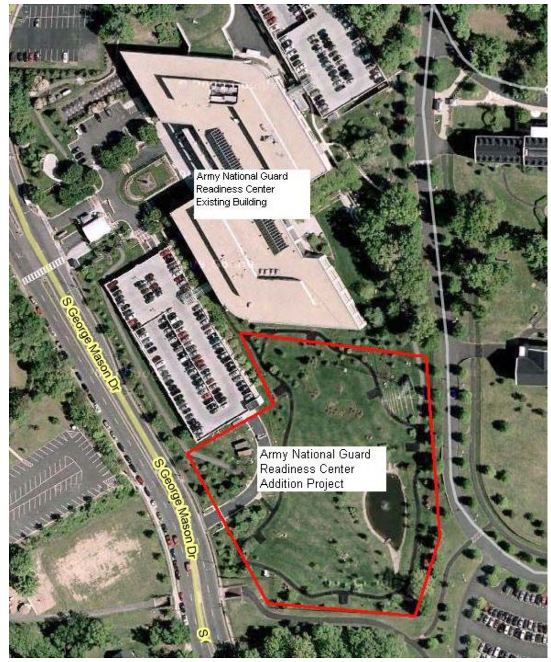

Figure 2: Google Image - ArNG Site Location

Building Occupancy:
Main – Class B (Business)
Accessory – A-1 (Assembly)

Building Function:
Joint Headquarters Administrative Building

Size: 251,000 Sq Ft

Number of Stories:
8 Levels – 3 below grade and 5 above grade
(Not including mechanical penthouse)

Dates of Construction:
December 2008 – January 2011

Cost Information:
Contract Value - $100,000,000

Project Delivery Method:
Design-Bid-Build with Lump Sum

Project Team		
Owner	Army National Guard	www.nationalguard.com
General Contractor/CM	Tompkins Builders	www.tompkinsbuilders.com
Architect/Engineer	DMJM Design H&N, Inc.	www.dmjmhn.aecom.com
Architect/Engineer	AECOM	www.aecom.com
Geotechnical Consultant	CH2M Hill	www.ch2m.com
Landscape Architect	LPDA Associates	www.lpda.net
Testing & Inspection	ECS Mid-Atlantic, LLC	www.ecslimited.com

Zoning:
The Army National Guard Readiness Center site is located within Arlington County's District S3-A special zoning district. This district designates land that has distinct and unique site advantages or other desirable features. Uses permitted include, but are not limited to, public parks, playgrounds, recreational areas, public buildings, cultural properties, and service buildings.

EXISTING CONDITIONS

Architecture

The Army National Guard Readiness Center Addition is an 8-story Joint Headquarters administrative building addition. The building is compromised of a Plaza component, which consists of three levels below ground level, and a Tower component, which consists of five levels above ground as well as a mechanical penthouse. The building will also feature emergency power generator and uninterruptible power supply (UPS) backup, special administrative areas within sensitive compartmented information facilities (SCIFS) and telecommunication center; joint operations center, general officer suites, auditorium, conference rooms, and training rooms. The addition embraces some of the same architectural features of the existing facility and will be comprised of mainly precast concrete panels and glass façade. The project also includes a one-story bridge connecting the new facility with the existing. Physical security measures have also been incorporated into the design including maximum standoff distance, internal bracing to prevent progressive collapse, blast walls, progressive collapse mitigation, berms, and bollards. The building is topped with a unique steel Tricorn.

Building Enclosure

- Façade
  - The façade of the Army National Guard Readiness Center Addition is comprised of a unique combination of battered and ribbed precast concrete panels as well as a glazed aluminum curtain wall system. The exterior architecture of the addition will mirror the architecture of the existing building. The curtain wall panels are constructed with clear glass, fritted glass, and spandrel glass. The curtain wall is attached to the edge of a concrete slab on metal decking floor system. The design had to compliant with the requirements of Force Protection Building classification and standoff distances as well as Department of Defense requirements. Therefore, the glazing, metal wall panels, and frames must work as one unified system to ensure that the hazard mitigation is effective.

- Roofing
  - There are two roofing systems used on this project, the main roofing of the tower and the green canopy at the plaza level. The main roofing system is rigid insulation topped with ballast over a single-ply waterproofing membrane. The roofing material has not yet been finalized but is expected to have an Solar Reflectance Index (SRI) equal to or greater than 78 to meet LEED standards. The flashing will be the same membrane as the single-ply membrane material. The parapet wall sheathing will be a glass-mat gypsum wall sheathing. The intensive green roof area is designed to sustain a wide variety of plant species including shrubs and small trees. This roofing includes a rubberized membrane that contains an inert clay filler and crumb rubber that enables the product to be resistant to acids from fertilizers. It will also contain polyester fabric reinforcing sheet, reinforced flashing membrane, a fiberglass root barrier protection course, a water

retention mat, and filter fabric below an appropriate thickness of soil to sustain the growth of approved vegetation.

Sustainable Features

The Army National Guard Readiness Center addition is expected to achieve a LEED Silver certification level. This will be accomplished through multiple methods. Starting with site conditions such as strict erosion and sediment control standards and a detailed construction waste management program that comply with LEED standards. Sustainable materials will be utilized throughout the building. Premium wood, carpet systems, and applied coatings are required for the building finishes, which must be low emitting. The building will also include a chilled water system and an air-to-air energy recovery system. The addition utilizes natural daylighting as well as automatic lighting shades and lighting technologies such as Solarscreen coated glass. An intensive green roof area is also a main feature of the building. It is located near the plaza area and will be accessible.

Codes & Standards

The following documents were either furnished for review or otherwise considered for this report:

- ACI 318-08 *Building Code Requirements for Structural Concrete* published in January 2008 by the American Concrete Institute

- AISC 13th Edition (LRFD) *Steel Construction Manual* Published in December 2005 by the American Institute of Steel Construction, Inc.

- ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* published in 2006 by the American Society of Civil Engineers

- IBC 2006 *International Building Code* published in January 2006 by the International Code Council, Inc.

- *Notes on ACI 318-08 Building Code Requirements for Structural Concrete* Published in 2005 by the Portland Cement Association

- Construction Documents originally dated August 25, 2008 by DMJM H&N, Inc.

Deflection Criteria

Floor Deflection Criteria:

Typical Live Load Deflection limited to L/360

Typical Total Deflection limited to L/240

Maximum Deflection limited to ¾"

Lateral Deflection Criteria:

Total Story Wind Drift limited to H/400

Total Allowable Seismic Drift limited to $0.020h_{SX}$

Material Specifications

These materials, their grades, and strengths were the materials that the current Army National Guard Readiness Center Addition is utilizing. All materials were listed on the drawings, general notes, of the specifications. These material properties are summarized in the following table.

Material Properties		
Material	**Grade**	**Strength**
Concrete		
Foundation	-	$f'_c = 4,500$ psi
Slab on Grade	-	$f'_c = 4,000$ psi
Columns	-	$f'_c = 4,000$ psi
Shear Walls	-	$f'_c = 4,500$ psi
Floor Slabs	-	$f'_c = 4,000$ psi
HSS Rectangular	A500 - Gr. B	$F_y = 46,000$ psi
HSS Circular	A500 - Gr. B	$F_y = 46,000$ psi
Reinforcing Bars	ASTM A615 - Gr 60	$f_y = 60,000$ psi
Steel Deck	ASTM A625 - Gr. 33	$F_y = 33,000$ psi
CMU	Type 1 - Gr. N Med Wt.	$f'_m = 2,500$ psi
Grout	C270 Type S	-

Table 1: *Material Properties*

Building Systems

- Construction
  - Excavation for the Army National Guard Readiness Center Addition began on December 1, 2008 and the anticipated substantial completion date is January 2011. Tompkins Builders, Inc., a subsidiary of Turner Construction Company is the General Contractor. The project delivery method is Design-Bid-Build with the General Contractor holding lump sum contracts with subcontractors. The Army National Guard Site is an expansive fifteen acres however, with the layout of the existing facility, two parking structures, and the simultaneous construction of a third parking structure, the actual site space is limited. This makes efficient coordination between subcontractors and the General Contractor crucial tower cranes were also utilized to help mitigate coordination issues and reduce traffic maintenance around the site.

- Mechanical System
  - Air-handling units (AHU's) are located in mechanical rooms on every level ranging from 1500 cfm to 2450 cfm. Individual variable air volume (VAV) terminals and fan coil units (FCUs) are also appropriately dispersed throughout each floor to control heating and ventilation in different spaces. A hydronic HVAC system distributes water to AHUs and VAV terminals on each floor as well as the energy recovery units in the mechanical penthouse level. The hydronic HVAC system consists of a 4-pipe heating and chilled water system. AHUs and VAVs are supplied by 100 percent outside air. A Building Automation System (BAS) regulates all individual units while monitoring the temperature in each space and controlling the FCUs. Emergency backup generators are also located in the mechanical penthouse with the energy recovery units.

- Electrical/Lighting Systems
  - Dominion Power Company supplies power to both the existing facility and the new Army National Guard Addition. It comes into the site at 35.4 kV and is stepped down by a switchgear and is then supplied to the building with 2 main feeders at 15 kV each. Within the main building, the feeders connect to substations and after being stepped down again to 480/277V 3 phase, 4-wire system is distributed throughout the building. Emergency energy is supplied by two 1500 kW, diesel powered generators located on the penthouse level down seven stories and cuts east-west at the second story to supply emergency power to the substation.

    The lighting elements throughout the new facility will be either fluorescent lamps (277 V) or incandescent lamps (120 V). The lighting system is fed by 208/120V 3 phase, 4 wire panel boards. Automatic controls cover most of the building with the exception of office spaces. The open office areas will be controlled by programmable lighting fixtures, which are provided in some of the smaller offices, located around the operations center.

- Transportation
  - There are two elevator pits in the new Army National Guard facility. Altogether there will be six- (6) machine room less (MRL) elevators. Three of the MRL elevators will be

gearless service elevators (cars 4-6) and the other three are passenger elevators (cars 1-3). All elevators service every floor and car 5 also services the penthouse level. Each elevator runs 350 feet per minute. The passenger cars have a platform size of 6'-8" Wide by 5'-5" Deep and the service elevators are 5'-8" Wide by 7' – 10 ½" Deep.

There are three stairwells in the building, two of which extend from the lowest level all the way to level 5T. The third set of stairs only reaches the three underground floors. Stair number 2 is the main stairwell. It is centrally located on the plaza levels and in the tower levels it is along the southern point of the triangular building. It also is a unique architectural feature with its triangular shape, complete glass enclosure and topped off by the steel tricorn above the building. This stair is to be constructed of structural steel and must be completed in conjunction with the cast-in-place concrete construction. This requires a great amount of coordination between subcontractors and the General Contractor.

- Telecommunication
  - The Army National Guard Readiness Center houses administrative and resource functions that provide liaison and support to the National Guard in all 50 States and to the federal government. This requires multiple communication systems that are extremely secure and therefore there are at least two IT/Telecommunication rooms on each floor. There will be 100% access flooring in all IT/Telecommunication rooms, conference rooms, and offices spaces to simplify coordination and removes items from already cramped ceiling spaces. All the telecommunication systems will be fed through floor boxes installed in the access flooring system.

- Fire Protection
  - Water services are available from two existing fire hydrants located on the West side of the building along George Mason Drive and at the northeast corner between the new and existing facility. The hydrants provide 1520 gallon per minute flow rate. The building was designed for both light hazard areas, which require 0.20 GPM over 3,000 sq ft. Most of the building is sprinkled with an automatic wet-pipe system with concealed sprinklers and vertical sprinkler risers in the stairwells. FM 200 system, which is a clean system, is used in the main server room where extremely sensitive electrical equipment is stored that would be easily damaged by water. The FM 200 system is a colorless, non-toxic gas stored in two 300-gallon cylinders, which will release into the room and extinguish the fire within 10 seconds of detection. All stairwells and elevator cores are two-hour fire rated as well as the provided areas of refuge. The corridors, mechanical rooms, electrical rooms and IT/Telecommunication rooms are all 1 hour fire-rated. The building is also fitted with a digital, addressable fire alarm system which will have manual station, heat detectors, duet smoke detectors, verified automatic alarm operations, automatic alarm operations, automatic sprinkler system water flow, fire extinguishing systems, and fire standpipe system.

- Security
  - Due to the sensitive nature of this building advanced security systems were a necessary part or the design of the Army National Guard Readiness Center Addition. Part of this

system includes intrusion detection. This protection will detect intrusion through protected areas throughout the building as well as through the building envelope. It also covers surge protection to sensitive equipment, card key access to secure areas and controllers, annunciators, pull boxes and other system components.

EXISTING STRUCTURAL ELEMENTS

Foundation

CH2M Hill performed the geotechnical report engineering survey on April 21, 2008. In this study, it was found that a relatively high water level of approximately 6 feet to 10 feet below the existing surface was anticipated. As much as 35 feet of excavation was required to reach the building grades therefore, drilled in soldier piles with wood lagging and tied-back anchors was recommended for temporary excavation support as well as the installation of dewatering well points. CH2M Hill noted that, with proper ground water management and control, the existing subsurface is suitable for support of the building using a mat foundation system based on evaluation of allowable bearing capacity and anticipated settlement. The recommended allowable bearing capacity for the new building location was 4800 lbs/ft^2 for a mat footing. As a result a 43-inch concrete mat foundation was designed.

Columns

A reasonably consistent column layout exists throughout the building even with the changes in the shape of the floors between level 3P and 1T. The typical interior gravity column is a 22-inch by 22-inch, reinforced normal weight concrete column. The strength of all columns is 4,000 pounds per square inch. While the size and shape of the column is monolithic on each floor, there are three changes in reinforcement. For levels 3P to 1P columns are reinforced with sixteen No. 10 vertical bars. These change after the 1P level where the tower component of the building begins. For levels 1T and 2T columns are reinforced with sixteen #8 vertical bars. The reinforcement changes again at the 3T level up to the 5T level; these columns are reinforced with eight #8 vertical bars. #3 ties are located 12 inches on center at every level.

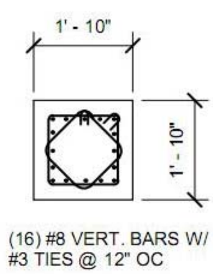

Figure 3: *Typical Interior Column*

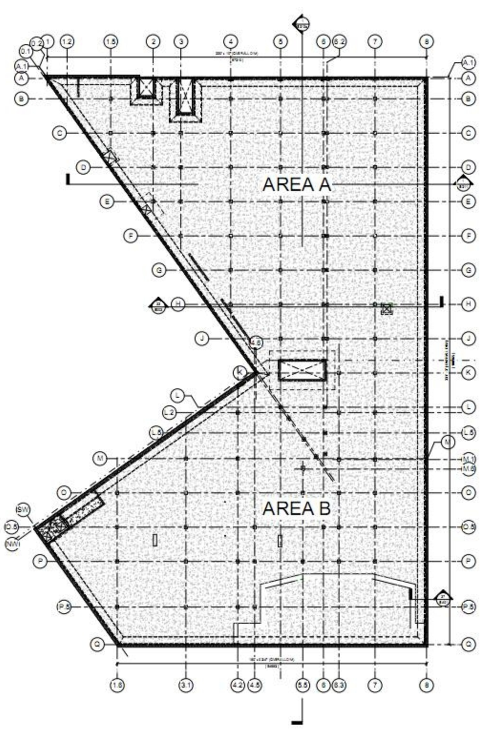

Figure 4*: Typical Column Layout for Underground Levels*

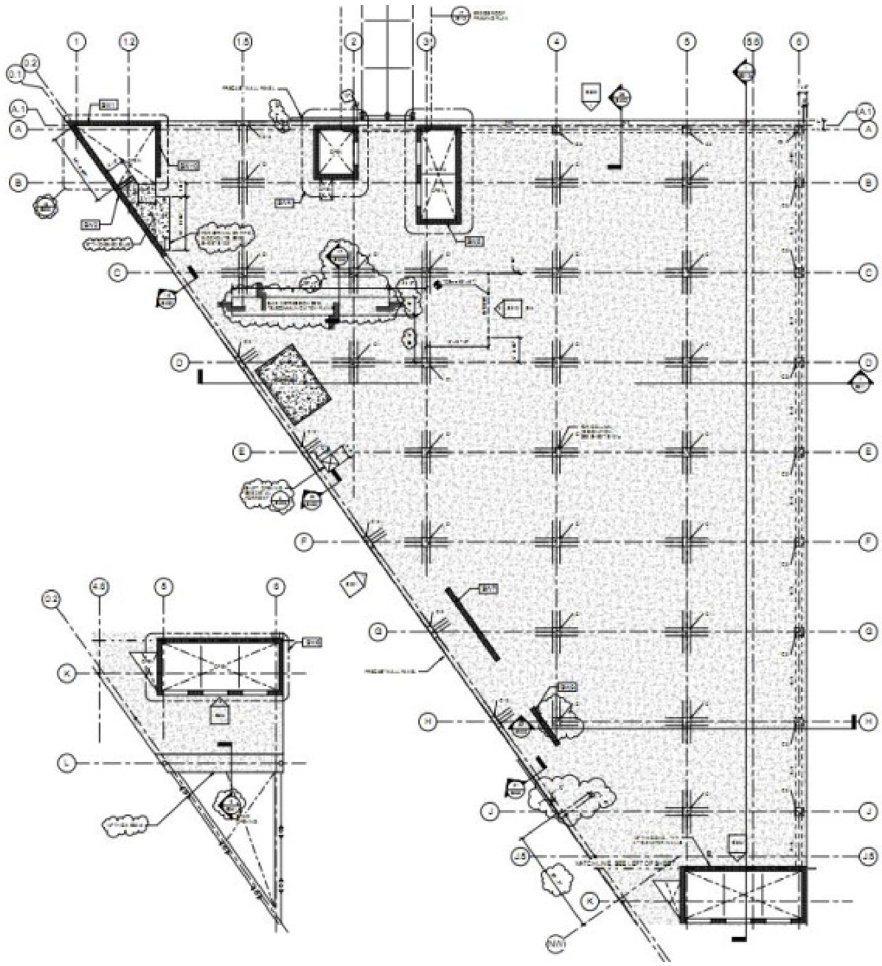

Figure 5: *Typical Column Layout for Tower Levels*

Floor Systems

The Army National Guard Readiness Center Addition utilizes a concrete structural system. All of the floors are a two-way flat slab with column strips and edge beams along the eastern and northern walls of the Tower component. The typical concrete strength is 4,000 pounds per square inch. The typical slab thickness is nine inches however; this changes in areas where the access flooring changes and for drainage areas in mechanical and electrical rooms. No. 6 and No. 8 bars are typically used for reinforcement in the floor systems.

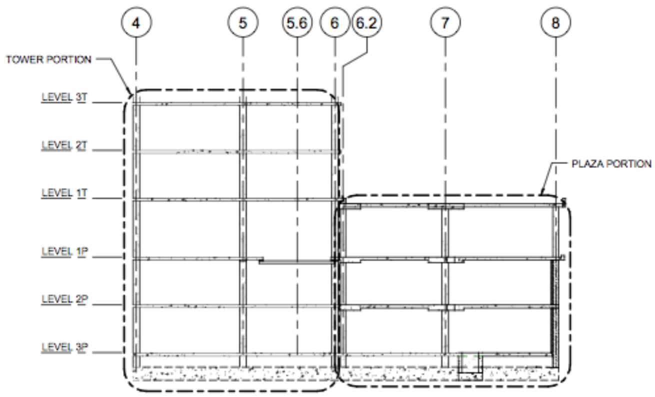

Figure 6: *Elevation showing location of expansion joint and relationship between Plaza portion and Tower portion*

Due to the irregular shape of the building and the change in shape from the underground portion of the building to the tower component, a two-inch expansion joint is located at the 3P to 1T levels along column line 6.2. This expansion joint makes the building act as almost two separate building, the tower portion and the plaza portion. The tower portion extends from level 3P to 5T while the plaza portion is comprised of the sub grade levels and topped of with an intensive green roof.

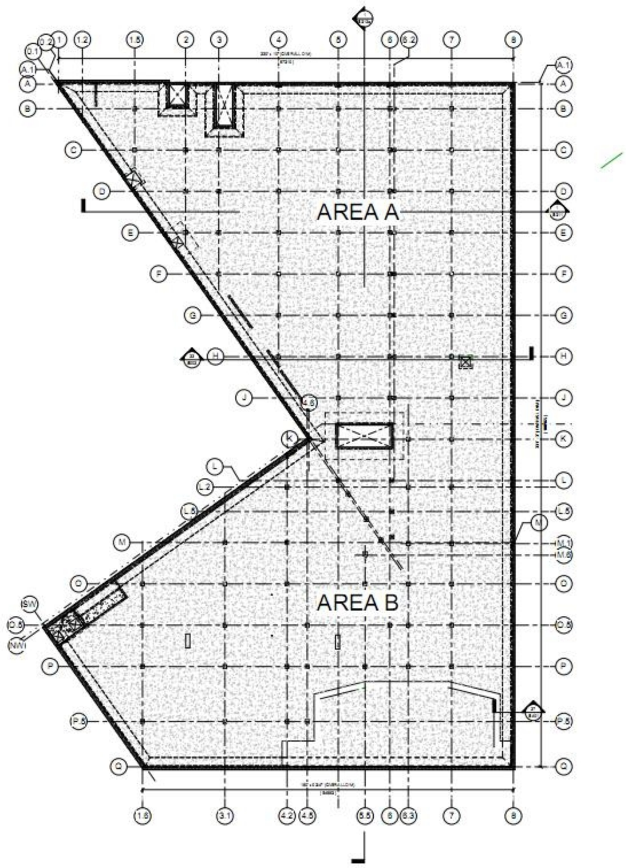

Figure 7: *Location of Expansion Joint*

Roof Systems

The penthouse roof of the tower is a two-way flat slab. The slab is 10" thick with a concrete strength of 4,000 pounds per square inch. This roof was designed to hold a 30 pounds per square foot snow load and is reinforced with #5 bars at 12 inches on center and 18 inches on center. A large skylight over the northern stairs required steel framing, which consists of beams ranging from W12x14 to W12x26.

The plaza roof is also a two-way slab with drop panels. The slab thickness ranges from eight inches to sixteen inches with a concrete strength of 4,000 pounds per square inch. This roof will act as an intensive green roof and therefore had to be designed to carry a 100-pound per square foot roof garden load. It is reinforced with #6 bars and includes a two-inch expansion joint where the roof abuts the floor of the first tower level (1T), as do the floors below.

Lateral System

The lateral system for the ArNG Readiness Center consists of reinforced concrete shear walls. These walls have a thickness of twelve inches and a concrete strength of 4,500 pounds per square inch. The numbers of shear walls varies between levels due to the building's change in footprint. Typical shear wall locations can be seen in figures 10 and 11 below. This system resists lateral loads in the north-south and east-west direction depending upon the orientation of the wall.

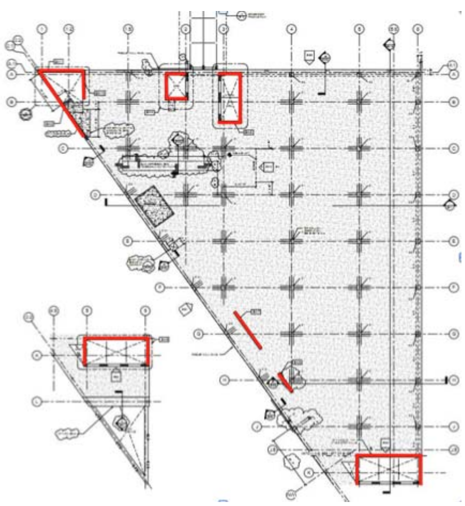

Figure 8: *Typical Shear Wall Locations in Below Grade Levels*

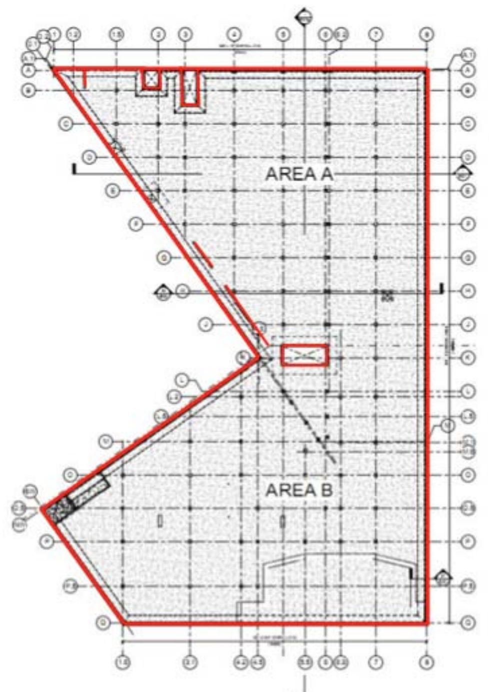

Figure 9: *Typical Shear Wall Locations in Tower Levels*

PROPOSAL SUMMARY

Proposal Statement

The Army National Guard Readiness Center Addition was designed as a cast-in-place concrete building with two-way flat slabs and normally reinforced shear walls. From the analysis in previous technical reports, it was confirmed that the structural elements were sufficiently designed to carry the gravity and lateral loads. Another main consideration for structural engineers during the design process was the possibility of terrorist attacks and other threats due to the significance of this building. Progressive collapse design and blast loading have become standard practice for structural engineers designing government buildings and high-rise structures since the September 11th attacks. For the purpose of this thesis, a new steel structural system will be designed for the Army National Guard Readiness Center. Blast loading and progressive collapse design will also be investigated.

Proposal Solution

For the purpose of this thesis, the Army National Guard Readiness Center Addition will be redesigned as a steel framing system and will include an analysis of progressive collapse and blast loading. Only the ground level will be considered for the blast loading analysis due to ease of accessibility. The redesign will include both a gravity and lateral system.

The proposed gravity system will be a composite metal deck flooring system with steel framing. A composite metal deck will be an advantageous choice due to its utilization of the benefits from both the steel and concrete. The deck will act as permanent formwork and the interlocking design of the steel and concrete will allow the concrete to carry the appropriate loading while the steel deck will serve as reinforcement.

Various lateral systems will be investigated before an appropriate system is chosen for the redesign of the lateral system. An optimum layout will be determined based on the torsional effects created by the wind and seismic loads. The proposed structural changes from a concrete system to a steel system will result in a lighter structural system overall and therefore an analysis of the foundation will be required. In Technical Report III it was determine, by inspection, that uplift and overturning were not issues for the current concrete structure due to the weight and soil friction. However, with the reduction of weight when changing from concrete to the steel structure, overturning and uplift will need to be investigated to determine if the current flat slab foundation must be redesigned for the proposed system.

Solution Methods

The new gravity system of the Army National Guard Readiness Center Addition will be considered first in the redesign. Vulcraft's *Steel Roof and Floor Deck* design manual will assist in the proposed floor design. Beams, girders, and columns will be sized using the 13th Edition of AISC's Steel Construction Manual. The live loads used in the redesign process will be taken from chapter 4 of ASCE 7-05. A model will be generated using RAM Structural System to help analyze the proposed framing system. Hand calculations will be performed to compare sizes of members determined by the RAM model.

The proposed lateral system will be considered next in the redesign process. Multiple lateral systems will be investigated and the most efficient system for this project will be chosen and designed. As in previous reports, the lateral system will be designed using standards set forth by ASCE 7-05. Load combinations will be taken from chapter 2, chapter 6 will be used for wind loading, and seismic loading will be determined using chapters 11, 12 and 22 of ASCE 7-05. The RAM model will also assist in the lateral system design and will be utilized to control the design and obtain the forces so the drift of the building meets the requirements of ASCE 7-05.

The progressive collapse analysis will include the hypothetical loss of a primary structural element. Critical perimeter and interior members will be 'lost' and a structural analysis will be performed without the critical component. Since he structural element was 'lost' due to a blast or destructive means, specific loading criteria will be utilized for the structural analysis. The results of the analysis will be compared to the ultimate strength of the structural system to determine its efficiency if there were to be an attack or damage to the structural system. Department of Defense Guidelines for blast loading and for progressive collapse design will be researched. The "GSA – Progressive Collapse Guidelines" will be exploited for the progressive collapse and blast analysis.

Breadth Topics

Acoustics Study

Noise transmission and potential acoustical issues could arise with the introduction of a steel structural system to the layout of the Army National Guard Readiness Center Addition. The proposed steel framing system will reduce the concrete thickness of the penthouse floor causing increased vibration of the floor from the mechanical equipment. The increased vibrations could potentially lead to increased noise levels transferred from the mechanical penthouse level to the office spaces located on the 5T level directly below the mechanical penthouse. "Architectural Acoustics" by M. David Egan was referenced for this study. The acoustical analysis will determine the sound pressure levels of the mechanical equipment located on the penthouse level and then the sound transmitted into the office spaces on the 5T level will be calculated to determine if they are appropriate. If necessary, additional acoustical materials will be introduced to keep the sound level within an acceptable range for office spaces.

Construction Management Study

This breadth topic will focus on the scheduling impact and cost-related issues that will be affected by the proposed structural changes. The study will include complete investigation into the cost and construction methods in order to compare the proposed steel system with the existing concrete structure. A construction schedule will be generated for the steel system. The critical path of the proposed construction schedule will then be compared to the critical path for the existing concrete structure to determine major scheduling impacts of the structural changes. A cost analysis of changing the structural system will also be conducted. The detailed cost analysis will be performed using R.S. Means. The goal of this study is to determine if the proposed structural steel system is an economical and efficient alternative to the existing concrete structural system.

DESIGN GOALS

To determine the feasibility of changing the structural system of the Army National Guard Readiness Center form a cast in place concrete system to steel framing system, a set of goals and criterion was created. The validity of the proposed changes will be tested against the set goals and criterion. This will assist in determining the final recommendations at the end of this report.

- Provide a structural steel solution that has little to no effect on the existing architecture of the Army National Guard Readiness Center since the aesthetics are a crucial element of the building
- Choose a single lateral system that will work effectively for this building
- Provide a steel solution that will reduce the overall building costs
- Reduce the construction schedule by designing a steel structure that can be erected more efficiently than the current concrete design
- Design the structural steel system for progressive collapse mitigation

GENERAL CRITERIA

Deflection Criteria

Live Load Deflections

- Typical live load deflections limited to: L/360
- Typical total deflections limited to: L/240
- Maximum deflections limited to: ¾”

Lateral Deflections

- Total allowable wind drifts limited to: H/500
- Total story wind drift limited to: H/400
- Total allowable seismic drift limited to: $0.015h_x$

Load Combinations

Referencing ASCE 7-05 Chapter 2, the following combinations were considered for combining factored loads for gravity and lateral load analysis. In general, load combination two normally governs for gravity load analysis. Depending on the magnitude of the lateral loads, load combinations four or five may govern. All of these combinations are based on the LRFD design method.

1. 1.4(D+F)
2. 1.2(D+F+T)+1.6(L+H)+0.5(L_r or S or R)
3. 1.2D+1.6(L_r or S or R)+(L or 0.8W)
4. 1.2D+1.6W+L+0.5(L_r or S or R)
5. 1.2D+1.0E+L+0.2S
6. 0.9D+1.6W+1.6H
7. 0.9D+1.0E+1.6H

STRUCTURAL DEPTH

Introduction

The Army National Guard Readiness Center was designed as a cast-in-place concrete structural system with two-way flat slab flooring and ordinary reinforced concrete shear walls. Long spans and multiple bay sizes make up the unique triangular shape of the Army National Guard Readiness Center. In considering an appropriate structural solution for the redesign, it was determined that a steel system will be most appropriate. Steel was chosen because of its numerous benefits such as short erection time, lower building weight, and high tensile strength. A composite steel framing system was chosen out of the possible steel solutions. A composite metal decking floor system will keep the ceiling cavity at an acceptable depth allowing for an ideal floor to ceiling height. This system will also be able to stretch the long spans and have a minimal impact on the existing architecture of the building. Possible steel lateral systems will be investigated to determine a steel lateral system and layout that would be most feasible for the Army National Guard Readiness Center. A progressive collapse analysis must also be performed to determine if the proposed structural steel system can mitigate progressive collapse in the event of a blast or forced damage. Results from the study of the steel system will then be compared to the existing cast-in-place concrete system. The feasibility of the steel structure will be determined from this comparison.

Figure 10: *Composite Metal Floor System*

Design Implications

Changing the structural system of the Army National Guard Readiness Center to a steel framing system will most definitely induce several other changes in the building that must also be taken into consideration. One of the main changes that will have several effects on the remainder of the building is the overall weight. The existing concrete structure will outweigh the proposed steel structure. The building's foundation will be one of the features impacted by the change in the structure's weight. With the reduced weight, uplift and overturning could pose a few problems with the foundation design and must be checked once the structural system is redesigned. The reduction in weight will also change the seismic loads. The seismic loads will reduce with the weight of the building. The loads could potentially decrease enough for the wind loading cases to control the lateral design of the building therefore the seismic loads must be recalculated and compared to the wind loads. Impacts of the structural change to the cost and scheduling should also be thought-out. Proposed steel system could potentially induce issues with sound transmission in the office spaces below the mechanical equipment, which needs to be considered. Changing the system to steel will also create issues with fire protection. Fire proofing materials and techniques will need to be considered and included in the changes to the cost and scheduling.

Design Procedures

The redesign of the Army National Guard Readiness Center as a structural steel system followed the same column grid and bay size since the W-shapes were capable of spanning the necessary lengths. Live loads were determined first according to Chapter 4 of ASCE 7-05. The metal deck was then designed based on the determined live and dead loads and utilizing Vulcraft's *Steel Roof and Floor Deck Design Guide.* Initial composite beam and girder sizes were designed to support the deck by performing hand calculations. Gravity columns were also initially sized for the Army National Guard Readiness Center redesign. Once initial sizes were determined a computer model was generated using RAM Structural System. This computer model was utilized to create a typical floor plan and structural layout. The member sizes that were designed by the computer program were compared to the initial sizes found from the hand calculations. Overall, the number of shear studs, beam sizes, and depths found by RAM resembled those found using hand calculations relatively closely.

The lateral system was designed next. Lateral design loads were derived using Chapter 6 for wind and Chapters 11, 12 and 22 for seismic from ASCE 7-05. It was determined that special moment frames would be utilized as the lateral system for the steel redesign. This was determined due to its minimal impedance on the architecture of the building as well as the moment connections, which will assist in progressive collapse mitigation. Moment frames were placed at the exterior of the Army National Guard Readiness Center and member were designed using RAM Structural Systems. The member sizes were checked using hand calculations. Drift and other serviceability criteria were checked last along with the foundation design.

Gravity and Lateral Loads

Live Loads

The live loads for the Army National Guard Readiness Center were calculated in accordance with IBC 2006, which references ASCE 7-05, Chapter 6. The loads that were determined from these references are noted in Table below.

Live Loads		
Occupancy	Design Load	ASCE 7-05 Load
Office	50 psf + 15 for partitions	50 psf
Lobbies	100 psf	100 psf
First Floor Corridor	100 psf	100 psf
Corridors (Above First Floor)	80 psf	80 psf
Fitness Center	100 psf	100 psf
Roof	20 psf	20 psf
Roof Garden	100 psf	100 psf

Table 2: *Live Loads*

Dead Loads

The dead loads used for the design of the Army National Guard Readiness Center were noted on the structural drawings for this project. These occupancy types and loading are summarized in Table below.

Dead Loads	
Typical Floor Dead Loads	
Occupancy	Design Loads
6" Raised Floor	43 psf
24" Raised Floor	20 psf
Slab + Deck	75 psf
MEP/Ceiling	15 psf
CMU Partitions	Actual Weight
Typical Roof Dead Loads	
Occupancy	Design Loads
Slab + Deck	75 psf
MEP/Ceiling	15 psf
Roofing Finish	4 psf

Table 3: *Dead Loads*

Snow Loads

The flat roof snow load for the Army National Guard Readiness Center was calculated in accordance with Chapter 7 of ASCE 7-05. A summary of the snow load factors that were used can be found in the table below.

Snow Load Criteria	
Ground Snow Loads	P_g=25 psf
Snow Exposure Factor	C_e=0.9
Snow Importance Factor	I=1.2
Thermal Factor	C_t=1.0
Flat Roof Snow Load	p_f=19 psf
Minimum Required Load	Pf=20 psf

Table 4: *Snow Loads*

Wind Loads

In accordance with IBC 2006, the provisions of ASCE Chapter 6 determined the wind loads on the building. To examine the lateral wind loads in both the North/South and East/West direction, Method 2, the analytical method, was used. From Figure 6-1 in ASCE 7-05 it was found that the basic wind speed in Arlington, VA was 90 mph. This method does not take into account any apparent shielding afforded by other building to reduce wind velocity. This could be crucial due to the relative proximity of the new facility with the existing structures that surround the building. For this report, a few assumptions were made to simplify the procedure. The main assumption was the Army National Guard Readiness Center Addition was to be considered a regular-shaped building. Using the commentary within ASCE 7-05 the approximate fundamental frequent of the building was calculated. It was determined from this that the building is flexible in nature and the Gust Factors were calculated accordingly (Refer to Appendix B for calculations). Figures below summarize the story forces and shear in both the north-south and east-west direction. Appendix B contains detailed spreadsheets, calculations, and criteria that were determined to ascertain the wind forces.

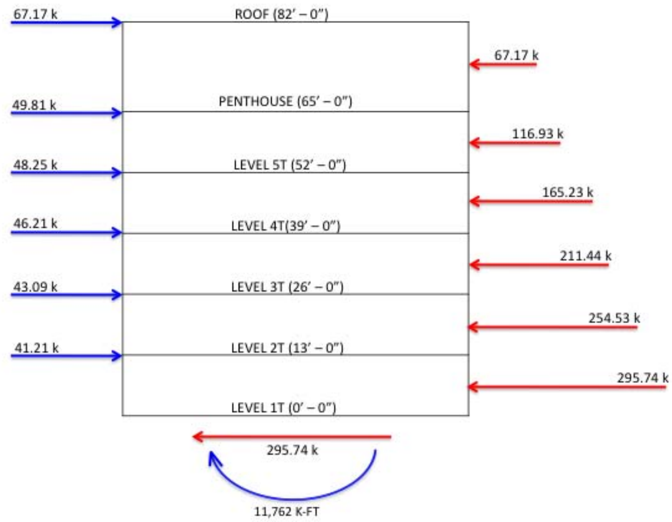

Figure 11: *Story Forces and Shear in the North-South Direction*

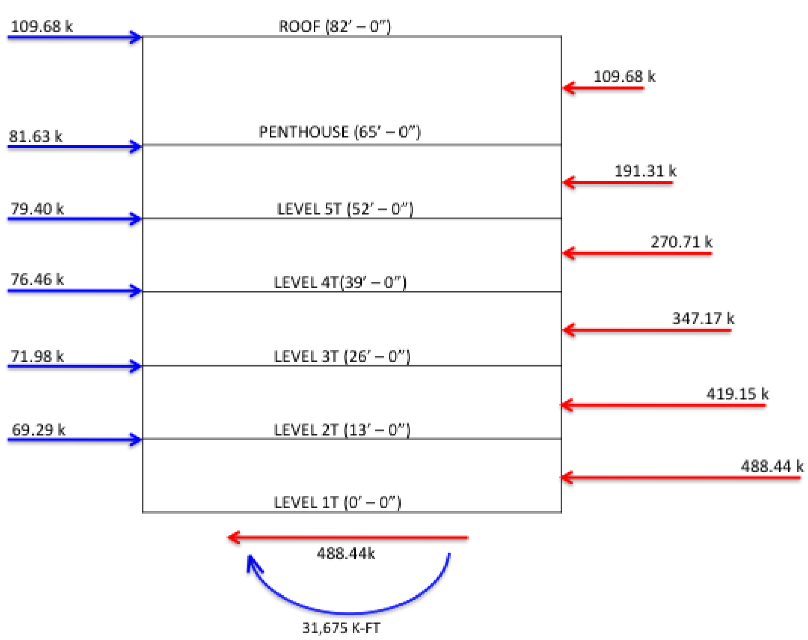

Figure 12: *Story Force and Shear in the East-West Direction*

Seismic Loads

Chapters 11 and 12 of ASCE 7-05 were referenced in order to calculate the seismic forces on the Army National Guard Readiness Center. It was assumed that the ArNG Readiness Center employed a rigid diaphragm, which allowed for the use of the Equivalent Lateral Force Procedure (ELF) found in section 12.8 of ASCE 7-05 standards. Upon investigation of the geotechnical report provided by CH2M Hill, it was determined that the Army National Guard Readiness Center falls under Site Class D. S_S and S_1 were then determined using the United State's Geological Surveying (USGS) website. All design variables and site parameters that were used in determining the seismic loads can be found in Appendix C along with detailed calculations and spreadsheets that were utilized to obtain the building weight, base shear, and overturning moment. Figure 14 is a loading diagram that summarizes the story forces, base shear, and overturning moment acting on the Army National Guard Readiness Center due to seismic loads.

Seismic Loads									
Level	Height h_X (ft)	Tributary Height (Ft)	Story Weight w_X (Kips)	h_x^k	$w_x h_x^k$	C_{VX}	Lateral Force F_X (kips)	Story Shear V_X (kips)	Moments M_X (ft-kips)
Roof	82	8.5	144	82.00	11808.00	0.03	7.90	0.00	0.00
Penthouse	65	13	1814	65.00	117910.00	0.34	78.87	7.90	67.15
5T	52	13	1810	52.00	94120.00	0.27	62.95	86.76	698.31
4T	39	13	1810	39.00	70590.00	0.20	47.22	149.72	2235.5
3T	26	13	1810	26.00	47060.00	0.14	31.48	196.93	4488.78
2T	13	13	298	13.00	3874.00	0.01	2.59	228.41	7253.62
1T*	0	6.5	0	0.00	0.00	0.00	0.00	**231**	**10224**
$\sum(w_i h_i^k)$ = 345,362		**$\sum(F_X)$ = V = 231 Kips**			**$\sum M_X$ = 10,224 'k**				
Total Building Weight(Above Grade) =9,495 kips									
* The Level 1T story weight is only weight of the columns whose base is at the ground floor. Weights of slabs, beams, and superimposed deads loads are not considered at the ground floor because the base shear is related only to the levels above grade and the components mentioned are at grade level.									

Table 5: *Seismic Loads*

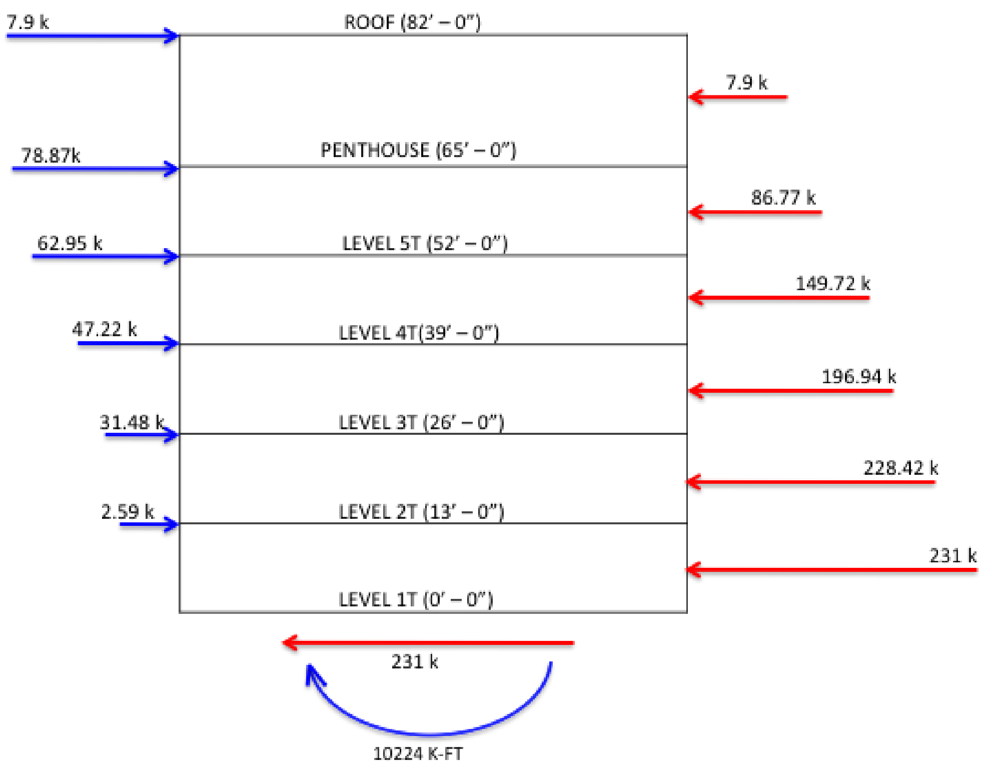

Figure 13: *Story Forces and Shear from Seismic Loads*

Conclusions

Wind loads

North-South:

Base Shear = 295.74 k → Apply 1.6 Factor → *473.2 k*
Overturning Moment =12788 'k → Apply 1.6 Factor → *20460.8 'k*

East-West:

Base Shear = 488.4 k → Apply 1.6 Factor → *781.44 k*
Overturning Moment = 21006 'k → Apply 1.6 Factor → *33609.6 'k*

Seismic Loads

Base Shear = 231 k → Apply 1.0 Factor → *231* k
Overturning Moment = 10224 'k → Apply 1.0 Factor → *10224 'k*

Factored Loads are determined by applying the factor of 1.6 to the wind forces and a factor of 1.0 to the seismic forces. When the factors are applied, the wind loads in both directions have a greater magnitude than the seismic force with a 1.0 factor. Therefore, the wind loads control the design of the Army National Guard Readiness Center lateral system. Lateral system spot checks will be determined using the wind load only since it is the governing lateral load.

Gravity System Redesign

Initial Considerations

The composite steel structure was chosen after careful consideration of multiple systems. Originally, a prestressed floor system seemed to be a good choice since it would minimize the slab thickness. After further investigation of prestressed concrete systems however, the disadvantages such as slab penetration issues and construction difficulty, began to outweigh the advantages of the system. Another possibility that was considered once it was decided that a steel structure would be used was a non-composite structural system. This was eliminated because it would have required deeper members than a composite system therefore interfering more with the MEP systems. It is possible, and will be most likely necessary to reduce the floor-to-floor height or increase the overall building height to avoid interference with the MEP systems. The overall building height can be increased since there are no zoning restrictions on height in this area of Arlington, Virginia. However, it is more efficient to maximize the floor-to-ceiling height and reduce the floor depth therefore the composite system was chosen for the structural redesign.

Beam, Girder, and Slab Design

The existing bay layout was used for the devolving layout of the new structure. This was an important aspect fort he architecture of the building especially considering the unique shape and aesthetic design of the Army National Guard Readiness Center. From this layout, the infill beams would need to run perpendicular to the span of the composite metal deck. Another consideration for the metal decking was fireproofing. A 3VLI deck would be chosen from Vulcraft's *Steel Roof and Deck* Catalog with a 3.5" lightweight concrete slab. This type of deck would ensure a two-hour fire rating for the slab without requiring any additional fireproofing. It was determined that a 3VLI deck would be used to satisfy the 14'-0" typical spacing to the infill beams. This results in an overall slab thickness of 6.5" inches with a 3" metal deck and 3.5" of concrete above the slab.

It was determined that the beams would be designed to act compositely. Smaller sections could be utilized by designing the beams this way due to some of the shear forces being taken by the concrete instead of all it being applied to the beams. The provisions listed in AISC's 13th Edition of the Steel Construction Manual were used to determine the number and size of shear studs required. Load combinations were then checked to establish the controlling combination. The gravity system was controlled by the load combination 1.2D+1.6L for the typical floors and $1.2D+1.6L_r+L$ for the roof. Once the load combinations were calculated and applied, typical members were sized using Load and Resistance Force Design (LRFD) Method and the AISC Steel Construction Manual. Member sizes were chosen based on the moment capacities from Table 3-19 of the Steel Construction Manual and the deflection criteria previously listed.

After preliminary sizes were selected, a computer model was generated using RAM Structural System. Typical floor plans were designed using the RAM model and then compared to the typical hand calculations. The W-shapes determined by the RAM output closely resembled the W-shapes that were determined by hand calculations. The RAM model was then used to optimize the structure. Duplicating the beam and girder sizes in similar bays on each floor reduced the number of different member sizes. Limiting the number of different member sizes can improve the overall efficiency of the construction process. Less size variations will save money from fabrication costs and make erection on site easier and less likely that there would be any mistakes. The final layout included W14's for infill beams and W18's for girders on the typical floor levels. A typical floor plan can be seen in the figure below. Hand calculations, RAM output, and typical floor plans can be found in Appendix D.

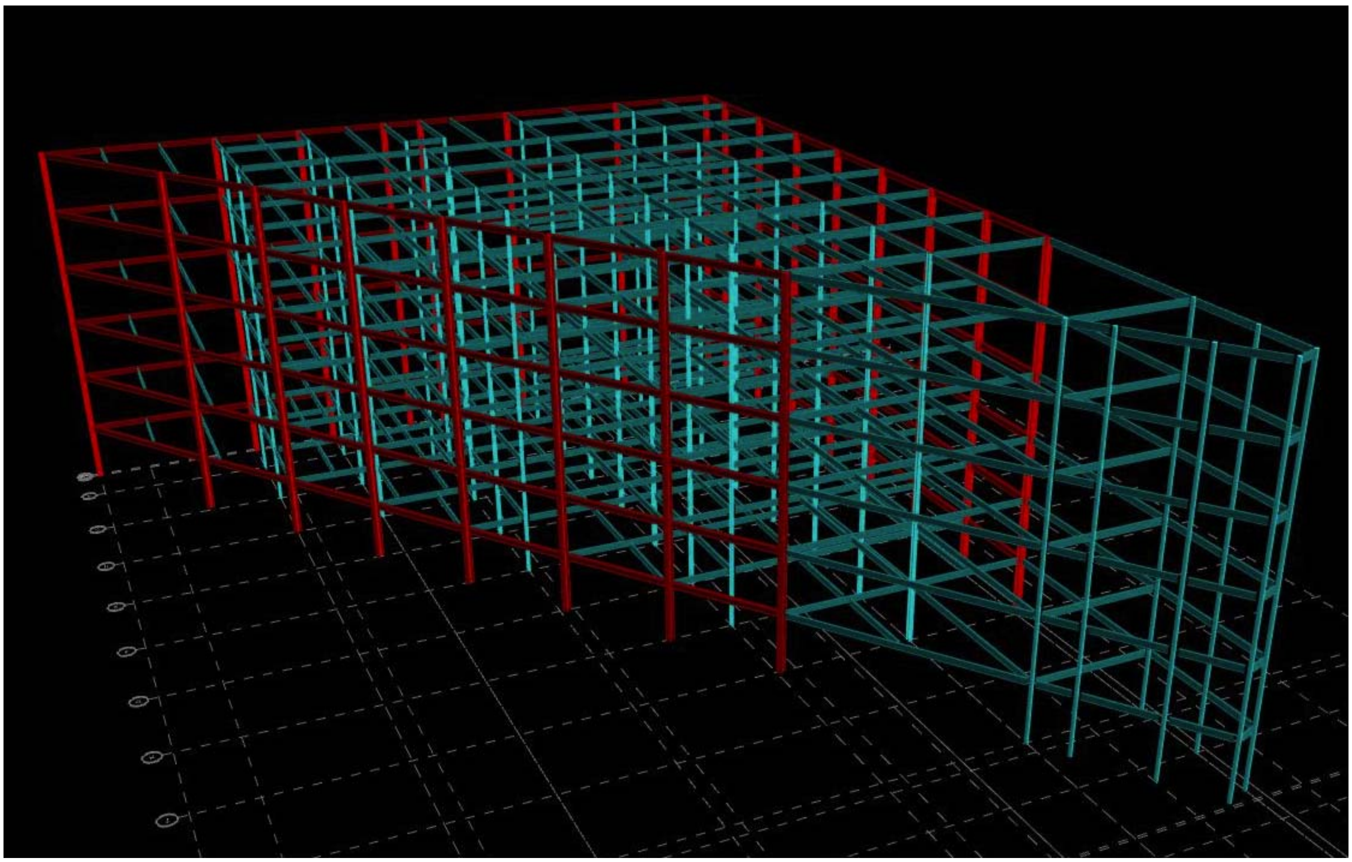

Figure 14: *RAM Model – Red members indicate lateral elements and green members indicate gravity elements*

Column Design

Gravity columns were designed once preliminary decking, beam sizes, and girder sizes were determined. A typical lad path used when considering the design of the gravity columns starts with the deck and slab carrying the gravity loads from the building to the infill beams. The beams then carry the loads to the girders, which in turn transfer the load to the columns. To begin the column design, the building's live loads were reduced in accordance with section 4.8 and 4.9 of ASCE 7-05. The tributary area for a given column was determined and then used along with the reduced live loads and the building dead loads to calculate the axial loads that act on a column at each level. All columns were designed for the axial load and gravity induced moments that were determined. AISC's 13th Edition Steel Construction Manual was used to choose the preliminary column sizes. Columns were sized for strength by using Table 4-1 for axial compression and Table 6-1 for combined axial and bending.

Once preliminary column sizes were determined the RAM structural model was used to optimize the structure and limit the number of different column sizes. Reducing the number of different sections and increasing repetition makes construction and erection easier saving precious time in the field and costs back fabrication costs. Column sizes were limited to W12's in order to minimize the effects on the existing architecture. For constructability purposes the columns were spliced at every other building level. The resulting design for the gravity loading was comprised of seven different W10 shapes and the existing HSS shapes located in the stair tower. The picture below shows a typical elevation with gravity column sizes indicated. Hand calculations can be found in Appendix D.

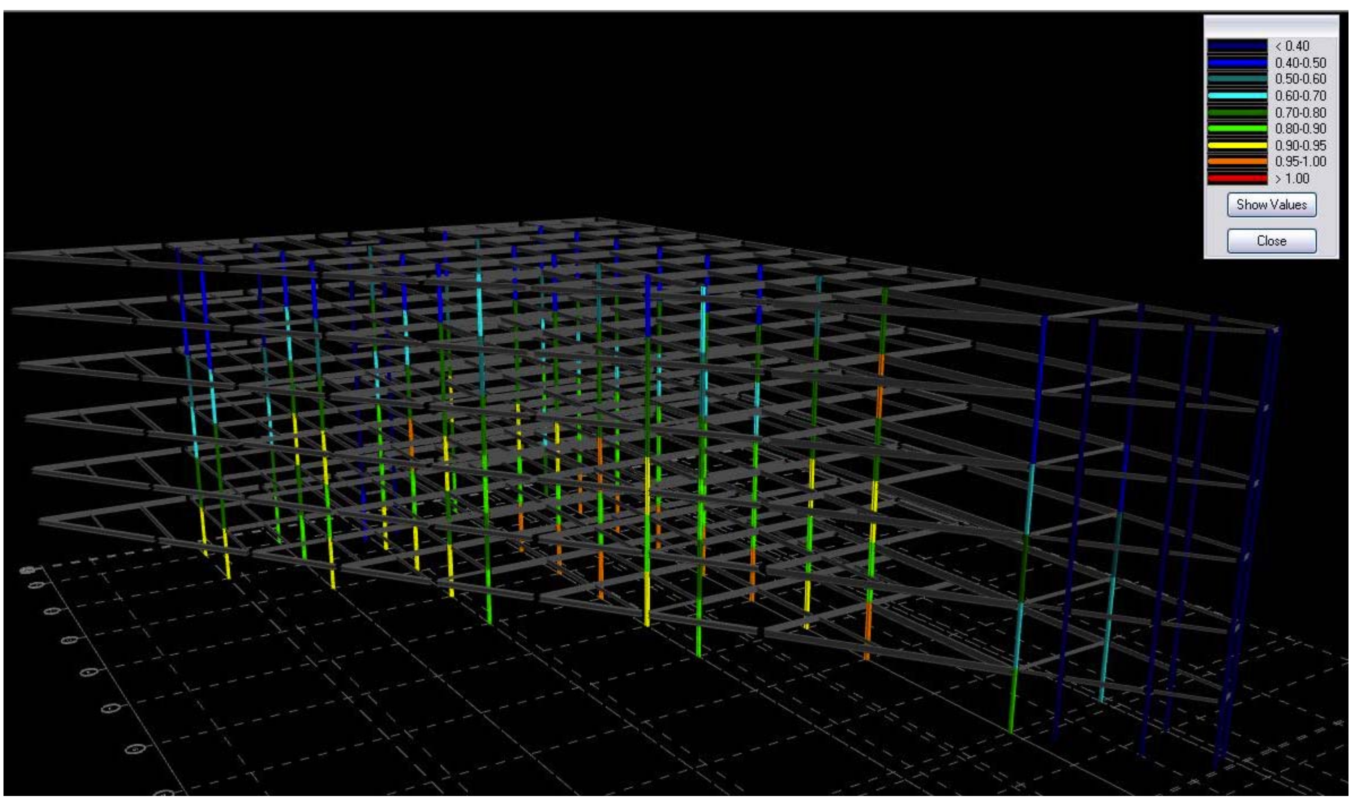

Figure 15: *RAM Model illustrating the Interaction Diagrams for Gravity Columns*

Final Gravity Design

The final layouts and member sizes for the redesigned gravity system of the Army National Guard Readiness Center can be found in this section.

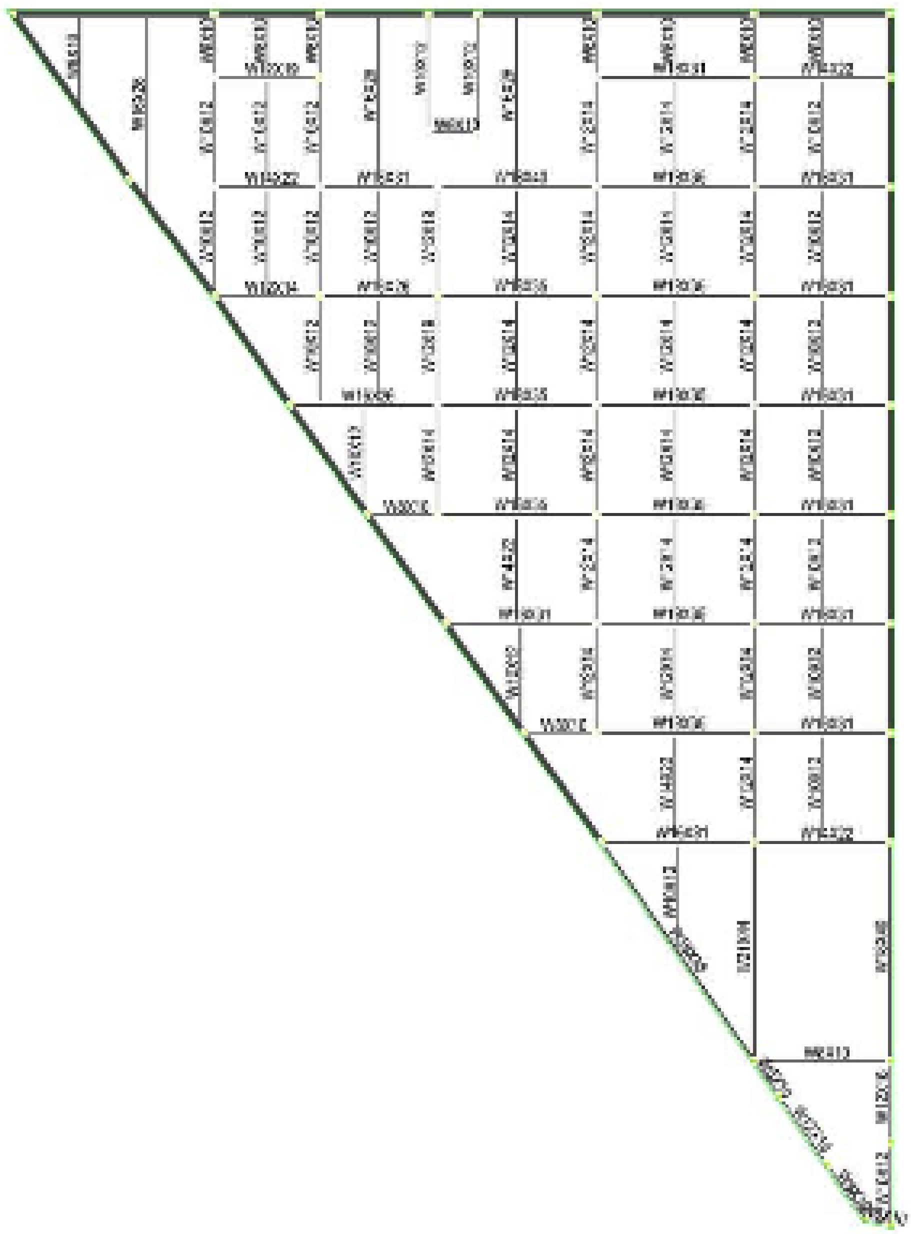

Figure 16: *Typical Gravity Girder and Beam Layout*

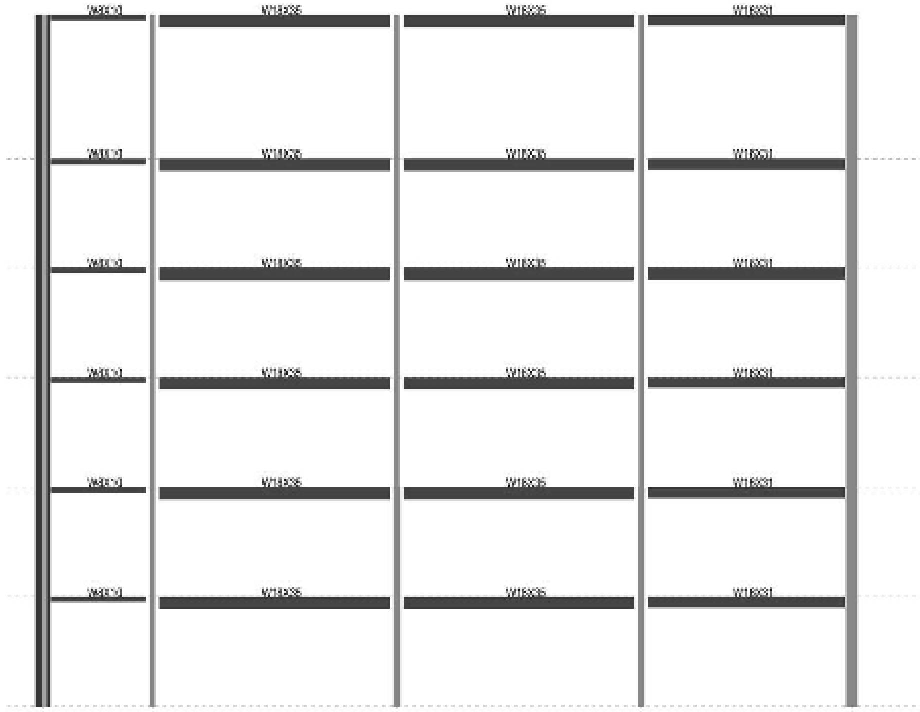

Figure 17: Typical Elevation Depicting Standard Girder Sizes

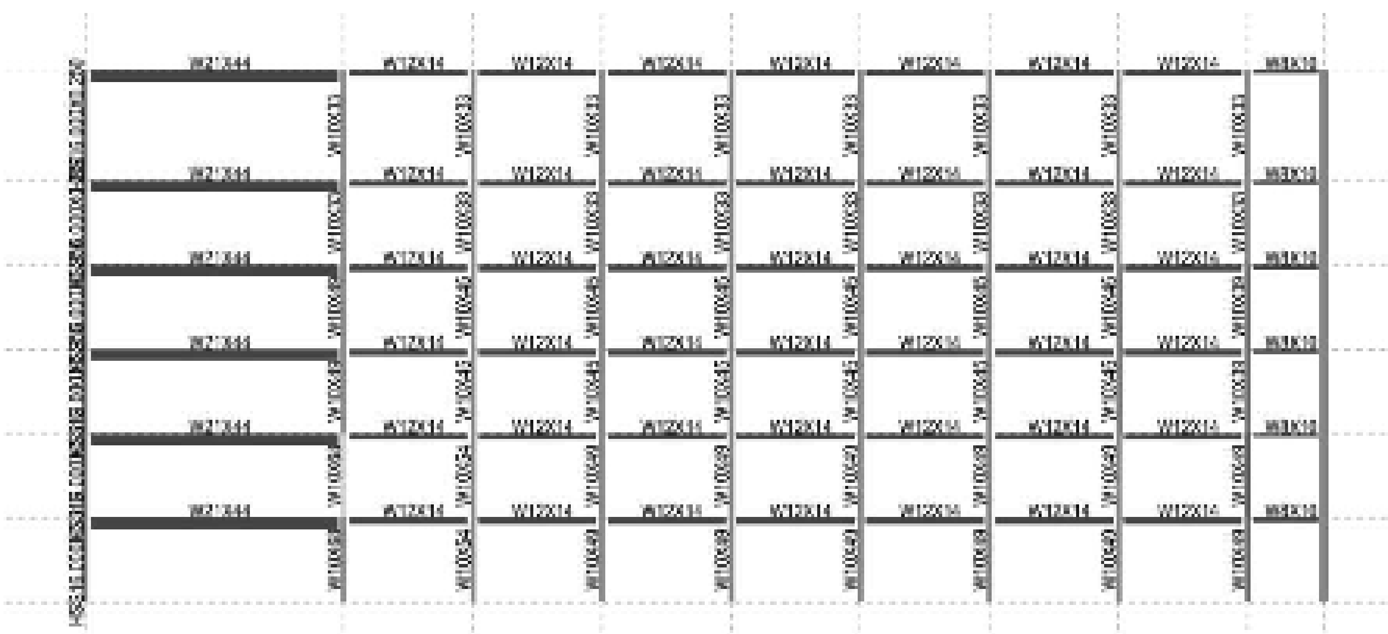

Figure 18: *Typical Elevation Depicting Gravity Column Sizes*

Lateral System Redesign

Initial Considerations

Several lateral force-resisting systems were considered before a final conclusion could be made on the lateral system that would be most efficient for the Army National Guard Readiness Center. Three steel lateral systems were thoroughly investigated before it was concluded that the system used would be steel moment-resisting frames. Buckling restrained braced frames (BRBF) were the first system considered. This type of frame is fairly new but rapidly gaining popularity due to a number of structural performance advantages over conventional braced frames. These systems are unique because of the configuration of the brace elements, which develop uniform axial strains in tension and compression. This is a result of the steel core resisting axial stresses and the outer concrete filled steel casing resisting buckling stresses. In a BRBF plastic hinges associated with buckling cannot form and the post-buckling load imbalance, inherent with conventional braced frames are eliminated by the near equal tension and compression capacities. This is extremely beneficial in high seismic regions; however, for this building buckling restrained braced frames did not seem to be appropriate. These frames have several disadvantages that outweigh the disadvantages in this case. Some of the disadvantages of the buckling restrained braced frames include their steep prices, the fact that many details and connections are subjected to U.S. Patent Laws, all details must be finalized prior to production and assembly cannot be modified later, and modeling can be extremely complex especially managing drift control.

Special Concentrically Braced Frames were also considered. These types of frames are extremely popular lateral force resisting systems for medium to low rise steel buildings. They are relatively simplistic and have many possible configurations. The different configurations can be helpful when working around architectural restrictions. The diagonal brace members dissipate seismic energy through yielding in tension and inelastic buckling in compression. Due to this cyclical yielding and buckling, significant loads are applied to connections. Because of the large loading, connections are required to be detailed much stronger than the nominal cross-sectional capacity of the brace members. Detailing the connections to provide this level of strength can be rather difficult, particularly when dealing with HSS, which are the most preferred because of their efficiency in carrying compressive loads, their pleasing aesthetic appearance, and the wide range of section sizes readily available in the U.S.. A final disadvantage of special concentrically braced frames is the issue of placing the diagonal bracing. While there are multiple configurations possible, the braces tend to get in the way unless there is a clear area available for them in the plan.

Finally, special moment-resisting frames were looked into as a possible system for the Army National Guard Readiness Center. Special moment frames allow for very open floor plans and were immediately put to the top of the list because of the limited impact it would have on the existing architecture and layout. The coast of the special moment frames tend to be lower than braced frames due to the minimum number of members required. A properly detailed special moment frame is one of the most ductile lateral force resisting systems. ASCE 7-05 recognizes the ductility with a high response modification coefficient (R=8), which yields lower design forces, smaller foundation forces, and reduced diaphragm forces in the structure. The lower design forces for special moment frames however result in a relatively high deflection amplification factor (C_d). The high deflection amplification factor can cause problems with drift controlling which can result in frames being much heavier to meet drift limits than if just designed for strength. Another disadvantage of special moment frames is the significant connections, which must be prequalified. At this time there are a limited number of approved connections. These types of frames usually require deeper beams, which could potentially occupy most or more than the ceiling cavity creating coordination and architectural issues. The final advantage of using moment frames over the other possibilities was the issue of progressive collapse. Designing braced frames for progressive collapse mitigation would be difficult for connection design and detailing. Moment frames, however, do not need special connections or other details when designing for progressive collapse.

Layout and Location of Lateral Elements

Once it was determined that special moment frames would be the best lateral force resisting system for the layout and purpose of the Army National Guard Readiness Center, placement of the lateral system needed to be determined. When considering the need for progressive collapse mitigation, it was decided that the moment frames would be placed along the perimeter of the building. Since there are no braces required for moment frames this seemed to be the perfect solution for the lateral system layout without having a significant impact on the façade or architecture of the building. A dual system was also a possible solution. Placing braced frames or more moment frames in the interior spans could have reduced the required size of the lateral members and potentially reduced the torsional effects of the building. The perimeter moment frames worked fairly efficiently and were ideal for progressive collapse design and therefore that layout was employed for the final design.

Figure 19: *Proposed Location and Layout of Lateral Force Resisting Moment Frames*

Modeling Assumptions

Once the necessary loads were determined and a preliminary gravity system was designed, a computer model was generated using RAM Structural System. Both the lateral and gravity systems were modeled in order to optimize the overall structure. The following modeling assumptions were taken into account:

- Proper load combination were generated in accordance with ASCE 7-05 standards

- A rigid diaphragm was assumed at every floor

- Both inherent and accidental torsion effects were considered

- Lateral forces were applied at the center of mass along with a moment due to accidental torsion from a standard 5% eccentricity

- Lateral beams were assumed to be fixed at both ends

- Lateral columns were assumed pinned at the bottom and fixed at the top

- P-Delta effects were automatically taken into consideration in the RAM model

- Rigid End Offsets were considered

- The structure was assigned a fixed base due to the concrete mat slab foundation

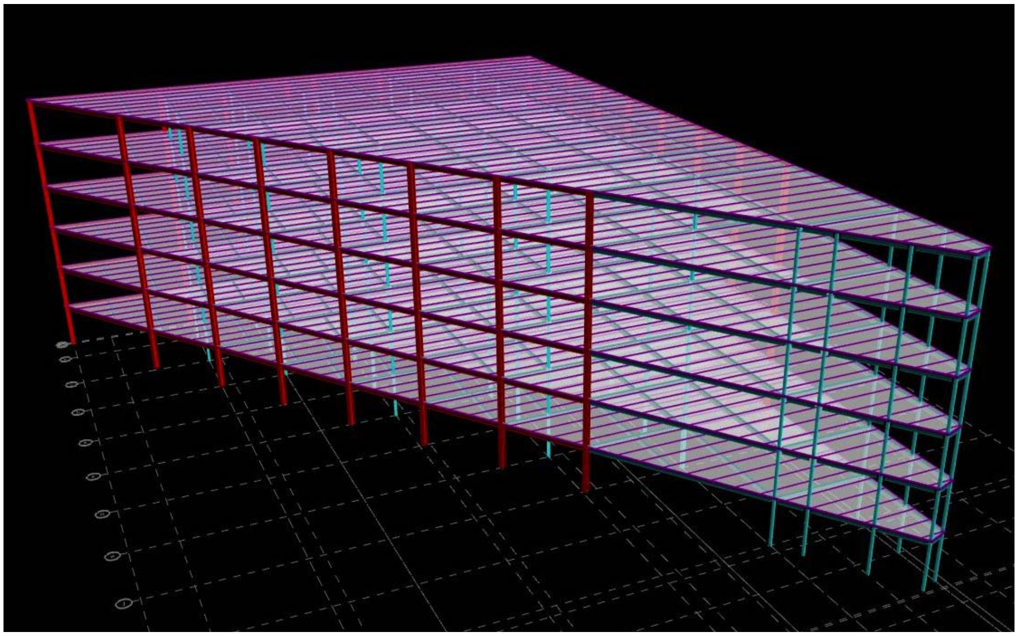

Figure 20: *RAM Structural Systems Model*

Serviceability

Drift and Displacement are serviceability issues for buildings. Taken this into account was one of the last steps of the overall, standard design process. Lateral drift and displacement should be limited by the following criteria set by ASCE 7-05:

Seismic Drift → $0.020\ h_x$

Wind Drift → h/400

Deflection values were taken from the RAM model. Three perimeter points and the center of mass were looked at and the controlling deflection was used for the analysis. Deflection was a controlling factor in the East-West direction but not in the North-South direction. After multiple iterations it was possible to get limit the building deflections to the acceptable drift and displacement ranges. A sample drift and displacement analysis can be seen in the table below. More comparisons and calculations can be found in Appendix E.

Story Drift in East - West Direction

Level	Story Height (ft)	Story Drift (in.)	Allowable Drift (h/400)	Result
Roof	17	0.2966	0.51	Good
Penthouse	13	0.2770	0.39	Good
5T	13	0.2483	0.39	Good
4T	13	0.2380	0.39	Good
3T	13	0.2100	0.39	Good
2T	13	0.2048	0.39	Good

Table 6: *Story Drift in East-West Direction*

Displacement in East-West Direction

Level	Height (ft)	Displacement (in.)	Allowable Drift (H/400)	Result
Roof	82	0.2308	2.46	Good
Penthouse	65	1.8473	1.95	Good
5T	52	1.3890	1.56	Good
4T	39	1.[illegible]252	1.17	Good
3T	26	0.1632	0.78	Good
2T	13	0.0418	0.39	Good

Table 7: *Story Drift in North-South Direction*

Torsion

In accordance with ASCE 7-05 chapter 12 section 8.4.2, any diaphragm not modeled as flexible must account for both inherent and accidental torsion. First the cent of mass and center of rigidity for each floor were located using the RAM output and were verified by a visual inspection of the floor plans and structural layout. The eccentricity could then be determined using these two points on the diaphragm. The eccentricity of the diaphragm controls the torsional moment on each floor. The figure below shows a typical diaphragm with location of the center of mass (COM) and center of rigidity (COR) indicated.

Inherent torsion is created when the center of rigidity and center of mass are not located at the same exact point. This torsion acts on the diaphragm, which is then carried to the lateral elements. An analysis was performed in each direction to determine the torsional moment effects at each level. Values can be seen in the table below.

Accidental torsion is caused by seismic forces acting at a 5% eccentricity in each direction from the center of mass. The necessary amplification factor to calculate accidental torsion was determined in accordance with chapter 12 of ASCE 7-05 as a value of 1.0. Sample calculations can be seen in Appendix E however values were not necessary since seismic loads and deflections do not control the design.

Fireproofing

During a fire, the structure can see temperature reaching 800-1,000 degrees Fahrenheit. For the existing cast-in-place structure this would not be an issue due to the properties and characteristics of concrete. The proposed steel structure however, will need to be appropriately protected from high temperatures. As the temperatures increase, the strength of the steel rapidly decreases. This causes the steel to bend and buckle under the load of the building and speeding up the failure of structural members. Spray applied intumescent paint and fire resistive materials can cling to the steel and preserve the strength of the steel in the event of a fire. By preventing heat damage to the steel, these materials add valuable evacuation time for both occupants and firefighters and can save property as well as lives.

Figure 21: *Example of spray on fireproofing applied to beams and girders.*

During the preliminary design process the deck and slab thickness were chosen and designed such that the decking does not require any spray on fireproofing, the deck and slab have a two-hour fire rating. While this greatly reduces the overall area threat needs to be protected, the steel beams, girders, and columns must still be considered. Eliminating the deck area, will save an enormous amount of time and money to fireproof the steel system and makes it more comparable to the concrete structure. After considering several types of fireproofing, it was determined that a cementitious plaster based material would be the most effective and efficient choice. The cementitious base would include a combination of Portland cement and lightweight aggregates. The columns would most likely require a greater thickness than the beams and girders because the columns are typically more critical. The lateral system would also need to be sprayed and would require more fireproofing due to the larger member and the criticality of connections. The fireproofing costs and time were considered when determining the overall structural cost and schedule. Results can be found in the construction management analysis and Appendix H.

Foundation

Due to the decreased building weight and the considerable wind forces, it was necessary to check the foundation design for any overturning moment problems. The original foundation was a 42" mat slab. The overturning moment due to the wind forces was calculated in both the East-West direction and the North-South direction. Only the overturning in the East-West direction was checked however because it was the controlling case and it was determined by inspection that the seismic loads would have no effect on the foundation from overturning. The building weight of the steel structural system was then determined. Once the building weight had been determined the existing

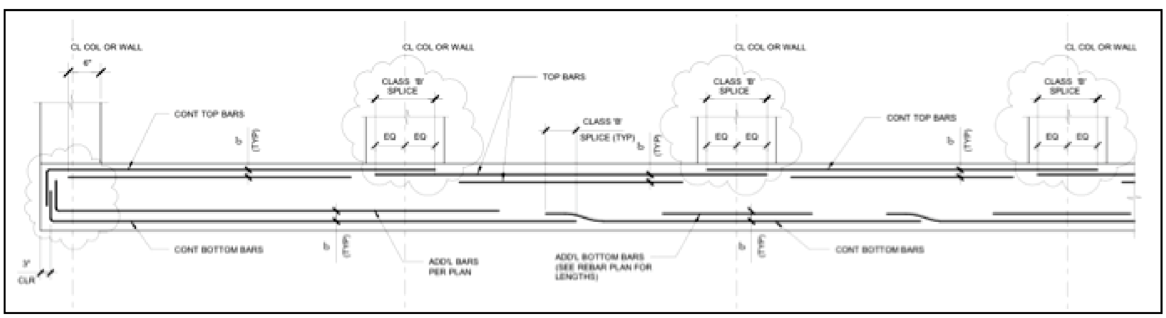

Figure 22: Detail of existing mat slab foundation

Progressive Collapse Analysis

Background

In April of 1995 the Alfred P. Murrah Federal Building in Oklahoma City was bombed. It was the most significant act of terrorism on American soil prior to the attacks of September 11th, 2001. The bombing claimed the lives of 168 and injured more than 680. The majority of injuries and deaths were a result of the partial collapse of the building and not the explosion. This was the most devastating progressive collapse incident in US history. Following this event, legislation was passed to increase protection around federal buildings and guidelines were developed for the design of buildings to mitigate the risk of progressive collapse.

Figure 23: *The Murrah Federal Building in Oklahoma City after the devastating bombing*

Progressive collapse is defined as a situation where a local failure of a primary structural element results in the collapse of adjoining members, which in turn leads to additional collapse. Essentially it is a domino effect where the extent of the total damage is disproportionate to the initial cause of collapse. Progressive collapse incidents have historically been the result of abnormal loading. There are four general types of abnormal loading: accidental impact, faulty construction practices, foundation failure, and violent change in air pressure like the explosion that was used in the Oklahoma City Bombing.

There are a number of building codes that reference progressive collapse design and analysis. The problem is that the majority of these existing codes provide vague guidance to defining the key issues that must be addressed in progressive collapse analysis or design. This lack of guidance has lead to conflicting interpretations; United States government agencies have developed design criteria and guidelines for progressive collapse mitigation. Both the General Services Administration (GSA) and the Department of Defense (DoD) have extensively studied progressive collapse and have presented guidelines that include preventing collapse in new buildings and methods for assessing the risk of collapse in existing buildings. The GSA guidelines, "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects", meets the provisions set forth by the Interagency Security Committee (ISC). DoD facilities must meet the requirements set forth by the Unified Facilities Criteria (UFC) presented in (UFC) 4-023-03 "Design of Buildings to Resist Progressive Collapse".

Design Approach

There are two general design approaches to reduce the potential for progressive collapse. These include Direct Design and Indirect Design.

1. *Direct Design* – This approach requires all primary structural elements be capable of resisting abnormal loading.

    - Alternate Path Method – Localizes failure by allowing the failure of a primary structural element but requiring the structure be capable of bridging over the missing structural element.

    - Specific Local Resistance Method – this method requires the members be designed to resist a specified abnormal load.

2. *Indirect Design* – This approach requires consideration of minimum strength, reinforcement continuity, ductility of connections, etc. for resisting progressive collapse. Theoretically, if the minimum values are met, the structure should be able to withstand any abnormal loading, and if an element should fail, alternate load paths will be available through continuity.

These do not provide any strict criteria or codes that must be adhered to; they are simply approaches to design and analysis of progressive collapse mitigation. Existing building codes with provision for progressive collapse may reference some of these approaches, however, the UFC and GSA Progressive Collapse Guidelines are the most complete sets of criteria currently. Both guidelines provide usable guidance to analysts and designers.

Design Strategies

The Army National Guard Readiness Center is part of the Department of Defense and therefore the existing structural system was designed to resist progressive collapse using the indirect method from the (UFC) 4-023-03 "Design of Buildings to Resist Progressive Collapse". For the purpose of this thesis both guidelines will be used to look at each approach to progressive collapse design. The Army National Guard Readiness Center will be considered a DoD building with a low level of protection (LLOP) and the Indirect Design Method will be utilized as was for the existing concrete structure design. For the direct method, the Army National Guard Readiness Center will be considered a GSA facility with a threat level identified as a high level of protection.

Direct Method – GSA Requirements

To perform a direct analysis procedure on the Army National Guard Readiness Center, the analysis prescribed by the GSA was utilized. This analysis assumes the loss of a column, particularly along the exterior perimeter of the structure. When the column is considered 'lost' plastic hinges form and moment redistribution is allowed. The linear elastic analysis set forth by the GSA works from the existing structural design, allowing the existing design to be maintained with only slight modifications to the member sizes. The GSA requirements state that it is critical that beams and girders are designed to span two full bay lengths. In a typical analysis for exterior consideration of a steel framed structure, the GSA guidelines recommend that an analysis be performed for the instantaneous loss of a column one floor above grade located at each of the following:

1. The middle of the short side of the building
2. The middle of the long side of the building
3. A corner of the building

For the purpose of this thesis, however, the loss of only one vertical member will be considered. The area that was analyzed was chosen because it had the longest spans and would therefore need to span the furthest in the event of a lost column. The GSA criterion determines the distribution of demand loads due to the devastating loss of a critical structural element. The requirements of primary and secondary elements are indicated by Demand-Capacity Ratios (DCR).

$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

Where Q_{UD} is the demand force determined in the component and Q_{CE} is the expected ultimate, unfactored capacity of the components. The GSA developed a step-by-step procedure, seen below, which was followed to analyze the Army National Guard Readiness Center.

Step-by-step procedure for conducting the linear elastic, static analysis follows.

Step 1. Remove a vertical support from the location being considered and conduct a linear- static analysis of the structure.

Step 2. Determine which members and connections have *DCR* values that exceed the acceptance criteria. If the *DCR* for any member end connection is exceeded based upon shear force, the member is to be considered a failed member. In addition, if the flexural *DCR* values for both ends of a member or its connections, as well as the span itself, are exceeded (creating a three hinged failure mechanism), the member is to be considered a failed member. Failed members should be removed from the model, and all dead and live loads associated with failed members should be redistributed to other members in adjacent bays.

Step 3. For a member or connection whose Q_{UD}/Q_{CE} ratio exceeds the applicable flexural *DCR* values, place a hinge at the member end or connection to release the moment. This hinge should be located at the center of flexural yielding for the member or connection. Use rigid offsets and/or stub members from the connecting member as needed to model the hinge in the proper location. For yielding at the end of a member the center of flexural yielding should not be taken to be more than 1/2 the depth of the member from the face of the intersecting member, which is usually a column.

Step 4. At each inserted hinge, apply equal-but-opposite moments to the stub/offset and member end to each side of the hinge. The magnitude of the moments should equal the expected flexural strength of the moment or connection, and the direction of the moments should be consistent with direction of the moments in the analysis performed in Step 1.

Step 5. Re-run the analysis and repeat Steps 1 through 4. Continue this process until no *DCR* values are exceeded. If moments have been re-distributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region, the structure will be considered to have a high potential for progressive collapse.

Figure 24: *Step-by-step procedure for conducting the linear elastic analysis*

A plastic analysis was conducted on the indicated frame. Acceptable DCR values were taken from a chart published by the GSA and used to determine the expected ultimate capacity. The expected ultimate was then used to choose the appropriate member sizes according to the plastic capacity. The final design can be seen in the figures below and further hand calculations can be referenced in Appendix F.

Indirect Method – DoD Requirements

For a Low Level of Protection (LLOP) facility such as the Army National Guard Readiness Center, the Tie Force method can be used to meet Department of Defense requirements. This method requires the structure be mechanically tied together in order to enhance continuity, ductility, and development of alternate load paths. Existing structural elements and connections, designed using conventional design procedures to withstand standard building loads can typically provide the required tie force. There are several horizontal tie requirements that must be provided depending on the type of construction. Some of the horizontal tie requirements include internal ties, peripheral ties, and ties to edge columns, corner columns, and bearing walls. Vertical ties are required in columns and load bearing walls. For any vertical tie forces that do not meet the tie force capacity, an Alternate Load Path analysis is required. Typical tie forces for a steel framing structure such as the proposed Army National Guard structure can be seen in the figure below.

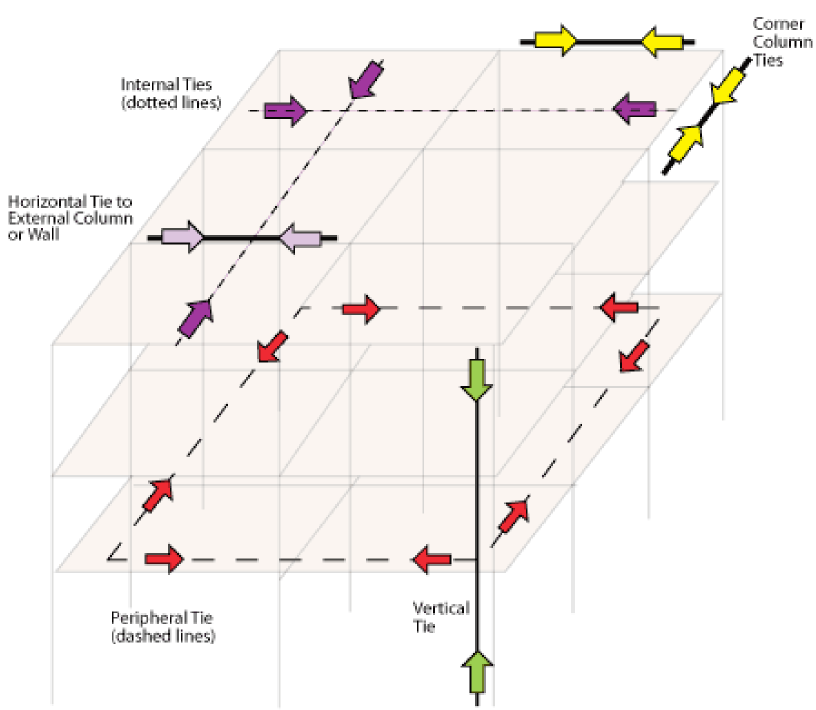

Figure 25: *Typical Tie Forces*

The design tie strength provided by a member of its connections to other members is taken as the product of the strength reduction factor, ϕ, and the nominal tie strength R_n, following the Load and Resistance Factor (LRFD) approach. The nominal tie strength, R_n, is calculated in accordance with requirements sand assumptions of applicable material-specific factors, such as the overstrength factor, Ω. As per the Load and Resistance Factor approach, the design strength must be greater than or equal to the required tie force capacity. Required tie forces can be seen in the table below and hand calculations can be found in Appendix F.

Tie Force Requirements	
Internal Tie Force	40.92 kips
Peripheral Tie Force	13.64 kips
Horizontal Tie Force	40.92 kips
Vertical Tie Force	164 kips

Table 8: *Tie Force Requirements*

Structural Depth Summary

The purpose of this depth study was to investigate the feasibility of redesigning the structural system of the Army National Guard Readiness Center to a steel system as well as the progressive collapse capabilities for the redesigned steel structure. Upon completion of this analysis it was determined that the structure could be successfully designed as a steel structure without any large negative impacts on other aspects of the building. During the design process, close attention was paid to the existing architectural elements and facets of the building. For the most part, the proposed steel design can accommodate the current layout and architecture. The existing column grid was consulted when determining the layout for the proposed steel system. The final steel solution nearly mimics the present column grid. Steel gravity columns were also limited to W12 shapes as to not impede on the spaces and functions of the original building layout. The only architectural impact of the redesign was the floor-to-ceiling heights. The proposed steel framing system has a larger floor depth than the current two-way concrete slab. This requires an increase in the ceiling cavity to ensure that there is no interference with the mechanical, electrical, or plumbing equipment running through the ceiling cavity. There are two viable solutions to this issue. First, the ceiling could be lowered the necessary amount since the building presently has eleven foot floor-to-ceiling heights and a two foot ceiling cavity. Lowering the ceiling heights would have an affect on the interior architecture of the Army National Guard Readiness Center however there would be no additional costs. The second option would be to simply increase each story to the necessary height to maintain the eleven-foot heights while still providing adequate space for the mechanical, electrical, and plumbing systems. Raising the story heights would ultimately increase the overall building height and the façade area, which would drive up costs. There are no height restrictions in the zoning code for this region of Arlington County so increasing the building height would not be a zoning issue. While both option are possible solutions, it was assumed for further analysis that the ceiling heights would be lowered and therefore no addition façade costs were considered.

The lateral system was redesigned to use moment frames at the perimeter of the building. The proposed steel structure reduced the overall building weight by a significant amount and therefore reduced the seismic loads enough to force the wind loads to control the lateral design of the building in both the East-West and North-South directions. Drift was the controlling factor in the East-West direction and it took multiple iterations to get adequate members to limit the drift to the standards set by ASCE 7-05. It is possible that a more efficient lateral system could have been developed using braced frames through the interior of the building in addition to the moment frames. Adding braced frames would also add costs however it would have limited member sizes of the moment frames by taking some of the lateral loads as well as potentially reducing the eccentricity of the diaphragms, in turn reducing the torsional moment effects. Braced frames could have also made the building stiffer and the drift may not have been the controlling factor of the design in the East-West direction. For the purpose of this thesis, however, the moment frames were a viable solution and worked well for the progressive collapse analysis.

For the progressive collapse analysis, two methods were utilized from two different standards for progressive collapse mitigation. This analysis provided necessary information to upgrade the size of the moment frame members in order to adequately alleviate any type of progressive collapse situation given a blast of other damaging event that would result in the instantaneous loss of a critical structural element. Since the Army National Guard Readiness Center is a secure building and all landscape, glazing, and surroundings have already been thoroughly analyzed and designed to mitigate any terroristic attacks, the progressive collapse analysis was limited to the perimeter members of the ground level of the building. The Army National Guard Readiness Center is a part of the Department of Defense and the initial design was analyzed using the guidelines for a Low Level of Protection (LLOP) Department of Defense facility. For the purpose of this thesis, both the Department of Defense and the U.S. General Services Administration guidelines were considered for the analysis. After investigating both processes, it appears the direct method using the General Services Administration guidelines for federal buildings is easier for analysis purposes and more conservative. The structural elements were upgraded as appropriate to provide adequate strength against possible progressive collapse.

Redesigning the structure of the Army National Guard Readiness Center to a steel framed system had the potential to create issues with other aspects of the building, some of which were considered in this section of the report. The first feature that was looked at was the effects of the reduced building weight on the foundation. The main concerns with the foundation were the issues of overturning and uplift. A brief analysis was completed and it was concluded that the weight of the steel structural system was sufficient in resisting the overturning moment caused by the wind force. The second main issue with the steel system is the fireproofing. The existing concrete structure does not require any fireproofing, but due to the material properties of steel it is imperative the structural steel members are protected from the high temperatures of fire. During the design process the metal deck and concrete slab thickness were chosen to meet a two-hour fireproofing. By designing the floor slabs to meet this criterion it cut back the area of steel that required fireproofing and hence reduced the cost and time required for fireproofing. After some research into various types, a cementitious plaster based spray-on fireproofing was chosen. All beams, girders, and columns of both the gravity and lateral moment frames required application. The fireproofing was considered in both the schedule and cost portions of the construction management analysis.

BREADTH STUDY 1: ACOUSTICAL ANALYSIS

Introduction

This breadth topic focuses on the noise transmission and potential acoustical issues that may become apparent when changing the structural system of the Army National Guard Readiness Center. With the introduction of a steel structural system noise transmission issues be induced in the office spaces on the 5T level below the mechanical penthouse. The proposed steel framing system will reduce the concrete thickness of the penthouse floor causing increase vibration from mechanical equipment and potentially transmitting more noise to the offices below. This analysis will determine the sound pressure levels of the mechanical equipment located above the 5T level and then calculate the sound transmitted into the office spaces. From this it will be determined if additional acoustical materials are necessary to keep the sound level within the preferred range of noise in an office area. If necessary, additional acoustical materials could be introduced to keep the sound level within an acceptable range for an office space.

Analysis

The main area that was analyzed for this study was the area below the two cooling towers, which are located directly above a large open office space. This are was chosen because it was known from earlier research that the cooling towers would generate the highest sound pressure levels and are located at a prime location over the offices. "Architectural Acoustics" by M. David Egan was used to reference sound pressure levels, absorption coefficients, background noise levels, and sound transmission coefficients as well as to design and analyze the floor system.

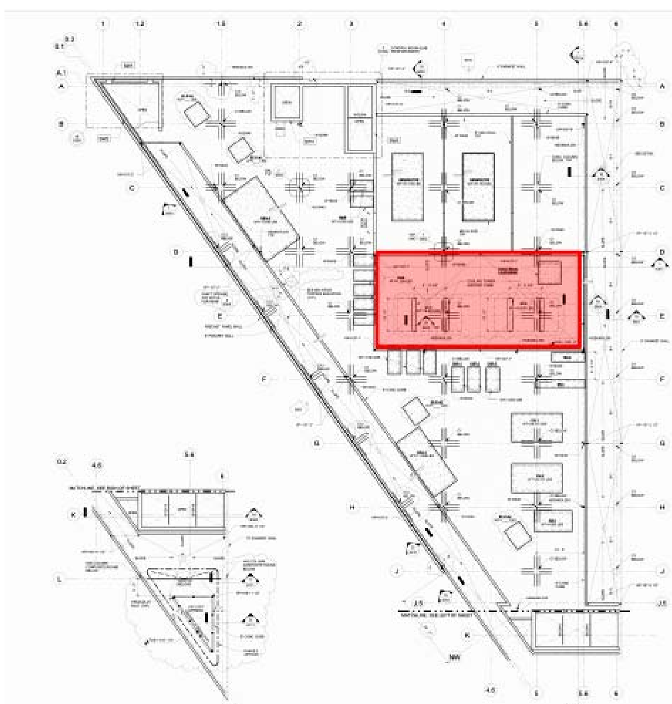

Figure 26: *Area Under Cooling Towers Considered for Acoustic Analysis*

The existing floor system beneath the cooling towers consists of an additional six inches of concrete at minimum, above the nine-inch concrete floor slab. While this construction does not include high impact isolation effectiveness, the combined fifteen inches of concrete is effective enough. There is also a gypsum ceiling suspended from the bottom of the concrete slab to add additional sound absorption. The ceiling was initially neglected in this analysis to determine if the floor system alone would absorb the sound. The proposed floor system is comprised of a 19 gage metal deck and only 4 ¼ inches below the minimum six-inch concrete base. Sketches of both the current rooftop system and the proposed rooftop system can be seen below.

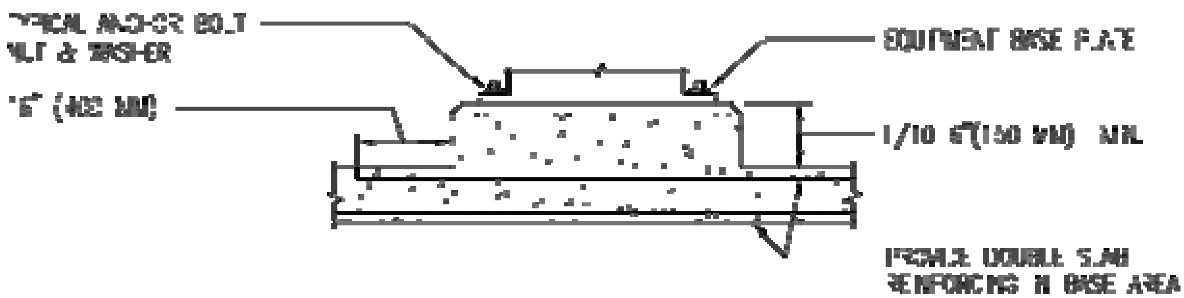

Figure 27: *Current Mechanical Penthouse Floor System*

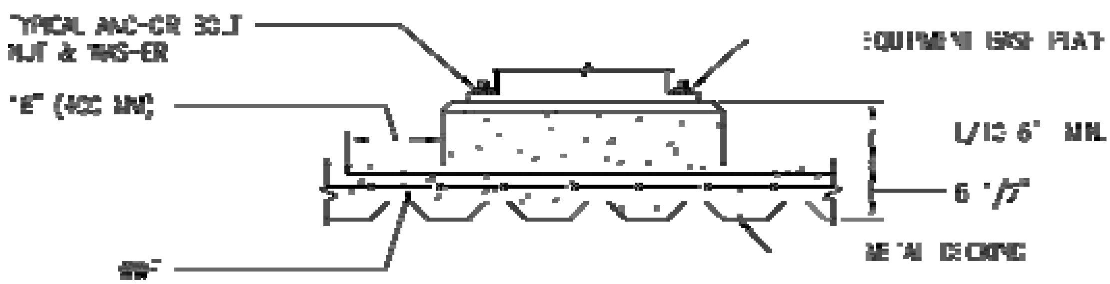

Figure 28: *Proposed Mechanical Penthouse Floor System*

After calculating the required transmission loss and the actual transmission loss it was apparent that the 10 ¼" of concrete thickness alone accounts for the necessary transmission loss. The following table shows values calculated for required noise reduction, required noise transmission loss, and total transmission loss. Since the transmission loss is greater than the calculated required transmission loss, the office space below the mechanical penthouse has no sound penetration from the cooling towers with the new structural system. The ceiling tiles and ceiling insulation therefore are only required to absorb the sound produced by the building systems running through the ceiling. Additional calculations can be found in Appendix G.

Acoustics Analysis						
Floor Design Criteria	Sound Pressure Level (dB)					
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Likely Noise from Cooling Towers	102	97	94	90	88	84
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Required Noise Reduction	57	57	59	60	63	64
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Total Room Absorption	4.95	4.95	9.9	9.9	9.9	9.9
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Required Transmission Lost	**77**	**77**	**76**	**77**	**80**	**81**
Floor Design Check	Sound Pressure Level (dB)					
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
6" Reinforced Concrete Slab	38	43	52	59	67	72
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4" Reinforced Concrete Slab	48	42	45	56	57	66
Actual Transmission Lost	**103**	**107**	**123**	**145**	**159**	**179**

Table 9: *Acoustics Analysis*

Conclusion

The equipment in the mechanical penthouse would emit a maximum sound pressure of 102 decibels (102 dB) that could potentially penetrate the offices space on the 5T level. The estimated background noise for the office space was 45 decibel therefore the required noise reduction to keep the equipment noise out of the office space is 57 decibel. A 19 gage metal deck and 10” of concrete are enough to provide a transmission loss of 103 decibel. Therefore, this floor system will prevent any sound penetration to the office spaces on the 5T level.

BREADTH STUDY 2: CONSTRUCTION MANAGEMENT ANALYSIS

Introduction

This section of the report includes a detailed assessment of both the existing concreter structure and the proposed steel structure to determine if the new structural steel system would be more economical and efficient from a construction viewpoint. The construction process for the existing cast-in-place structure is different than the construction process for the proposed steel structure and the differences must be considered. The two main aspects that must be considered when changing the structure of a building are the coast and project schedule. The efficiency of a structural system will be determined by these two factors. The duration of construction along the material, equipment and labor costs will be determined for both structural systems and then compared. From the comparison conclusions will be made about the constructability and feasibility of the proposed steel framing solution.

Figure 29: Site Excavation

Construction Methods

One of the main goads of any building project is to make the construction process as quick and efficient as possible. Steel construction will reduce the erection time die to the ease of fabrication when compared to a cast-in-place concrete structure. Another way to reduce construction times as well as costs is member repetition. This reduces the number of different sections require, which in turn cutes down on material costs and reduces the amount of coordination time in the field. Throughout the design process for the structural steel system this was taken into consideration and the framing system was optimized using RAM Structural System to increase member repetition. Less member sizes also reduces the chance of making mistakes during erection. Floor-to-floor construction will be used to analyze both structural systems, as it is one of the most common and basic methods and widely used in the Arlington, Virginia region. This construction method entails construction one floor at a time for the entire building instead of in sections.

Figure 30: Reinforcement being placed in the Mat Slab

Costs

To determine how the structural system redesign would affect the overall cost of the building a detail estimate of both systems was necessary. First the existing structure was analyzed. R.S. Means was used to obtain the prices for building components therefore concrete and reinforcement takeoffs had to be performed. For the concrete building, concrete, formwork, and reinforcement were considered when estimating column and slab costs. Concrete finishing for the slabs was also included in the pricing. RAM was used for the takeoffs of the weight of steel members and shear studs. Framing, metal decking, concrete, slab finishing, welded wire fabric shear studs, and fireproofing were determine for the steel structure cost estimation. The costs and comparisons can be seen in the tables below and more detailed breakdowns can be found in Appendix H.

Concrete Cost Summary				
Building Components	Cost			
	Material	Equipment	Labor	Total
Concrete	$ 1,009,014.00			$ 1,009,014.00
[illegible]	$ [illegible]		$ [illegible]	$ [illegible]
Reinforcement	$ 967,950.00		$ 298,950.00	$ 1,266,900.00
[illegible]		$ [illegible]	$ [illegible]	$ [illegible]
Slab Finish			$ 47,925.00	$ 47,925.00
[illegible]		$ [illegible]	$ [illegible]	$ [illegible]
[illegible]	$ [illegible]	$ [illegible]	$ [illegible]	$ 5,869,697.00

Table 10: Concrete Cost Summary

Steel Cost Summary				
Building Components	Cost			
	Material	Equipment	Labor	Total
Steel Framing	$ 4,156,250.00	$ 191,520.00	$ 525,350.00	$ 4,873,120.00
[illegible]	[illegible]	[illegible]	$ [illegible]	$ [illegible]
Concrete	$ 334,512.00			$ 334,512.00
[illegible]		$ [illegible]	$ [illegible]	[illegible]
Welded Wire Fabric	$ 118,584.00		$ 102,900.00	$ 221,484.00
[illegible]			[illegible]	$ [illegible]
Fireproofing			$ 106,499.00	$ 106,499.00
[illegible]	$[illegible]	$392,240.00	$[illegible]	$6,972,992.00

Table 11: Steel Cost Summary

Schedule

A project schedule was generated for each of the structural systems using the time acquired from crew labor and unit amounts. The floor plan was broken down into four construction zones based on the limited area of a single concrete slab pour. The zones are indicated in the figure below. As state before, a floor-by-floor method was utilized for this schedule analysis. A detailed breakdown of tasks and durations as well as schedules for both the existing concrete structure and the proposed steel structure can be found in Appendix H. The overall estimated construction duration for the concrete structural system was 337 days. This was determined using only the number of crews provided by R.S. Means. By considering multiples crews or larger crew sizes, the construction process could be shortened however it would cause the total cost to increase.

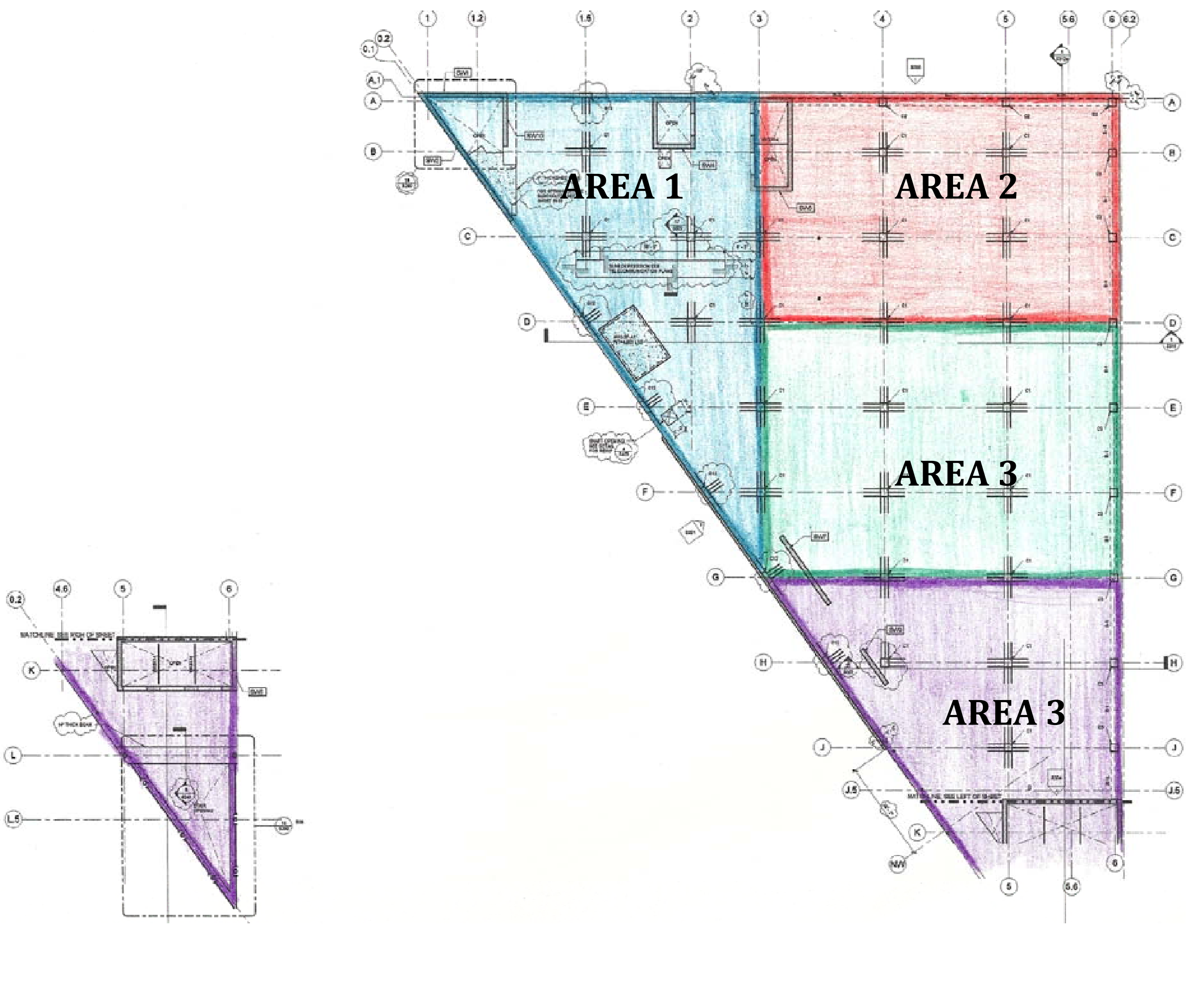

Figure 31: *Building Construction Zones*

For the proposed steel structural system, the building zones remained the same for convenience of this analysis, however the number of zones could have been reduced and the area of each zone increase. Again, a floor-by-floor construction method was employed and a project schedule can be found in Appendix H to view the order of tasks completed. The overall estimated construction duration for the steel structure was 171 days.

Construction Management Summary

Through the completion of this breadth analysis, a comparison of the efficiency and constructability of the existing concrete structure and the proposed steel structure could be evaluated. A summary of the results can be seen in the following table and the detailed information and calculations can be found in Appendix H. The cost estimate for the current concrete structure was determined to be approximately $5.87 million, which ended up being less than the $6.97 million that was estimated for the proposed steel framing structure. Although the estimated costs were over a million dollars more for the steel structure, it took less than half the time for erection and construction than the existing concrete structure.

Structural System Comparison			
Exisiting Concrete Structure		**Proposed Steel Structure**	
Time		Time	
Days	337	Days	171
Cost		Cost	
Material	$3,273,794.00	Material	$5,487,464.00
Labor	$435,501.00	Labor	$392,240.00
Equipment	$2,160,402.00	Equipment	$1,092,788.00
TOTAL	$5,869,697.00	TOTAL	$6,972,392.00

Table 12: *Structural System Comparison*

CONCLUSIONS

Goal Evaluation

To evaluate the success of the structural redesign the design goals set forth at the beginning of this report were revisited to assess. The original goals are listed below with conclusions and arguments to support whether or not the design goals have been successfully achieved. After the design goals evaluated, final conclusions and recommendations can be determined.

Goal 1: Design a steel structure that has little impact on the existing architecture of the Army National Guard Readiness Center.

This goal was achieved by continually considering the architecture during the design of the steel structure. The perimeter moment frames do not change the façade and allow the interior to be open where necessary. The original column grid was consulted when configuring the new grid for the steel structure. The only aspect that may change would be the floor-to-ceiling height however there are two solutions that would resolve this problem.

Goal 2: Choose a lateral system and layout that will work effectively for the Army National Guard Readiness Center and loads determined in accordance with ASCE 7-05.

The new lateral system consists of moment frames along the perimeter of the building. This system probed to be a good choice for the Army National Guard Readiness Center because it left the original architecture intact while providing adequate progressive collapse mitigation.

Goal 3: Design a structural steel structure that will reduce the overall building costs

This goal was met by designing a composite steel deck flooring system with composite steel beams and girders. Steel columns were designed to carry the appropriate gravity loads. Steel moment resisting frames were determined to be a suitable lateral solution.

Goal 4: Analyze the proposed steel structural system to meet progressive collapse mitigation.

> *Researching both the U.S. General Service Administration and the Department of Defense requirements for progressive collapse mitigation and applying the analysis guidelines for both the proposed steel system accomplished this goal. Once the analysis procedure was completed, member sizes were upgraded to meet the progressive collapse standards.*

Conclusions

This thesis report was conducted in order to determine the feasibility or redesigning the Army National Guard Readiness Center as a steel framed structure. After taking into account all of the outcomes of the redesign, it was concluded that that the pros and cons of both systems weigh each other out and either of the structural systems would be a viable and practical solution for the building. Through this analysis a better appreciation for the overall building design and how all aspects of a building work together was gained as well as a better understanding of progressive collapse mitigation design and guidelines.

For the depth of this thesis report, the structural system of the Army National Guard Readiness Center was redesigned as a composite metal deck and concrete slab floor system with composite beams and girders and steel columns. This was a complete redesign from the two-way concrete flat slab and concrete columns that act as the current structural system. The lateral system was converted from ordinary reinforced shear walls to steel moment frames located at the perimeter of the building. The perimeter moment frames were also analyzed for progressive collapse mitigation in accordance with both the GSA guidelines used for federal buildings and the DoD guidelines, which were used by the structural engineers for the original design. A RAM Structural Systems model was generated once the preliminary gravity system was laid out and sized. The model was used to optimize that gravity system and increase the redundancy of member sizes to cut down on fabrication costs as well as decreasing the complexity of erection in the field. The RAM model also assisted in the design of the lateral system members and layout.

Two breadth studies were performed along with the depth analysis to investigate other aspects of the building that were affected by the structural redesign of the Army National Guard Readiness Center. The first breadth study was an acoustical analysis to determine if the change to a steel system would negatively impact the acoustical aspects of the office spaces located on the 5T level. An area of the mechanical penthouse which houses two large cooling towers was the focus of this study due to the sound pressure caused by the towers and their location directly over the open office area. Once the sound pressure of the cooling towers was determined the background noise of an office area was listed and then the required transmission loss could be calculated. Once the required transmission loss was known it was determined that the actual transmission loss of the proposed floor system was sufficient in reducing the sound pressure to an acceptable level for office spaces.

A construction management analysis was the second breadth topic for this thesis report in order to determine the constructability and feasibility of the redesign for the Army National Guard Readiness Center. A cost analysis for the existing concrete structure was performed and then compared to a cost analysis that was performed for the proposed steel structure. It was determined that the steel framing system would be approximately one million dollars more than the current structure. The construction schedules for both systems were also generated for this breadth study. From the construction schedules it was concluded that it would take twice as long to construct the concrete structure as it would the steel structure therefore posing both an advantage and disadvantage of the structural redesign.

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