Final Report

Rendering provided by DCS Design

Kingstowne Section 36A
5680 King Center Drive
Kingstowne, VA 22315

James Chavanic

Structural Option

Advisor: Dr. Boothby

April 3, 2013

Kingstowne Section 36A

5680 KING CENTER DRIVE
KINGSTOWNE, VA 22315

James Chavanic — Structural

Project Team

Owner:
Kingstowne Office 36 LP
General Contractor:
L.F. Jennings Inc.
Architect:
Davis, Carter, Scott Ltd.
Civil Engineer:
Tri-Tek Engineering
Mechanical Engineer:
Jordan & Skala Engineers
Structural Engineer:
Cagley & Associates

Building Overview

Occupancy:	Office Parking Garage Retail
Size:	202,145 GSF
# Of Stories:	8 Total 4 Parking/Retail, 4 Office
Height:	86'-11" From Average Grade
Cost:	$19 Million
Delivery:	Design-Bid-Build


Source: DCS Design

Structure

Foundation:
- Spread Footings and Mat Foundations bearing on Geopiers

Office Levels:
- Wide-flange beams and columns supporting a composite floor
- Braced frames and moment frames transfer lateral loads

Parking Garage Levels:
- Sloped, 8 inch thick, two-way flat slab with drop panels
- 12" thick concrete shear walls transfer lateral loads


Source: James Chavanic 8-10-12

Architecture

When completed, Kingstowne Section 36A will be part of a master planned development for retail and office space. The appearance of this development can be characterized by a rectilinear footprint, pink velour brick, aluminum storefront with glass of blue/black appearance, and precast concrete bands around the circumference of the building.

Electrical

- 480/277V 3 phase for mechanical and lighting loads
- 208/120V 3 phase for receptacle and other loads


Source: DCS Design

Mechanical

- Four rooftop units with natural gas fired heating ranging in total CFM from 19,500 to 21,500 provide heating and cooling to the office levels
-Two 5.0 kW electric unit heaters providing 350 CFM each in the retail space
-Four 5.0 kW electric unit heaters providing 350 CFM each at the highest level of parking
-Three split system heat pumps ranging in total CFM from 600 to 1,800 provide cooling to the lobby and retail space

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EXECUTIVE SUMMARY

Kingstowne Section 36A (KT36A) is a 200,000 SF mixed use building currently being constructed in Fairfax County Virginia. When completed, the lower half of the building will serve as a parking garage serving the office tenants of the upper half of the building. The parking garage levels utilize flat slab concrete construction while the office levels use a composite steel construction. A more thorough description of the existing structure can be found in the first half of this report.

The goal of this thesis was to use one type of structural system (reinforced concrete) throughout the height of the building, simplifying coordination of construction and hopefully reducing the cost of the overall structure. Once the office levels and roof level were designed for gravity and lateral loads, the structure was analyzed and designed to resist progressive collapse following guidelines adopted by the U.S. Department of Defense. According to the guidelines, a thorough design for progressive collapse also incorporates assessing elements outside of the building structure, including the surrounding site and exterior building façade. Following through with this, a site layout redesign was conducted to reduce the risk of events that could initiate progressive collapse. Finally, new glazing for the building would be sized to resist pressures caused by a specified explosive charge, with the goal of maintaining the thermal performance of the existing systems.

Design loads on KT36A were calculated in accordance with ASCE 7-10. From here, structural design for both the gravity and lateral systems was completed using ACI 318-11. A three-dimensional model of the building structure created in ETABS was also used to aid in the design of the lateral system. Once the structure was designed for gravity and lateral loads, a design for progressive collapse was conducted following UFC 4-023-03 and GSA guidelines for designing against progressive collapse. The resulting design consists of 8" thick slabs with drop panels of the same thickness at the columns, 24" wide X 28" deep edge beams spanning the E-W direction, 24" wide X 31" deep edge beams spanning the N-S direction, columns of varying size. Shear walls are all 12" thick and primarily reinforced with #4 @ 12" O.C. in each face for the vertical and horizontal reinforcement.

Using the United States General Services Administration (GSA) Site Security Design Guide, modifications to the site design layout were implemented to reduce the risk of building and structural damage associated with vehicular impact and exterior explosion. Structural bollards, hardened site furniture, large planters, and security booths were all applied to the site to reduce the possible associated risks.

New glazing for the parking levels and office levels and an aluminum frame support system were designed to withstand the maximum wind pressures and pressure resulting from 80 lbs of TNT exploding at a standoff distance of 35' away. Parking level glazing remained as an uninsulated system, but was increased in thickness to 5/8". Glazing for the office levels also required a thicker system, which remained an insulating glass unit (IGU). Heat transfer analyses were conducted for both the existing and newly designed IGU's. The results found that the new glazing allowed more heat gain in both the summer and winter. While this could be desired in the winter months, it is not desirable during the summer months.

ACKNOWLEDGEMENTS

To my loving fiancée, Mom, Dad, sister, and Grandpa; who have taught me so much about life and helped make me who I am today. My gratitude for your endless support through this journey we call pursuing an AE degree cannot be described in words. Thank you for believing in me.

I would also like to thank my friends for keeping me sane and teaching me that you must have fun every now and then.

I would like to extend a special thanks to the following individuals for their knowledge, advice, and support in making meaningful completion of this senior thesis a possibility.

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- Professor Parfitt
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AE Graduate Students:

- David Tran
- Ryan Solnosky

Past Graduates

- Michael Kostick

Cagley & Associates

- Frank Malits
- Nehemias Iglesias

Halle Companies

- Rich Rounds

DCS Design

- Carmencita Calong

Building Introduction

Kingstowne Section 36A (KT36A) is a 200,000 ft^2, 8 story office building to be located in Fairfax County Virginia. It will contain 4 levels of concrete structure parking garage and 4 levels of composite steel construction office space. Floor space has also been allocated for about 5,000 square feet of retail area on the ground floor (Parking Level 1). KT36A will be 86'-11" in height when measured from the average grade. The reason the building height is measured from average grade is because there is a significant grade elevation change from the south side of the building to the north side, on the order of 26'-8" (See Figure 1). This poses unique challenges in the structural design of the building since the geotechnical report states the soil placing a load of 60psf/ft in depth below grade surface on the structure. This means that there is more than 1600 psf of soil load on the foundation walls at the lowest slab levels. This load alone had enough impact on the building that six 12" thick shear walls had to be constructed at parking level 1 to transfer the loads safely.

When completed, KT36A will be part of a master planned development for retail and office space owned by the Halle Companies. Being a part of a master planned development, the building was designed to match the appearance of the surrounding buildings. This appearance can be characterized by a rectilinear footprint, pink velour brick, aluminum storefront with glass of blue/black appearance, and precast concrete bands around the circumference of the building.

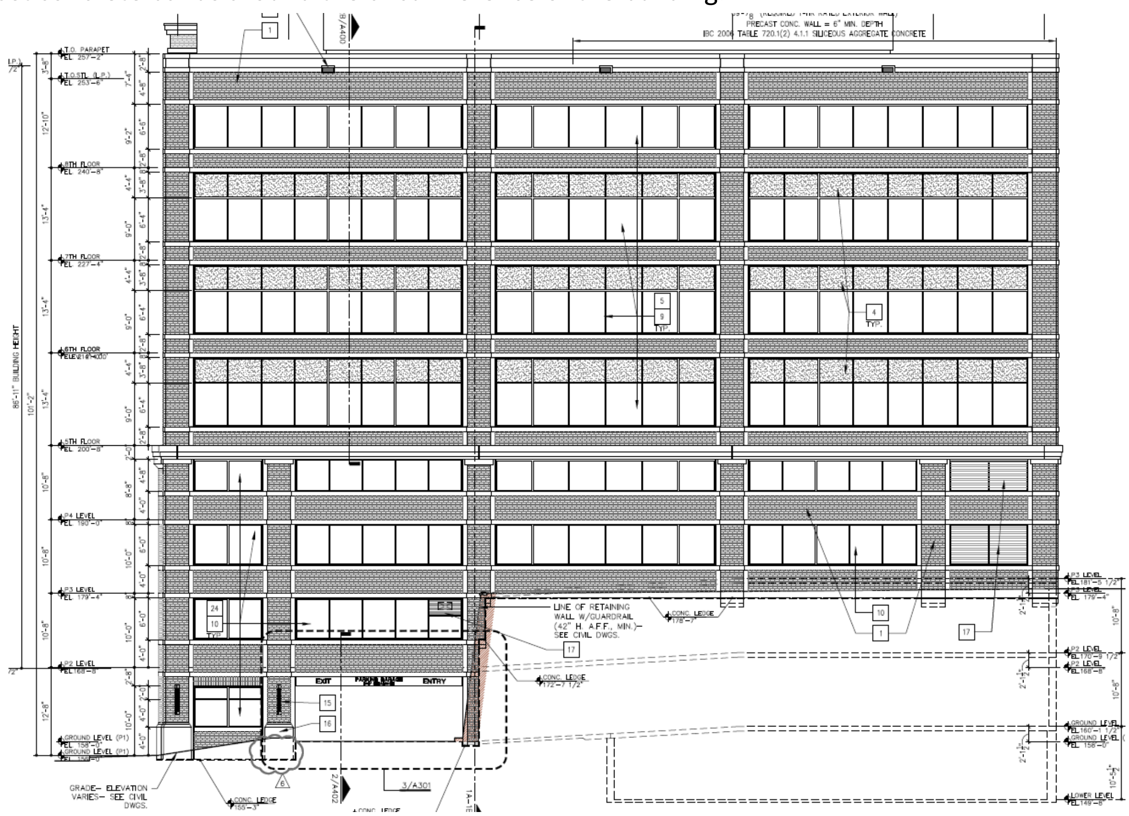


Figure 1: Elevation Looking East Showing Grade Differences (Source: DCS Design Drawing A-301)

EXISTING STRUCTURE OVERVIEW

Kingstowne Section 36A consists of two different primary structural systems; cast-in-place concrete for the lowest four floors of the building and a composite steel system for the remaining four floors. The concrete floors are used for the parking garage and retail space while the steel system is used at the office occupancy levels. Lateral forces in the concrete levels are resisted with 12" thick concrete shear walls of varying height. When the building transitions to steel construction, lateral forces are transferred to the concrete columns and shear walls through concentrically braced frames, eccentrically braced frames, and rigid moment frames. Per sheet S-001, components such as steel stairs and curtain wall/window systems were not included in the scope for the structural design of this building.

FOUNDATIONS

In their report submitted August of 2009, Burgess & Niple, Inc. (B&N) advised that shallow foundations not be used on this project due to settlement concerns based on subsurface conditions. They performed five new soil test borings, ranging from 45 to 100 feet in depth below the grade surface. In addition, they reviewed 14 borings from previous investigations, ranging in depth from 10 to 55 feet below grade surface.

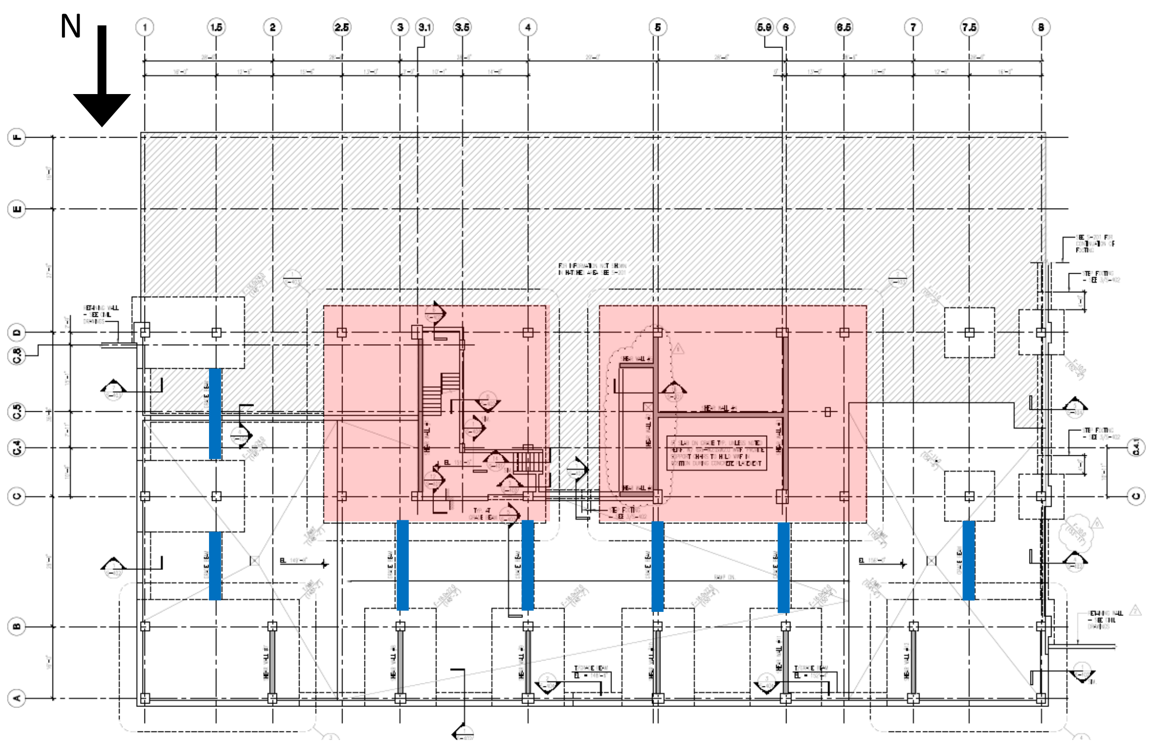


Figure 2: Foundation Plan (Level P0) Showing 48" Thick Mat Foundations Shaded in Red (Source: Cagley & Assoc. Drawing S-200)

Each of the borings found lean clay and fat clay fills with varying amounts of sand, residual soils consisting of lean to fat clay, and clayey to silty sands. Based on the fill materials being encountered between 4 and 48 feet below grade, B&N offered two foundation options. An intermediate foundation system consisting of spread and strip footings bearing on rammed aggregate piers (Geopiers) was chosen for KT36A over the alternate option of a deep system consisting of spread and strip footings bearing on caissons. Geopier diameters typically range from 24 to 36 inches and are compacted using a special high-energy impact hammer with a 45-degree beveled tamper. Per B&N report, footings supported by Geopier elements can be designed using a maximum bearing pressure of 7,000 psf.

Using the information provided by B&N, Cagley & Associates designed spread footings ranging from 27" to 44" in depth to support the columns of KT36A. 48" thick mat foundations bearing on Geopiers are located at the central core of the building to transfer forces in the main shear walls to the soil (See Figure 2). Grade beams (Blue lines in Figure 2) of 30" depth are used throughout level P0 to also transfer forces from the shear walls to the column footings. Foundation walls are supported by continuous wall footings designed for an allowable bearing pressure of 2,500 psf. All foundations are to bear a minimum of 30" below grade unless stated otherwise.

GARAGE LEVELS

FLOOR SYSTEM

As previously mentioned, KT36A utilizes cast-in-place concrete for the support structure in the garage. With the exception of the 5" thick slab on grade, this system consists of 8" thick two-way, flat slab construction with drop panels that project 8" below the bottom of structural slab. The drop panels are continuous between grid lines C and D to help the slab span the increased distance of 36'-6" in this bay, otherwise, they are typically 10'-0" x 10'-0" in size. Due to the need for vehicles to circulate vertically throughout the parking garage levels, the floor is sloped on 3 sides of the central core to achieve this.

Since a two-way, flat plate concrete floor system is subjected to both positive and negative moments, reinforcing steel is required in the top and bottom of the slab. The typical bottom mat of reinforcement in KT36A consists of #4 bars spaced at 12" on center in each direction of the slab. Additional bottom reinforcement in certain middle strips and continuous drop panels is also noted on the drawings. Top reinforcement is comprised of both #5 and #6 bars, both oriented in the same fashion as the bottom mat, with the #6 bars typically being used in the column strips to resist the larger negative moments present there (see Figure 3 for a typical bay layout). A typical bay size for the concrete levels is 28'-6" x 29'-0".

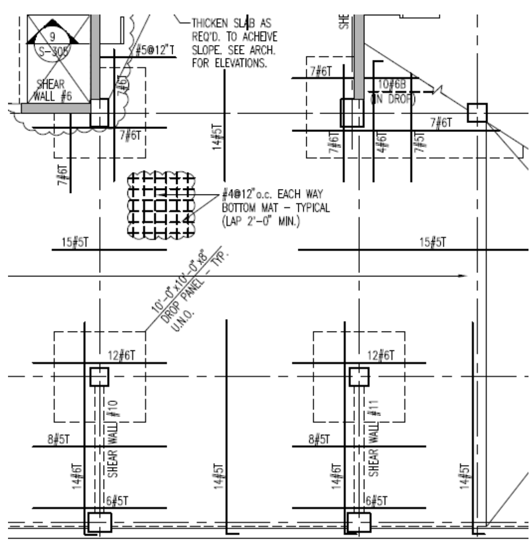


Figure 3: Partial Plan Level P1 (Source: Cagley & Assoc. Drawing S-201)

FRAMING SYSTEM

Supporting the floor slabs are cast-in-place concrete columns constructed of 5000 psi concrete. The most common column size is 24" x 24" reinforced with a varying number of #8 bars and either #3 or #4 ties. Columns of this size primarily account for the gravity resisting system of KT36A. The largest columns used are 36" x 30" reinforced with a varying number of #11 bars and #4 stirrups. The larger columns are located at the ends of the large shear walls in the central core of the building. A small number of concrete beams are also present in the project, typically at areas of the perimeter where additional façade support was needed and at large protrusions in the floor slab.

LATERAL SYSTEM

Cast-in-place concrete shear walls resist the lateral forces present in the parking garage levels of KT36A. All of the twelve walls present in the building are 12" thick and cast using 5000 psi concrete. Six of the shear walls (#1 - #6, see Red lines in Figure 4) extend 4-5 stories from the 48" thick mat foundations to office level 1 which is also the 5th elevated floor of the building. Three of the six walls are oriented to resist lateral forces in the N-S direction while the other three walls are oriented in the E-W direction. The remaining six walls (#7 - #12, Green lines in Figure 4) are only one story tall and are oriented to best resist the unbalanced lateral soil load placed on KT36A.

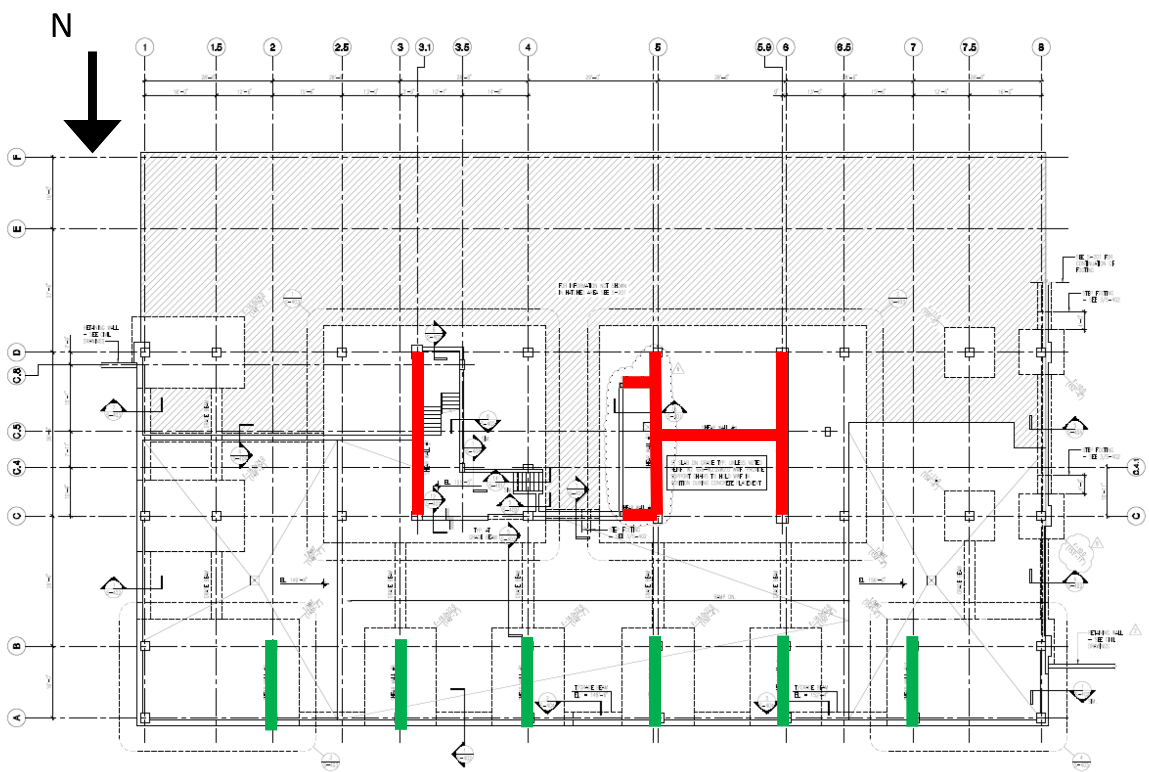


Figure 4: Foundation Plan (Level P0) Showing Shear Walls (Source: Cagley & Assoc. Drawing S-200)

Office Levels

FLOOR SYSTEM

Office level 1 is constructed of the same cast-in-place style of construction as the garage floors below it with the exception of the top of slab elevation being uniform throughout the floor. The remaining floors are constructed using a composite steel system. This system is comprised of 3 ¼" thick lightweight concrete on 2" x 18 gage galvanized composite steel decking. The 3000 psi lightweight concrete (115 pcf) coupled with the decking yields a total slab thickness of 5 ¼". Reinforcement for the slab is provided by 6x6-W2.1xW2.1 welded wire fabric.

According to sheet S-001, all decking should meet the three span continuous condition. The decking typically spans 9'-6" perpendicular to cambered beams of varying size. Shear studs of ¾" diameter placed along the length of the beams make this a composite system capable of more efficiently carrying the loads when compared to a non-composite system. The studs must be minimum length of 3 ½" but no longer than 4 ½" to meet designer and code requirements.

FRAMING SYSTEM

The composite floor system mentioned above is supported by structural steel framing comprised of primarily wide flange shapes. W21's and W18's account for most of the beams while the columns range in size from W12x40 to W14x109. A majority of the beams in KT36A are cambered between ¾" and 1 ¼", a function of the span and load demand on the beams. With the exception of four W30x99 sections cambered 1", most of the girders fall within the same size range as the beams. The four W30x99 girders each span 44'-0" which warrants the use of the camber to satisfy the total deflection criteria. The columns are all spliced just above the 7th floor (office level 3) where they are reduced in size to more economically carry the lighter axial loads. See Figure 5 below for a typical office floor level layout.

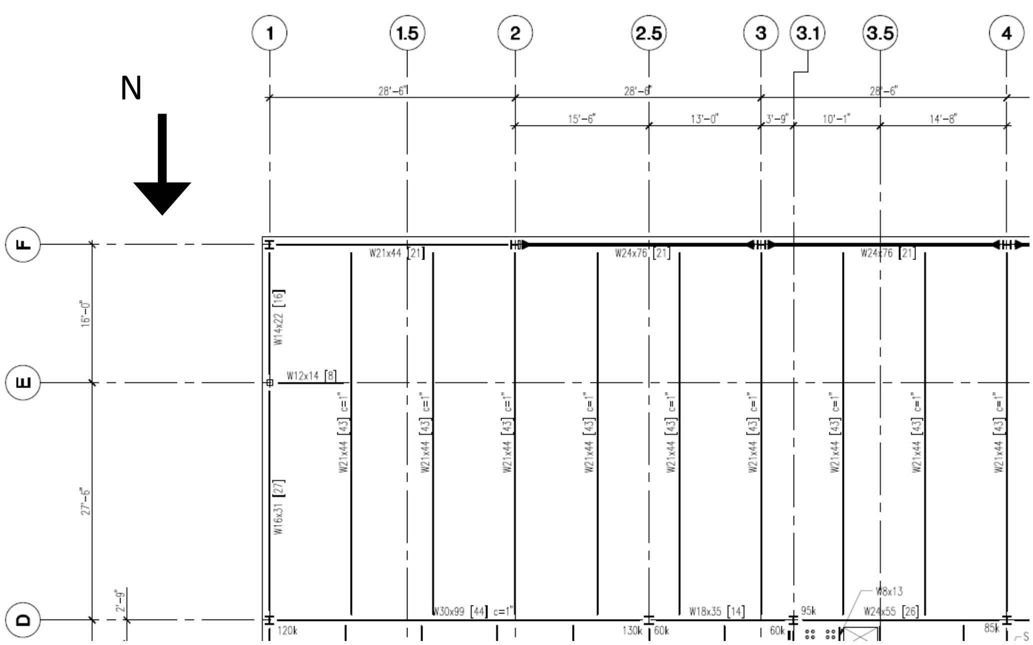


Figure 5: Typical Composite Slab Partial Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

LATERAL SYSTEM

Lateral forces at the office levels are transferred to the concrete shear walls through three different frame systems. Concentrically braced (Green Line) and eccentrically braced frames (Purple Lines) work in the north – south direction while ordinary steel moment frames (Orange Lines) resist the loads in the east – west direction. See Figure 6 for their location and orientation within the building. The eccentrically braced frames were necessary to maintain enough clearance for a corridor in that area of the building. Diagonal bracing for the frames consists of either HSS10x10 or HSS9x9 of varying thickness. Moment frames were most likely chosen for the east – west direction so as not to obstruct the occupants view to the exterior and lower lateral loads acting on the building in this direction.

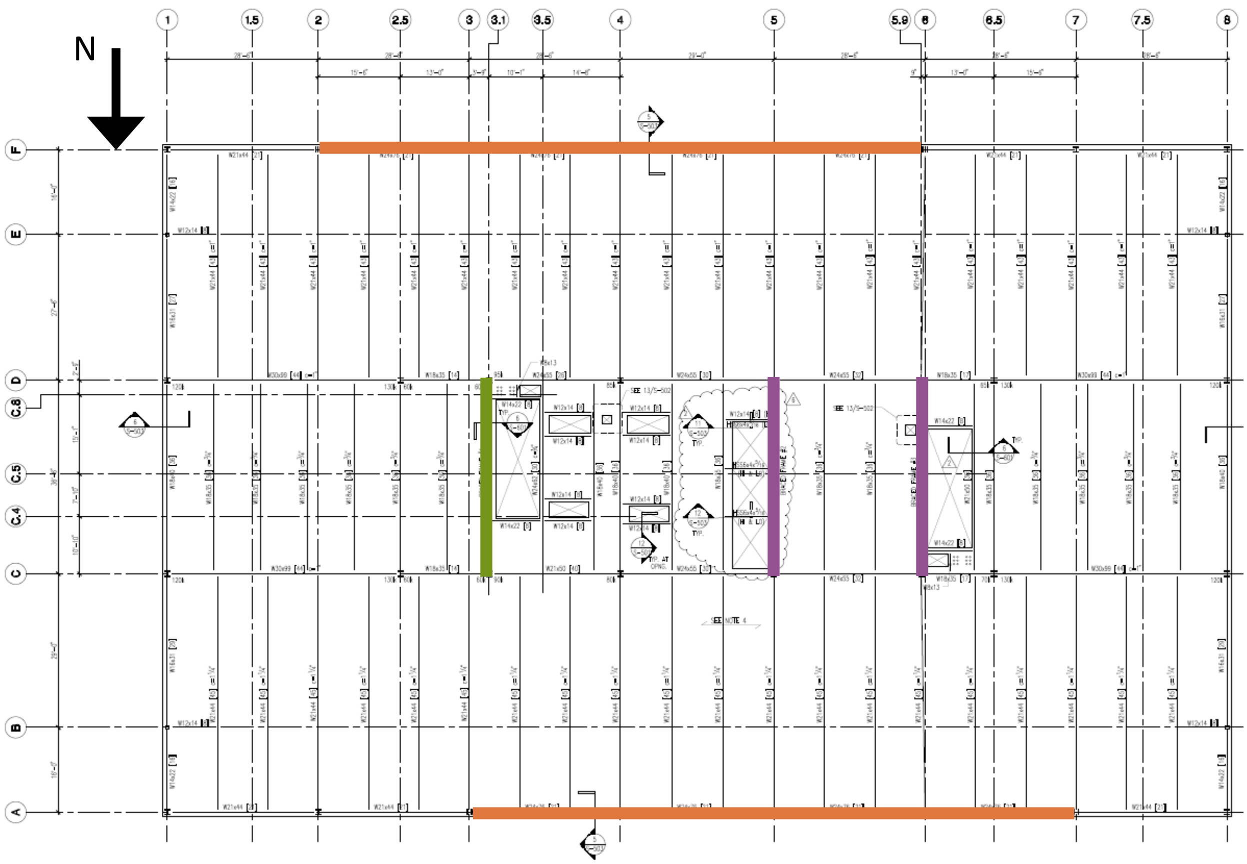
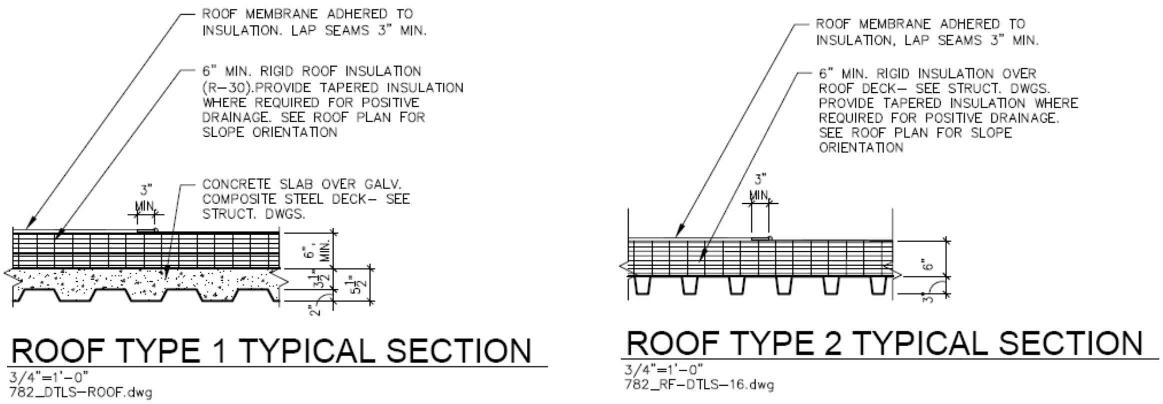


Figure 6: Typical Composite Slab Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

Roof System

The roofing system consists of a white EPDM membrane fully adhered over 6" minimum of R-30 continuous rigid roof insulation. The seams of the membrane must be lapped a minimum of 3" to ensure a watertight seal. Where mechanical equipment is located (see Figure 9), the roofing materials are supported by 2"x 18GA galvanized composite steel deck with a 3.25" thick light-weight concrete topping. The load carrying capacity that this type offers is required to support the four 17,000lb roof top mechanical units needed to condition the air for the building occupants. In all other areas of the roof, the system is supported by 3"x 20GA type N roof deck. Each of the roof types are supported by steel W-shapes that are sloped to achieve proper drainage.



Figures 7 and 8: Typical Roofing Details (Source: DCS Design Drawing A-410)

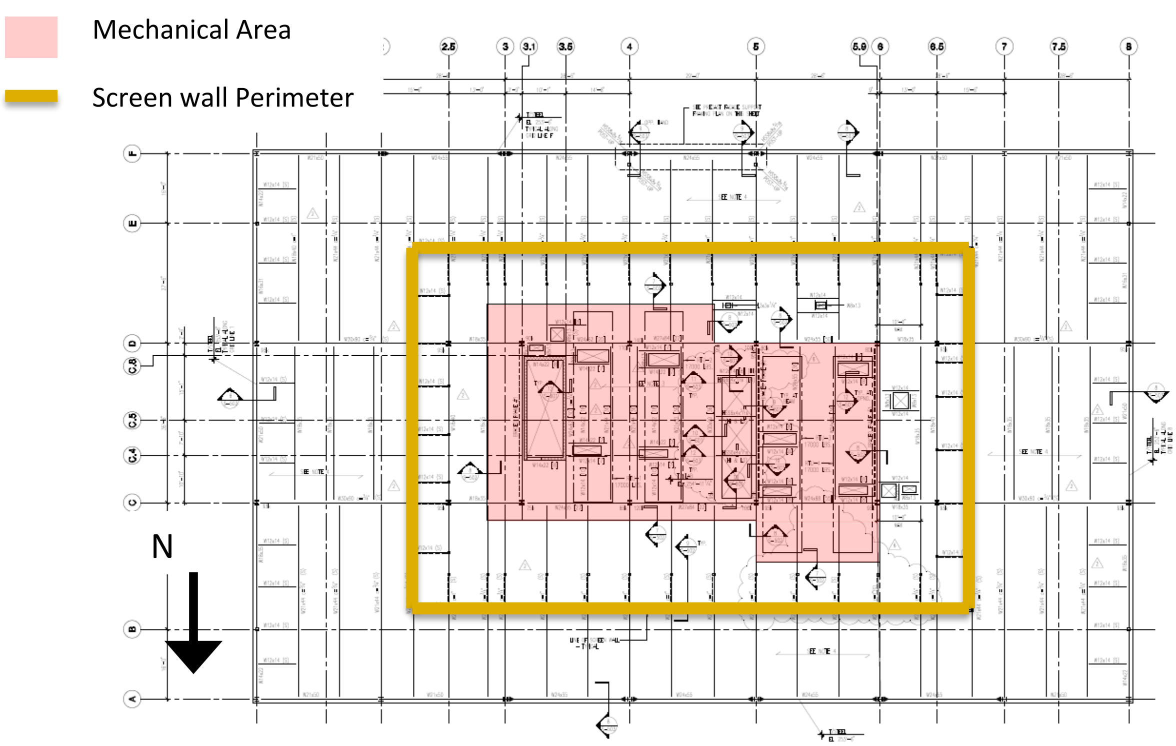


Figure 9: Structural Roof Plan (Source: Cagley & Assoc. Drawing S-209)

THESIS PROPOSAL

STRUCTURAL DEPTH

PROBLEM STATEMENT

As previously stated in the structure overview, Kingstowne 36A is constructed of two completely different structural systems. Since the construction practices for the two systems are also different, separate trades are required to complete the work. This leads to increased costs since separate labor forces need to be mobilized and more complex construction sequencing.

In addition to the increased costs of bringing different trades to the site, Technical Report 2 revealed that the existing composite steel system at the office levels is the most expensive of the considered floor systems. After comparing the existing and alternate floor systems, the cast-in-place concrete flat slab already being used in the garage levels was found to be one of the least expensive options. Considering this cost reduction and the previously mentioned factors, changing the structural system of the office levels to cast-in-place concrete flat slab could lead to a lower building cost and faster completion time.

PROBLEM SOLUTION

Cast-in-place concrete creating a flat slab structural system will be used to redesign the existing composite steel structure at the office levels of Kingstowne 36A. In their current configuration, the office levels have fewer column lines than the parking garage levels below. This is due to the steel system being able to efficiently span farther distances than the concrete system. Having greater span lengths and fewer columns in the office space allows a more flexible layout for the tenant, which is likely the reasoning for switching to the steel construction at the office levels. This impact on the architecture and function of the interior layout will be considered acceptable for the purposes of the proposed analysis. A design for the first office floor level is contained in the provided structural drawings. Considering the remaining three office floors are identical to the first one, the concrete redesign will focus on the roof level where large mechanical equipment loads are located.

Upon being informed that the building would be entirely constructed of concrete now, a governmental agency has accepted tenancy in the building. Adhering to the guidelines of the United States General Services Administration, the building must now be designed to resist progressive collapse. Edge beams will be added to the perimeter of the building at the office floor levels to help transfer the loads in the event of removal of a critical structural component. In order to analyze the effects of a progressive collapse scenario, SAP2000 will be utilized to implement the alternate load path method for analysis in accordance with UFC-4-023-03 (Design of Buildings to Resist Progressive Collapse). Depending on the results of the analysis, a perimeter transfer girder system may be added at the roof level to aid in transferring the load to adjacent supporting elements.

Considering the fact that the concrete system will weigh significantly more than the existing steel system, increased dead load will be placed on the existing concrete columns and foundation systems. The current designs will be evaluated and adjusted based on the new loading conditions.

Breadth Topics

BREADTH 1: SITE LAYOUT REDESIGN

One of the best ways to protect against a progressive collapse situation is to reduce the risk of it happening in the first place. This is accomplished through site layouts that minimize potential risks such as explosions and vehicular impacts through strategic site logistics and landscape architecture. Modifications will be made to the existing site plan for Kingstowne 36A to minimize the potential risks. The modifications can include, but are not limited to; increasing stand-off distance, installing barriers, and employing energy deflection shields. The modified site plan will be presented showing the measures taken to create a safer building perimeter.

BREADTH 2: BUILDING ENVELOPE AND FAÇADE STUDY

Kingstowne 36A is currently clad in a precast-concrete panel, combined with thermal glass and plain glass, façade. This system, however, is most likely not resistant to blast loading. Cladding the building in a blast resistant façade will help to further mitigate the risks that can potentially cause a progressive collapse scenario. The current system will be evaluated with a heat transfer and performance analysis to determine the effectiveness of the façade. This analysis will then be used as the basis to design an alternative façade system that is blast resistant. An additional goal to obtain with the new façade system is to, at a minimum, match the performance of the existing façade.

MAE Requirements

To meet the MAE curriculum requirements for the proposed senior thesis, knowledge and skills acquired from AE 530, Computer Modeling of Building Structures; AE 538, Earthquake Engineering; and AE 542, Building Enclosure Science and Design will be applied. Redesign of the existing structure to entirely cast-in-place concrete construction will be modeled in ETABS to aid in the analysis and design of the structure. Design methods presented in AE 538 will be used to design the new shear walls that will be added and determine if the existing shear walls have enough capacity to resist the seismic loads, considering seismic loads are expected to control the lateral design due to the increased weight of the structure. Material covered in AE 542 will be used to evaluate the existing façade system and design a replacement that is blast resistant.

General Design Provisions

Police services in Fairfax County Virginia have decided to make KT36A their dispatch headquarters for the surrounding areas. This escalates the building to Risk Category IV which has significant impacts on its' design. Risk Category IV has been assigned considering the facility must maintain safe functionality in a time of natural disaster (rare and powerful earthquake) or emergency crisis situations. Now considered a high risk building with critical functions, the design team has decided to design KT36A to satisfy the requirements set forth by the General Services Administration and Unified Facilities Criteria.

Design Codes And Standards

Per sheet S-001, Kingstowne Section 36A was designed in accordance with the following codes:

- 2006 International Building Code
- 2006 Virginia Uniform Statewide Building Code (Supplement to 2006 IBC)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- ACI Manual of Concrete Practice, Parts 1 through 5
- Manual of Standard Practice (Concrete Reinforcing Steel Institute)
- Building Code Requirements for Masonry Structures (ACI 530, ASCE 5, TMS 402)
- Specifications for Masonry Structures (ACI 530.1, ASCE 6, TMS 602)
- AISC Manual of Steel Construction, 13th Edition
- Detailing for Steel Construction (AISC)
- Structural Welding Code ANSI/AWS D1.1 (American Welding Society)
- Design Manual for Floor Decks and Roof Decks (Steel Deck Institute)

Codes / Manuals referenced for the purposes of this report:

- IBC 2009 - International Building Code, 2009 Edition
- ASCE 7-10 – Minimum Design Loads For Buildings and Other Structures, 2010 Edition
- ACI 318-11 – Building Code Requirements For Structural Concrete, 2011 Edition
- ASCE 41-06 – Seismic Rehabilitation of Existing Buildings, 2006 Edition
- UFC 4-023-03 – Design of Buildings to Resist Progressive Collapse, 2009 Edition
- The Site Security Design Guide – General Services Administration
- Progressive Collapse Analysis and Design Guidelines – General Services Administration
- ASTM E1300-12a – Standard Practice for Determining Load Resistance of Glass in Buildings
- ASTM F2248-12- Standard Practice for Specifying an Equivalent 3s Duration Design Loading for Blast Resistant Glazing

MATERIAL PROPERTIES

Minimum Concrete Compressive Strength	
Location	**28 Day f'c (psi)**
Footings	3000
Grade Beams	3000
Foundation Walls	5000
Shear Walls	5000
Columns	5000
Slabs-on-Grade	3500
Reinforced Slabs	5000
Reinforced Beams	5000
Elevated Parking Floors	5000
Light Weight on Steel Deck	3000

Max. Concrete W/C Ratios	
f'c @ 28 Days (psi)	**W/C (Max)**
$f'c \leq 3500$	0.55
$3500 < f'c < 5000$	0.50
$5000 \leq f'c$	0.45
Elevated Parking	0.40

Reinforcement:

- Deformed Reinforcing Bars — ASTM A615, Grade 60
- Welded Wire Reinforcement — ASTM A185

Masonry:

- Concrete Masonry Units — Light weight, Hollow ASTM C90, Min. f'_m = 1900 psi
- Mortar — ASTM C270 – Type M (Below Grade)
Type S (Above Grade)
- Grout — ASTM C476 – Min. f'c @ 28 days = 2000 psi
- Horizontal Joint Reinforcement — ASTM A951 – 9 Gage Truss-type Galvanized

Structural Steel:

- Wide Flange Shapes and Tees — ASTM A992, Grade 50
- Square/ Rectangular HSS — ASTM A500, Grade B, F_y = 46 ksi
- Base Plates and Rigid Frame Continuity Plates — ASTM A572, Grade 50
- All Other Structural Plates and Shapes — ASTM A36, F_y = 36 ksi
- Grout — ASTM C1107, Non-shrink, Non-metallic
f'c = 5000 psi

LOADS

DEAD LOADS

Superimposed Dead Loads	
Plan Area	**Load (psf)**
Office Floors	15
Roof	30
Parking Garage Floors	5

Dead loads resulting from system self-weights were calculated and estimated based on required dimensions of structural elements. The self-weight dead loads can be found throughout the body of this report and the appendices as they are dependent on specific structural elements. All reinforced concrete self-weights are based on a density of 150 pcf which includes an allowance for the weight of the rebar. Considering the planned redesign of the existing façade to withstand blast loading, a 100 psf average façade load was estimated based on the 58 psf average façade load used in evaluating the existing design of the building in Technical Report 1.

LIVE LOADS

(IBC Load used for concrete redesign)

Live Loads			
Plan Area	**Design Load (psf)**	**IBC Load (psf)**	**Notes**
Lobbies	100	100	
Mechanical	150	N/A	Non-reducible
Offices	80	80	Corridors used, otherwise 50 psf
Office Partitions	20	15	Minimum per section 1607.5
Parking Garage	50	40	
Retail	100	100	Located on first floor
Stairs and Exitways	100	100	Non-reducible
Storage (Light)	125	125	Non-reducible
Roof Load	30	20	

SNOW LOAD

Snow loads for KT36A were calculated using ASCE 7-10. According to Figure 7-1 in this code, Kingstowne Virginia is located in a 25 psf ground snow load area. After applying equation 7.3-1 in ASCE 7-10, this equates to a 21 psf flat roof snow load which is higher than the 17.5 psf load used in the original design of the building. This is solely attributable to the snow importance factor of 1.2 used as a result of the Risk Category IV classification. Considering the elevated parapet above the entrance at the north side of the building and the screen wall present on the roof, unbalanced (drift) snow load can be of importance in these areas. Drift on the leeward side of the parapet can add an additional 15” of snow to the roof balanced snow load while a drift occurring on the windward side of the screen wall can add an additional 12” to the balanced snow load. The drift at the screen wall may be further reduced depending on the final decision of how much gap to leave between the bottom of the screen wall and the top of the finished roof. Snow load calculations can be found in Appendix A.

Load Cases And Combinations

Section 2.3.2 of ASCE 7-10 lists seven different load combinations for LRFD strength design. The combinations are used to determine the factored ultimate loads on the building for combined gravity and lateral loading. Combination 2 was found to control when only gravity loads were being considered. Also considering dead load, live load, and snow load, combination 4 controlled for wind and combination 5 for seismic. Below are the ASCE 7-10 combinations:

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

Per ASCE 7-10 2.3.2:
Include H with factor of 1.6 when it adds to the primary load effect.

Include H with a factor of 0.9 when it resists the primary load effect

Within the controlling wind load case, four sub-cases must be investigated according to Chapter 27 of ASCE 7-10. Application of the four cases is necessary to understand how wind pressures acting on the building in any direction that is not parallel to the main orthogonal axes affect the building structure. Since the wind load on the north side of KT36A is different from that of the south side, combinations three and four each had to be considered for the different directions. Figure 10 below shows the criteria used for calculating the different load cases.

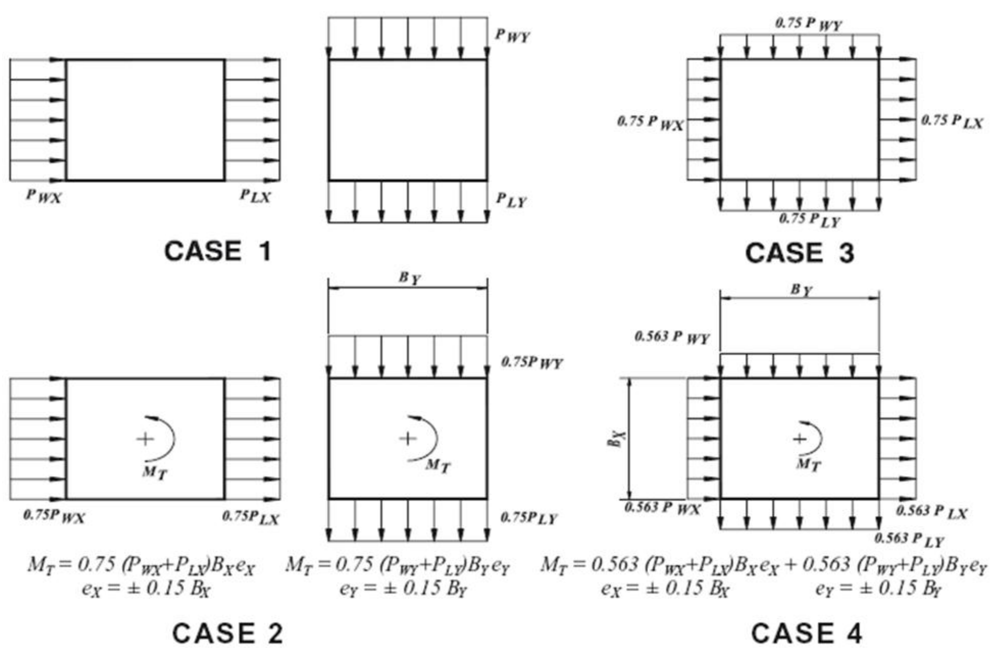


Figure 10: Design Wind Load Cases (Source: ASCE 7-10 Figure 27.4-8)

Gravity Design

In Technical Report 2, the existing floor systems and possible alternatives were investigated to determine feasible and efficient structural systems for the function of Kingstowne Section 36A. The existing composite steel construction office levels were found to be the most expensive system at $25.38 per square foot based on general assembly costs for this system. Considering cost is almost always a major driving force on construction projects, it was decided that designing an entirely concrete structure had the potential to significantly reduce the cost of the structure based on the assembly cost of $16.60 per square foot determined for the existing structure of the garage levels. A comparison of the possible structural systems can be found in Appendix B.

Constructing KT36A with an entirely concrete structure will have some significant impacts on the building as a whole. First, the structural depth required for a flat slab concrete structure is less than that required of a steel system. Considering the 17" of clear space provided below the steel structure of the office levels, 24" of clear space is provided below the 8" concrete slab, the 7" extra used to make up for the space that was available between the steel beams. Factoring in the 9'-0" ceiling height of the current design, an 11'-8" floor-to-floor height results at the office levels. This removes 20" of floor-to-floor height for each floor of the original building design, resulting in a total decrease in building height of 7'-8". Another opportunity for significant cost savings considering building facades are typically a significant cost to owners. Second, the building self-weight will significantly increase, impacting the loads on the existing columns and foundations, likely requiring capacity increases in the new design. A check of the existing foundations can be found on Page 39 of this report. Also impacted is the flexibility of tenant space in the office levels. The two-way flat slab system requires smaller bay sizes than the steel system so more columns will be located throughout the floor area to accommodate this.

Edge Beam Design / Office Levels Slab Check

In anticipation of designing the building against progressive collapse failure, edge beams were added to the floor slabs at all levels to essentially create moment frames around the perimeter of KT36A. Provisions for designing against progressive collapse call for removing perimeter columns at strategic locations. Perimeter moment frames will better allow the structure to adequately bridge the gap created by the removed column. Per the recommendations of the GSA in Appendix B.3 of the Progressive Collapse Analysis and Design Guidelines, the edge beams were designed for a load combination of 2(DL+0.25LL). A trial size for the beam was chosen as 24" wide to match the width of the perimeter columns and 20" deep. This size provides an α_f value of 2.48 which is more than enough for the 0.8 required for the beam to be considered an edge beam. The GSA load combination results in a load of 5.46 klf on the beam at the office levels. This design load was used for designing beams at all levels of the building since it is the highest load that any of the edge beams will see. The moment coefficient method of ACI 318-11 Section 8.3.3 was used to determine the design moments along frames A and F. A frame analysis completed in RAM Elements with pattern live loading was used to determine the loads along frames 1 and 8. The resulting beam designs are shown in Figure 11 on the following page. Typical 1.5" clear cover on all beam reinforcement.

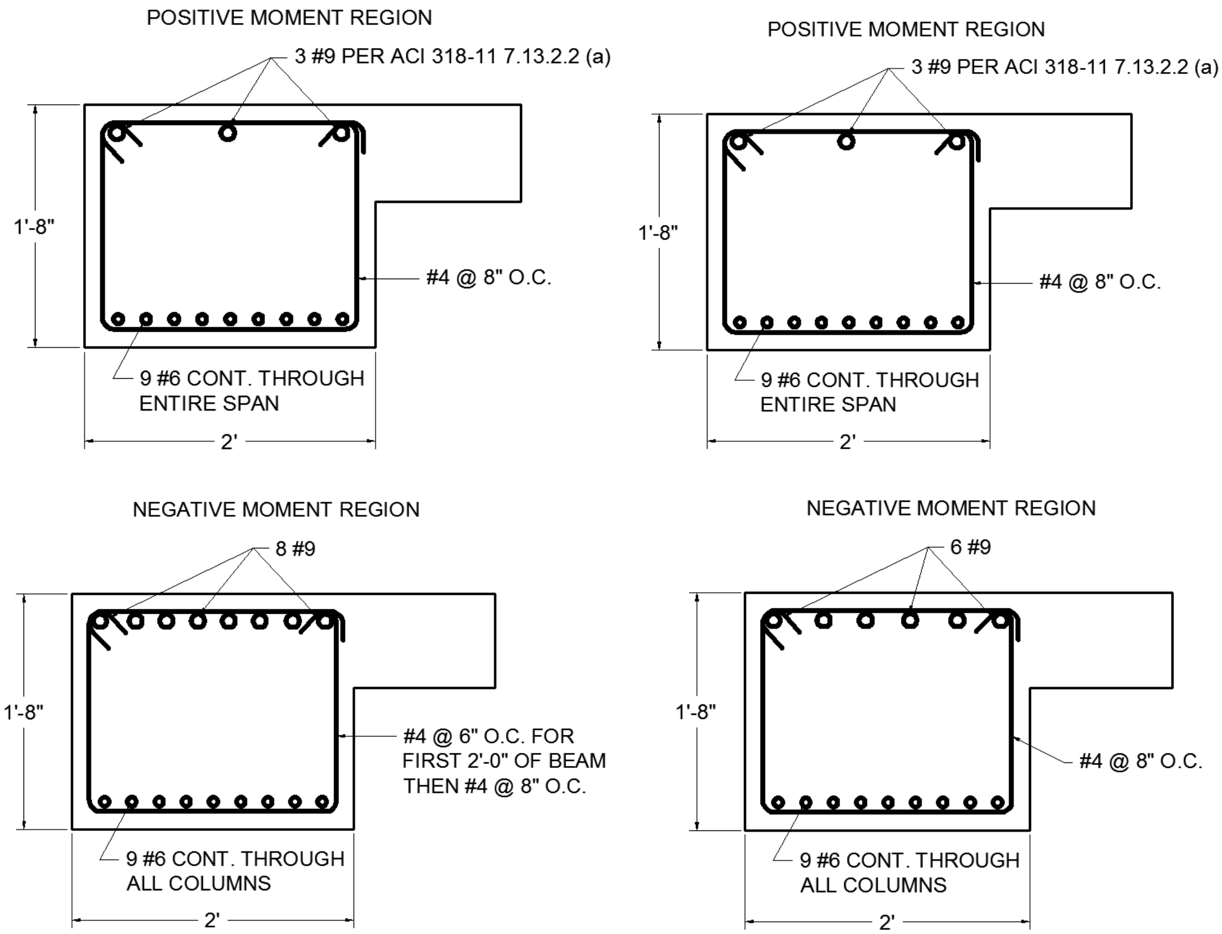


Figure 11: Beam Designs (N-S on Left)   (E-W on Right)

Design of the office floor systems was adopted from the design already completed for OL1 in the original design of the building. OL1 was the last concrete floor in the original structure. The existing design of this floor had to be evaluated for adequacy in the newly designed building. The edge beams added extra negative moment resistance at the end conditions so the provided top reinforcement in these regions had to be checked for appropriate capacity. Upon looking at the existing drawings, the 15 #5 bars provided at the edge condition regions of middle strips are more than adequate since minimum reinforcement for temperature and shrinkage controlled. The 15 #6 bars provided at the column strips are also adequate without considering the GSA load combination of 2(DL+0.25LL). This load combination is intended to be used when starting a design from scratch in attempt to reach a preliminary design that is closer to the final design meeting progressive collapse provisions. Reinforcement in all slabs of the parking and office levels will likely change once provisions for designing to resist progressive collapse are considered. A zoomed in view of the existing slab design at OL1 can be seen on the following page in Figure 12.

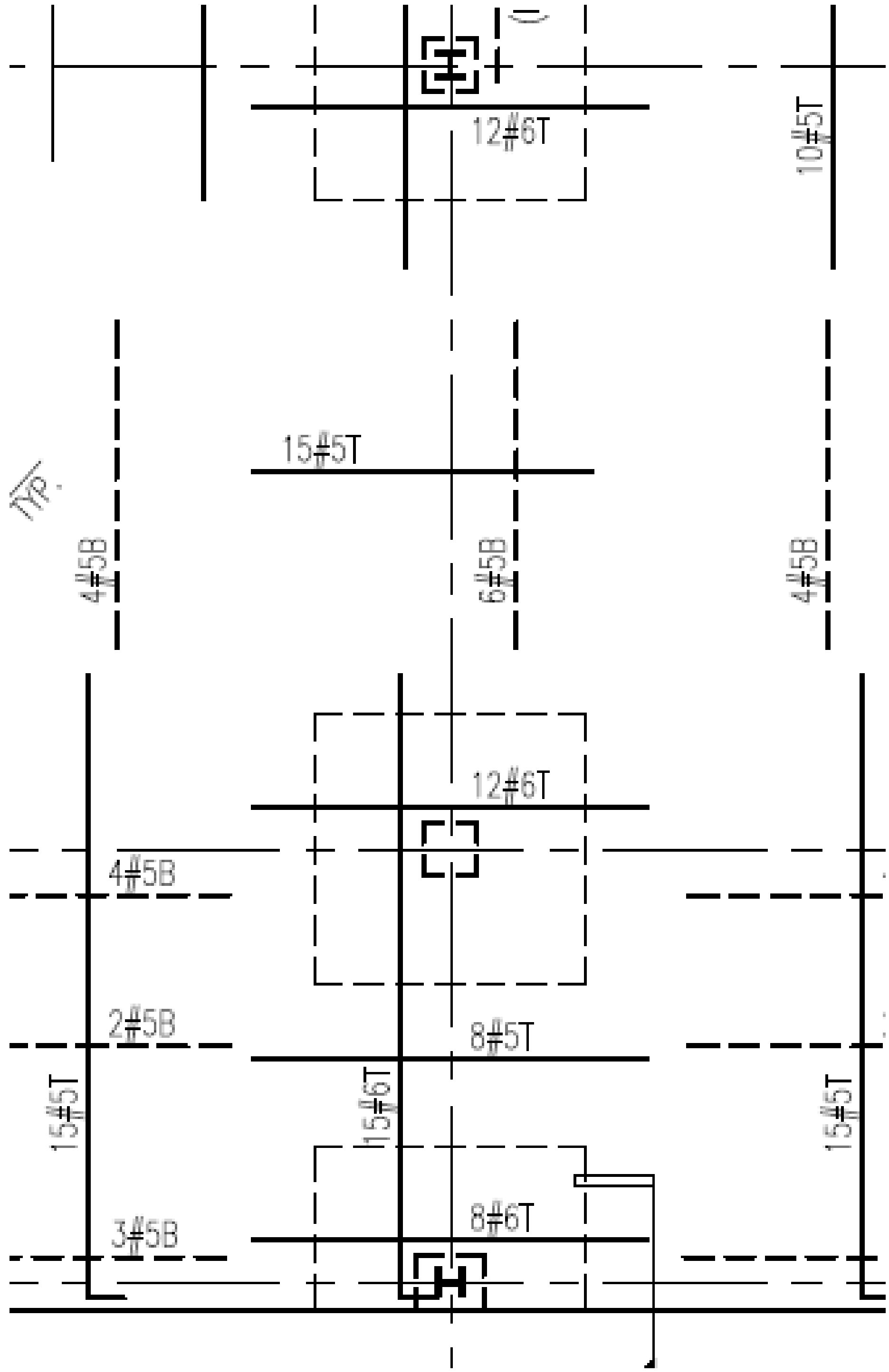


Figure 12: Existing Rebar Layout Used To Check Design of Office Levels (Source: Cagley & Assoc. Drawing S-205)

Once the gravity design of the upper floors was complete, the existing columns that remained at the parking garage levels had to be checked for adequate capacity for carrying the increased building loads. SP column was used to evaluate the capacity of columns A, B, and C along column line 5. In general, reinforcement was increased by approximately 30% while maintaining the same column cross sections.

LATERAL DESIGN

With the gravity design complete for the building, an understanding of the modified story heights and building self-weight is now known. This information is crucial for the accurate calculation of wind and seismic loads on KT36A. Since this structural redesign is a continuation of part of the existing structure, the existing shear wall layout is maintained through the final height of the structure. It is reasonable to maintain this level of lateral resistance considering lateral loads will be significantly higher than the original design due to increased building weight and the influences of the risk category IV classification.

Analysis of the lateral force resisting system will be completed with the use of a three dimensional structural model created using ETABS computer modeling software. Forces, moments, and displacements obtained from the analysis will then be used to design the individual shear walls while ensuring that serviceability requirements are satisfied.

SOIL LOADS

As previously shown in Figure 1, KT36A is exposed to a significant lateral earth pressure on the north side of the building due to the topography of the chosen site. According to the geotechnical report for KT36A completed by Burgess and Niple, Inc., elements exhibiting an at-rest condition should be designed for an equivalent lateral fluid pressure of 60 psf / foot of wall height. The at-rest condition is true of the foundation walls for the building following the assumption that the walls are supported by the building structure which has sufficient stiffness to allow for minimal deflections. This will be confirmed by the displacement output from the ETABS model. For application of the soil load to the building, it is idealized as acting at a uniform depth of 26’-8” across the width of the building. Input of the loads into the ETABS model is executed by placing a 40k load on each column line at the P3 level and a 320k load on each of the six, one-story shear walls applied at the P2 level (see Figure 13). Per ASCE 7-10 2.3.2, the soils loads (H) are applied with a factor of 1.6 when acting in conjunction with other lateral loads and a factor of 0.9 when resisting other lateral loads. Soil load calculation can be found in Appendix A.

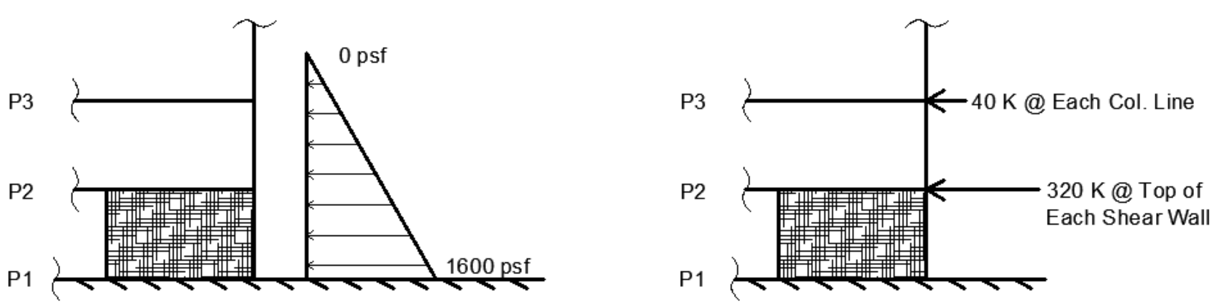


Figure 13: Soil Loads on One Story Tall Shear Walls

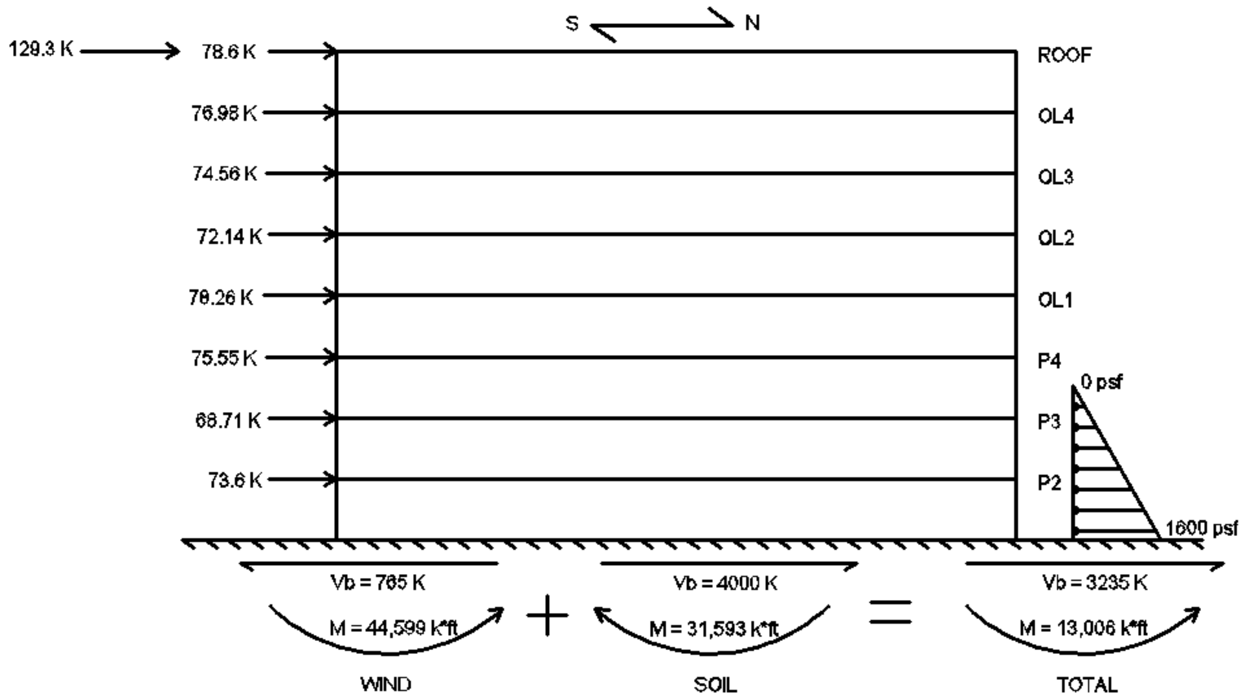
Wind Loads

Wind loads for KT36A were calculated using the MWFRS directional procedure outlined in Chapter 27 of ASCE 7-10. Considering the difference in grade elevation from the South side to the North side of the building, wind pressures are calculated for a North or South wind in addition to the East-West wind. In all cases, two internal pressure coefficients are used in determining the wind loads. This is based on the difference in function of the building, with the parking levels considered "partially enclosed" and the office levels considered "enclosed". The parking levels are considered as partially enclosed based on the two entrances to the garages always being open. Under the assumptions that the windows at the office levels are inoperable and the glazing is impact resistant, the office levels can be treated as an enclosed building.

Wind loads on the screen walls shown in Figure 9 are also taken into consideration. Since the main wind force resisting elements of the building do not extend above the roof line, the loads from the screen walls are transferred to the resisting elements through the roof slab. To represent this in the analysis of the building, two resultant point loads are applied at the roof level in the direction of the prevailing wind. Figures 15, 16, and 17 on the following pages show the results of the wind load calculations and the corresponding lateral force diagram for the given wind direction. Figures 15 and 16 regarding the North and South winds, respectively, also show the effects of the soil load on the North side of the building. Figure 14 gives a summary of the parameters used in finding the wind loads on KT36A. See Appendix A for wind load calculations.

Wind Parameter Summary	
Velocity	120 MPH
Risk Category	IV
Exposure	B
K_d	0.85
K_{zt}	1.00
Gust Factor G	0.85
GC_{pi} (Office Levels)	+/- 0.18
GC_{pi} (Garage Levels)	+/- 0.55
Flexible or Rigid?	Rigid

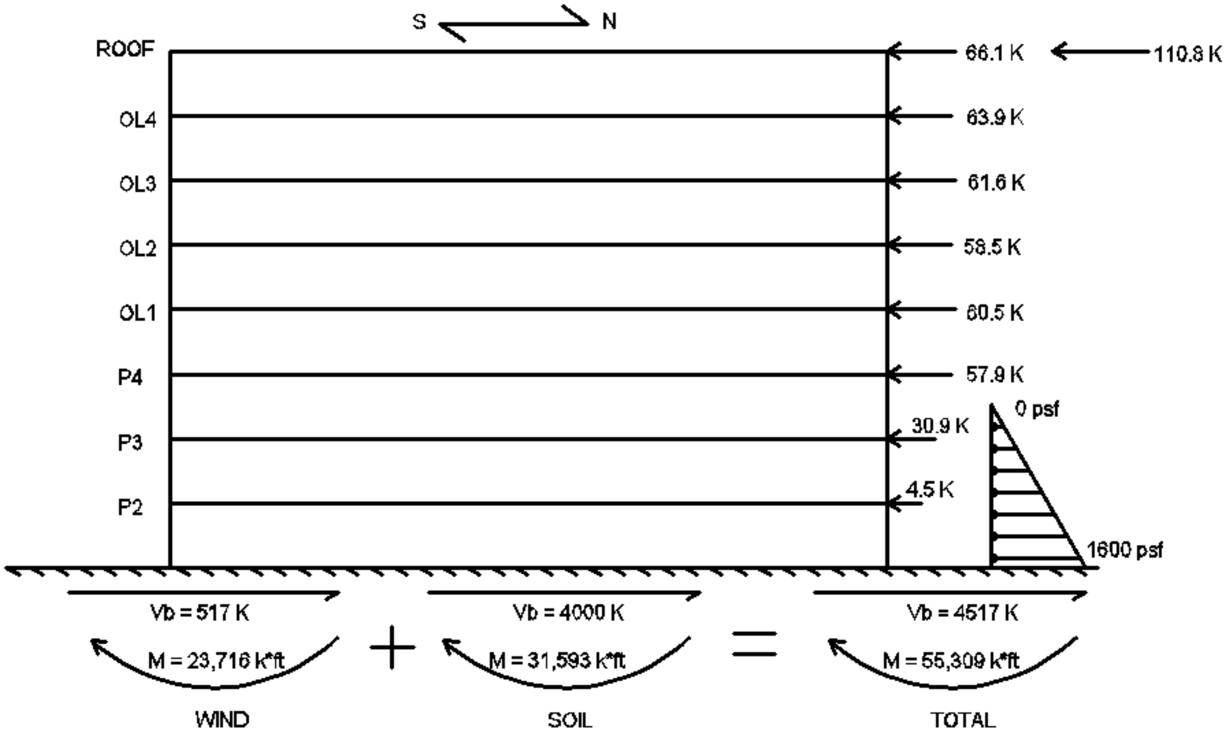
Figure 14: Design Wind Load Parameters



North - South (MWFRS) - North Wind										
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Side Walls (psf)	Tributary Area (ft2)	Force (k)
Ground (P1)	156	0	0.57	17.86	30.21	28.8		-1.4	1280	36.8
P2	168.67	12.67	0.57	17.86	30.21	28.8		-1.4	2559	73.6
P3	179.33	23.33	0.647	20.27	30.21	30.4	-3.0	-1.4	2153	68.7
P4	190	34	0.724	22.69	30.21	32.0	-3.0	-1.4	2155	75.6
5 (OL1)	200.67	44.67	0.783	24.53	30.21	33.3	-3.0	-1.4	2155	78.3
6 (OL2)	212.33	56.33	0.835	26.16	30.21	23.2	-7.4	-12.5	2355	72.1
7 (OL3)	224	68	0.882	27.64	30.21	24.2	-7.4	-12.5	2357	74.6
8 (OL4)	235.67	79.67	0.93	29.14	30.21	25.3	-7.4	-12.5	2357	77.0
Roof	247.33	91.33	0.964	30.21	30.21	26.0	-7.4	-12.5	2355	78.6
Screen Wall	260.83	104.83	1.002	31.40	30.21	47.1	-31.4	-12.5	1647	129.3

Σ=	765	kips
Σ OT Moment=	44599	k*ft

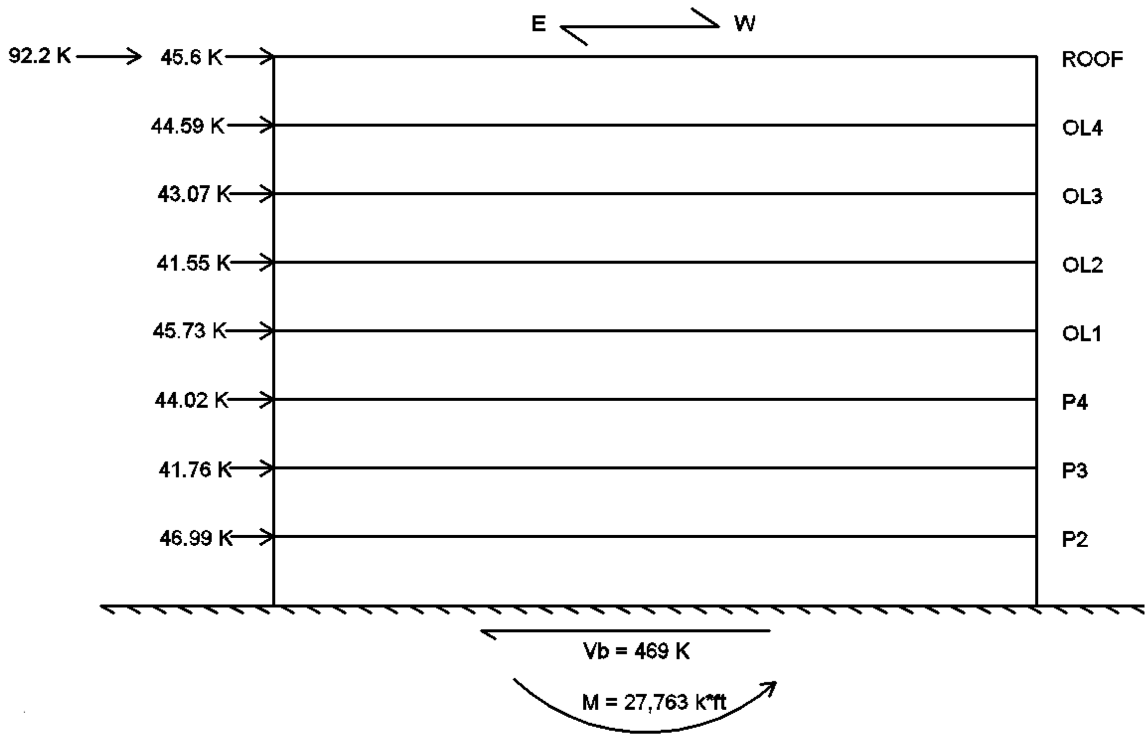
Figure 15: Design Wind Loads, North Wind



North - South (MWFRS) - South Wind										
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Side Walls (psf)	Tributary Area (ft2)	Force (k)
P1	156	0			25.38		-1.8	-1.1	1280	2.3
P2	168.67	0			25.38		-1.8	-1.1	2559	4.5
P3	179.33	0	0.57	16.40	25.38	25.1	-1.8	-1.1	2155	30.9
P4	190	10.67	0.57	16.40	25.38	25.1	-1.8	-1.1	2155	57.9
5 (OL1)	200.67	21.34	0.631	18.16	25.38	26.3	-1.8	-1.1	2155	60.5
6 (OL2)	212.33	33	0.718	20.66	25.38	18.6	-6.2	-10.5	2355	58.5
7 (OL3)	224	44.67	0.784	22.56	25.38	19.9	-6.2	-10.5	2357	61.6
8 (OL4)	235.67	56.34	0.835	24.03	25.38	20.9	-6.2	-10.5	2357	63.9
Roof	247.33	68	0.882	25.38	25.38	21.8	-6.2	-10.5	2355	66.1
Screen Wall	260.83	81.5	0.935	26.91	25.38	40.4	-26.9	-10.5	1647	110.8

∑=	517	kips
∑ OT Moment=	23716	k*ft

Figure 16: Design Wind Loads, South Wind



East - West (MWFRS)										
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Side Walls (psf)	Tributary Area (ft2)	Force (k)
Ground (P1)	156	0	0.57	17.86	30.21	28.8	-0.4	-1.4	805	23.5
P2	168.67	12.67	0.57	17.86	30.21	28.8	-0.4	-1.4	1609	47.0
P3	179.33	23.33	0.647	20.27	30.21	30.4	-0.4	-1.4	1354	41.8
P4	190	34	0.724	22.69	30.21	32.0	-0.4	-1.4	1355	44.0
5 (OL1)	200.67	44.67	0.783	24.53	30.21	33.3	-0.4	-1.4	1355	45.7
6 (OL2)	212.33	56.33	0.835	26.16	30.21	23.2	-4.8	-12.5	1481	41.6
7 (OL3)	224	68	0.882	27.64	30.21	24.2	-4.8	-12.5	1482	43.1
8 (OL4)	235.67	79.67	0.93	29.14	30.21	25.3	-4.8	-12.5	1482	44.6
Roof	247.33	91.33	0.964	30.21	30.21	26.0	-4.8	-12.5	1481	45.6
Screen Wall	260.83	104.83	1.002	31.40	30.21	47.1	-31.4	-12.5	1175	92.2

∑=	469	kips
∑ OT Moment=	27763	k*ft

Figure 17: Design Wind Loads, East-West Wind

Seismic Loads

Upon starting the redesign of KT36A, one of the largest expected increases was the amount of seismic base shear the building would be designed for. Since this structural redesign keeps the building on its' existing site, the site soil classification remained as Site Class D per the recommendation of the geotechnical report. Instead of reading the spectral response acceleration parameters from the maps in ASCE 7-10, the values were obtained using the USGS Seismic Design Maps application from www.usgs.gov. The resulting parameters classify the building as Seismic Design Category C, which is mainly influenced by the Risk Category IV classification.

As previously noted, accurately calculating building weight is critical for obtaining the seismic base shear and distributing it through the height of the building. To achieve this information, structure self-weight loads are calculated based on volume of concrete present at each level, while a 5 psf mechanical load and 100 psf façade load are assumed. A 100 psf façade load was chosen based on the current 54 psf façade being redesigned to resist blast loading. The resulting effective seismic weight of KT36A is 39,017k, which is about 55% higher than the effective seismic weight of the original design. Calculation of this value is detailed in Figure 18 below.

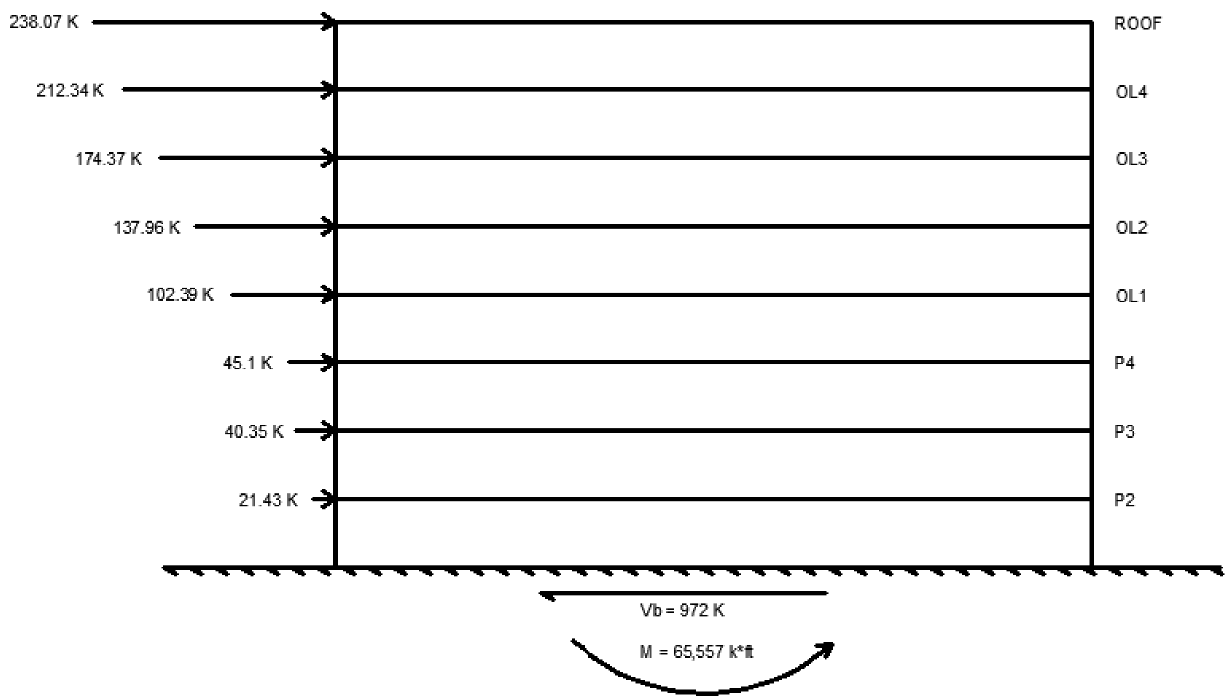
Floor Self Weight Calcs											
	Area (ft^2)	Perimeter (ft)	Height (ft)	Slab (psf)	Drops (psf)	Framing (psf)	Mech. (psf)	Façade (psf)	Shear Wall (k)	4 RTU @ 17k	Total (kips)
Ground Level (P1)	25116	658	0	100	21	17	5	100	241.4	0	4250
P2	25103	658	12.67	100	21	17	5	100	276.5	0	4634
P3	25235	658	10.66	100	21	17	5	100	252.8	0	4563
P4	11192	658	10.67	100	21	17	5	100	252.9	0	2555
5th Floor (OL1)	25299	658	10.67	100	21	17	5	100	264.7	0	4617
6th Floor (OL2)	25299	658	11.67	100	21	17	5	100	276.6	0	4662
7th Floor (OL3)	25299	658	11.67	100	21	17	5	100	276.6	0	4662
8th Floor (OL4)	25299	658	11.67	100	21	17	5	100	276.6	0	4662
Roof	25299	658	11.67	100	21	17	13	100	138.3	68	4410
										∑=	39017 kips

Figure 18: Calculation of Floor Self-weights

Calculating the seismic loads using the equivalent lateral force procedure in Chapters 11 and 12 of ASCE 7-10 yields a seismic base shear of 972 k, increasing approximately 45% over the base shear calculated for the original design. This base shear is the same in both orthogonal directions of the building since the lateral force resisting system, ordinarily reinforced concrete shear walls, is the same in both directions. See Figure 19 for a summary of the parameters used in determining the seismic loads on KT36A. A summary of the calculated loads and how they were determined can be seen in Figure 20 on the following page. Appendix A details the seismic load calculations.

Seismic Parameter Summary	
Site Class	D
Risk Category	IV => I=1.5
S_l	0.051
S_s	0.12
S_{Dl}	0.082
S_{Ds}	0.127
Seismic Design Category	C
Ordinarily Reinforced Concrete Shear Walls	R = 5
	Ω_o = 2.5
	C_d = 4.5
C_s	0.0249
Building Weight	39,017 k

Figure 19: Seismic Design Load Parameters



T= 0.9879 s
k 1.244
V_b= 972 kips

Floor	Elevation (ft)	Story Height h_x (ft)	Floor Weight w_x (kips)	$w_x*h_x^k$	C_{vx}	Story Force (kips)	Story Shear (kips)
Ground (P1)	156	0	4250	0	0	0.0	972
P2	168.67	12.67	4634	109096.3	0.0220	21.4	972.00
P3	179.33	23.33	4563	229587.4	0.0464	45.1	950.57
P4	190	34	2555	205410.3	0.0415	40.4	905.47
5 (OL1)	200.67	44.67	4617	521223.7	0.1053	102.4	865.12
6 (OL2)	212.33	56.33	4662	702284.7	0.1419	138.0	762.73
7 (OL3)	224	68	4662	887634.2	0.1794	174.4	624.78
8 (OL4)	235.67	79.67	4662	1080945.3	0.2185	212.3	450.41
Roof	247.33	91.33	4410	1211934.9	0.2449	238.1	238.07

Overturning Moment (k*ft)	65557

Figure 20: Calculation of Seismic Story Force and Shear

Referencing ASCE 7-10 Section 12.8.4.2, accidental torsion due to seismic loading should be considered when loading the building. Accidental torsion is applied to account for any possible differences in the center of mass or center of rigidity of the building from their anticipated locations. When applied, this torsion causes additional shear load in some of the lateral resisting elements. The inherent eccentricity of the building is used to determine which direction to apply the accidental torsion so as to cause the maximum effect on the building. Calculations of the accidental torsion at each floor of the building can be seen in Figure 21. Considering the building falls in SDC C, ASCE 7-10 12.8.4.3 requires that the accidental torsion moment shall be amplified if Type 1a or 1b torsional irregularity is present as defined by ASCE 7-10 Table 12.3-1. Location of the shear walls in the north-south direction causes an extreme torsional irregularity (Type 1b), thus the accidental torsion moments are amplified by a factor of 1.912 in the Y direction. A torsional irregularity does not exist in the east-west direction.

Seismic Loading Torsion E-W Direction (X)							
Floor	**Story Force (k)**	**COR Location**	**COM Location**	**e (ft)**	$M_{inherent}$ (k-ft)	M_{acc} (k-ft)	M_{total} (k-ft)
RF	238.07	64.669	62.497	2.172	517.089	1487.9	2005.0
OL4	212.34	64.669	62.497	2.172	461.201	1327.1	1788.3
OL3	174.37	64.679	62.497	2.182	380.465	1089.8	1470.2
OL2	137.96	64.604	62.497	2.107	290.673	862.2	1152.9
OL1	102.39	64.432	62.497	1.935	198.121	639.9	838.0
P4	45.10	64.087	62.497	1.590	71.709	281.9	353.6
P3	40.35	63.575	62.497	1.078	43.498	252.2	295.7
P2	21.43	62.851	62.497	0.354	7.586	133.9	141.5

Seismic Loading Torsion N-S Direction (Y)								
Floor	**Story Force (k)**	**COR Location**	**COM Location**	**e (ft)**	$M_{inherent}$ (k-ft)	M_{acc} (k-ft)	Amped M_{acc} (k-ft)	M_{total} (k-ft)
RF	238.07	106.278	99.75	-6.528	-1554.124	-2380.7	-4551.9	-6106.0
OL4	212.34	106.179	99.75	-6.429	-1365.128	-2123.4	-4059.9	-5425.1
OL3	174.37	106.057	99.75	-6.307	-1099.723	-1743.7	-3333.9	-4433.6
OL2	137.96	105.911	99.75	-6.161	-849.945	-1379.6	-2637.7	-3487.7
OL1	102.39	105.702	99.748	-5.954	-609.620	-1023.9	-1957.7	-2567.3
P4	45.10	105.328	99.748	-5.580	-251.657	-451.0	-862.3	-1114.0
P3	40.35	104.47	99.748	-4.722	-190.535	-403.5	-771.5	-962.0
P2	21.43	102.708	99.748	-2.960	-63.435	-214.3	-409.8	-473.2

Figure 21: Calculation of Accidental Torsion

Computer Model

To efficiently analyze the effects of the lateral loads on the building as a whole, a three-dimensional structural model was created using ETABS. ETABS is a modeling and analysis program commonly used by the structural engineering industry to obtain an accurate and comprehensive analysis of the building lateral systems. After applying the appropriate property modifiers and structural considerations to the building, member forces and story displacements/drifts can be easily obtained for the controlling load case(s). For this analysis, gravity load and lateral load carrying elements were modeled since lateral loads alone impart significant axial loads on the gravity load carrying columns. See Figure 22 on the following page for a three-dimensional view of the structural system model in ETABS.

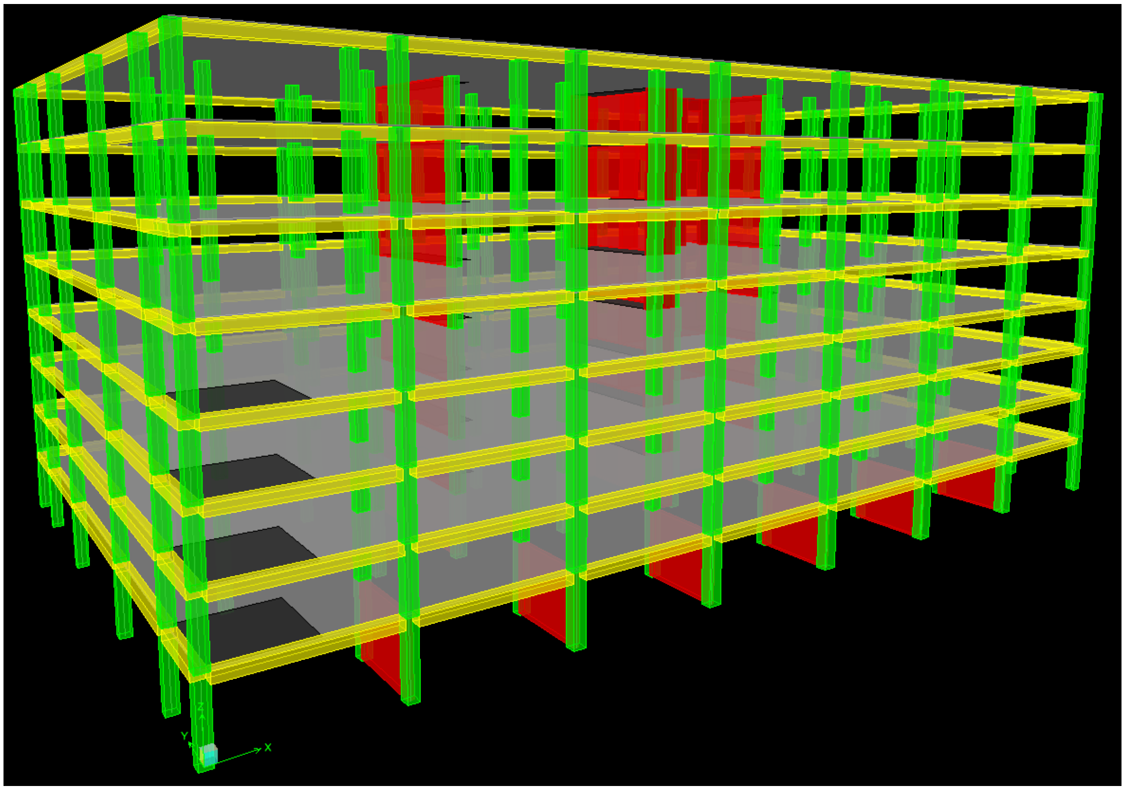


Figure 22: View of North-East Corner of ETABS Model

In order to accurately predict the realistic behavior of the structure, the following assumptions and considerations were made when defining the model:

- Per ASCE 7-10 Section 12.7.3
    - Effects of cracked concrete considered in accordance with ACI 318-11 8.8.2
        - Column moment of inertia modified by $0.7*I_g$ about both axes
        - Beam moment of inertia modified by $0.35*I_g$ about both axes
        - Shear wall moment of inertia modified by $0.35*I_g$ (The cracked modifier was used here due to the significant moments on the shear walls)
- Each floor level was modeled as a rigid diaphragm
- A rigid diaphragm constraint was also assigned to all points intersecting each rigid diaphragm
- All shear walls were modeled as membrane elements so as not to resist out of plane forces
- All columns occurring at the ends of shear walls were modeled centered with the plane of the shear wall, even though many have a slight offset when viewed in plan. This accounts for the increased stiffness the columns provide through working with the shear walls.
- All material self-weights were applied as a distributed mass over the area of each floor
- Lateral loads were calculated by hand (as previously seen) and directly applied into the model
- All concrete column and shear wall base restraints were modeled as fixed connections
- Shear walls were "meshed" with a maximum size of 18" x 18" to properly account for shear deformations in both axes of the plane of the wall

Using the controlling load combinations, 12 combination load cases were created in ETABS to observe the effects of combined lateral loading on the building. The first two combinations considered the earthquake loading (considering accidental torsion) acting simultaneously with the lateral soil load (H), in the respective orthogonal directions. An orthogonal combination of the seismic loads acting in the "X" and "Y" directions together is typically considered; however, according to ASCE 7-10 12.5.3, this was not required for this analysis since KT36A is located in seismic design category C and horizontal structural irregularity Type 5 is not present in the structure. Even though the seismic loads are dynamic in nature, they were treated as a constant static load, concurrent with the procedures of the equivalent lateral force method. The 10 wind load cases created were based on Table 27.4-8 in ASCE 7-10. A summary of the load cases defined in the ETABS model can be seen in Figure 23.

ETABS Case Name	Description
EQXTSOIL	E-W Seismic load + Accidental Torsion + Soil
EQYTSOIL	N-S Seismic load + Accidental Torsion + Soil
ETABS Case Name	Description
CASE1NW	Case 1 North Wind + 0.9 H
CASE1SW	Case 1 South Wind + 1.6 H
CASE1EWW	Case 1 East-West Wind + 0.9 H
CASE2NW	Case 2 North Wind + 0.9 H
CASE2SW	Case 2 South Wind + 1.6 H
CASE2EWW	Case 2 East-West Wind + 0.9 H
CASE3NW	Case 3 North Wind + East-West Wind + 0.9 H
CASE3SW	Case 3 South Wind + East-West Wind + 1.6 H
CASE4NW	Case 4 North Wind + East-West Wind + 0.9 H
CASE4SW	Case 4 South Wind + East-West Wind + 1.6 H

Figure 23: ETABS Load Cases

Story Drifts and Displacements

Story drifts were calculated for KT36A based on the floor deflections obtained from the ETABS model. Each of the seismic loading combinations controlled for its' respective direction when considering total building deflection. This is expected since the base shear is much higher for the seismic loads. As mentioned earlier, it was not necessary to examine other seismic loading combinations. Controlling wind cases are CASE2NW for story drift and CASE2SW for total building deflection.

In the seismic loading drift calculations, the story drifts were checked against a limit of 0.010 h_{sx} for a risk category IV building in accordance with ASCE 7-10 12.12.1. It is also important to note that the seismic displacement values obtained from ETABS were amplified by a factor of (C_d/I) as specified in section 12.8.6 of ASCE 7-10. This amplification factor was found to be equal to 3 (4.5/1.5), which ironically works out to be the same as it was in the original design of the building. Referencing the ASCE 7-10 commentary, wind load story drifts were checked against a limit of H/400 with H being the height of the story being analyzed.

The following figures display the drift values for the controlling load cases and their corresponding directions. It can be seen that all of the story drifts are well below their allowable limits, on the order of

10% of the allowable for the seismic drifts and 20% of the allowable for the highest wind drifts. This signifies that shear wall layout provided is stiffer than what is required and may have room for optimization. However, this was not included in the scope of this senior thesis.

Seismic Displacement and Drift E-W						
Story	Story Ht. (ft)	X Disp. (in)	X Disp. Amped (in)	Amped X Story Drift (in)	Allow. Drift (in)	Acceptable?
OL4	11.6667	0.5704	1.7112	0.2451	1.400	YES
OL3	11.6667	0.4887	1.4661	0.2871	1.400	YES
OL2	11.6667	0.393	1.1790	0.3180	1.400	YES
OL1	11.6667	0.287	0.8610	0.3015	1.400	YES
P4	10.6667	0.1865	0.5595	0.1878	1.280	YES
P3	10.6667	0.1239	0.3717	0.1584	1.280	YES
P2	10.6667	0.0711	0.2133	0.1290	1.280	YES
P1	12.6667	0.0281	0.0843	0.0843	1.520	YES

Seismic Displacement and Drift N-S						
Story	Story Ht. (ft)	Y Disp. (in)	Y Disp. Amped (in)	Amped Y Story Drift (in)	Allow. Drift (in)	Acceptable?
OL4	11.6667	0.638	1.9140	0.2985	1.400	YES
OL3	11.6667	0.5385	1.6155	0.3084	1.400	YES
OL2	11.6667	0.4357	1.3071	0.3063	1.400	YES
OL1	11.6667	0.3336	1.0008	0.2898	1.400	YES
P4	10.6667	0.237	0.7110	0.2394	1.280	YES
P3	10.6667	0.1572	0.4716	0.2067	1.280	YES
P2	10.6667	0.0883	0.2649	0.1668	1.280	YES
P1	12.6667	0.0327	0.0981	0.0981	1.520	YES

Figure 24: Calculation of Seismic Drifts

Wind Displacement and Drift CASE2NW					
Story	Story Ht. (ft)	Y Displacement (in)	Y Story Drift (in)	Allowable Drift (in)	Acceptable?
OL4	11.6667	0.4107	0.0684	0.350	YES
OL3	11.6667	0.3423	0.0697	0.350	YES
OL2	11.6667	0.2726	0.0684	0.350	YES
OL1	11.6667	0.2042	0.0646	0.350	YES
P4	10.6667	0.1396	0.054	0.320	YES
P3	10.6667	0.0856	0.0473	0.320	YES
P2	10.6667	0.0383	0.0336	0.320	YES
P1	12.6667	0.0047	0.0047	0.380	YES

Figure 25: Calculation of Wind Drifts

Wind Displacement and Drift CASE2SW					
Story	Story Ht. (ft)	Y Displacement (in)	Y Story Drift (in)	Allowable Drift (in)	Acceptable?
OL4	11.6667	0.4218	0.0642	0.350	YES
OL3	11.6667	0.3576	0.0653	0.350	YES
OL2	11.6667	0.2923	0.0643	0.350	YES
OL1	11.6667	0.228	0.0611	0.350	YES
P4	10.6667	0.1669	0.0517	0.320	YES
P3	10.6667	0.1152	0.0465	0.320	YES
P2	10.6667	0.0687	0.0405	0.320	YES
P1	12.6667	0.0282	0.0282	0.380	YES

Figure 25 (Cont.): Calculation of Wind Drifts

Member Design

When defining the membrane area elements of the shear walls, only the thickness of the area element is specified. ETABS does not include reinforcement in area elements, thus the stiffness of the building is based solely on the concrete strength and the geometric properties of the walls. This allows the shear walls to be sized based on deflection and drift criteria without designing the reinforcement in the walls.

Concurrent with the findings of the drift and displacement analysis, seismic loads in each of the orthogonal directions were found to also control the forces in the shear walls oriented in the corresponding directions. As previously stated, ordinarily reinforced shear walls were chosen to resist the lateral loads in the building. Design of the walls was carried out in accordance with Section 11.9 and Chapter 14 of ACI 318-11. Figure 26 shows the layout of the designed shear walls.

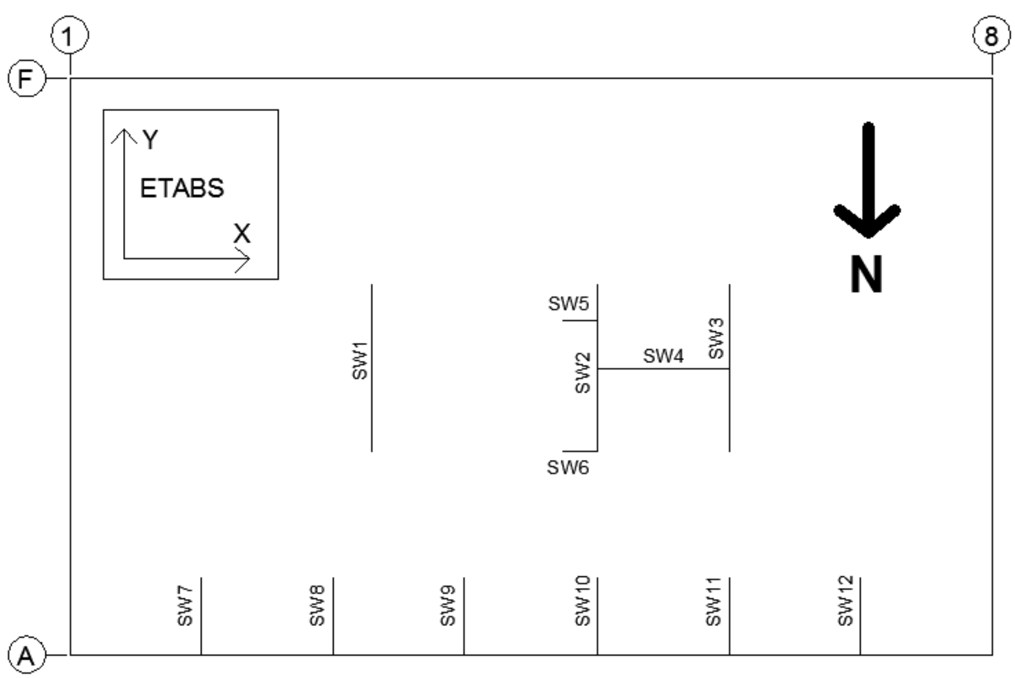


Figure 26: Designed Shear Wall Layout

Shear walls SW7 through SW12 were all designed for the worst case loading seen by this group of walls. Loads of 493 k in shear and 6268 k*ft in moment were found to control in SW7 due to the EQYTSOIL loading case. Design of the wall is based on the assumption that the concrete slab above the wall is detailed in such a way to transfer some axial load into the wall. This allowed the use of equations incorporating axial load, but the axial loads were conservatively entered as "0" into the equations. Each of the walls are bounded at the ends by the previously sized columns. The tied reinforcement in the columns was treated as boundary elements for the walls. The tension capacity of the reinforcing in one of the boundary elements was then taken about a moment arm equal to the distance from the tension zone centroid to the compression zone centroid to find the moment capacity of the wall. The walls were then checked for adequate shear capacity using Section 11.9 of ACI 318-11. It was determined that the concrete alone did not provide enough capacity, so it was necessary to add reinforcement steel. Following the provisions of ACI 318-11 Section 14.3, minimum reinforcement was calculated for the walls based on the thickness of the walls and the spacing of the reinforcement. Maximum spacing of the reinforcement in the vertical and horizontal directions is controlled by the 18" minimum and #4 bars spaced at 12" O.C. in each face of the wall satisfied the minimum reinforcement ratio of 0.0025 for both the longitudinal and transverse bars. SW1 was also designed in the same manner as SW7. Maximum loads in shear and moment on SW1 were found to be 842 k and 43,470 k*ft, respectively. Again, minimum reinforcement in the wall was found to provide plenty of additional shear capacity. Calculations for the shear wall designs can be found in Appendix E. Figure 27 below shows the typical design of shear walls SW7 through SW12.

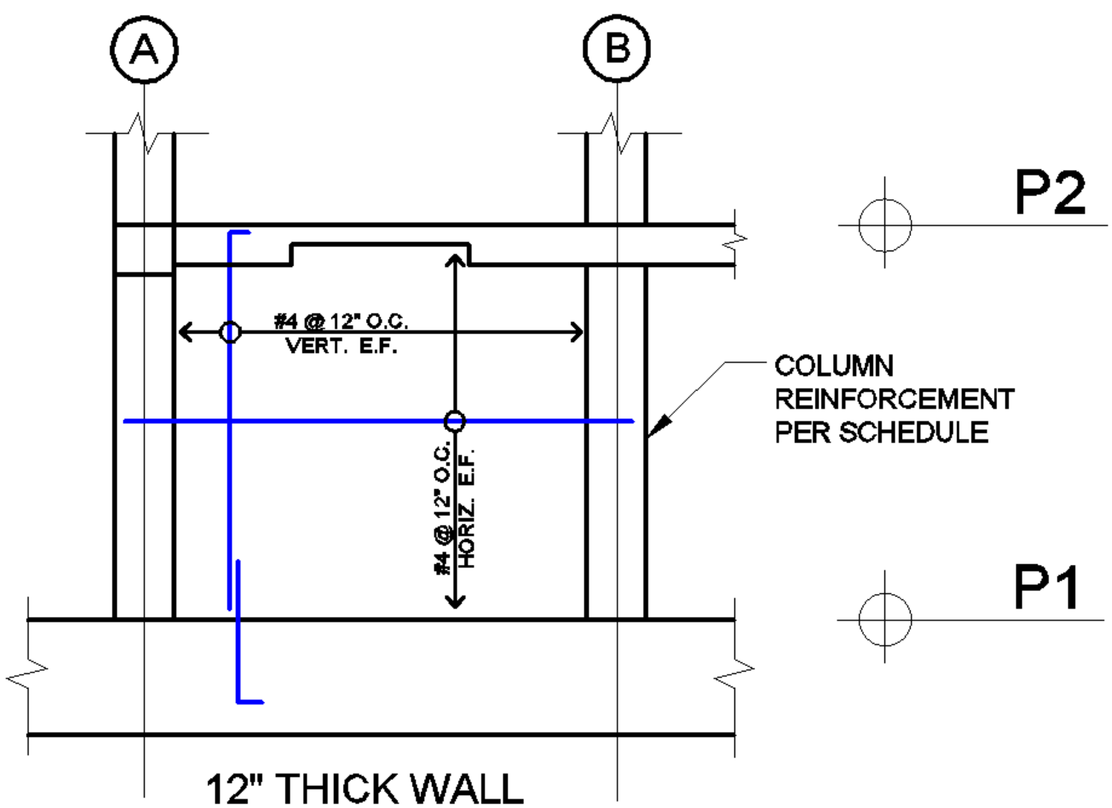


Figure 27: Designed SW7 – SW12

Unlike the rest of the shear walls, SW4 does not have a continuous cross-section throughout its height. In the original design of the building, SW4 ended at the floor slab of OL1. Above this level, restrooms are located in the space as can be seen in the Figure 28 below. This poses a conflict in continuing the shear wall to the roof level of the building. SW5 and SW6 also act to resist lateral forces in the same direction of the building; however, SW5 and SW6 are significantly less stiff than SW4 and are not capable of carrying the load on their own.

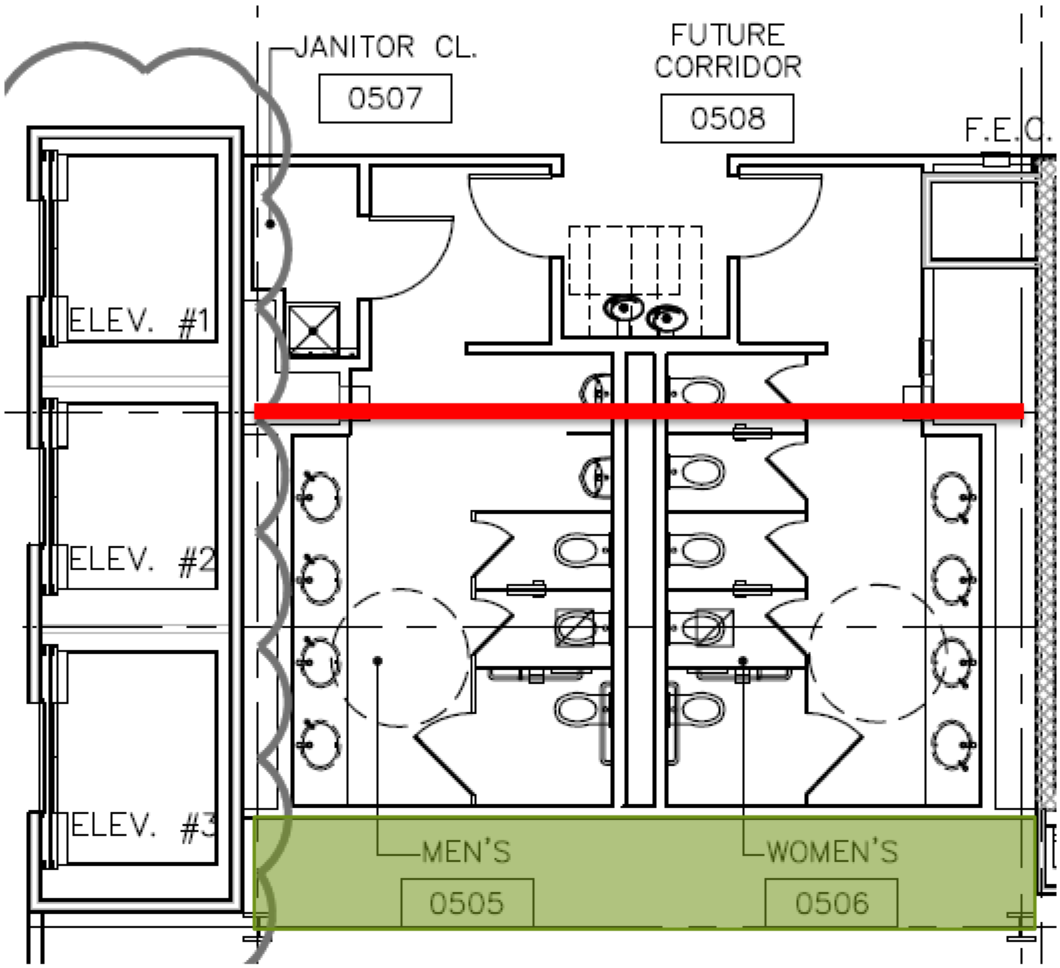


Figure 28: SW4 Conflict in Restrooms

In order to keep the office space as open as possible, adding shear walls between other column lines was considered undesirable. Without using concrete moment frames, this left the solution at continuing SW4 through the remaining levels to the roof and casting openings in it to maintain access to the restrooms at each of the office levels. In order to achieve this, the restroom assembly shifted towards column line C by 2’-6”, effectively consuming an area that would likely only be used for cabinet space (see green shaded region in Figure 28). The other impact is a mechanical shaft coming through the ceiling of OL4. This could be relocated to column line D which places the shaft at a symmetric location on the other side of the shear wall. The location of the designed SW4 can be seen in blue in Figure 29.

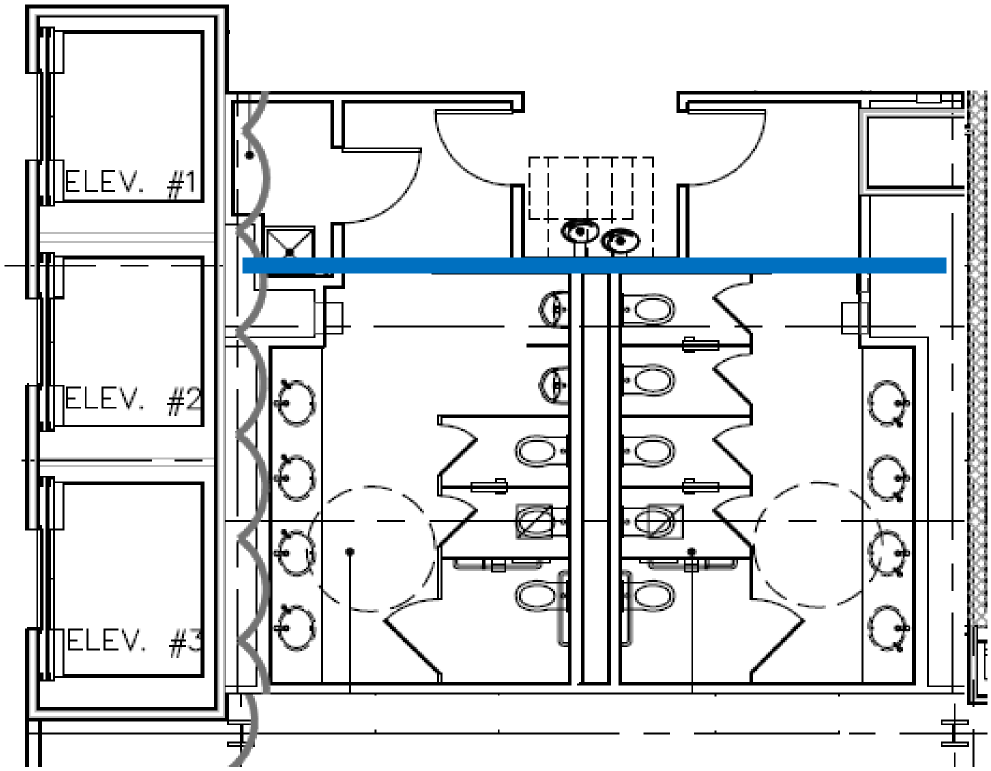


Figure 29: Adjusted Architecture for SW4

Design of SW4 was completed in accordance with Chapters 11, 14, and 21 of ACI 318-11. The cross section of the wall is consistent through the slab of OL1. From the foundation to this level, the wall was designed in the same manner as SW7 and SW1, with the exception of the boundary elements. In SW4, the boundary elements reside within the intersection of the perpendicular shear walls. 4 #9 bars are needed for each boundary element in order to resist the overturning moment of 5157 k*ft on the wall. This moment is significantly lower than walls of similar size because SW4 acts compositely with SW2 and SW3, essentially working as a large wide-flange section. The key to achieving the composite action is to adequately develop the shear interface at the intersection of the 2 walls. This is accomplished by continuing the reinforcement in SW4 into the perpendicular walls and casting the walls monolithically together. In order to maintain access to the restrooms, openings are incorporated into the formwork which create coupling beams between the openings at each floor level. Even though the provisions of Chapter 21 are not required due to the building classified as seismic design category C, sections of this chapter in ACI 318-11 were used to design the coupling beams. Per ACI 318-11 Section 21.9.7.2, groups of diagonal reinforcement equating to 2 #4, 4 #9, and 2 #6 bars are required in the coupling beams due to the amount of shear present in them. Confinement for the diagonal bars is provided by transverse reinforcement placed throughout each coupling beam (red lines in Figure 30). This option was chosen considering it greatly simplifies field placement of the rebar and laborers in this region are likely not experienced in placing rebar in heavily reinforced coupling beams. Figure 30 shows the design of SW4.

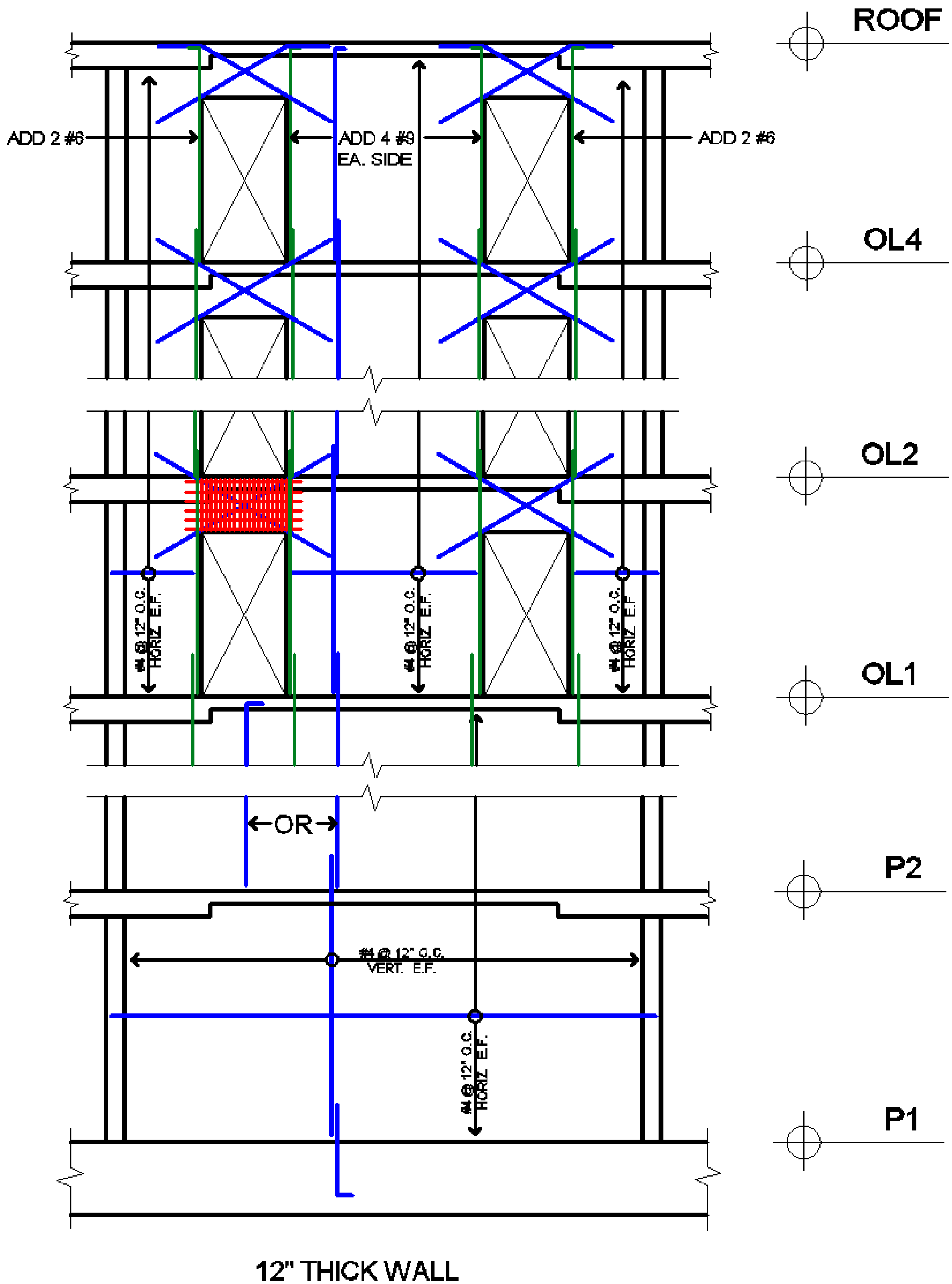


Figure 30: SW4 Reinforcement Layout

IMPACT ON FOUNDATIONS

Once the reinforced concrete structure was appropriately designed for the expected loads, the ability of the existing foundation system to carry the design loads needed evaluated. With an increase in building weight of approximately 55% over the original design, it was expected that an alteration to the existing foundation system would be necessary. As previously mentioned, KT36A was originally designed with a foundation bearing on Geopiers, which are rammed aggregate piers. According to the geotechnical report completed by Burgess and Niple, spread footings bearing on Geopiers can be designed with an allowable bearing pressure of 7000 psf with each 30" diameter Geopier having a capacity of 100 kips. Most of the columns in the original design are supported by spread footings of varying size and depth, with the exception of the central core columns which are located around the shear walls. The foundation of this central core consists of a massive 48" thick concrete mat foundation.

For the purposes of this senior thesis design, assessment of the existing foundations would be based on a typical 11'-0" x 11'-0" x 36" deep spread footing. The axial loads on column C-1.5 were chosen for design of a typical footing since this location sees the highest load on a typically sized footing. Using the ASD load combo of D+0.75L+0.75S, an axial load of 1165 kips rests on the footing. Lateral loads were found to have negligible effect on the axial load in this column, leaving the controlling load at 1165 kips. This results in 12 Geopiers of 30" diameter needed under the footing. The required area of the footing based on the 7000 psf allowable bearing pressure is approximately 166 ft^2. A 13'-0" x 13'-0" footing satisfies the needed area, however, the geotechnical report recommends proportioning the footings based on the number of Geopiers required below each footing. Considering this, a spread footing of 12'-0" x 16'-0" in plan was chosen with the Geopiers arranged in a 3 x 4 grid. A plan view of the footing design can be seen in Figure 31.

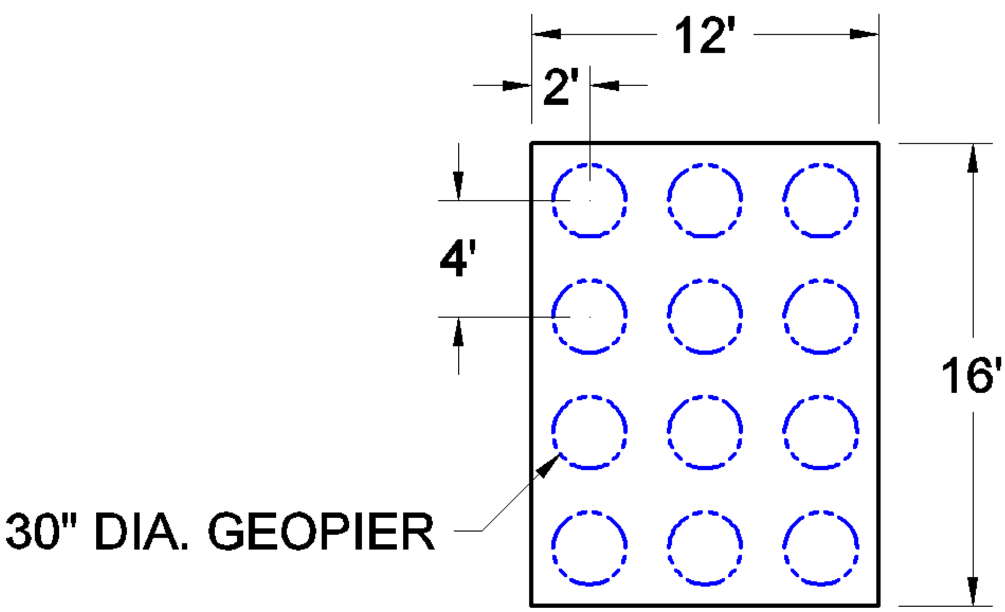


Figure 31: Plan View of Designed Typical Footing

Progressive Collapse Analysis and Design

Progressive collapse is defined in ASCE 7-10 as the spread of initial, local failure from element to element, eventually resulting in the collapse of a large portion of the structure, or worse, the entire structure. Design guidelines and provisions regarding design of building structures to resist progressive collapse started to become an important design consideration shortly after the Oklahoma City Bombing event in 1995. Since then, guides such as the Unified Facilities Criteria (UFC) 4-023-03 – Design of Buildings to Resist Progressive Collapse and the General Services Administration (GSA) – Progressive Collapse Analysis and Design Guidelines have been created and adopted by the U.S. Government for use in the design of critical function buildings.

For the purposes of the analysis and design completed in this report, more emphasis was placed on using the UFC 4-023-03 Guide since this is the most recent document pertaining to design against progressive collapse failure. Two design methods are specified in the UFC, direct design and indirect design. Direct design consists of the Alternative Path Method (requiring the structure to bridge over a missing structural element) and the Enhanced Local Resistance Method, which requires increased strength capacity for perimeter columns. Indirect design consists of the Tie-Force Method, which requires a minimum tensile capacity in the structural elements to mechanically tie the structure together and enhance its ductility, continuity, and redundancy.

Requirements for which individual or combination of design methods to implement in the design are based on the Risk Category of the building. As previously stated in the General Design Provisions of this report, Kingstowne Section 36A is now considered a Risk Category IV structure considering the prospective use of the building. The Risk Categories of ASCE 7-10 translate directly to the Occupancy Categories defined in the UFC. Per Section 2-2.4 of UFC 4-023-03, an Occupancy Category IV building must be designed for progressive collapse through completion of the Tie-Force Method, Alternative Path Method, and Enhanced Local Resistance Method. Progressive collapse design of KT36A will proceed in this order, starting with the Tie-Force Method.

Tie Force Method

The idea behind the Tie-Force Method is to allow loads to be redistributed to adjacent members upon the loss of a critical structural element. In order to accomplish this, Section 2-2.4.1 of the UFC states that adequate internal, peripheral, and vertical tie-force capacity shall be provided. The UFC lays out the Tie-Force Method in Section 3-1. Of particular note in this section are that the gravity designed slab reinforcement can be used to satisfy tie-force requirements, and that peripheral ties are to be placed within 3'-0" of the perimeter of the structure and cannot be placed above flexural elements.

The three types of tie-forces are calculated using the same principle, design tie strength must be greater than or equal to the calculated tie force ($\boldsymbol{\phi R_n \geq F_t}$).

The design tie force is a function of the type and amount of steel being used, noted in this equation:

$$\phi R_n = \phi\ \Omega\ A_s\ F_y$$

Here, the strength reduction factor **ϕ** = 0.75 per Section 4-3 in the UFC, and the material over strength factor **Ω** = 1.25 per Table 6-4 in ASCE 41-06. A_s is ultimately what is solved for in finding the amount of steel required to provide adequate tie force capacity. Calculating the required tie force for each of the three types of tie forces is based off of the load combination **W_f = 1.2D + 0.5L**. However, the equations used to find the required tie force are different depending on the type of tie being designed.

For both the longitudinal and transverse directions, the internal tie forces are calculated using the following equation where L_i equals the greater distance between centers of columns supporting any two adjacent floor spaces in the considered direction:

$$F_i = 3*W_f*L_i$$

Peripheral tie forces are calculated for the perimeter of the building and at any slab openings in the building using the following equation. It is important to note here that the dead load (D) used in finding W_f includes the façade load if a perimeter peripheral tie force is being calculated. L_1 equals the greater distance between centers of columns at the perimeter of the building in direction of loading, or is equal to the length of a slab opening in direction under consideration. L_P equals 3 feet following the provision that the peripheral ties must lie within 3 feet of the perimeter.

$$F_i = 6*W_f*L_1*L_P$$

Vertical tie forces are resisted by the longitudinal bars found within columns. The necessary reinforcement required by traditional design of the building is typically more than adequate for resisting the required vertical tie forces. Vertical tie forces are calculated using the following equation where A_T is the tributary area of the specified column.

$$F_v = A_T*W_f$$

Completion of the Tie-Force Method required more reinforcement in the concrete slabs than what was calculated in the base design. In the North-South direction, #6 bars @ 12” O.C. are required while #6 bars @ 15” O.C. are required in the East-West direction. This combination of required reinforcement will have the most effective use if placed as a continuous bottom mat in the slabs. The tie-force bars will then replace the bottom reinforcement in the slabs, with the exception of where the required reinforcement exceeds the amount provided by the tie-force bars. Additional bars will be added to the typical bottom reinforcement here to satisfy the demands of the design loads. Required vertical tie forced required 4 #8 bars which is satisfied by all columns found within KT36A. Calculations for the Tie-Force Method can be found in Appendix G. Figure 32 on the following page shows typical tie-force reinforcement in a corner zone of the slabs.

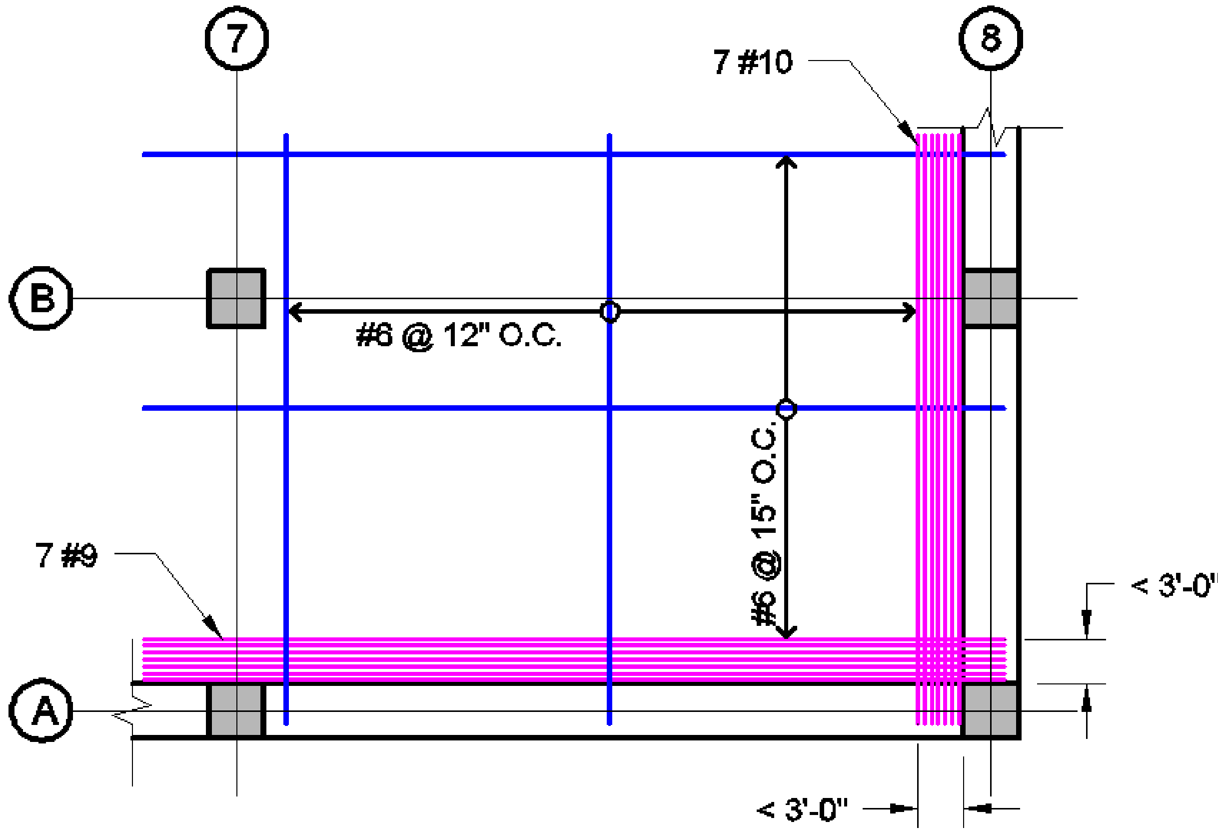


Figure 32: Typical Tie Force Bar Layout at Perimeter of Building

ALTERNATIVE PATH METHOD

The next required method in designing the building for progressive collapse is the Alternative Path Method. This method is a way of directly designing the building for a progressive collapse scenario by strategically removing columns (one at a time) in order to replicate an event that would lead to progressive collapse of the structure. All column removal locations are standardly considered along the perimeter of the building, with interior column removal required in the analysis if public access is available to the interior of the building. Although there is parking within the building, availability to the space will be restricted to only employees of the police headquarters building as detailed in the Site Layout Redesign of this report. The retail space at the first floor of KT36A will be used as an equipment check and storage area for the officers; therefore it was not viewed as being a threat to the internal space of the building.

The strategic column locations mentioned above include removal at the middle along the long side of the building, at the middle along the short side of the building, and at a chosen corner of the building. At each of the removal locations, the analysis must be performed at the first story above grade (PL1), the story at mid-height of the building (PL4 / OL1), the story above the level where column splices occur

(OL1), and the story directly below the roof (OL4). It is critical that the column is only removed between the lateral supports of the column at that level, as stated in the UFC. This is illustrated in Figure 33.

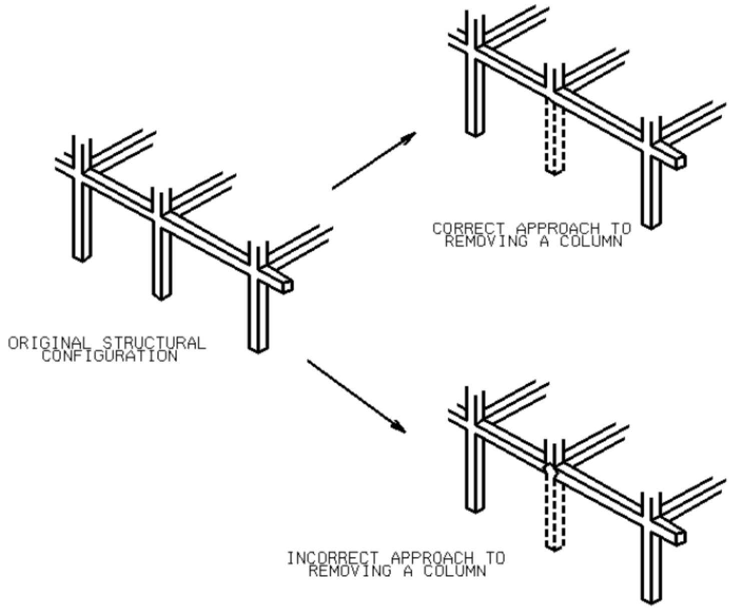


Figure 33: Correct Removal of Column (Source: UFC-023-03)

There are three analysis options within the Alternative Path Method: Linear Static, Nonlinear Static, and Nonlinear Dynamic. Nonlinear Static was chosen for the analysis completed in this report to obtain more realistic results over the linear static analysis without the performing the time intensive nonlinear dynamic analysis. The UFC specifies a gravity load combination to use in the nonlinear static analysis:

$$\mathbf{G = (0.9 \text{ or } 1.2)*D + (0.5*L \text{ or } 0.2*S)}$$

This load was found to be 280 psf at the parking levels, 312 psf at the office levels, and 240 psf at the roof level. In the bays immediately surrounding the column removal location, the UFC requires the gravity loads to be multiplied by a dynamic amplification factor to account for the effects of the dynamic response of the structure (the acceleration of the area above the removed column will cause greater forces in the surrounding members). UFC also requires a notional lateral load equal to 0.2% of the gravity load seen at each floor level. This notional lateral load was applied at each floor level as a series of point loads at each grid location, totaling the calculated load for each floor level. A separate load case was created for each of the four orthogonal directions, North, South, East and West.

In order to analyze the structure, a three dimensional model was created using SAP 2000. The same modeling philosophies used in creating the ETABS model were also used in creating this structural model, which follows the guidelines of ASCE 41-06 for modeling of building structures. Plastic hinges were defined using the requirements for life safety found in Table 4-1 of UFC 4-023-03 and assigned to the immediate beams and columns involved in the collapse area. The rotational limit for the hinges was calculated at 0.03 radians.

All loads were applied as nonlinear staged construction load cases in SAP 2000. In each load case, stage 1 consisted of loading the entire structure while stage 2 was defined as removal of the particular column at the location being analyzed. Failure of the members is defined as any hinge exhibiting a rotation greater than 0.03 radians, appearing as light blue, green, yellow, orange, or red in the images of frame A shown below.

The model was analyzed for each of the column removal locations considering the four different lateral load application directions. Members not satisfying the rotational limit were redesigned by modifying the amount of reinforcement in the beam and/or increasing the cross-section of the member. Through a trial and error process, the members were considered adequate when hinges still formed, but exhibited a rotation less than 0.03 radians. This is portrayed as "blue" hinges in Figure 34 which equates to the purple region on the scale. Calculations for this method can be found in Appendix G.

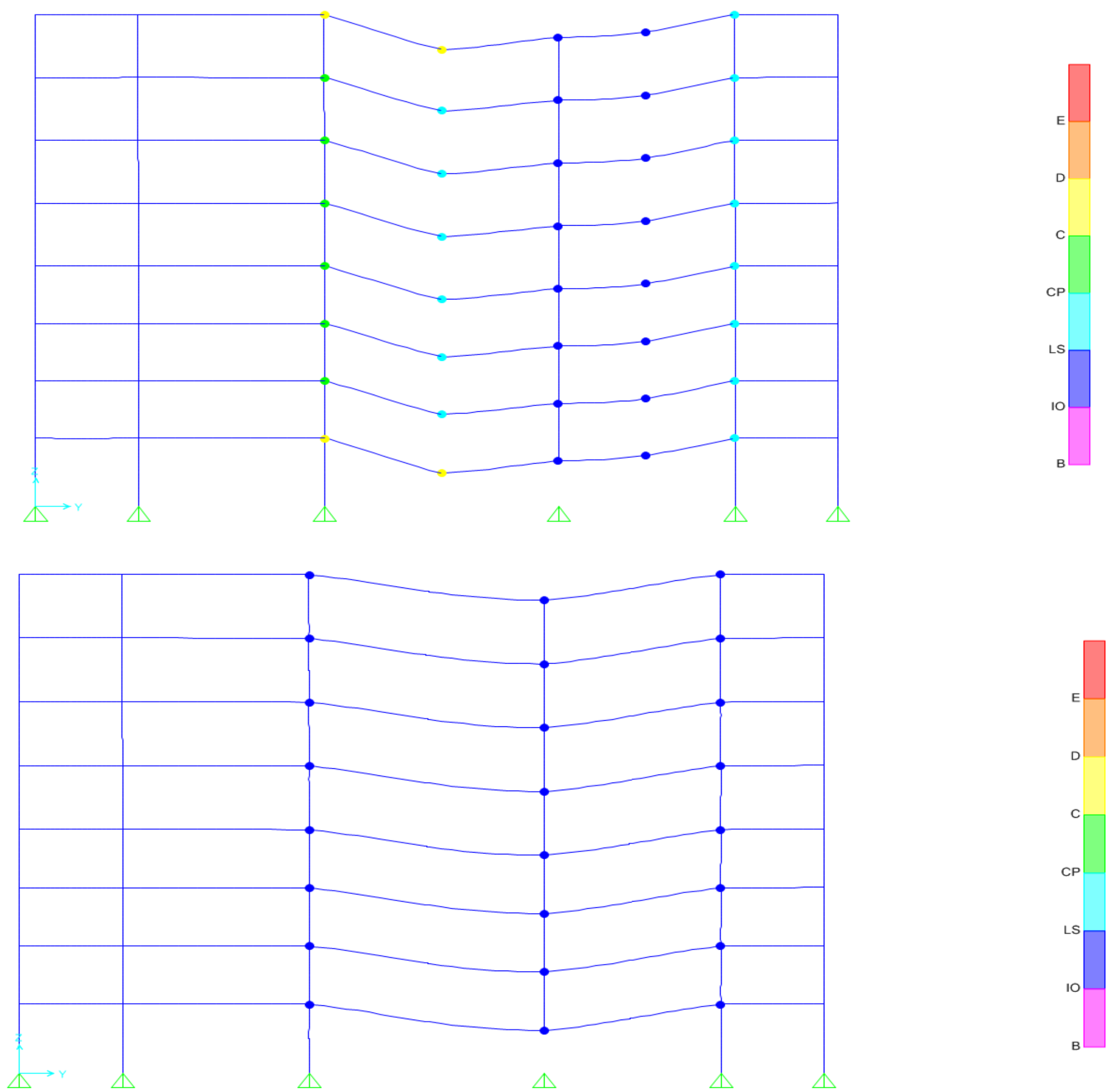


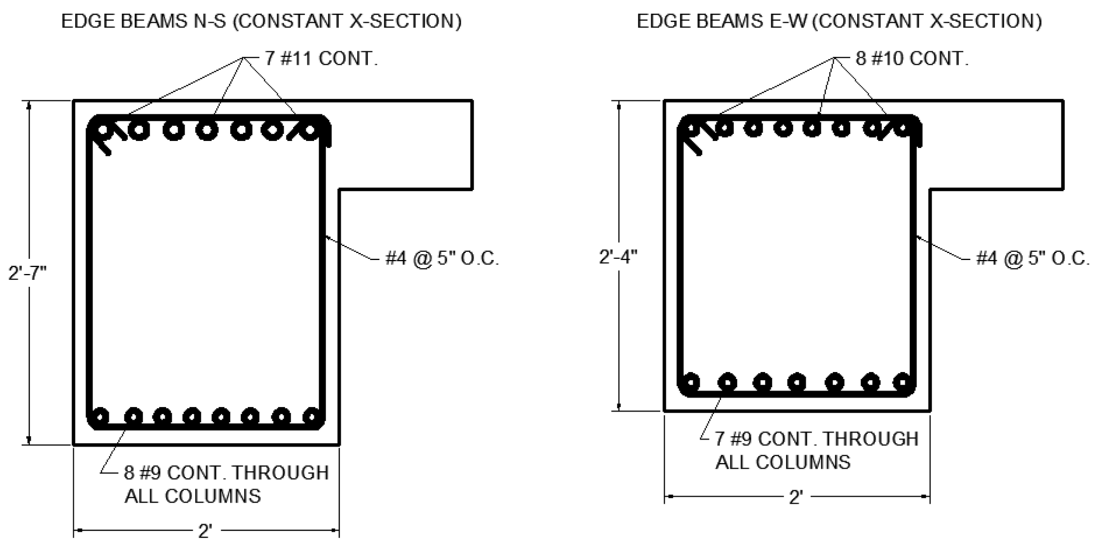
Figure 34: Images of Failing (Top) and Passing (Bottom) Frame 1

Enhanced Local Resistance Method

According to the UFC, enhanced local resistance must be considered at all perimeter columns for the first two stories above grade in an occupancy category IV building. Enhanced local resistance criteria requires than the specified columns must be designed to either have twice the flexural capacity of the traditional design, or satisfy the flexural demand found in the Alternative Path Analysis. Since none of the columns exhibited failure in the Alternative Path Analysis, the perimeter columns would be designed for two times the flexural resistance of their original design.

The flexural resistance of the traditionally designed columns was evaluated for a zero axial load condition considering this is the controlling moment condition on a column that does not see a net tension load. The designed perimeter columns contain 8 #9 bars and have a moment capacity of 370 k*ft about the X axis and 465 k*ft about the Y axis. In satisfying the enhanced local resistance criteria, the columns were able to maintain the same plan dimensions, but reinforcement increased to 12 #11 bars spaced evenly on all faces of the column. This configuration provides 870 k*ft of capacity about the X axis and 1042 k*ft of capacity about the Y axis.

Resulting Member Designs



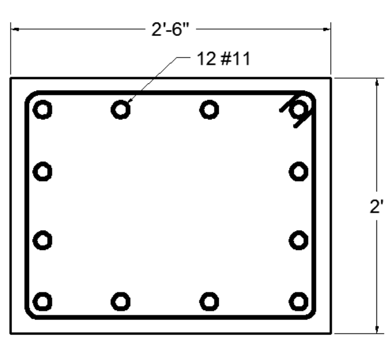


Figure 35: Final Typical Section Designs

Breadth 1: Site Layout Redesign

While designing a building structure to resist progressive collapse is excellent to implement, reducing the risk of an event that could lead to a progressive collapse situation is a more effective and smarter design approach according to the Unified Facilities Criteria. Risks such as vehicular impacts and explosions should be evaluated and mitigated through design of the landscape architecture and exterior of the building. This is considered an indirect design approach and will be the focus of this breadth analysis, concentrating on building / site security and improvements.

According to the GSA Site Security Design Guide, applicable threats and risks to the building must be identified and prioritized first. Then possible solutions for improving the site layout and building security are evaluated for how effective they can be in conjunction with the impact they may have on the aesthetics and function of the building. From here, the best possible solutions are selected and applied to the site design.

Evaluation of Kingstowne Section 36A and its surrounding site found many areas for improvement. Major improvement areas are highlighted in Figure 36. Once the site deficiencies were determined, an action plan for mitigating the risks was created based on the suggestions for site improvement in the Site Security Design Guide. One of the most important deficiencies to consider is the public access that is currently available to the parking garage. As mentioned in the progressive collapse design, parking within the building must be controlled in order to not present the risk of explosion within the building. Under the assumption that an employee of the headquarters building would not want to cause harm to his/her coworkers, access to the parking garage must be restricted to employees only. This is achieved by building a security booth at both of the entrances to the parking garage. Operators in the booths will control both entrance to and exit from the garage through identification screening. Entrance and exit lanes will each contain a collapsible traffic barrier controlled by the booth operators and are to be separated by a structurally sound barrier.

Also of high concern is the proximity of outdoor public parking to the perimeter of the building. Upon analyzing standoff distances for the current site, a 10’ standoff distance was discovered at the south side of the building where available parking is closest. The chosen solution here was to essentially move this parking area 25’ to the south, increasing the standoff distance to 35’. Closely spaced structural bollards placed on the building side of the relocated sidewalk provide a barrier for vehicles targeted at impacting the building at high speed. Parking areas to the north and east of the building are also of concern due to close standoff distance; however both of the areas are intended for use by other buildings. This poses limitations on what can be done to reduce the risks associated with the parking areas. Risk present at the east side of the building was reduced by removing the available parking closest to the building and replacing it with a new sidewalk area and green space to increase the standoff distance. Structural bollards were again used here to protect the building from vehicular impact. Since the parking found at the north side of the building was likely commonly used by patrons of the nearby Kohl’s department store, modifications to the parking lot were not desirable. Instead, 4’-0” tall hardened site furniture and planters were implemented as a barrier against potential vehicular impact and explosions.

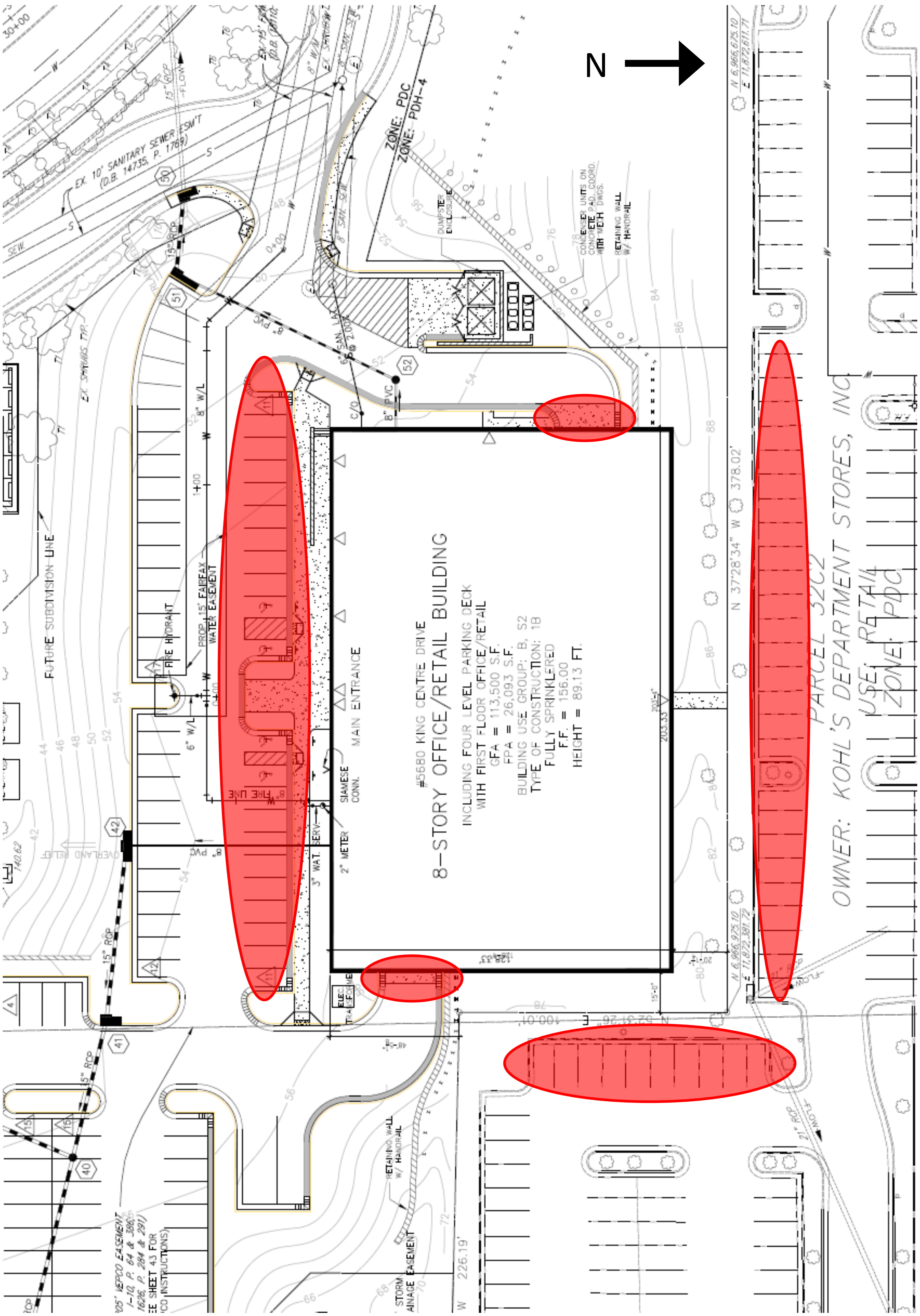


Figure 36: Identified Areas for Improvement

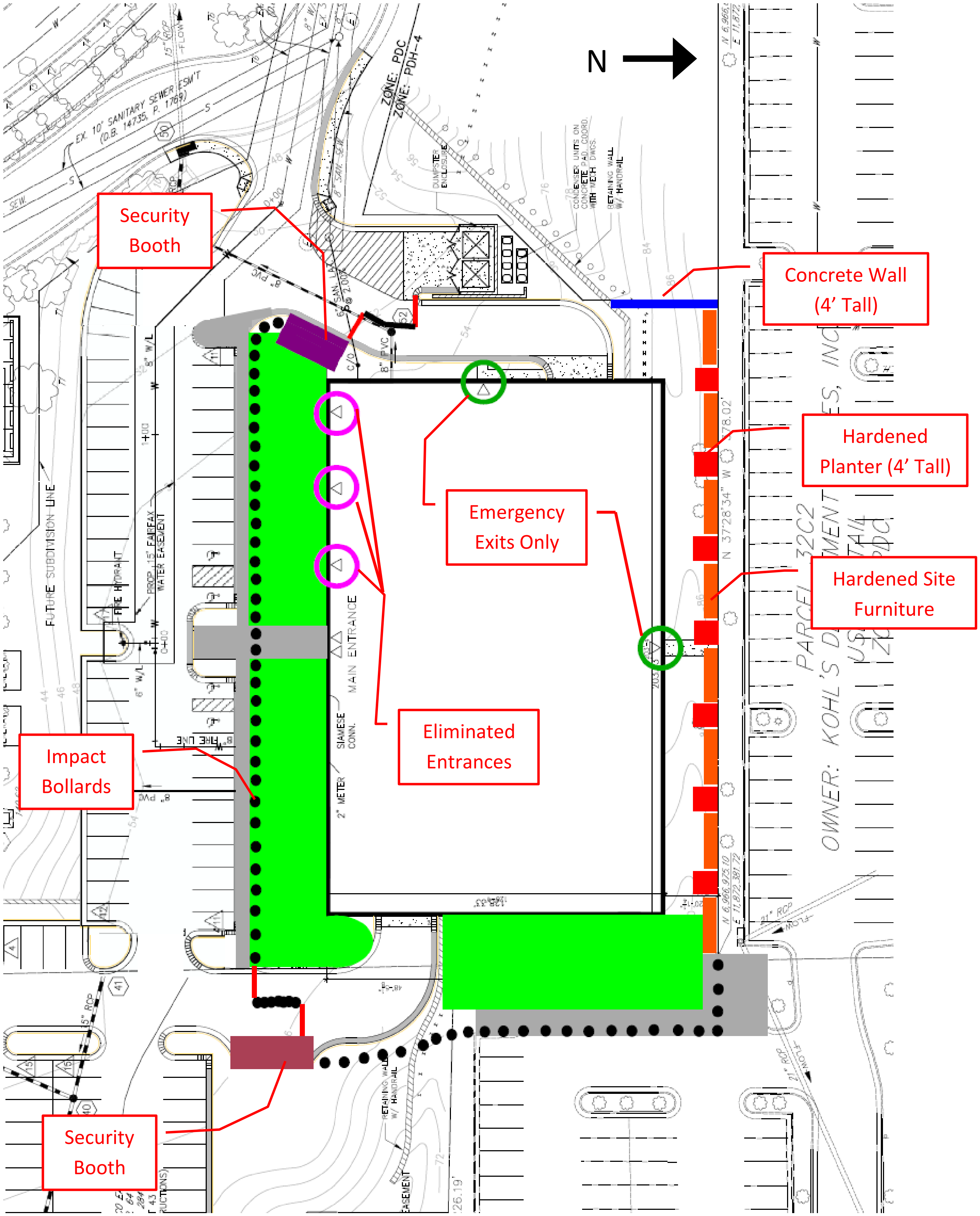


Figure 37: Redesigned Site Plan to Reduce Risk

BREADTH 2: BUILDING ENVELOPE AND FAÇADE STUDY

Kingstowne Section 36A was originally designed with three main different types of facades with a few exceptions. All of the brick work on the building is actually precast concrete panels with a ½" thick thin brick veneer, backed by 6" metal stud wall stuffed with R-19 batt insulation. The brick veneer is of a color and style to match the surrounding buildings in the Kingstowne development. The remaining two primary facades on the building are the uninsulated glazing panels found at the parking levels and the insulated glazing units (IGU) found at the office levels. The goals of this analysis were to design glazing for the building that is resistant to blast loads while meeting or exceeding the performance of the originally designed glazing.

The glass façade of the parking levels consists of two ¼" thick panes of clear glass placed back-to-back. This system is not insulated considering the parking garage levels are not conditioned. Glazing at the office levels consists of two ¼" thick panes of glass with a ½" thick air gap between the panes adding up to a 1" IGU. All glass panels are supported by a Kawneer Trifab aluminum storefront system consisting of mullions and transoms of 2" x 4 ½" in size. For the purposes of this analysis, the glass facades and aluminum mullions/transoms will be sized for wind and blast loads.

Based on the wind loads used for the MWFRS design in the lateral analysis, components and cladding loads would be higher but likely not higher than the pressure due to blast on the glazing systems. Using Figure 1 in ASTM F2248 and Table 2 in ASTM E1300-12a, non-factored blast loads of 100 psf for the office level glazing and 90 psf for parking level glazing were calculated. Based on architectural elevations, glass panes are all 5'-0" wide and vary in height from 6'-0" at parking levels to 6'-6" at office levels. Referencing Figures A1.34 and A1.10 in ASTM E1300-12a results in a ¾" thick PVB laminated inner lite and a 5/8" thick monolithic outer lite at the office levels. The two panes separated by a ½" "gas" gap create the IGU used for the office levels. The 5/8" thick lite is also acceptable for the redesigned glazing at the parking levels. Blast loads resulted in needing a mullion 4" x 7" x 0.25" in cross section, much larger than the 2" x 4 ½ " found on the original façade design. The heat transfer analysis found that the designed façade does not perform as well as the existing one in the summer.

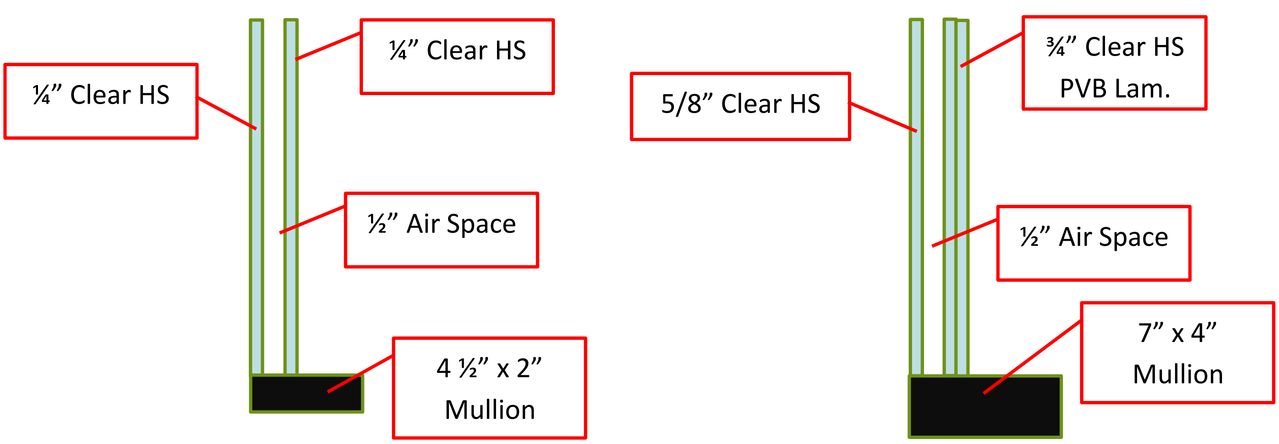


Figure 38: Existing (Left) and Designed (Right) Glazing at Office Levels

MAE Requirements

To meet the MAE curriculum requirements for the proposed senior thesis, knowledge and skills acquired from AE 530, Computer Modeling of Building Structures; AE 538, Earthquake Engineering; and AE 542, Building Enclosure Science and Design were all applied. Redesign of the existing structure to entirely cast-in-place concrete construction was modeled in ETABS to aid in the analysis and design of the structure. Also, an advanced SAP 2000 model was created to perform a non-linear analysis used in designing the building for progressive collapse. Design methods presented in AE 538 were implemented to design the new shear walls and determine if the existing shear walls have enough capacity to resist the seismic loads. Material covered in AE 542 was used to evaluate the existing façade system and design a replacement that is blast resistant.

Conclusion

Kingstowne Section 36A is currently programmed as an office building coupled with publically available parking. Considering a hypothetical new tenant requiring a more robust than average building, the structure of KT36A was successfully designed as a monolithic concrete skeleton capable of resisting scenarios conducive to causing a progressive collapse style failure. The structural design consisted of adding edge beams to the structure at the perimeter to create moment frames capable of spanning a missing column, checking the existing design of OL1 to determine if it is adequate enough to resist the loads considering stiffness difference caused by adding the edge beams, designing the roof level for the heavy mechanical equipment there, designing shear walls to resist lateral load on the building, and stiffening the structure via three different methods to resist progressive collapse. This was considered direct design of the building. Indirect design to resist progressive collapse was implemented by reducing risks found within the site layout and designing glazing to withstand a specified explosion.

Using the United States General Services Administration (GSA) Site Security Design Guide, modifications to the site design layout were implemented to reduce the risk of building and structural damage associated with vehicular impact and exterior explosion. Structural bollards, hardened site furniture, large planters, and security booths were all applied to the site to reduce the possible associated risks.

New glazing for the parking levels and office levels and an aluminum frame support system were designed to withstand the maximum wind pressures and pressure resulting from 80 lbs of TNT exploding at a standoff distance of 35' away. Parking level glazing remained as an uninsulated system, but was increased in thickness to 5/8". Glazing for the office levels also required a thicker system, which remained an insulating glass unit (IGU). Heat transfer analyses were conducted for both the existing and newly designed IGU's. The results found that the new glazing allowed more heat gain in both the summer and winter. While this could be desired in the winter months, it is not desirable during the summer months.

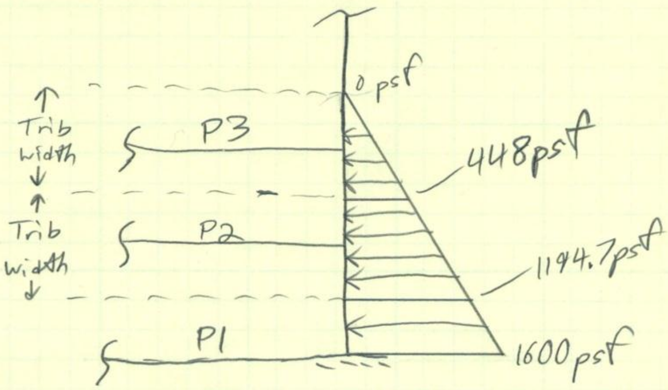
APPENDIX A: Load Calculations

Soil Loading | Tech 3

Per Geotech Report

Soil load = 64 psf/ft depth

Average Grade is about 25 ft above the P1 Parking Level



Force resisted by each floor level = Area · width of building

$R_{P3} = \frac{1}{2}(7\,ft \cdot 448\,psf) \cdot 200\,ft$ ↳ building width in this direction

$R_{P3} = 313.6$ Kips

≈ 40k @ each integer column line

$R_{P2} = \frac{1}{2} 11.6667 (448 + 1194.7) \cdot 200\,ft$

$R_{P2} = 1916.5$ kips

≈ 320k to each shear wall

Using ASCE 7-10 2.3.2

- Apply H with load factor of 1.6 when H adds to the primary load effect
- Apply H with load factor of 0.9 when H resists the primary load effects

General Notes

Future Floor to Ceiling Height = 9'-0"

Office Space Floor to Floor Height = 13'-4"

Result = 4'-4" of Structure + MEP Space
= 52"

Deepest structure consumes 35 1/4" of space
∴ 16 3/4" of space below structure

Provide 24" of clearance below proposed concrete flat slab system (not including drops) for a floor to floor height of 11'-8". This removes 20" (1'-8") from the F-F height at each of the office levels for a total reduction of 80" (7'-8") in the height of the building

Building is now importance category III
Say Police operations facility

From ASCE 7-10 Table 1.5-2

$I_{snow} = 1.20$

$I_{seismic} = 1.50$

Snow loads

Fix Prop. Mods

ASCE 7-10

$$P_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot P_g$$

$C_e = 1.0$ Exposure B (Partially) Table 7-1

$C_t = 1.0$ Table 7-2

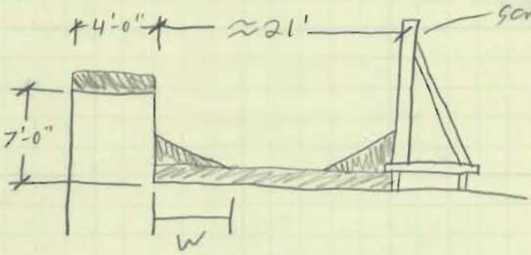
$I_s = 1.20$ risk cat IV

$P_g = 25$ psf (figure 7-1)

$P_f = 0.7 \cdot 1.0 \cdot 1.0 \cdot 1.2 \cdot 25$ psf

$P_f = 21$ psf

Drift Load check



$w = 4h_d$ b/c $h_d < h_c$ $\quad 4h_d = 4 \cdot 1.25' = 5'$

$$h_b = \frac{P_f}{\gamma} = \frac{21\,psf}{(0.13 \cdot 25 + 14)} = 1.22' \approx 14.5''$$

$l_u = 4'-0''$ ⇒ use 20' for calc per figure 7.9

Leeward $h_d = 1.25'$ for $l_u = 20$ ft and $P_g = 25$ psf

windward $h_d = 0.75 \cdot 1.25' = 0.9375'$

leeward drift controls

apply average drift load of 11 psf over a width of 5'-0" around parapet perimeter and screen wall perimeter.

Wind Loads

Calculated using ASCE 7-10

$V = 120$ MPH (Figure 26.5-1B) Risk Cat. IV

Exposure B : (Surface roughness B > 2600ft)

$k_d = 0.85$ (Table 26.6-1)

$k_{zt} = 1.0$ (Section 26.8.2)

Gust Factor

flexible if f_{nat} is < 1.0 Hz

$$f_{nat} = \frac{1}{T_a} = \frac{1}{0.5811\,s} = 1.72 \text{ Hz}$$

T_a → from seismic load determination

1.72 Hz > 1.0 Hz ∴ rigid building

$G = 0.85$

$GC_{pi} = \pm 0.18$ (Enclosed Building) office levels

$GC_{pi} = \pm 0.55$ (Partially Enclosed Building) Parking Levels

K_z for Exposure B

0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04

North Wind

z	k_z
0	0.57
12.67	0.57
23.33	0.647
34.00	0.724
44.67	0.783
56.33	0.835
68.00	0.882
79.67	0.93
91.33	0.964
104.83	1.002

South wind

z	k_z
0	0.57
10.67	0.57
21.34	0.631
33.00	0.718
44.67	0.784
56.34	0.835
68.00	0.882
81.50	0.935

$$q_z = 0.00256 K_z K_{zT} K_d V^2$$

$$p = q G C_p - q_i (GC_{pi})$$

high grade @ approx. 183'-5" → use P3 level (slightly Cons.)

Wind Loads Cont

N-S Direction "South Wind" or "North Wind"

$L/B = 127/202 = 0.6287$

WW$_{wall}$ $C_p = 0.8$ (use w/ q_z)

LW$_{wall}$ $C_p = -0.5$ (use w/ q_h)

Screen Wall is 122'×87'

$L/B = 87/122 = 0.713$

WW $C_p = 0.8$ (q_z) LW $C_p = -0.5$ (q_h)

E-W Direction

$L/B = 202/127 = 1.59$

WW$_{wall}$ $C_p = 0.8$ (use w/ q_z)

LW$_{wall}$ $C_p = -0.4$ (use w/ q_h)

Screen Wall is 122'×87'

$L/B = 122/87 = 1.4$

WW $C_p = 0.8$ (q_z) LW $C_p = -0.4$ (q_h)

all side walls

$C_p = -0.7$ use with q_h

Seismic Loads

Site Class D (Geotech) Location: Kingstowne, VA

Risk Category IV

$I = 1.50$

$R = 5$ $\quad \Omega_0 = 2\frac{1}{2}$ $\quad C_d = 4\frac{1}{2}$ Table 12.2-1 in ASCE 7-10

$S_1 = 0.051$
$S_s = 0.119$ > USGS Report

$S_{ms} = F_a S_s = 1.6 \cdot 0.119 = 0.1904$ Table 11.4-1

$S_{m1} = F_v S_1 = 2.4 \cdot 0.051 = 0.1224$ Table 11.4-2

$S_{DS} = \frac{2}{3} S_{ms} = 0.127 \rightarrow$ Seis. Cat. A

$S_{D1} = \frac{2}{3} S_{m1} = 0.082 \rightarrow$ Seis. Cat. C $\rightarrow$ controls

Per table 12.6-1

Equivalent Lateral Force Analysis is permitted

$V = C_s \cdot W$

$T_a = C_t h_n^x$ $\quad T_L = 8s$

$C_t = 0.02$

$x = 0.75$

$T_a = 0.02 \cdot 89.33^{0.75}$

$T_a = 0.5811s$

Coeff. for upper limit on T_a

$C_u = 1.7$ (Table 12.8-1)

$T_{max} = C_u T_a = 1.7 \cdot 0.5811s$

$T_{max} = 0.9879s$

$$C_s = \min \begin{cases} \frac{S_{DS}}{(R/I_e)} = \frac{0.127}{(5/1.5)} = 0.0381 \\ \frac{S_{D1}}{T(R/I_e)} = \frac{0.082}{0.9879(5/1.5)} = 0.0249 \end{cases}$$

controls

Base shear

$> 0.044 S_{DS} I_e$ ✓

> 0.01 ✓

$V_b = C_s \cdot W$

C_s = same in both directions

$V_b = 0.0249 \cdot 39017\ K$

$V_b = 972K$

12.8.4.3 Amp. of Accidental Torsional Moment

$$A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

$$Y_{direction} \quad A_x = \left(\frac{0.4518}{1.2\,\frac{(0.4518+0.002601)}{2}}\right)^2 = 1.412$$

* calced @ roof

$$X_{direction} \quad A_x = \left(\frac{0.5744}{1.2\left(\frac{0.5744+0.4289}{2}\right)}\right)^2 = 0.9105$$

* calced @ roof

< 1.0 ∴ No tors. irreg.

12.5.3 SDC = C

Type 5 irregularity DNE

∴ provisions of 12.5.2 apply (SDC = B)

No need to apply combo of seismic loading

Load Combos

$1.4D$

$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

$1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$

$1.2D + 1.0E + L + 0.2S$

$0.9D + 1.0W$

$0.9D + 1.0E$

APPENDIX B: Floor System Comparison

		Systems					
		Office Levels (80 psf LL)				Garage Levels (40 psf LL)	
		Existing	Alternatives			Existing	Alternative
Consideration		Composite Steel Deck on Composite Beams and Girders	One-way Concrete Pan Joists on Wide Girders	Non-Composite Steel Deck on Open Web Joists		2-Way Flat Slab With Drop Panels	Precast, Prestressed Double Tees on Precast, Prestressed Beams
Sub-Options				LH Series Joists (4' O.C.)	K-Series Joists (2' O.C.)		
System Stats							
	Slab Weight	44 psf	125 psf	45 psf	45 psf	100 psf	67 psf
	System Weight	49 psf	207 psf	50 psf	52 psf	121 psf	72 psf
	Slab Depth	5.25"	10"	4"	4"	8"	4"
	System Depth	35.25"	30"	39"	36.5"	16"	30"
	Assembly Cost	$25.38/SF	$20.97/SF	$15.32/SF	$18.89/SF	$16.60/SF	$23.25/SF
Architectural							
	Bay Size	28'-6" x 45'-0"	28'-6" x 45'-0"	28'-6" x 45'-0"	28'-6" x 45'-0"	28'-6" x 29'-0"	28'-6" x 45'-0"
	Fire Rating	2 HR - UL Assembly	2 HR	2 HR - UL Assembly	2 HR - UL Assembly	2 HR	< 2 HR
	Other	Additional fire-proofing needed to protect framing members	Decrease in floor to floor height Structure changes to concrete	Increase in floor to floor height, however, may be offset by running mechanical entities through joists	Increase in floor to floor height, however, may be offset by running mechanical entities through joists	Smaller bays needed to make system economical	Provides ability to eliminate 2 column lines in the garage levels Increased floor to floor height over existing slab
Structural							
	Gravity System Alterations	No Change	Concrete joists w/ wide beam girders Concrete columns	Beams become open web joists	Beams become open web joists	No Change	Precast beams, girders, and columns
	Lateral System Alterations	No Change	Extend shear walls from garage levels up to office levels	Little to no change	Little to no change	No Change	Little to no change
	Foundation Alterations	No Change	Significant Impact Increased footing sizes Deep foundation system may be necessary	Little to no change	Little to no change	No Change	May be able to reduce footing sizes due to reduced self-weight
Construction							
	Formwork Required	Minimal	Yes	Minimal	Minimal	Yes	None
	Constructability	Technical	Technical	Easy	Easy	Technical	Technical
	Lead Time	Standard	Standard	Standard	Standard	Standard	Long
Serviceability							
	Vibration Control	Mediocre	Great	Mediocre	Mediocre	Great	Good
Feasible		Yes	No	Yes	Yes	Yes	Yes

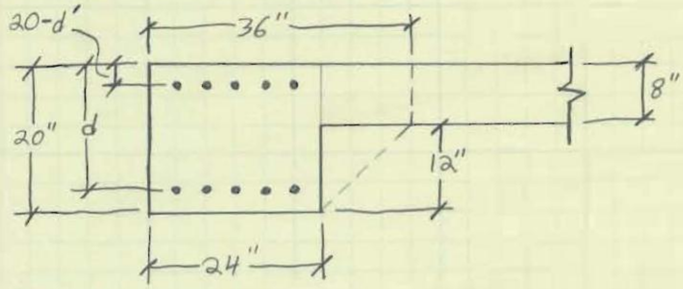
APPENDIX C: Edge Beam Design

Edge Beam Design

- Beams will be 24" wide to keep same width as columns
  - slab is 8" deep per existing design
  - size beam depth @ $\approx 2.5 \cdot$ slab depth Page 734 of W+M

$\approx 2.5 \cdot 8'' = 20''$

Edge beam section



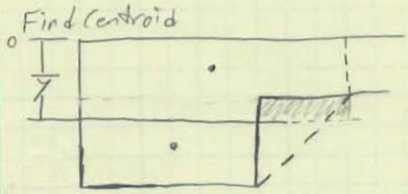
Per GSA B.3

Design primary and secondary structural elements to resist $2(DL+0.25LL)$ applied in both the downward and upward directions.

$$\alpha_f = \frac{E_{cb} \cdot I_b}{E_{cs} \cdot I_s}$$

E_{cb} and E_{cs} are the same since $f'_c = 5000$ psi for both

Find Centroid



$$\bar{y} = \frac{(8'' \cdot 36'') \cdot 4'' + (12'' \cdot 24'') \cdot 14''}{8'' \cdot 36'' + 12'' \cdot 24''}$$

$\bar{y} = 9''$ from top

$I_b = \Sigma \frac{1}{3} b h^3$ provided h touches N/A

$I_b = \frac{1}{3} \cdot 24 \cdot 11^3 + \frac{1}{3} \cdot 36 \cdot 9^3 - \frac{1}{3} \cdot 12 \cdot 1^3$

$I_b = 19392 \text{ in}^4$

I_{slab} = varies ∴ check grids

1-1.5 $b = 12'' + \frac{16'}{2} = 108''$

1-2.0 $b = 12'' + \frac{28.5'}{2} = 183''$

E-F $b = 12'' + \frac{16'}{2} = 108''$

$I_{slab\ 1\text{-}1.5,\ E\text{-}F} = \frac{1}{12} \cdot 108'' \cdot 8''^3$

$= 4608 \text{ in}^4$

$\alpha = 4.2083 > 0.8$ ✓

$I_{slab\ 1\text{-}2.0} = \frac{1}{12} \cdot 183'' \cdot 8''^3$

$= 7808 \text{ in}^4$

$\alpha = 2.4836 > 0.8$ ✓

Edge Beam Design

$f_y = 60{,}000$ psi

∴ In table 9.5(c) of ACI 318-11

Slab $h_{min} = \frac{\ell_n}{36} = \frac{34.5' \cdot 12''}{36} = 11.5''$

↳ however, the $\ell_n = 34.5'$ occurs in an area with continuous drop panels

☆ Even so, deflections will need to be checked on the existing design.

Design Edge Beams for all levels

Dead Loads

Beam Self $= 150\,lb/ft^3 \cdot 1' \cdot 2' = 300\,lb/ft$

Slab $= 150\,lb/ft^3 \cdot \frac{8''}{12''} \cdot 9' = 960\,lb/ft$

Facade $= 100\,lb/ft^2 \cdot 11'8'' = 1167\,lb/ft$

↳ since facade will be redesigned later, an average facade load of 100 psf will be applied to the building based on percentage coverage of precast panel and glass ratio.

SIDL $= 15\,psf \cdot 9' = 135\,lb/ft$

MEP $= 5\,psf \cdot 9' = 45\,lb/ft$

$DL = 2.55\,klf$

Live Loads

office $80\,psf \cdot 9' = 720\,lb/ft$

$LL = 0.720\,klf$

Per GSA guidelines Design load $= 2(DL + 0.25LL)$

Static $2(DL + 0.25LL) = 5.46\,k/f$

Dynamic $(DL + 0.25LL) = 2.73\,klf$

Hand check against RAM Elements Design

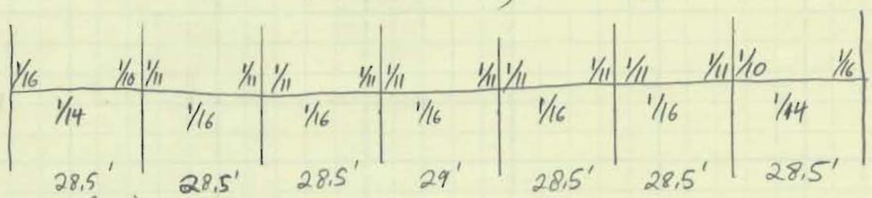
use ACI Moment Coefficients for finding loads

Edge Beam Design

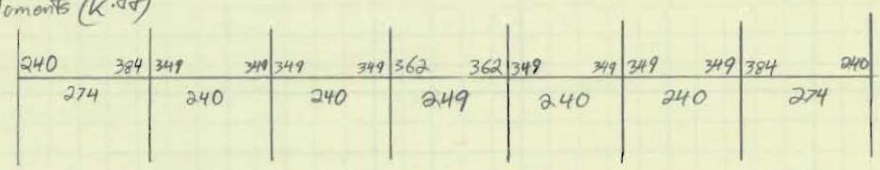
Beam check (Worst case along CL-A)

Controlling $w_u = 2(DL + 0.25LL) = 5.46\,klf$

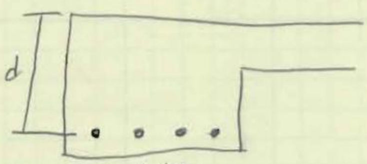
8.3.3 ACI Moment coefficients



Moments (k·ft)



ACI Coefficients are larger than frame analysis #s



$d = 20'' - 1\tfrac{1}{2}'' - \#4 - d_b/2$ → say #6

$d = 17.625''$

Midspan Central Bay

$A_s \approx \frac{M_u}{4d} = \frac{249\,k\cdot ft}{4 \cdot 17.625''}$

$A_s \approx 3.53\,in^2$

Midspan Exterior bay

$A_s \approx \frac{274}{4 \cdot 17.625}$

$A_s \approx 3.89\,in^2$

→ Try 9 #6 bars $A_s = 3.96\,in^2$ → satisfies all positive moment conditions

Ram specs 12 #4 bars

$\phi M_n = \phi A_s f_y (d - a/2)$

$a = \frac{A_s f_y}{0.85 f'_c \cdot b}$

$a = \frac{3.96\,in^2 \cdot 60{,}000\,psi}{0.85 \cdot 5000\,psi \cdot 36''}$

$a = 1.553'' < 8''$ ∴ acts as rect beam

$\beta_1 = 0.80$ ($f'_c = 5000\,psi$)

$c = a/\beta_1 = 1.553''/0.8$

$c = 1.941''$

$\varepsilon_t = \frac{0.003}{c}(d - c)$

$\varepsilon_t = \frac{0.003}{1.941}(17.625 - 1.941)$

$\varepsilon_t = 0.0242$ $(2.42\%) > 0.005$

∴ $\phi = 0.9$

$\phi M_n = 0.9 \cdot 3.96 \cdot 60\,ksi\,(17.625'' - 1.553/2)$

$\boxed{\phi M_n = 300\,k\cdot ft}$

$\phi M_n \geq M_u$ ✓ good for length of continuous beams

per ACI 318-11 7.13.2.2 (b)

keep at least 1/4 $A_{s\,req}$ continuous through all beams (> 2 bars)

∴ 3 #6 bars continuous

Edge Beam Design

Design for max $^{(-)}$ moment at support columns now $M_u = 384 k\cdot ft$

$A_s \approx \frac{M_u}{4d} = \frac{384 k\cdot ft}{4 \cdot 17.625''}$ ↳ assuming #6 bars

$A_s \approx 5.445 in^2$

Try 6 #9 bars
$A_s = 6.00 in^2$

Design as a doubly reinforced beam $A_s' = 3\#6 = 1.32 in^2$

$d' = 20'' - d$
$d' = 2.375''$

check beam assuming Case 1 $\varepsilon_s \geq \varepsilon_y$
$\varepsilon_s' \geq \varepsilon_y$ ✗ N.G.

$a = \frac{5.72 \cdot 60000 - 1.32 \cdot 60000}{0.85 \cdot 5000 \cdot 24''}$ ✗

$a = 2.59''$ $\beta_1 = 0.80$ $c = 3.238$ ✗

$\varepsilon_s' = \frac{0.003}{3.238}(3.238 - 2.375'')$
$\varepsilon_s' = 0.0008 > 0.002$ ✗
∴ case 2

$$0.85 f_c' \cdot \beta_1 \cdot c \cdot b + A_s' \frac{0.003}{c}(c - d')E_s = A_s f_y$$

$$(0.85 f_c' \cdot \beta_1 \cdot b)c^2 + (A_s' \cdot 0.003 \cdot E_s)c - A_s' d' \cdot 0.003 \cdot E_s = (A_s \cdot f_y)c$$

$$(0.85 f_c' \cdot \beta_1 \cdot b)c^2 + (A_s' \cdot 0.003 \cdot E_s - A_s f_y)c - A_s' d' \cdot 0.003 \cdot E_s = 0$$

$$81{,}600 c^2 + (-228{,}360)c - 272{,}745 = 0$$

$$c = 3.702'' \rightarrow a = \beta_1 \cdot c \quad a = 2.962''$$

$$\phi M_n = \phi\left[A_s' \cdot \frac{0.003}{c}(c - d') \cdot E_s \cdot (d - d') + 0.85 f_c' \beta_1 \cdot c \cdot b \cdot \left(d - \frac{\beta_1 c}{2}\right)\right]$$

$$\phi M_n = \phi\left[1.32 \cdot \frac{0.003}{3.702} \cdot (3.702 - 2.375) \cdot 29000000 \cdot (17.625 - 2.375) + 0.85 \cdot 5000 \cdot 0.8 \cdot 3.702 \cdot 24 \cdot \left(17.625 - \frac{2.962}{2}\right)\right] \cdot \frac{1 ft}{12''}$$

$$\phi M_n = \phi[458.7 k\cdot ft]$$

$$\phi M_n = 0.9(458.7 k\cdot ft)$$

$$\boxed{\phi M_n = 412.8 k\cdot ft}$$

$\phi M_n \geq M_u$ ✓ good for length of continuous beams

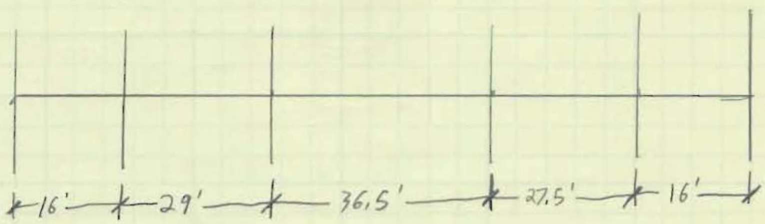
$\varepsilon_s = \frac{0.003}{c}(d - c)$
$= \frac{0.003}{3.702}(17.625 - 3.702)$
$= 0.0113 > 0.005$ ∴ $\phi = 0.9$

$\varepsilon_s' = \frac{0.003}{c}(c - d')$
$= \frac{0.003}{3.702}(3.702 - 2.375)$
$= 0.00108 < 0.002$ ✓

Edge Beam Design

Beam check Worst case along CL-1

controlling $w_u = 2(DL + 0.25LL) = 5.46$ Klf



ACI 8.3.3 Moment coefficients do not apply $\frac{l_{adj}}{l_1} > 20\%$

∴ need to do frame analysis w/ pattern loading

Alternating checkerboard live load applied

resulting in $M^-_{max} = -514.5$ k·ft

$M^+_{max} = 275$ k·ft

Design for max (+) moment @ midspan (occurs @ central span)

$A_s \approx \frac{M_u}{4d} = \frac{275 \text{ k·ft}}{4 \cdot 17.625''}$ ↳ assuming #6 bars

$A_s \approx 3.53 in^2$ → know from from design in opposite direction that 9 #6 bars provides a capacity of 300 k·ft

$\boxed{\text{∴ use 9 \#6 bars with 3 bars continuous through all supports}}$

Design for max (−) moments @ column supports (occurs @ central span)

$A_s \approx \frac{M_u}{4d} = \frac{514.5 \text{ k·ft}}{4 \cdot 17.5''}$ ↳ assuming #8 bars

$A_s \approx 7.35 in^2$

Try 8 #9 bars $A_s = 8 in^2$

check beam assuming case 1 $\varepsilon_s \geq \varepsilon_y$

$\varepsilon_s' \geq \varepsilon_y$ ✗ N.G.

a) $a = \frac{A_s f_y - A_s' f_y}{0.85 f'_c b}$

$a = \frac{7.8 \cdot 60000 psi - 1.32 in^2 \cdot 60000 psi}{0.85 \cdot 5000 \cdot 24''}$

$a = 3.812''$ $c = a/\beta_1$ $c = 4.765''$

Design beam as Doubly reinforced

$A_s' = 3\#6 = 1.32 in^2$

$d' = 2.375''$

$\varepsilon_s' = \frac{0.003}{c}(c - d')$

$= \frac{0.003}{4.765}(4.765'' - 2.375'')$

$\varepsilon_s' = 0.001505 > 0.002$ ✗

∴ case 2

Edge Beam Design

$$0.85 f'_c \cdot \beta_1 \cdot c \cdot b + A_s' \frac{0.003}{c}(c-d')E_s = A_s f_y$$

$$(0.85 f'_c \cdot \beta_1 \cdot b)c^2 + (A_s' \cdot 0.003 \cdot E_s - A_s f_y)c - A_s' d' \, 0.003 \cdot E_s = 0$$

$$81600c^2 + (-353160)c - 272745 = 0$$

$$c = 5.00 \text{ in}$$

$$\phi M_n = \phi\left[A_s' \cdot \frac{0.003}{c}(c-d') \cdot E_s \cdot (d-d') + 0.85 f'_c \cdot \beta_1 \cdot c \cdot b\left(d - \frac{\beta_1 c}{2}\right)\right]$$

$$= \phi\left[1.32 \cdot \frac{0.003}{5.0}(5.0-2.375) \cdot 29{,}000{,}000 \cdot (17.5625-2.375) + 0.85 \cdot 5000 \cdot 0.8 \cdot 5.0 \cdot 24'' \cdot \left(17.5625 - \frac{0.8 \cdot 5}{2}\right)\right]\frac{1 ft}{12''}$$

$$\phi M_n = \phi \cdot 605.4 k \cdot ft$$

$$\phi M_n = 0.9 \cdot 605.4 k \cdot ft$$

$$\boxed{\phi M_n = 545 \, k \cdot ft}$$

$\phi M_n \geq M_u$ ✓ good for length of continuous beams

$$\varepsilon_s = \frac{0.003}{c}(d-c)$$

$$= \frac{0.003}{5}(17.5625-5)$$

$$\varepsilon_s = 0.0075 > 0.005 \therefore \phi = 0.9$$

$$\varepsilon'_s = \frac{0.003}{c}(c-d')$$

$$= \frac{0.003}{5}(5-2.375'')$$

$$\varepsilon'_s = 0.001575 < 0.002 ✓$$

Continue 3 #7 bars through entire length of beam   ACI 318-11 7.13.2.2(a) requires 1/6 As req

Shear Design

$$V_{u(34.5' span)} = \frac{1}{2} \cdot 5.46 klf \cdot 34.5'$$

$$V_u = 94k$$

$$V_{u@d} = 94k - 5.46klf \cdot \frac{17.5''}{12''}$$

$$V_{u@d} = 86k$$

$$V_c = 2\lambda\sqrt{f'_c}\, b_w \cdot d$$

$$V_c = 2 \cdot 1.0\sqrt{5000} \cdot 24'' \cdot 17.5''$$

$$V_c = 59.4k \quad \phi V_c = 44.5k$$

$$\tfrac{1}{2}\phi V_c = 22.3k < 86k$$

$\therefore$ need reinforcing

$$V_{s\,req} = \frac{V_u}{\phi} - V_c = \frac{86k}{0.75} - 59.4k$$

$$V_{s\,req} = 55.3k \leq 8\sqrt{f'_c}\, b_w \cdot d$$

$$\leq 237.6k ✓$$

$$\leq 4\sqrt{f'_c}\, b_w \cdot d$$

$$\leq 118.8k ✓$$

$$\therefore s_{max} = \min\begin{cases} d/2 = 17.5/2 = 8.75'' \rightarrow \text{use } 8'' \\ 24'' \end{cases}$$

using #4 $\quad s = A_v \cdot f_{yt} \cdot \frac{d}{V_s} \leq 8''$

$A_v = 0.4 in^2$

$$s = 0.4in^2 \cdot 60ksi \cdot \frac{17.5''}{55.3k}$$

$$s = 7.6'' \rightarrow \text{use } 6''$$

$0.4 in^2 \geq A_{v\,min} \therefore$ ok

Edge Beam Design

find when spacing can go to 8" ← (max spacing)

V_s provided using 8" spacing

$$V_s = A_v \cdot f_{yt} \cdot \frac{d}{s}$$
$$= 0.4\,in^2 \cdot 60\,ksi \cdot \frac{17.5''}{8''} = 52.5\,k$$

$$V_u \text{ discontinue} = V_{u_{dis}} = \phi(V_s + V_c)$$
$$= 0.75(52.5k + 59.4k)$$
$$= 83.9\,k$$

$$83.9\,k = 94k - 5.46\,klf \cdot \frac{x}{12''}$$

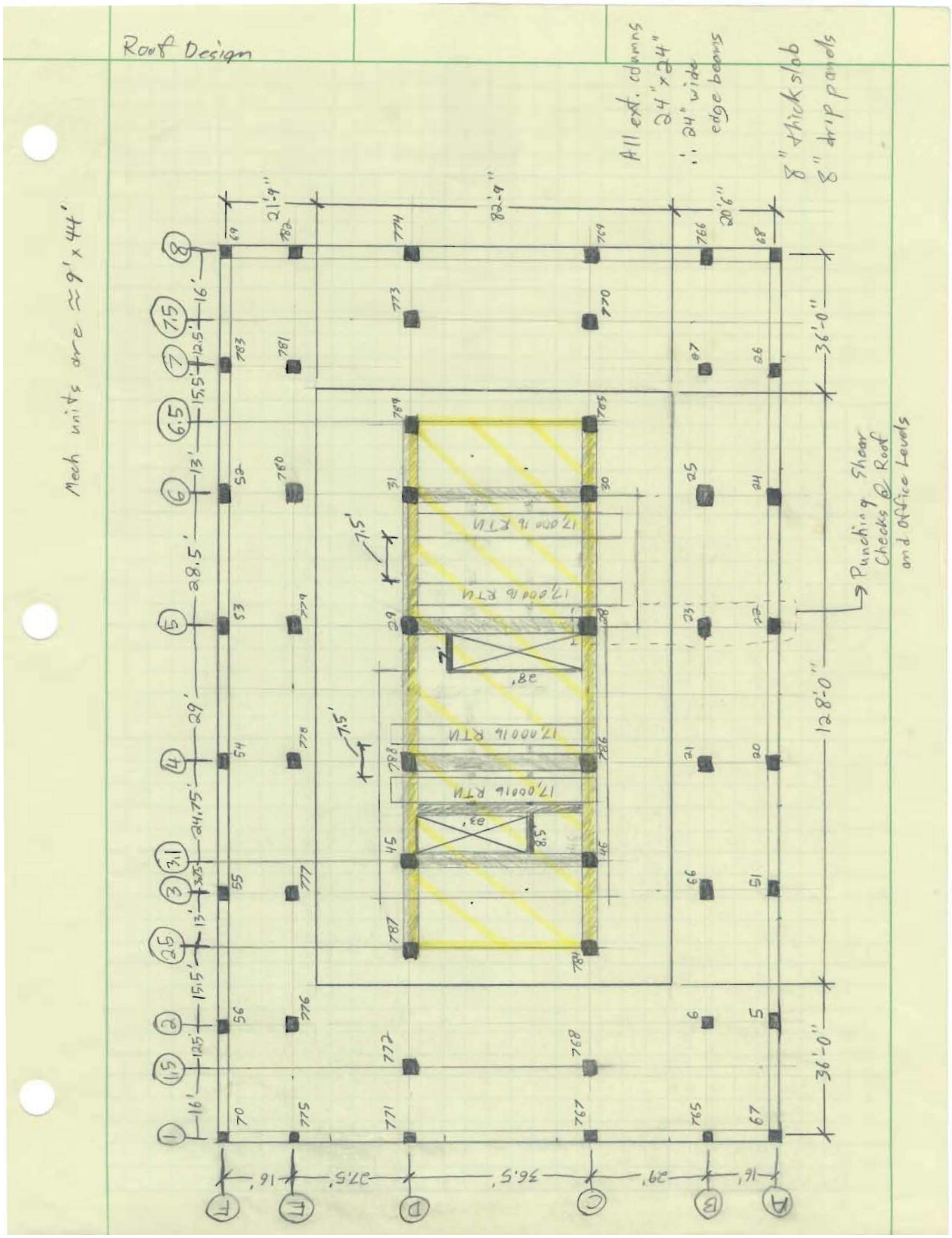
$x = 22.2''$ into beam → use #4 @ 6" O.C. for first 2'-0" of beam from each column

then use #4 @ 8" O.C. for remaining 30'-6"

All remaining spans are less than 30'-6"

∴ #4 @ 8" O.C. will satisfy shear requirements for all other spans

APPENDIX D: Roof and Column Design

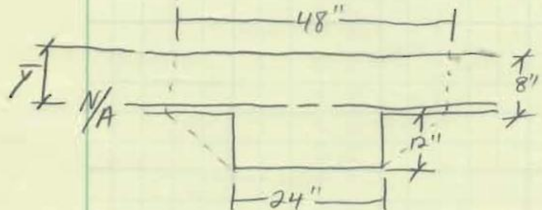


Roof Design

All beams spanning between columns are 24" wide and are 20" deep including slab depth.

Control Bay will be controlling bay (C-D, 5-6) for Mechanical area stiffened with beams. Effects of interior beams will be neglected for finding the α values.

Try 8" thick slab to keep similar formwork as floors below



$$\bar{Y} = \frac{8" \cdot 48" \cdot 4" + 24" \cdot 12" \cdot 14"}{8 \cdot 48 + 24 \cdot 12}$$

$$\bar{Y} = 8.3"$$

$I_b = \Sigma \frac{1}{3} b h^3$ → h touches N/A

$I_b = \frac{1}{3} \cdot 24" \cdot 11.7^3 + \frac{1}{3} \cdot 48 \cdot 8.3^3 - \frac{1}{3} \cdot 24 \cdot 0.3^3$

$I_b = 21{,}961 \text{ in}^4$

For areas where beam is bordered by a slab opening
$I_b = 19{,}392 \text{ in}^4$

$I_{s\,5,C-D} = \frac{1}{12} \cdot \left(\frac{28.5}{2} + 1\right) \cdot 12" \cdot 8"^3 = 7808 \text{ in}^4 \quad \alpha_{5\,C-D} = \frac{19392}{7808} = 2.484$

$I_{s\,6\,C-D} = \frac{1}{12} \cdot \left(\frac{28.5+13}{2}\right) \cdot 12" \cdot 8"^3 = 10624 \text{ in}^4 \quad \alpha_{6\,C-D} = \frac{21961}{10624} = 2.067$

$I_{s\,C\,5-6} = \frac{1}{12} \cdot \left(\frac{36.5+29}{2}\right) \cdot 12" \cdot 8"^3 = 16768 \text{ in}^4 \quad \alpha_{C\,5-6} = \frac{21961}{16768} = 1.310$

$I_{s\,D\,5-6} = \frac{1}{12} \cdot \left(\frac{36.5+27.5}{2}\right) 12" \cdot 8"^3 = 16384 \text{ in}^4 \quad \alpha_{D\,5-6} = \frac{21961}{16384} = 1.340$

$$\alpha_{fm} = \frac{2.484 + 2.067 + 1.31 + 1.34}{4}$$

$$\alpha_{fm} = 1.80 \quad \therefore h_{min} = \frac{\ell_n \left(0.8 + \frac{f_y}{200{,}000}\right)}{36 + 5\beta(\alpha_{fm} - 0.2)}$$

$$= \frac{34.5 \cdot 12" \left(0.8 + \frac{60000}{200{,}000}\right)}{36 + 5 \cdot \frac{34.5}{26.5}(1.80 - 0.2)}$$

$h_{min} = 9.81"$

$\therefore$ use 10" slab in mechanical areas

by inspection α_{fm} will still be between 0.2 and 2

Roof Design

Loads

Dead

Self wt. = 125 psf @ 10" slab   100 psf @ 8" slab

MEP =
SI = } 30 psf (per what was used in original design)

Snow

Flat roof = 21 psf → controls over roof live

Dift @ screen = 11 psf (5' wide)

Drift @ Parapet = 11 psf (5' wide)

Live

Roof @ Mechanical = 20 psf since equipment weight is directly considered

Roof outside Mechanical = 20 psf

Mechanical Equip (Dead)

Each unit weighs 17,000 lbs

assume there are 8 supports for each unit, each carrying equal load

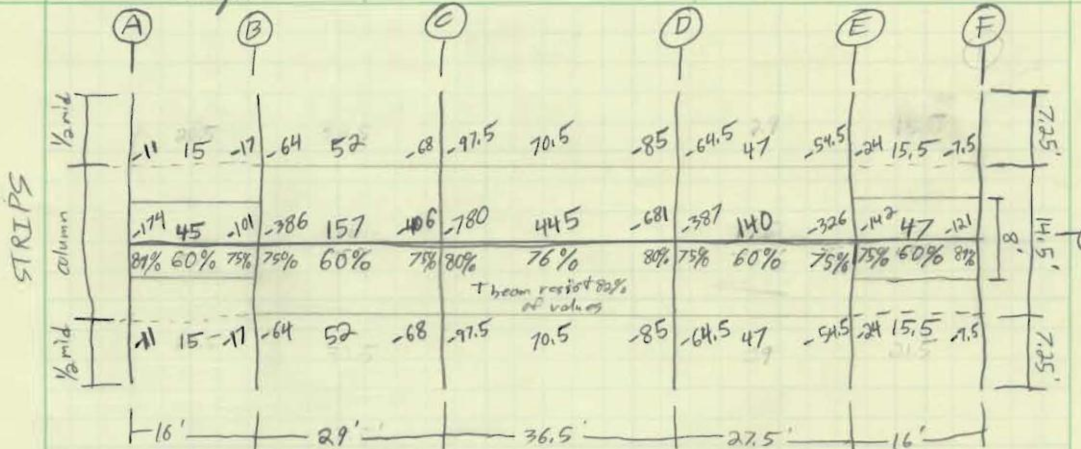
4 supports on each side, each supporting 2,125 lbs

Choose Design Method

Direct Design X   some panels act as one way slabs
ie $\ell_1/\ell_2 > 2$
certain spans differ more than 1/3 the longer span

∴ Equivalent Frame Method

Roof Design



Moment Distribution

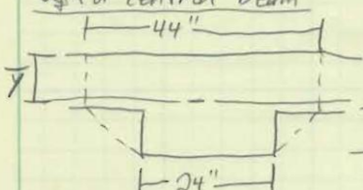
$$\%^-_{int} = 75 + 30\left(\frac{\alpha_{f1}\cdot \ell_2}{\ell_t}\right)\left(1-\frac{\ell_2}{\ell_1}\right)$$

$$\%^-_{ext} = 100 - 10\beta_t + 12\beta_t\left(\frac{\alpha_{f1}\ell_2}{\ell_1}\right)\left(1-\frac{\ell_2}{\ell_1}\right) \qquad \beta_t = \frac{E_{cb}\cdot C}{2E_{cs}\cdot I_s} \qquad C = \Sigma\left(1-0.63\frac{x}{y}\right)\frac{x^3y}{3}$$

$$I_s = \frac{\ell_2 h^3}{12}$$

$$\%^+ = 60 + 30\left(\frac{\alpha_{f1}\ell_2}{\ell_1}\right)\left(1.5-\frac{\ell_2}{\ell_1}\right)$$

α_f for central beam



$$\bar{y} = \frac{10\cdot 44\cdot 5 + 10\cdot 24\cdot 15}{10\cdot 44 + 10\cdot 24} \qquad I_s = \frac{1}{12}\cdot\left(\frac{28.5+15.25}{2}\right)\cdot 12''\cdot 10''^3$$

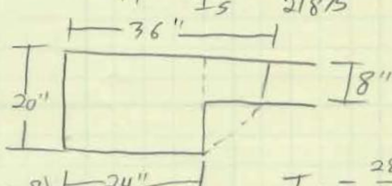
$$\bar{y} = 8.53'' \qquad I_s = 21{,}875\ in^4$$

$$I_b = \frac{1}{3}\cdot 24\cdot(20-8.53)^3 + \frac{1}{3}\cdot 44\cdot 8.53^3 + \frac{1}{3}\cdot 20\cdot(10-8.53)^3$$

$$I_b = 21196\ in^4$$

$$\alpha_{f1} = \frac{I_b}{I_s} = \frac{21198}{21875} = 0.969$$

β_t for edge beam



$$C = \Sigma\left(1-0.63\frac{x}{y}\right)\frac{x^3y}{3}$$

$$C = \left(1-0.63\cdot\frac{20}{24}\right)\frac{20^3\cdot 24}{3} + \left(1-0.63\frac{8}{12}\right)\frac{8^3\cdot 12}{3} \qquad I_s = \frac{28.5\cdot 12\cdot 8''^3}{12}$$

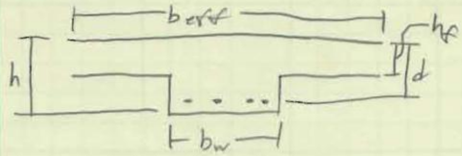
$$C = 31588\ in^4 \qquad I_s = 14592 \qquad \beta_t = \frac{31588}{2\cdot 14592} = 1.08$$

Roof Design

Since remaining roof loads are similar to the office levels, an 8" thick slab will be kept in this area.

<u>Column line 4 Analysis Direction</u>

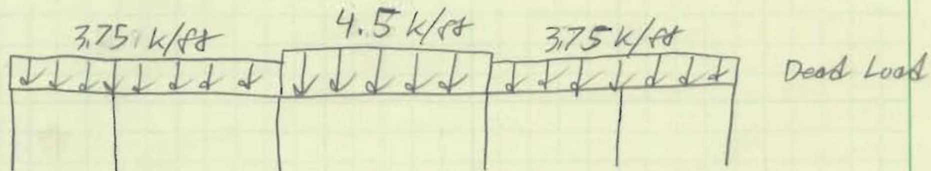
Beam between C and D will act as a T-beam

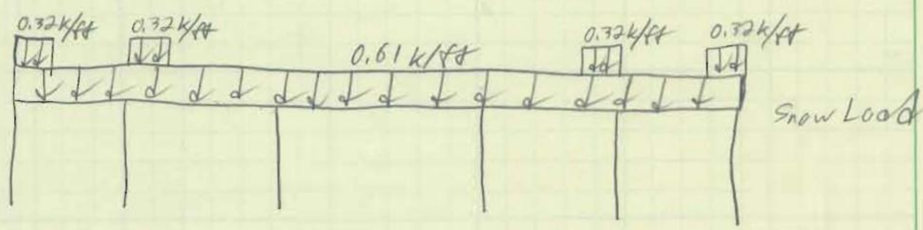


$$b_{eff} = \min \begin{cases} b_w + 16h_f = 24 + 16 \cdot 10 = 184 \\ b_w + 2(\tfrac{1}{2}\,\text{clr dist}) = 322.5 \\ \tfrac{1}{4}\,\ell = \tfrac{1}{4} \cdot 36.5 \cdot 12 = 109.5 \end{cases} \rightarrow b_{eff} = 109.5$$

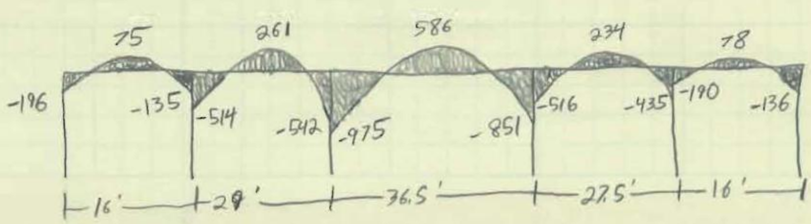
Trib width for moment calcs

$$= 28.75'$$





Controlling Load Combo = 2 DL + 0.5LL

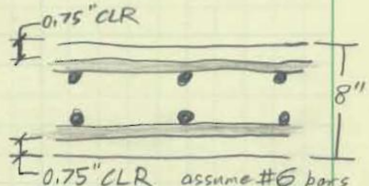


$\frac{0.136}{12} = \frac{0.2}{x}$

Roof Design

All moments Determined from equivalent frame method will be used to design slab sections of the found strip widths.

Code checks of slab reinforcement



7.12.2.1 $\rho_{min} = 0.0018 \Rightarrow$ #4 @ 16" O.C.

13.3.2 $S_{bars} \le 2t = 16"$

assume #6 bars
$\therefore d = 6.125"$ short dir
$d = 6.875"$ long dir

13.3.3 Positive moment reinforcement shall extend to edge of slab at discontinuous edge

Column line 4 Bay A-B

$A_s = \frac{M_u}{\phi f_y jd}$ $jd \simeq 0.95d$ $\phi = 0.9$

Column Strip 8' wide

Ext. A $A_s = \frac{174k \cdot ft \cdot 12"}{0.9 \cdot 60ksi \cdot 0.95 \cdot 6.125} = 6.65 in^2 \rightarrow$ #6 @ 6" O.C. 16 bars

Mid. $A_s = \frac{45 \cdot 12}{0.9 \cdot 60 \cdot 0.95 \cdot 6.125} = 1.72 in^2 \rightarrow$ #5 @ 16" O.C. 6 bars

Int. B $A_s = \frac{101 \cdot 12}{0.9 \cdot 60 \cdot 0.95 \cdot 6.125} = 3.86 in^2 \rightarrow$ #6 @ 10" O.C. 9 bars

½ Middle Strip 10.5' wide

Ext. A $A_s = 0.42 in^2 \rightarrow$ #4 @ 16" O.C. controls 8 bars

Mid $A_s = 0.57 in^2 \rightarrow$ #4 @ 16" O.C. controls 8 bars

Int. B $A_s = 0.65 in^2 \rightarrow$ #4 @ 16" O.C. controls 8 bars

Column line 4 Bay B-C

Column Strip 14.5' wide

Int. B $A_s = 14.74 in^2 \rightarrow$ #6 @ 5" O.C. 34 bars

Mid. $A_s = 6.00 in^2 \rightarrow$ (14) #6 @ 12" O.C.

Int. C $A_s = 15.5 in^2 \rightarrow$ (36) #6 @ 4" O.C.

½ Middle strip 7.25' wide

Int. B $A_s = 2.44 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Mid. $A_s = 2.00 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Int. C $A_s = 2.6 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Design Beam !!

Roof Design

Column line 4 Bay C-D (10" deep slab)

Column Strip ✱ Design for T-beam to resist 82% of moments
14.5' wide

Int. C $A_s = 9.14 in^2 \rightarrow$ (21) #6 in top of beam

Mid $A_s = 5.21 in^2 \rightarrow$ (6) #9 in bottom of beam

Int. D $A_s = 7.88 in^2 \rightarrow$ (19) #6 in top of beam

½ Middle Strip 7.25' wide

Int. C $A_s = 2.81 in^2 \rightarrow$ (7) #6 @ 12" O.C.

Mid $A_s = 2.03 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Int. D $A_s = 2.45 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Column line 4 Bay D-E

Column Strip 14.5' wide

Int. D $A_s = 14.8 in^2 \rightarrow$ (34) #6 @ 5" O.C.

Mid $A_s = 5.35 in^2 \rightarrow$ (14) #6 @ 12" O.C.

Int. E $A_s = 12.45 in^2 \rightarrow$ (29) #6 @ 6" O.C.

½ Middle Strip 7.25' wide

Int D $A_s = 2.46 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Mid $A_s = 1.79 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Int E $A_s = 2.08 in^2 \rightarrow$ (6) #6 @ 14" O.C.

Column line 4 Bay E-F

Column Strip 8' wide

Int. E $A_s = 5.42 in^2 \rightarrow$ #6 @ 6" O.C. 13 bars

Mid $A_s = 1.79 in^2 \rightarrow$ #5 @ 16" O.C. 6 bars

Ext. F $A_s = 4.62 in^2 \rightarrow$ (11) #6 @ 8" O.C.

½ Middle Strip 10.5' wide

Int. E $A_s = 0.92 in^2 \rightarrow$ (8) #4 @ 16" O.C. controls

Mid $A_s = 0.59 in^2 \rightarrow$ (8) #4 @ 16" O.C. controls

Ext. F $A_s = 0.29 in^2 \rightarrow$ (8) #4 @ 16" O.C. controls

Column Designs/checks (Gravity)

check typical int column @ lowest level of building (B-5)

$P = 1810K$ $(2D+0.5L)$ $P_u = 1373K$ 24"x24" 10 #10

$M_{unb} \approx 300k \cdot ft$

Current design fails GSA recommendation

$\phi P_n = 1640K$ passes strength design

Try 24"x24" w/ 12 #11 bars

$\phi P_n = 1820k \geq P_{GSA}$ ✓

$\geq P_u$ ✓

$\phi M_n @ P_u = 570$ ft·k

∴ Column is adequate for Gravity Loads

check typical ext. column @ lowest level of building (A-5)

$P = 1135k$ $(2D+0.5L)$ $P_u = 816K$ 30"x24" 8 #9

$M_{unb} \approx 250$ ft·k

*Note

column may become part of lateral system

Current design is adequate

$\phi P_n = 1820K \geq P_{GSA}$ ✓

$\geq P_u$ ✓

$\phi M_n @ P_u = 748$ ft·k (weak axis)

∴ 24"x30" w/ 8 #9 bars is adequate for gravity loads

check heavily loaded interior column @ lowest level (C-5)

$P = 2652K$ $(2D+0.5L)$ $P_u = 1992K$ 24"x36" 14 #11

$M_{unb} \approx 450$ ft·k

Current design fails GSA recommendation passes strength design

$\phi P_n = 2540K$

Try 24"x36" w/ 18 #11 bars

$\phi P_n = 2720K \geq P_{GSA}$ ✓, P_u ✓

$\phi M_n @ P_u = 1290$ K·ft (Strong Axis)

∴ 24"x36" w/ 18 #11 bars is adequate for gravity loads

Column Designs/checks (Gravity)

Column C-5 Design @ OL1 $\rho_{min} = 1\%$
$\rho_{max} = 8\%$

Maintain shape to keep formwork
→ ends up being highly inefficient

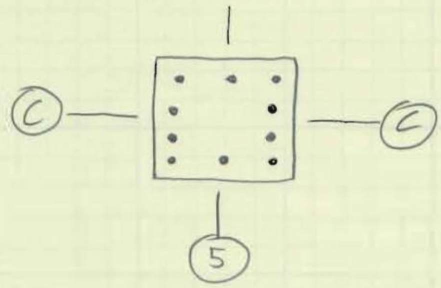
$P_{GSA} = 1192k$ > $M_{unb} \approx 450$ ft·k

$P_u = 872k$

SP Column → 24"×24" w/ 10 #9 bars

$\phi P_n = 1560K$

$\phi M_n @ P_u = 550k \cdot ft$



Column Designs/Checks (Gravity)

Column B-5 Design @ OL1

$\rho_{min} = 1\%$

$\rho_{max} = 8\%$

$P_{GSA} = 805\,k$

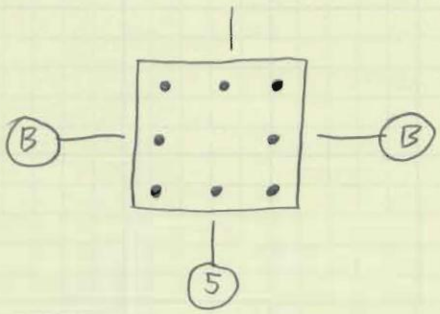
$P_u = 600\,k$

$M_{unb} \approx 300\,k\cdot ft$

SP Column ⟶ 24" × 24" w/ 8 #8 bars

$\phi P_n = 1460\,k$

$\phi M_n @ P_u = 580\,k\cdot ft$



Column A-5 Design @ OL1

$\rho_{min} = 1\%$

$\rho_{max} = 8\%$

$P_{GSA} = 480\,k$

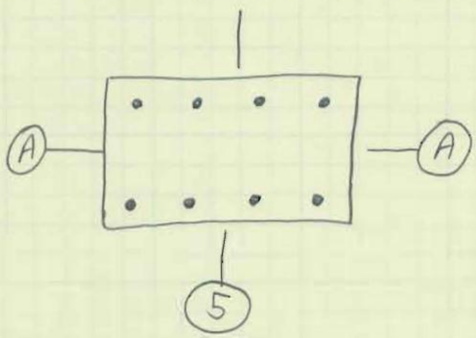
$P_u = 350\,k$

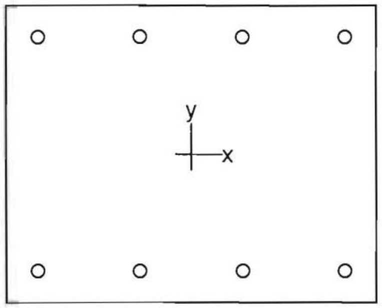
$M_{unb} \approx 250\,k\cdot ft$

SP Column ⟶ 30" × 24" w/ 8 #9 bars

$\phi P_n = 1825\,k$

$\phi M_n @ P_u = 635\,k\cdot ft$

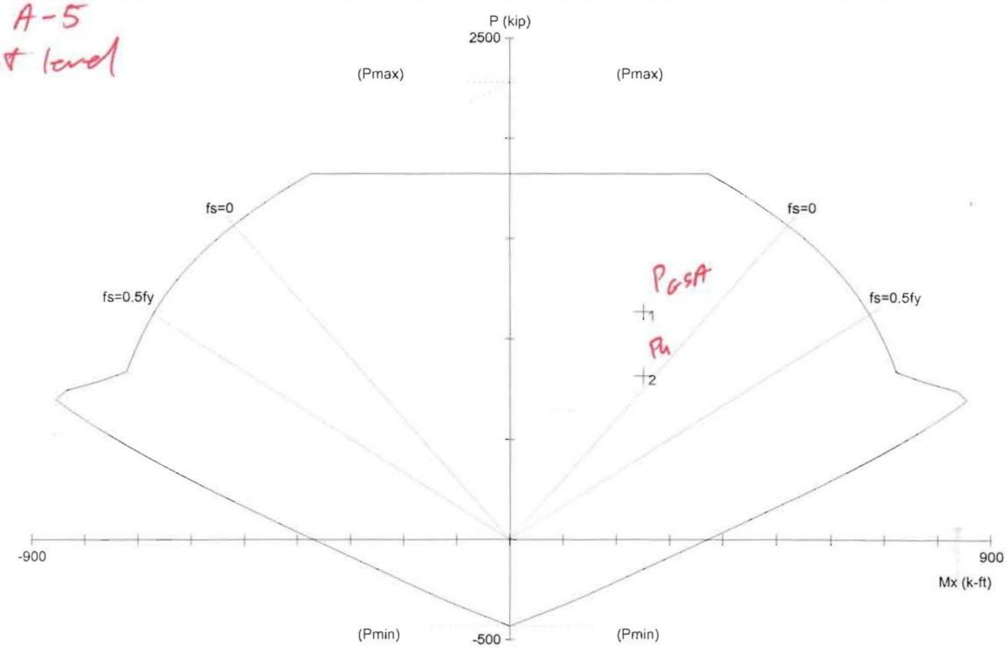




30 x 24 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 02/24/13
Time: 20:48:21

Column A-5
@ lowest level

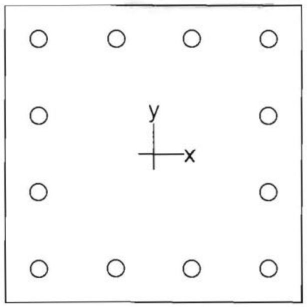


spColumn v4.81. Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68

File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\A-5 30 x 24 Cap Check.col
Project:

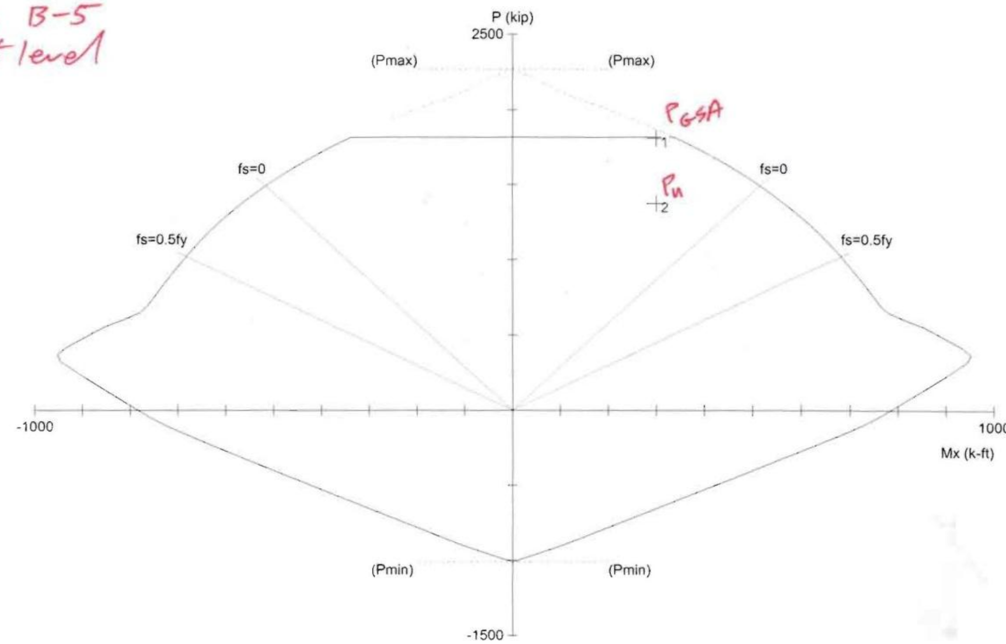
Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 720 in^2	8 #9 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 8.00 in^2	rho = 1.11%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 34560 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 54000 in^4
Beta1 = 0.8		Min clear spacing = 7.16 in	Clear cover = 2.00 in

Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65



24 x 24 in

Column B-5
@lowest level



Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 02/21/13
Time: 21:42:25

spColumn v4.81. Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68

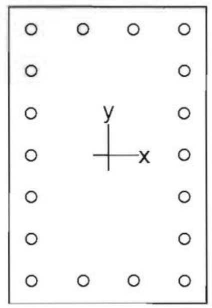
File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\24 x 24 Cap Check.col

Project:

Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 576 in^2	12 #11 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 18.72 in^2	rho = 3.25%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.8		Min clear spacing = 4.79 in	Clear cover = 2.00 in

Confinement: Tied

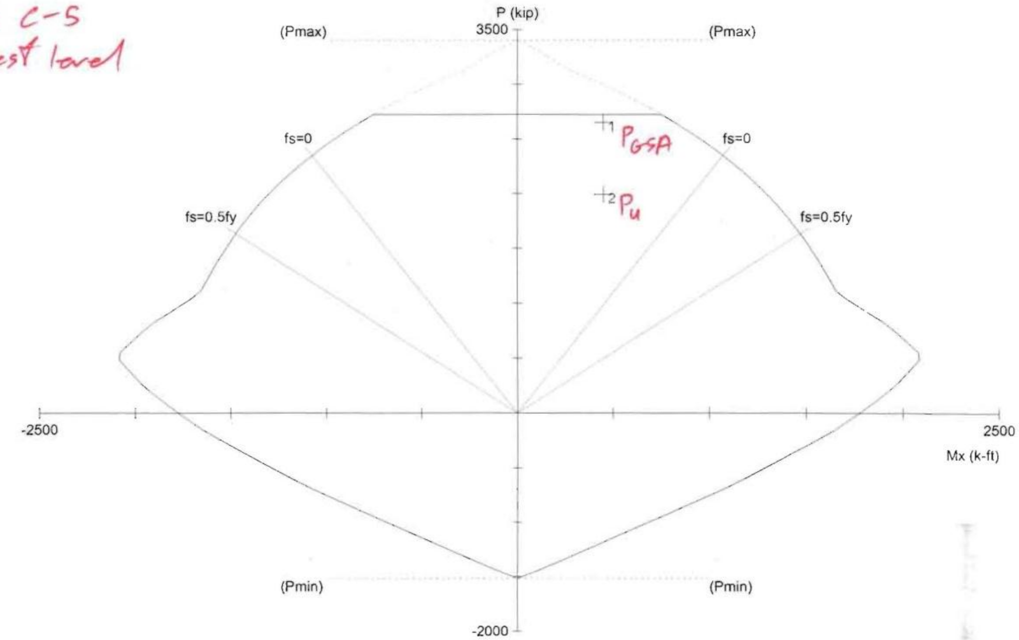
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65



24 x 36 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 02/24/13
Time: 19:46:42

Column C-5
@ lowest level



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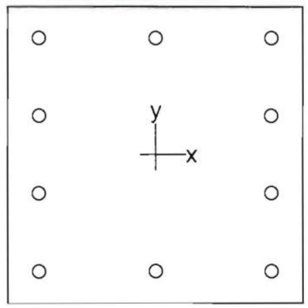
File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\C-5 24 x 36 Cap Check.col
Project:

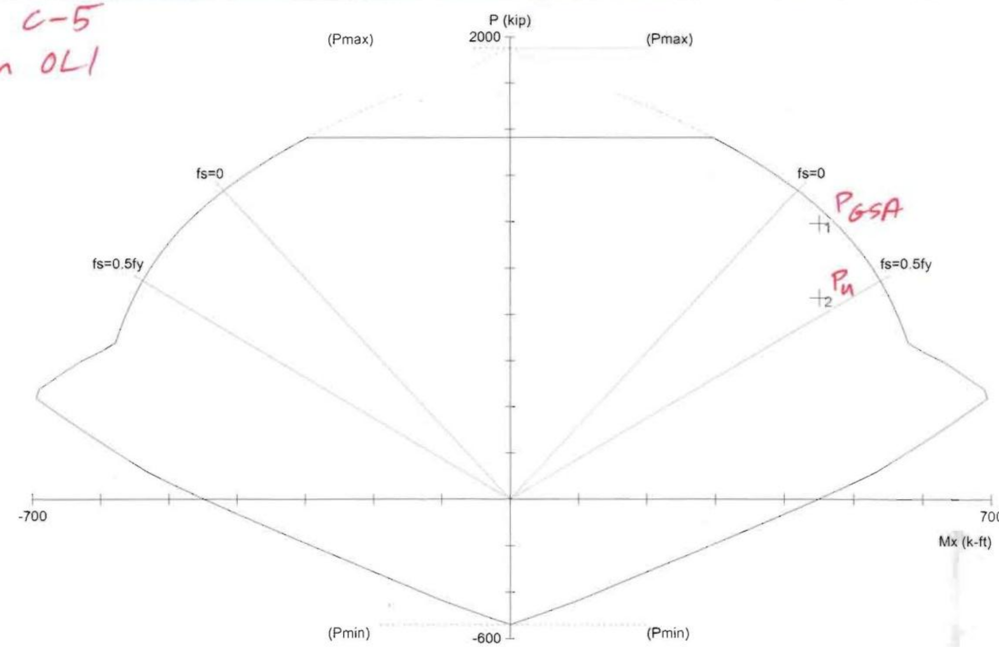
Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 864 in^2	18 #11 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 28.08 in^2	rho = 3.25%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 93312 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 41472 in^4
Beta1 = 0.8		Min clear spacing = 3.69 in	Clear cover = 2.00 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			



24 x 24 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Design
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 02/24/13
Time: 20:17:57

Column C-5
Design OL1



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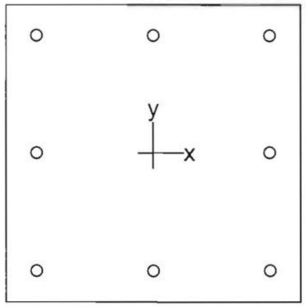
File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\C-5 24 x 36 Design @ OL1.col
Project:

Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 576 in^2	10 #9 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 10.00 in^2	rho = 1.74%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.8		Min clear spacing = 5.16 in	Clear cover = 2.00 in

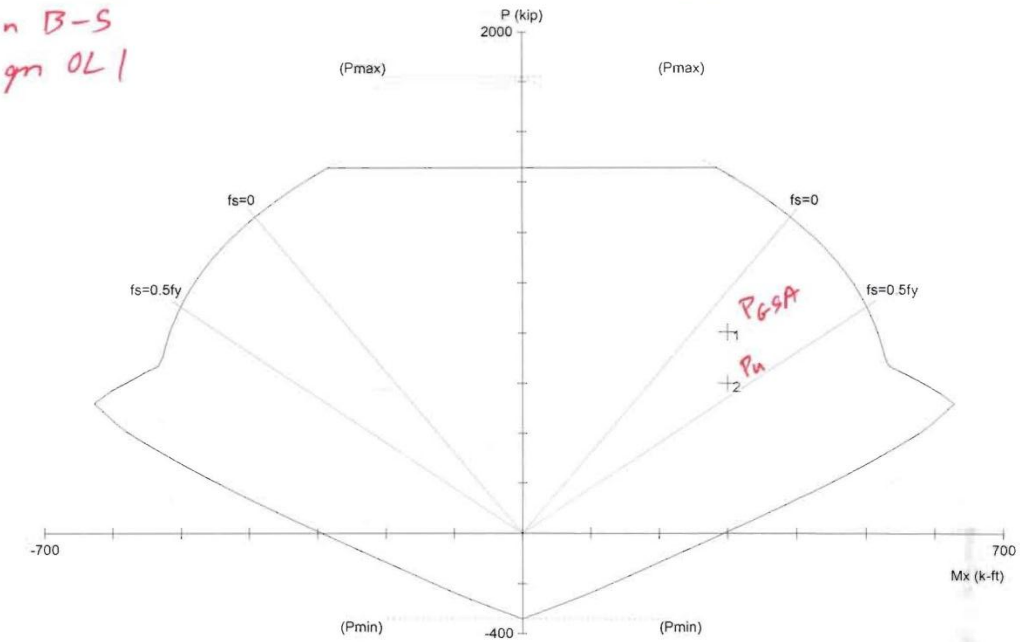
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65



24 x 24 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Design
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 02/24/13
Time: 20:51:59

Column B-5
Design OL1



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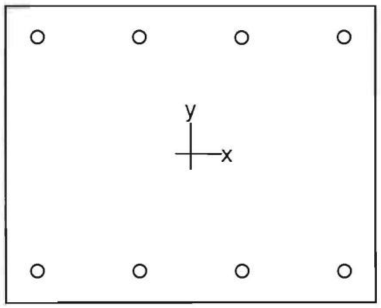
File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\B-5 24 x 24 Design @ OL1.col
Project:

Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 576 in^2	8 #8 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 6.32 in^2	rho = 1.10%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.8		Min clear spacing = 8.50 in	Clear cover = 2.00 in

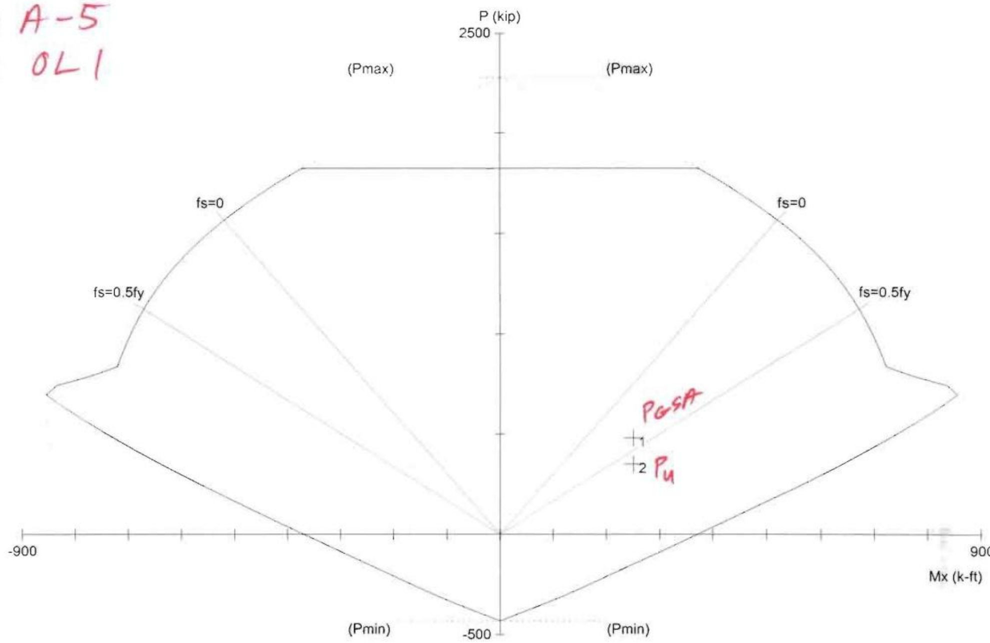
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65



30 x 24 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Design
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 02/24/13
Time: 20:42:15

Column A-5
Design OL1



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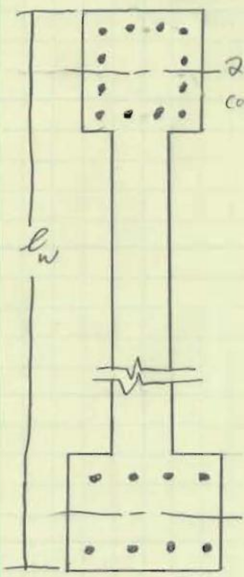
File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\A-5 30 x 24 Design @ OL1.col

Project:			
Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 720 in^2	8 #9 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 8.00 in^2	rho = 1.11%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 34560 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 54000 in^4
Beta1 = 0.8		Min clear spacing = 7.16 in	Clear cover = 2.00 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

APPENDIX E: Shear Wall Design

Shear wall Design | Worst Case of SW 7,8,9,10,11,12 = SW7 EQ Y T SOIL

Check that existing walls are adequate for new loads



24"x24" Column 12 #11 bars (See SP column Design)
compression for controlling case

$h_w = 12.67'$ $h_w/\ell_w = 0.704$

$\ell_w = 18.0'$

Max loads @ Base

$V2 = 493\,K$

$M3 = 6268\,k\cdot ft$

ρ_h = wall thickness

Tension for controlling case
30"x24" Column 8#9 bars (See SP Column Design)

Detail Reinforcement to put some axial load in walls
Check designs with $N_u = 0$ though
→ use equations that utilize N_u

Check moment strength of wall

$T = A_s \cdot f_y$

$T = 8 \cdot 1.0\,in^2 \cdot 60{,}000\,psi$

$T = 480\,K$

$d = 17' = (\ell_w - \frac{24''}{2})$

$a = \dfrac{T + \cancel{N_u}^{\,0}}{0.85 f'_c\, b}$

$a = \dfrac{480{,}000\,lb}{0.85 \cdot 5000 \cdot 24''}$

$a = 4.7'' < 24''$ ✓ ∴ eq. is valid

$M_n = T(d - a/2) + \cancel{N_u \left(\frac{\ell_w - a}{2}\right)}^{\,0}$

$c = a/\beta_1 = 5.875''$

$\beta_1 = 0.8$ (5000 psi concrete)

Is $5.875'' < 0.375d$?

$5.875'' < 76.5''$ ✓ ∴ $\phi = 0.9$

(Mtw pg. 960)

$M_n = 480K(17' \cdot 12''/ft - 4.7/2)$

$M_n = 8066\,k\cdot ft$

$\phi M_n = 0.9 \cdot 8066\,k\cdot ft$

$\phi M_n = 7259\,k\cdot ft \geq 6268\,k\cdot ft$ ✓

Sufficient Moment Strength

Shear Wall Design | Worst Case SW7 | EQYT SOIL

Check Shear Strength of Wall

- No Tension seen by walls
- continuing notion that wall sees axial load but designing with $N_u = 0$

$$V_c = \min \begin{cases} 3.3\lambda\sqrt{f'_c}\,hd + \dfrac{N_u d}{4 l_w} \nearrow 0 = 3.3 \cdot 1.0 \cdot \sqrt{5000} \cdot 12'' \cdot 17' \cdot 12'' = 571\,k \\ \left[0.6\lambda\sqrt{f'_c} + \dfrac{l_w\left(1.25\lambda\sqrt{f'_c} + 0.2\dfrac{N_u}{l_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}}\right] \cdot hd \end{cases}$$

$$\left[0.6 \cdot 1.0\sqrt{5000} + \frac{18' \cdot 12''\left(1.25 \cdot 1.0\sqrt{5000}\right)}{\left(\dfrac{6268\,k\text{ft} \cdot 12''}{493k} - \dfrac{18' \cdot 12''}{2}\right)}\right] \cdot 12'' \cdot 17' \cdot 12'' = 1140\,k$$

$\therefore V_c = 571\,k$ $\qquad \frac{1}{2}\phi V_c = \frac{1}{2} \cdot 0.75 \cdot 571\,K$

$\frac{1}{2}\phi V_c = 214\,K$

$493 > 214k \therefore$ need steel

$SDC = C \therefore$ No need to use $V_n = A_{cv}(\alpha_c \lambda\sqrt{f'_c} + \rho_t f_y)$ per ACI 318-11 21.1.1.5

No need to consider Chapter 21 of ACI 318-11 since walls are ordinary reinforced per 21.1.1.7(b)

Check if $\phi \frac{V_c}{2} < V_u \le \phi V_c = 428k$

$214k \le 493k < 428k$ ✗ $\therefore$ minimum ACI 11.9.9 reinforcement ~~may~~ close satisfy design

Horizontal Reinforcement

$\rho_t = 0.0025 = \dfrac{A_{s\,min}}{h\,s_2}$ $\qquad \frac{0.54}{18} = \frac{x}{12}$

$A_{s\,min} = 0.0025 \cdot 12'' \cdot 18'' = 0.54\,in^2/18''$ $= 0.36\,in^2/12''$

$$s_2 = \min \begin{cases} l_w/5 = \frac{18' \cdot 12}{5} = 43.2'' \\ 3h = 3 \cdot 12'' = 36'' \\ 18'' \rightarrow \text{controls} \end{cases}$$

use #4 @ 12" O.C. Each Face

$\rho_t = 0.002778 > 0.0025$ ✓

Vertical Reinforcement

$\rho_l = 0.0025 + 0.5\left(2.5 - \frac{12.67}{18}\right)(0.002778 - 0.0025)$

$\rho_l = 0.00275 = \dfrac{A_{s\,min}}{h \cdot s_1}$

$\hookrightarrow$ use #4 @ 12" O.C. Each Face

$$s_1 = \min \begin{cases} l_w/3 = \frac{18 \cdot 12}{3} = 72'' \\ 3h = 36'' \\ 18'' \rightarrow \text{controls} \end{cases}$$

$\rightarrow$ is $V_n \le 10\sqrt{f'_c}\,hd$

$V_c + V_s \le 1731\,K$

$860 \le 1731$ ✓

all spacing requirements met ✓

$V_s = \dfrac{2 \cdot 0.2\,in^2 \cdot 60\,ksi \cdot 18' \cdot 12''}{12''} = 432k$ $\quad \phi 432k + 428k = 752k$

$752k \ge 493k$ ✓

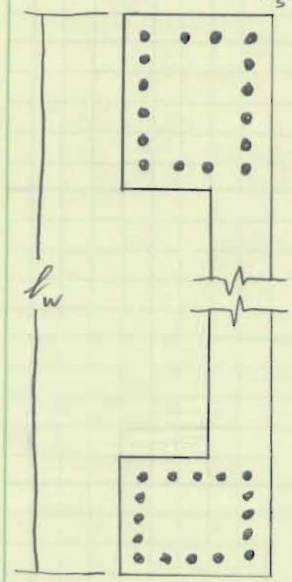
Shear Wall Design SW1

Basing Reinforcement increase off of Column C-5 Redesign

$A_{s\,redesign} = 129\%$ of original A_s

D 3.1 = 36×30
12#11

C 3.1 = 24×30
12#11



36"×30" Column

compression for controlling case

12 #11 bars · 129% = 21.83 in² → 16 #11 bars

$h_w = 91.33'$

$h_w/\ell_w = 2.34$

$\ell_w = 36.5' + 1.5' + 1' = 39'$

Max loads

$V2 = 842\,K$

$M3 = 43{,}470\ k\cdot ft$

24"×30" Column

Tension for controlling case

12#11 bars · 129% = 21.83 in² → 16 #11 bars

Detail Reinforcement to put axial load in walls

Check Moment Strength of Wall

$T = A_s \cdot f_y$

$T = 16 \cdot 1.41\,in^2 \cdot 60{,}000\,psi$

$T = 1353.6\,K$

$d = 38' \quad (\ell_w - 24''/2)$

$a = \dfrac{T + \cancel{N_u}^{\to 0}}{0.85 f'_c \cdot b}$

$a = \dfrac{1353.6 \cdot 1000\,lb}{0.85 \cdot 5000 \cdot 30''}$

$a = 10.62'' < 36''$ ✓ ∴ eq. is valid

$c = a/\beta_1 = 13.27''$

$\beta_1 = 0.8$

Is $13.27'' < 0.375d$?

$13.27'' < 171''$ ✓ ∴ $\phi = 0.9$

$M_n = T(d - a/2) + \cancel{N_u\left(\dfrac{\ell_w - a}{2}\right)}^{\to 0}$

$M_n = 1353.6\left(38' \cdot \dfrac{12''}{1ft} - \dfrac{10.62''}{2}\right)$

$M_n = 50{,}837\ k\cdot ft$

$\phi M_n = 0.9 \cdot 50837\ k\cdot ft$

$\phi M_n = 45{,}754\ k\cdot ft \geq 43{,}470\ k\cdot ft$ ✓

sufficient Moment Capacity

Shear Wall Design SW1 Cont.

Check Shear Strength of Wall

- No tension seen by wall
- continuing notion that wall sees axial load

check with $N_u = 0$ (cons.)

$$V_c = \min \begin{cases} 3.3\lambda\sqrt{f'_c}\,hd + \dfrac{N_u \cdot d}{4 l_w} \to 0 = 3.3 \cdot 1.0\sqrt{5000} \cdot 12'' \cdot 38' \cdot 12'' = 1276k \\ \left[0.6\lambda\sqrt{f'_c} + \dfrac{l_w\left(1.25\lambda\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right] \cdot hd \end{cases}$$

$$\left[0.6 \cdot 1.0\sqrt{5000} + \frac{38' \cdot 12''(1.25 \cdot 1.0\sqrt{5000})}{\frac{43479 \cdot 12''}{842} - \frac{38' \cdot 12''}{2}}\right] \cdot 12'' \cdot 38' \cdot 12'' = 733k$$

$\therefore V_c = 733k$ $\quad \tfrac{1}{2}\phi V_c = \tfrac{1}{2} \cdot 0.75 \cdot 733k$ $\quad \phi V_c = 550k$

$\tfrac{1}{2}\phi V_c = 275k$

$842k > 275k$ ∴ need steel

$842k > 550k$ ∴ need more than min. steel

$V_s \geq \left(\frac{V_u}{\phi} - V_c\right)$

$V_s \geq \left(\frac{842k}{0.75} - 733k\right)$

$V_s \geq 390k$ → Try 12" o.c. spacing

$V_s = \frac{A_v f_y d}{s}$

$390k = \frac{A_v \cdot 60ksi \cdot 38' \cdot 12''}{12''}$

$A_v = 0.1711\ in^2$

$A_{v_{min}} = 0.0025 \cdot 12'' \cdot 12'' = 0.36\ in^2$

using #4 @ 12" o.c. Each face

$V_s = \frac{2 \cdot 0.2 in^2 \cdot 60ksi \cdot 38' \cdot 12''}{12''}$

$V_s = 912k > 390k$ ✓

Horizontal Reinforcement

#4 @ 12" o.c. Each Face

Vertical Reinforcement

#4 @ 12" o.c. Each Face

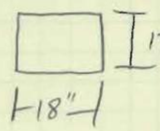
Shear Wall 4

Architectural interference at office levels with restrooms

Solution

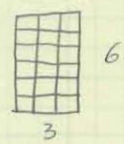
- slide entire restroom setup towards Grid C 2'-6"
- Knock holes in shear wall for entrances to restrooms
- ceiling height is 9'-0"

Mesh is



@ office levels

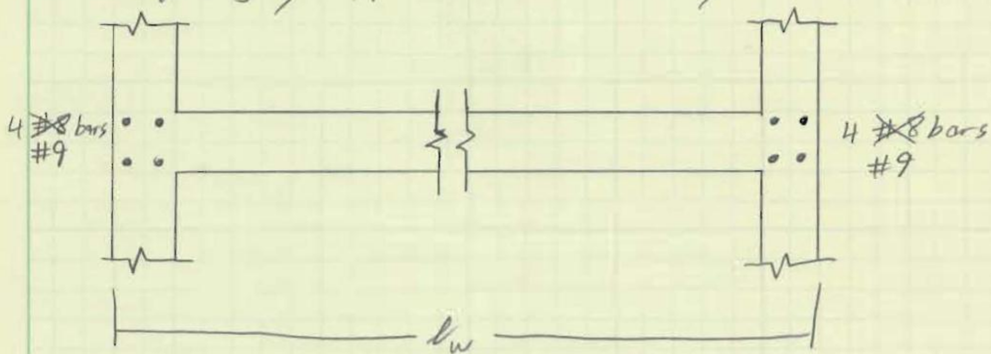
∴ remove



3×6 for each door starting to either side of 9 central meshes

Shear Wall Design | SW 4 | Bottom 4 floors

Using Cagley typ. Detail for Boundary element Reinforcing



$h_w = 91.33'$

$\ell_w = 29.5'$

Max loads

$V2 = 981k$

$M3 = 5157\,k\cdot ft$

Detail Reinforcement to put axial load in walls

Check Moment Strength of Wall

$T = A_s \cdot f_y$

$T = 4 \cdot 0.79 in^2 \cdot 60 ksi$

$T = 189.6\,k$

$d = 29'\ \left(\ell_w - \frac{12''}{2}\right)$

$M_n = T\left(d - a/2\right) + \cancel{N_u\left(\frac{\ell_w - a}{2}\right)} \to 0$

$M_n = 189.6k\left(29' \cdot 12''/1ft - \frac{3.72''}{2}\right)$

$M_n = 5469\,k\cdot ft$

$a = \frac{T + \cancel{N_u} \to 0}{0.85 f'_c \cdot b}$

$a = \frac{189600\,lb}{0.85 \cdot 5000 \cdot 12''}$

$a = 3.72'' < 12''$ ✓ ∴ eq is valid

$c = a/\beta_1 = 4.65''$

$\beta_1 = 0.8$

Is $4.65'' < 0.375d$?

$4.65'' < 130.5''$ ✓ ∴ $\phi = 0.9$

$\phi M_n = 4922\,k\cdot ft < 5155\,k\cdot ft$ ∴ N.G.

☆ Iteration! Try 4 #9 bars ☆

$T = 4 \cdot 1.0 in^2 \cdot 60 ksi$

$T = 240K$

$a = \frac{240 \cdot 1000\,lb}{0.85 \cdot 5000 \cdot 12}$

$a = 4.71'' < 12''$ ✓ ∴ eq is valid

$c = a/\beta_1 = 5.89''$ $\phi = 0.9$ still

$\phi M_n = 0.9 \cdot 240\left(29' \cdot 12''/1ft - \frac{4.71''}{2}\right)$

$\phi M_n = 6221\,k\cdot ft \geq 5157\,k\cdot ft$ ✓ sufficient moment capacity

Shear Wall Design | SW 4 Cont. | Bottom 4 Floors

Check Shear Strength of Wall

- No tension seen by wall
- Continuing Notion that wall sees axial load

$$V_c = \min \begin{cases} 3.3\lambda\sqrt{f'_c}\,hd + \dfrac{N_u \cdot d}{4 \cdot \ell_w}^{\nearrow 0} = 3.3 \cdot 1.0 \cdot \sqrt{5000} \cdot 12'' \cdot 29' \cdot 12'' = 974k \\ \left[0.6\lambda\sqrt{f'_c} + \dfrac{\ell_w\left(1.25\lambda\sqrt{f'_c} + 0.2\dfrac{N_u}{\ell_w \cdot h}\right)}{\dfrac{M_u}{V_u} - \dfrac{\ell_w}{2}}\right] \cdot hd \end{cases}$$

$$\left[0.6 \cdot 1.0\sqrt{5000} + \frac{29.5' \cdot 12''\left(1.25 \cdot 1.0\sqrt{5000}\right)}{\left(\dfrac{5157 \cdot 12}{981k} - \dfrac{29.5' \cdot 12''}{2}\right)}\right] \cdot 12'' \cdot 29' \cdot 12'' = -\#$$

$\therefore$ Doesn't control

$\therefore V_c = 974K$ $\quad \frac{1}{2}\phi V_c = \frac{1}{2} \cdot 0.75 \cdot 974k$ $\quad \phi V_c = 730K$

$\frac{1}{2}\phi V_c = 365K$

$981k > 365k$ $\therefore$ need steel

$V_s \geq \dfrac{V_u}{\phi} - V_c$

$V_s \geq \dfrac{981}{0.75} - 974$

$V_s \geq 334K$ $\rightarrow$ trying the minimum steel for the wall

$$V_s = \frac{2 \cdot 0.2 in^2 \cdot 60ksi \cdot 29.5' \cdot 12''}{12''}$$

$V_s = 708K \geq 334k$ ✓

$\therefore As_{min}$ provides enough additional strength

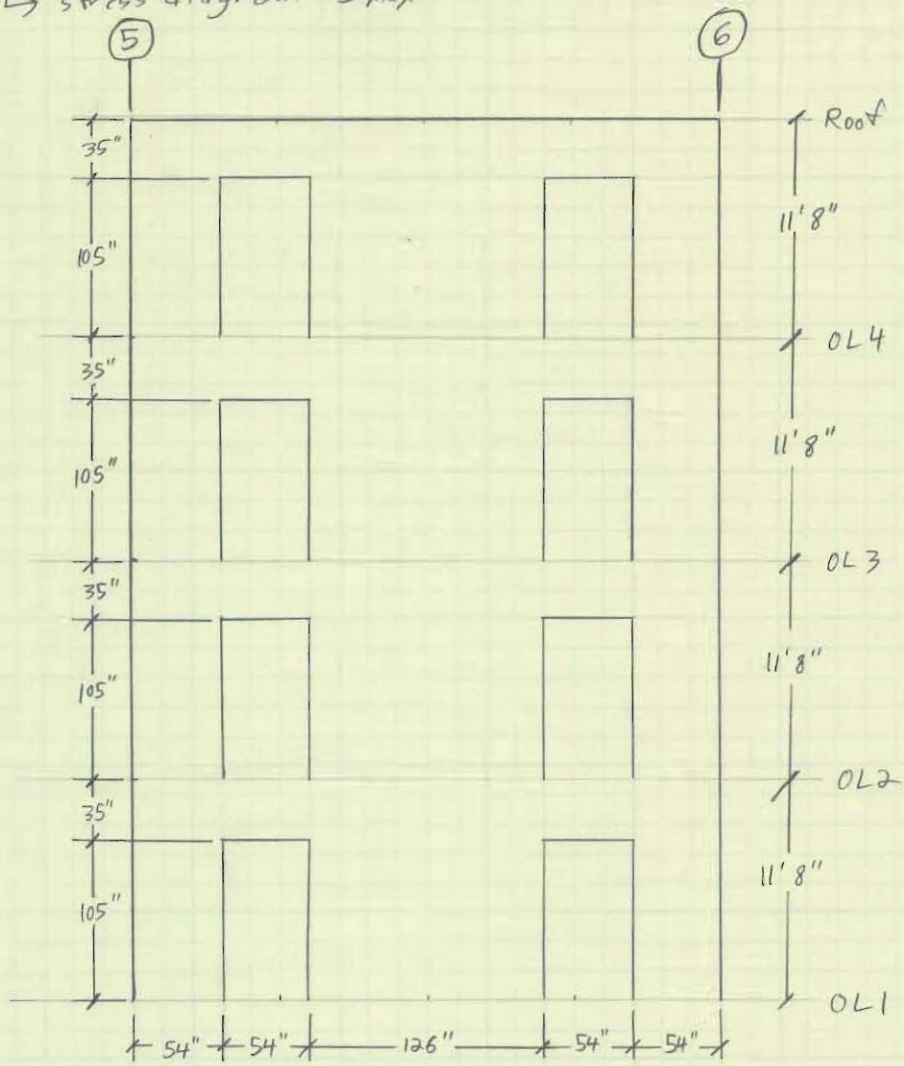
Horizontal Reinforcement	Vertical Reinforcement
#4 @ 12" O.C. Each Face	#4 @ 12" O.C. Each Face

Valid Design for lower half of wall

Shear Wall Design SW 4 Upper 4 Floors

$f_b = \frac{M}{S}$

↳ stress diagram "S_{max}"



$\frac{l_n}{h} = \frac{54''}{35''} = 1.543 \rightarrow$ ACI 318-11 21.9.7.2

↳ does V_u exceed $4\lambda\sqrt{f'_c}\,A_{cw}$?

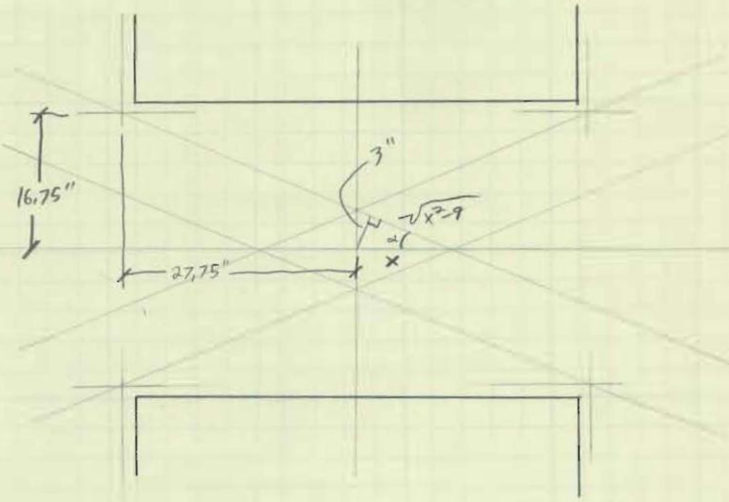
$V_u = 206k$ worst case

$4\lambda\sqrt{f'_c}\,A_{cw} = 4 \cdot 1.0\sqrt{5000} \cdot 12'' \cdot 35''$

$= 118.8k$

Yes → 2 intersecting groups of diagonal bars are req
at all levels except for roof

Shear Wall Design | SW 4 | Upper 4 floors



Develop all diagonals to $1.25\,\ell_d$ in tension 21.9.7.4(b)

$V_n = 2 A_{vd} f_y \sin\alpha$

$V_u \le \phi V_n$

$\underbrace{206K}_{max} \le \phi \cdot 2 \cdot A_{vd} \cdot f_y \sin\alpha$

$\tan\alpha = \dfrac{16.75}{27.75+x}$ $\quad \sin\alpha = \dfrac{3}{x}$ $\quad \cos\alpha = \dfrac{\sqrt{x^2-9}}{x}$

$\dfrac{3/\cancel{x}}{\sqrt{x^2-9}/\cancel{x}}$

$\alpha = 25.8^\circ$

$A_{vd} \ge \dfrac{206000\text{ lb}}{0.75 \cdot 2 \cdot 60{,}000\text{psi} \cdot \sin 25.8^\circ}$

$\dfrac{3}{\sqrt{x^2-9}} = \dfrac{16.75}{27.75+x}$

$A_{vd} \ge = 5.26\text{ in}^2$

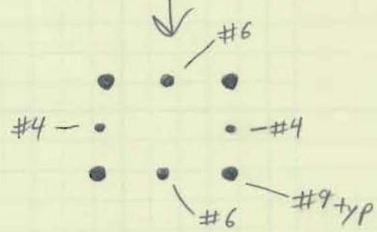
$3(27.75+x) = 16.75\sqrt{x^2-9}$

$9(770.06+55.5x+x^2) = 280.56(x^2-9)$

$6930.54+499.5x+9x^2 = 280.56x^2 - 2525$

$0 = 271.56x^2 - 499.5x - 9455.54 = 0$

$x = 6.892''$



$A_s = 4 \cdot 1.0\text{in}^2 + 2 \cdot 0.2\text{in}^2 + 2 \cdot 0.44\text{in}^2$

$A = 5.28\text{in}^2$

$V_n = 2 \cdot A_{vd} f_y \sin\alpha \le 10\sqrt{f'_c}\, A_{cw}$

$2 \cdot 5.28\text{in}^2 \cdot 60{,}000\text{psi} \cdot \sin 25.8^\circ \le 10\sqrt{5000} \cdot 35'' \cdot 12''$

$275{,}762\text{ lb} \le 296{,}985\text{ lb}$ ✓

Shear Wall Design SW4 Upper 4 floors

Transverse Reinforcement in Coupling Beam (21.9.7.4(d) → states crossties ≤ 8" o.c.)

21.6.4.2 crossties ≤ 14" o.c. alternate hooks

21.6.4.4
$$A_{sh} = 0.3\frac{s b_c f'_c}{f_{yt}}\left[\left(\frac{A_g}{A_{ch}}\right)-1\right]$$
$$A_{sh} = 0.09\frac{s b_c f'_c}{f_{yt}}$$

21.6.4.7 → keep clear cover on transverse reinf. ≤ 4"

Vertical bars (ties + hoops)

$$\frac{A_{sh}}{s} = 0.3\cdot\frac{(12''-2\cdot0.75'')\cdot5000psi}{60{,}000psi}\left[\frac{35''\cdot12''}{(35-2\cdot.75)(12-2\cdot0.75)}-1\right)$$

$$\frac{A_{sh}}{s} = 0.050933$$

$$\frac{A_{sh}}{s} = 0.09\cdot\frac{(12''-2\cdot0.75'')\cdot5000psi}{60000\,psi} = 0.07875 \rightarrow \text{controls}$$

say spacing = 3" (#4 bars) $A_{sh} = 0.23625$ minimum $3\#4 = 0.6\,in^2$

horizontal bars (ties + hoops)

$$\frac{A_{sh}}{s} = 0.3\,\frac{(35''-2\cdot0.75'')\cdot5000psi}{60{,}000psi}\left(\frac{35''\cdot12''}{(35-1.5)(12-1.5)}-1\right)$$

$$\frac{A_{sh}}{s} = 0.1625$$

$$\frac{A_{sh}}{s} = 0.09\cdot\frac{(35''-2\cdot0.75'')\,5000psi}{60000\,psi} = 0.25125 \rightarrow \text{controls}$$

say spacing ≈ 6" $A_{sh} = 1.51\,in^2_{\text{minimum}} \rightarrow 4\#5 + 2\#4 = 1.64\,in^2$ (2#4 → from hoop)

Longitudinal bars

$0.01A_g < A_s < 0.06A_g$

$4.2\,in^2 < A_s < 25.2\,in^2$

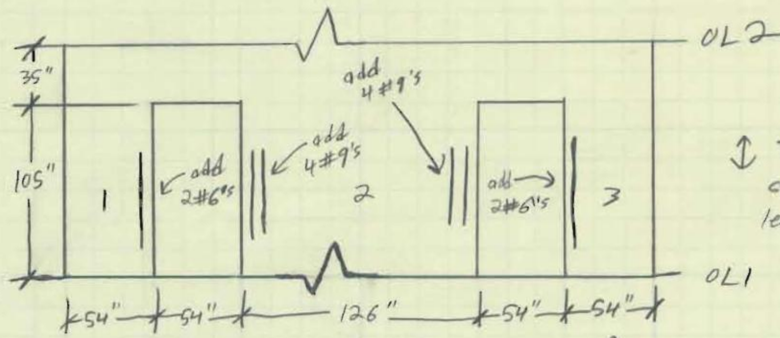
need 14 bars for spacing reqs.

→ 14 #6's $A_s = 6.16\,in^2$ ✓

Shear Wall Design | SW 4 | Upper 4 floors

Design wall "piers" between openings

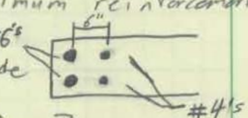
Design between OL1 and OL2



Trans. reinf. shall extend at least 12" above and below pier

21.9.8.1 $\ell_w/b_w = \frac{54}{12} = 4.5$ $\ell_w/b_w = 10.5$ $\ell_w/b_w = \frac{54}{12} = 4.5$

ACI 318-11 14.3.7
at least 2 #5's need in addition to minimum reinforcement shall be provided around openings ∴ provide



Pier 1
$V_u = 120k$
$M_u = 892 k\cdot in = 74.3 k\cdot ft$

Pier 2
$V_u = 301k$
$M_u = 1291 k\cdot ft$

Pier 3
$V_u = 116.5k$
$M_u = 293 k\cdot ft$

For Pier 1 and 3

Check Moment Strength

$T = A_s \cdot f_y = (2\cdot 0.2 + 2\cdot 0.44)\, 60ksi$

$T = 76.8k$ $d = 54"$

$\phi M_n = 0.9 \cdot 76.8 (54" - \frac{1.51}{2})$

$\phi M_n = 306\ k\cdot ft \geq 74.3 k\cdot ft$ ✓
$\geq 293\ k\cdot ft$ ✓

$a = \frac{T}{0.85 f'_c \cdot b} = \frac{76{,}800\ lb}{0.85 \cdot 5000 \cdot 12"} = 1.51"$

$c = a/\beta_1 = \frac{1.51}{0.8} = 1.89"$

Is $1.89" \leq 0.375 d$?
$\leq 0.375 \cdot 54" = 20.25"$ ✓ ∴ $\phi = 0.9$

For Pier 2

check Moment Strength Try 4 #9's

$T = A_s \cdot f_y = 4 in^2 \cdot 60 ksi$

$T = 240k$ $d = 126" - 5" = 121"$

$\phi M_n = 0.9 \cdot 240k (121" - \frac{4.71"}{2})$

$\phi M_n = 2135\ k\cdot ft \geq 1291\ k\cdot ft$

$a = \frac{T}{0.85 f'_c \cdot b} = \frac{240000}{0.85 \cdot 5000 \cdot 12} = 4.71"$

$c = a/\beta_1 = \frac{4.71}{0.8} = 5.88"$

$5.88" \leq 0.375 \cdot 121"$
$\leq 45.375"$ ✓ ∴ $\phi = 0.9$

used #9's since they were already needed @ ends of walls

Shear Wall Design SW 4 upper 4 floors

For Pier 1/3 $L_w = 60''$

$\left(\frac{M_u}{V_u}\right)_3 = \frac{293 \cdot 12}{116.5} = 30.2''$ → will give very large V_c by insp.

$\left(\frac{M_u}{V_u}\right)_1 = \frac{74.3 \cdot 12}{120} = 7.43''$ → will give negative V_c by insp.

$\therefore V_c = 3.3\lambda\sqrt{f'_c}\, hd + \frac{N_u \cdot d}{4 l_w} \rightarrow 0$ will control here

$= 3.3 \cdot 1.0\sqrt{5000} \cdot 12'' \cdot 54'' = 151.2k$

$\frac{1}{2}\phi V_c = \frac{1}{2} \cdot 0.75 \cdot 151.2k$

$= 56.7k \geq 120k$ ✗ ∴ reinforcement is required

check that min reinf. of #4 @ 12" o.c. each face will satisfy

$V_s \geq \frac{V_u}{\phi} - V_c$

$\geq \frac{120k}{0.75} - 151.2$

$V_s \geq 8800 lb$

$V_s = \frac{2 \cdot 0.2in^2 \cdot 60ksi \cdot 54''}{12''}$

$V_s = 108{,}000\ lb \geq 8800\ lb$

max spacing = min $\begin{cases} l_w/5 = \frac{60''}{5} = 12'' \rightarrow \text{controls} \\ 3h = 3 \cdot 12'' = 36'' \\ 18'' = 18'' \end{cases}$

∴ use #4 @ 12" o.c. each face for horizontal and vertical reinforcement

For Pier 2 $L_w = 126''$

$\left(\frac{M_u}{V_u}\right)_2 = \frac{1291 \cdot 12}{301} = 51.5''$ → will give negative V_c by inspection

$\therefore V_c = 3.3\lambda\sqrt{f'_c}\, hd + \frac{N_u \cdot d}{4 l_w} \rightarrow 0$ will control here

$= 3.3 \cdot 1.0\sqrt{5000} \cdot 12'' \cdot 121'' = 339k$

$\frac{1}{2}\phi V_c = \frac{1}{2} \cdot 0.75 \cdot 338k$

$= 127k \geq 301k$ ✗ ∴ reinforcement is req.

check that min. reinf. of #4 @ 12" o.c. each face will satisfy

$V_s \geq \frac{V_u}{\phi} - V_c$

$\geq \frac{301}{0.75} - 338$

$\geq 62.3k$

$V_s = \frac{2 \cdot 0.2in^2 \cdot 60ksi \cdot 121''}{12''}$

$V_s = 242k \geq 62.3k$

max spacing = min $\begin{cases} l_w/5 = \frac{121}{5} = 24'' \\ 3h = 36'' \\ 18'' \rightarrow \text{controls} \end{cases}$

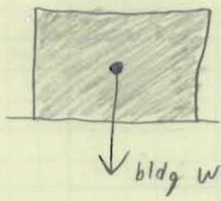
∴ continue using #4 @ 12" o.c. each face for horizontal and vertical reinforcement.

APPENDIX F: Foundation Check

Foundation Check

Check Overturning Moment

Worst Case Overturning Moment



bldg wt = 39,017k

Soil: 31,593 k·ft + Earthquake: 65,557 k·ft

= 97,150 k·ft

$M_{resisting} = \frac{2}{3}$ bldg wt. $\cdot \frac{\text{least dim}}{2}$

$= \frac{2}{3} \cdot 39{,}017k \cdot \frac{125'}{2} = 1{,}625{,}708\ k\cdot ft$

$F.S. = \frac{M_{resist}}{M_{over}} = \frac{1{,}625{,}708}{97{,}150} = 16.734 \therefore OK$

Check Laterally induced loads in columns (Axial)

Max Tension in Col. line A = −332 k A−2 EQ YT Soil
DL = 537.2k 0.9DL + 1.0E = 151 k (+) ∴ ok

Max Comp. in Col. line B = 332 k B−2 EQ YT Soil
S = 14.74k DL = 842.3k 1.2DL + 1.0E + 0.2S = 1346k → check cap.

Max Tension in Col. line C = −857k C−3,1 EQ YT Soil
DL = 977k 0.9DL + 1.0E = 22k (+) ∴ OK

Max Comp. in Col. line D = 1006 K D−3,1 EQ YT Soil
S = 17.04k DL = 977k 1.2 DL + 1.0E + 0.2S = 2182k → check cap.

Columns Considered in Design

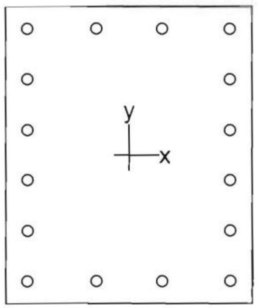
A−5 −242 k DL = 541.6 k → Designed for 1134k
B−5 230 k DL = 850.16k → Designed for 1808k
C−5 −413 k DL = 1251k → Designed for 2651k

→ M = 2150 k·in = 180k·ft

Design is more than adequate

See SP Column Design on next page

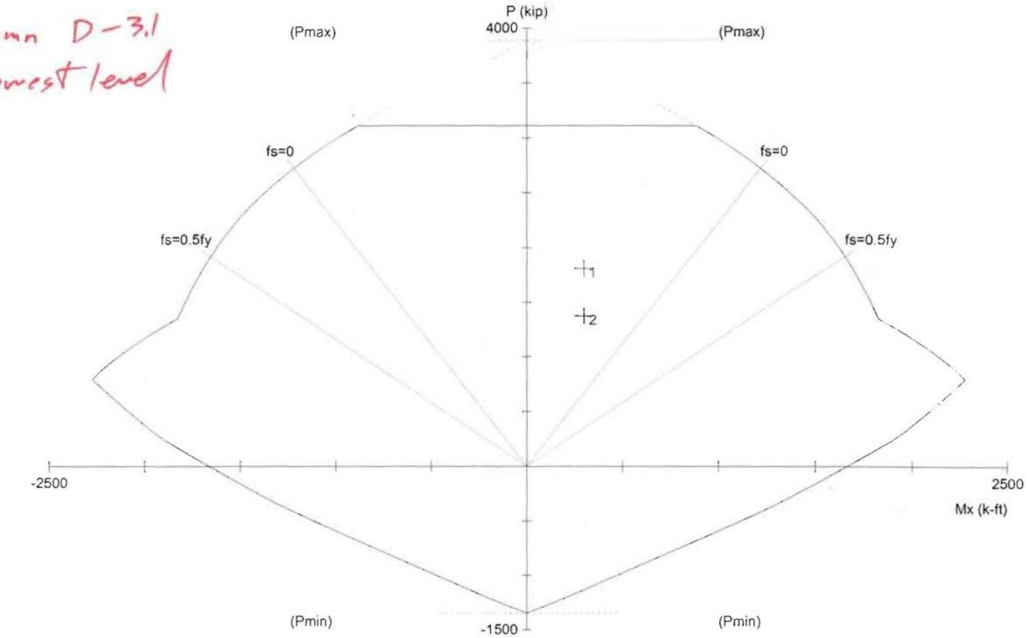
appears overdesigned, however amount of rebar is needed for moment capacity of shear wall



30 x 36 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 03/29/13
Time: 14:28:22

Column D-3.1
@ lowest level



spColumn v4.81. Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68

File: X:\Senior Thesis Kingstowne 36A\Spring Semester Models\D-3.1 36 x 30 Design @ P1.col

Project:

Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 1080 in^2	16 #11 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 24.96 in^2	rho = 2.31%
fc = 4.25 ksi		Xo = 0.00 in	Ix = 116640 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 81000 in^4
Beta1 = 0.8		Min clear spacing = 4.71 in	Clear cover = 2.00 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Foundation Check

Check capacity of existing footing size and Geopier Capacity

Existing Square Ftg. @ C-1.5 or D-1.5
is 11'-0" × 11'-0" × 36" deep

ASD Load Combo on Ftg

$D + 0.75L + 0.75S = 1165 \text{ kips}$

Per Geotech Report

Allowable bearing cap. of soil with Geopiers = 7000 psf

$$7000 \text{ psf} = \frac{1{,}165{,}000 \text{ psf}}{A_{req}}$$

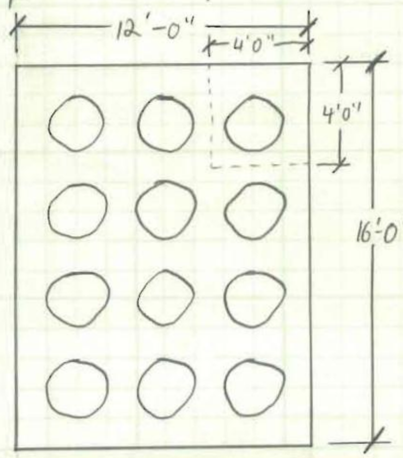
$A_{req} = 166.4 \text{ ft}^2$

Assuming square footing
need 13' × 13' Footing

Per Geotech Report

a 16' deep 30" DIA. can be expected to provide a capacity of 100 kips

$\therefore \frac{1165 \text{ kips}}{100 \text{ kips}} \approx 12$ piers needed



A footing this size provides 192 ft² of bearing area and 18" between each "Geopier"

APPENDIX G: Progressive Collapse Calculations

Progressive Collapse | Tie force Method

2-2.4.1 Provide adequate internal, peripheral, and vertical tie force cap.

3-1.1 LRFD Design of Tie forces

use overstrength factor of $\underset{(\Omega)}{1.25}$ in Table 6-4 of ASCE41

$\phi R_n \geq \sum \gamma_i Q_i$ $\quad \phi = 0.75$ (Tension Controlled) 4-3 in UFC

$\sum \gamma_i Q_i$ → Required Tie Strength

ϕR_n → Design Tie Strength $= \phi \Omega A_s \cdot f_y = 0.75 \cdot 1.25 \cdot A_{s_{min}} \cdot 60\text{ksi}$

$W_F = 1.2D + 0.5L$

$\phi R_n = 56.25 A_{s_{min}}$

Loads (Internal Ties)

Parking

$D = 143\text{psf}$ $\quad (100 + 21 + 17 + 5)$ (slab, drops, framing, SIDL)

$L = 40\text{psf}$

$W_F = 192\text{psf}$

Office

$D = 153\text{psf}$ $\quad (100 + 21 + 17 + 15)$ (slab, drops, framing, SIDL)

$L = 80\text{psf}$

$W_F = 224\text{psf}$

Roof

$D = 155\text{psf}$ $\quad (100 + 21 + 17/2 + 25)$ (slab, drops, framing, SIDL)

$L = 23\text{psf}$ (using Snow)

$W_F = 198\text{psf}$

Internal Ties Required Tie Strength $= F_i = 3 w_F L_1$

Parking N-S

$F_i = 3 \cdot 0.192\text{ksf} \cdot 36.5'$

$F_i = 21.024\text{ k/ft}$

$A_{s_{min}} = \frac{21.024\text{ k/ft}}{56.25} = 0.374\text{ in}^2/\text{ft}$ → #6 @ 12" o.c. $(A_s = 0.44\text{ in}^2/\text{ft})$

Parking E-W

$F_i = 3 \cdot 0.192\text{ksf} \cdot 29' = 16.7\text{ k/ft}$

$A_{s_{min}} = \frac{16.7\text{ k/ft}}{56.25} = 0.297\text{ in}^2/\text{ft}$

#6 @ 15" o.c.

$A_s = 0.352\text{ in}^2/\text{ft}$

Office N-S

$F_i = 3 \cdot 0.224\text{psf} \cdot 36.5' = 24.53\text{ k/ft}$

$A_{s_{min}} = \frac{24.53\text{ k/ft}}{56.25} = 0.436\text{ in}^2/\text{ft}$ → #6 @ 12" o.c.

Office E-W

$F_i = 3 \cdot 0.224\text{psf} \cdot 29' = 19.5\text{ k} \cdot \text{ft}$

$A_{s_{min}} = \frac{19.5}{56.25} = 0.347\text{ in}^2/\text{ft}$

#6 @ 15" o.c.

Roof N-S

$F_i = 3 \cdot 0.198\text{psf} \cdot 36.5' = 21.7\text{ k/ft}$

$A_{s_{min}} = \frac{21.7}{56.25} = 0.386\text{ in}^2/\text{ft}$ → #6 @ 12" o.c.

Roof E-W

$F_i = 3 \cdot 0.198\text{psf} \cdot 29' = 17.23\text{ k/ft}$

$A_{s_{min}} = \frac{17.23}{56.25} = 0.306\text{ in}^2/\text{ft}$

#6 @ 15" o.c.

Progressive Collapse | Tie force Method

Loads (Peripheral Ties) Must be located within 3' of perimeter beams

Parking $W_F = 192 psf + 1.2(100 psf \cdot \frac{10.67'}{3'}) = 619 psf$

Office $W_F = 224 psf + 1.2(100 psf \cdot \frac{11.67'}{3'}) = 691 psf$

Roof $W_F = 148 psf + 1.2(100 psf \cdot \frac{11.67'}{2 \cdot 3''}) = 431.4 psf$

Peripheral Ties Required Tie Strength $= F_i = 6 w_F \cdot L_1 \cdot L_p$

Parking/Office N-S

$F_i = 6 \cdot 691 \cdot 36.5' \cdot 3'$

$F_i = 454 k$

$A_{s min} = \frac{454k}{56.25} = 8.07 in^2$

Use 7 #10 bars ($A_s = 8.9 in^2$)

Parking/Office E-W

$F_i = 6 \cdot 691 \cdot 29' \cdot 3'$

$F_i = 361 k$

$A_{s min} = \frac{361k}{56.25} = 6.42 in^2$

Use 7 #9 bars ($A_s = 7.0 in^2$)

Roof N-S

$F_i = 6 \cdot 431.4 \cdot 36.5 \cdot 3'$

$F_i = 284 k$

$A_{s min} = \frac{284k}{56.25} = 5.05 in^2$

use 4 #10 bars ($A_s = 5.08 in^2$)

Roof E-W

$F_i = 6 \cdot 431.4 \cdot 29' \cdot 3'$

$F_i = 226 k$

$A_{s min} = \frac{226k}{56.25} = 4.02 in^2$

use 5 #9 bars ($A_s = 5.0 in^2$)

Parking level mechanical trace corners (N-S and E-W)

$F_i = 6 \cdot 192 \cdot 16' \cdot 3'$

$F_i = 56 k$

$A_{s min} = \frac{56k}{67.5} = 0.83 in^2$

use 3 #5 bars ($A_s = 0.93 in^2$)

Elevator and stair openings (all levels)

$F_i = 6 \cdot 224 \cdot 29' \cdot 3'$ → longest length the opening

$F_i = 117 k$

$A_{s min} = \frac{117k}{67.5} = 1.73 in^2$ → use 2 #9 bars ($A_s = 2.0 in^2$)

Vertical Column Ties (for all columns)

- use largest trib. area column (C4, C5)

$A_{trib} = 942 ft^2$

$F_i = A_{trib} \cdot W_F$

$= 942 ft^2 \cdot 224 psf$

$F_i = 211 k$

$A_{s min} = \frac{211k}{56.25} = 3.75 in^2$

→ 4 #8's min, → all columns satisfy this minimum

Progressive Collapse | Alternate Path Loads

Increase Gravity Load

– apply at all bays adjacent to and above removed column

Base Gravity Load = (0.9 or 1.2) D + (0.5L or 0.2S)

Parking

$D = 143\,psf$, $L = 40\,psf$ > $G = 192\,psf + 87.5\,psf = 280\,psf$

Office

$D = 153\,psf$, $L = 80\,psf$ > $G = 224\,psf + 87.5\,psf = 312\,psf$

Roof

$D = 155\,psf$, $L = 20\,psf$, $S = 23\,psf$ > $G = 196\,psf + \frac{87.5\,psf}{2} = 240\,psf$

$0.5L = 10\,psf$ $\quad 0.2S = 4.6\,psf$

Facade contribution over 16' interior width

$$D_{Facade} = 1.2 \cdot 100\,psf \cdot 11.67'/16' = 87.5\,psf$$

$$\text{Increased Gravity load} = \Omega_N\left[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)\right]$$

$$\Omega_N = 1.04 + \frac{0.45}{\left(\frac{\theta_{pra}}{\theta_y} + 0.48\right)}$$

$\theta_{pra} = 0.025$ (Table 4-1 UFC 4-023-03)

$$\theta_y = \frac{Z f_{ye} \cdot l_b}{6 \cdot EI_b}$$

where $EI_b = 0.5\,E_c I_g$ $\quad E_c = 57000\sqrt{f'_c} = 4030.5\,ksi$

$= 0.5 \cdot 4030.5\,ksi \cdot 19392\,in^4$

$= 39079728$

$$z = \frac{bh^2}{4} = \frac{24'' \cdot 20^2}{4}$$

$Z = 2400\,in^3$

$$\theta_y = \frac{2400\,in^3 \cdot 60\,ksi \cdot 36.5' \cdot 12''}{6 \cdot 39079728\,k\cdot in^2}$$

$\theta_y = 0.269$

$$\Omega_N = 1.04 + \frac{0.45}{\left(\frac{0.025}{0.069}\right) + 0.48}$$

$\Omega_N = 1.83$ → increase gravity loads by 83% in bays affected by removed column

Progressive Collapse   Alternate Path

Lateral Loads

$L_{lat} = 0.002\,\Sigma P$

★ will need 3/4" max aggregate to satisfy spacing

ΣP = sum of gravity loads @ particular level → use office levels since heaviest

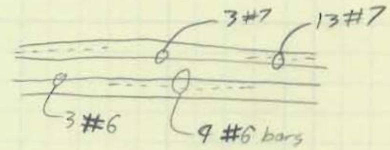
$= (153\,psf + 80\,psf) \cdot 25{,}299\,psf$

$+ \underbrace{100\,psf \cdot 11.67' \cdot (2 \cdot 200' + 2 \cdot 125')}$ → facade contribution to gravity loads

$\Sigma P = 6650\,kips$

$L_{Lat} = 0.002 \cdot 6650\,k$ $L_{Lat} = 13.3\,k$ / each level

$\rho = 9$ #6 bars
$\rho' = 13$ #6 bars > reversed at $-M$



$\dfrac{\rho - \rho'}{\rho_{bal}} = \dfrac{13 \cdot 0.6/24 \cdot 20 - 3 \cdot 0.44/24 \cdot 20}{0.034} = 0.4$ → controls limit

$\rho_{bal} = 0.85\beta_1 \dfrac{f'_c}{f_y} \cdot \dfrac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}$

$= 0.85 \cdot 0.8 \cdot \dfrac{5000}{60000} \cdot \dfrac{0.003}{0.003 + 0.002}$

$= 0.034$

$\dfrac{V}{b_w \cdot d\sqrt{f'_c}} = \dfrac{5.46 \cdot 36/2 \cdot 1000}{24" \cdot 17.5"\sqrt{5000}} = 3.3$

say transverse reinf. is NC (non-conforming)

Life safety rotation angle limit = 0.03 radians
→ limit used for plastic hinges in SAP2000

Results

Removal of columns along long side of building (A−5) 24"x 28" Beams
— Top Steel → 10 in² → 8#10's (10.16 in²) ✓
Bottom Steel → 7 in² → 7#9's (7.00 in²) ✓ > L.S. ≥ θ ≥ I.O.

Removal of columns along short side of building (D−1) 24"x 31" Beams
— Top steel ≈ 11 in² → Try 7#11's (10.92 in²) ✓
Bottom steel ≈ 8 in² → 8 #9's (8.00 in²) ✓ > L.S. ≥ θ ≥ I.O.

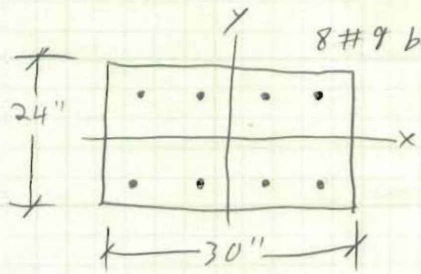
Removal of columns at corner of building (A−1)
— use design of beams on long side of building (Hinges will not develop)

Progressive Collapse | Enhanced Local Resistance

Per UFC, applies to all perimeter columns for the first 2 stories above grade for OC IV building

Flexural Resistance of current column

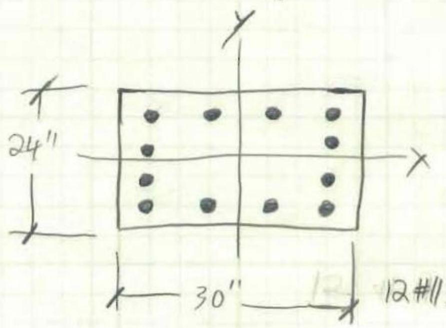
8 #9 bars



$M_{nx} = 370\ k\cdot ft$

$M_{ny} = 465\ k\cdot ft$

New Design



12 #11 bars

$M_{nx} = 807\ k\cdot ft$

$M_{ny} = 1042\ k\cdot ft$

$807 > 2\cdot 370$ ✓

$1042 > 2\cdot 465$ ✓

Shear Demand on Beams

$V_u = 155\ k$ (from SAP Model)

$V_c = 2\cdot 1.0\sqrt{5000}\cdot 24\cdot 28.5''$

$V_c = 96.7k \quad \phi V_c = 72.5k$

$V_{s\,req} = \frac{155k}{\phi} - 72.5k \quad V_{s\,req} = 134.2k \le 4\sqrt{f'_c}\cdot b_w\cdot d$

$\le 193k$ ✓

$s_{max} = d/2 = 14''$

$s = A_v\cdot f_{yt}\cdot \frac{d}{V_s}$

$= 0.2\cdot 2\cdot 60ksi\cdot \frac{28.5''}{134.2}$

$s = 5.09''$

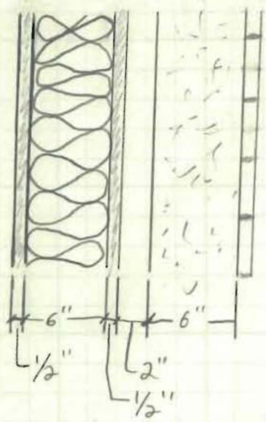
∴ use #4 @5" o.c.
Throughout beams N-S and beams E-W

APPENDIX H: Façade Design

Façade Design

All levels have precast concrete panels that are 6" thick with ½" thick brick veneer

2" air space then 6" metal stud wall with R-19 Batt Insulation



Panel Spans Bay width and is 40" Tall at office levels

56" Tall @ parking levels

Glazing Section Below at Parking Levels

≈ 5' wide panes

6'-0" Typical Span

Mullions and Transoms

- Kawneer Trifab VG 451T
- Sub-sill 2" sight line
- 4½" front Glaze
- Heavy Wt. Frame (463-029 Screw Spline)

→ ¼" Clear HS (VS-20 #2)

→ ¼" VS1-20 Monolithic HS

Viracon Vision Glass

½" Total Thickness

Glazing Section Below at Office Levels

6'-6" Typical span of clear 10'-0" span total

≈ 5' wide panes

1" Total thickness

→ ¼" Clear HS (VRE-46 #2)

→ ½" air space

→ ¼" Clear HS (VRE-46 #2) for vision glass

→ ¼" Clear HS (V948 Medium Gray Viraspan #4)

Viracon 1" VRE1-46 Insulating HS/HS Spandrel

Facade Design | ASTM E1300-12a

Loads

Wind Loads, Blast Loads
↳ not calculated, known blast would control

Blast Loading

$LR = 180\ psf$ 80 lb TNT (Figure 1 in ASTM F2248)
35' standoff
F2248-12

assuming no load sharing (conservative)

Now into ASTM E1300-12a

GTF — Table 2 (3s short duration load)

GTF → 1.8 for both lites @ office
2.0 for both lites @ parking

$LR = GTL \cdot NFL$
↳ non-factored load

$$NFL = \frac{LR}{GTL}$$

$NFL = 100\ psf$ @ office
$NFL = 90\ psf$ @ parking

$NFL = 4.785\ kPa$ @ office
$NFL = 4.31\ kPa$ @ parking

Figure A1.34 (Aspect Ratio of 1.3)
For 5'×6'6" Pane, Capacity of 5.8 kPa for 3/4" thick glass PVB Laminated inner lite, 4 sides simply supported

Figure A1.10 capacity of 5.8 kPa for 5/8" thick monolithic outer lite, 4 sides simply supported

Facade Design

Linear Interp. for determining capacity

for 5/8" Glass (Lam)

$$NFL_{cap} = \left(\frac{4-5}{1.5-1}\right)\left[\left(\frac{78}{60}\right)-1\right]+5 = 4.4\ kPa < 4.785\ kPa$$

∴ try next size

for 3/4" Glass Lam

$$NFL_{cap} = \left(\frac{5-7}{1.5-1}\right)\cdot\left[\left(\frac{78}{60}\right)-1\right]+7 = 5.8\ kPa > 4.785\ kPa\ ✓$$

Summary (Using all Heat Strengthened Glass)

Parking Levels

use 5/8" thick PVB Laminate Glass

Office Levels

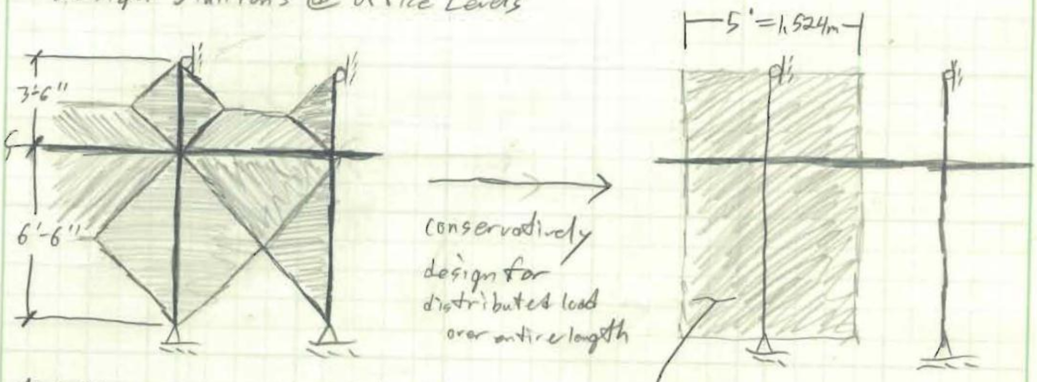
use 3/4" thick PVB Laminate for inner lite

use 5/8" thick monolithic Glass for outer lite

Mullion Design Must have twice the cap. of selected glass panel

T6 - Aluminum

Design Mullions @ Office Levels



* Mullions are secured to transoms which are continuously supported by the precast concrete panels

$\omega = 5.8\ kPa \cdot 1.524m$

$\omega = 8.84\ \frac{kN}{m}$

Facade Design

$$M_{max} = \frac{w\ell^2}{8} = 8.84\frac{kN}{m} \cdot \frac{(10' \cdot 0.3048)^2}{8}$$

$M_{max} = 10.3\ kN \cdot m$

For T6 aluminum

$\sigma_{bending} = 96 N/mm^2$

$= 96{,}000\ kN/m^2$

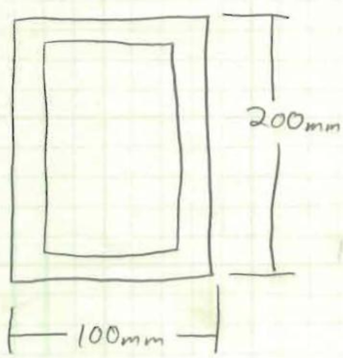
$$\sigma_{bend} = \frac{M_{max}}{S_{req}}$$

$$S_{req} = \frac{10.3\ kN \cdot m}{96{,}000}$$

$S_{req} = 0.0001073\ m^3$

$S_{req} = 107{,}292\ mm^3 = 6.55\ in^3$

Assume Hollow Rectangular Section



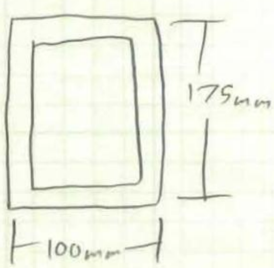
solve for required thickness

$$107292 \leq \frac{\frac{1}{12} \cdot 100 \cdot 200^3 - \frac{1}{12}(100-2t)(200-2t)^3}{100}$$

$671248600 \geq (100-2t)(200-2t)^3$

Using $T=5$

$S = 152{,}241$



with a 6 mm wall also works

$\boxed{\text{U.S. units } 4'' \times 7'' \times 0.25'' \text{ thick} \quad S = 9.78\ in^3 > 6.55\ in^3}$

Facade Design
Performance Analysis (Office Level Glazing Only)

~~APF~~

Freihaut Spreadsheet

June 21
air Temps
Low = 69.3
High = 89.2

December 21
air Temps
Low = 46.3
High = 63.2

Latitude = 38.87°
Longitude = 77.03°

Window Area
5'0" wide
6'6" high

Existing Design
U Value winter = 0.30
U Value summer = 0.27
SHGC = 0.28

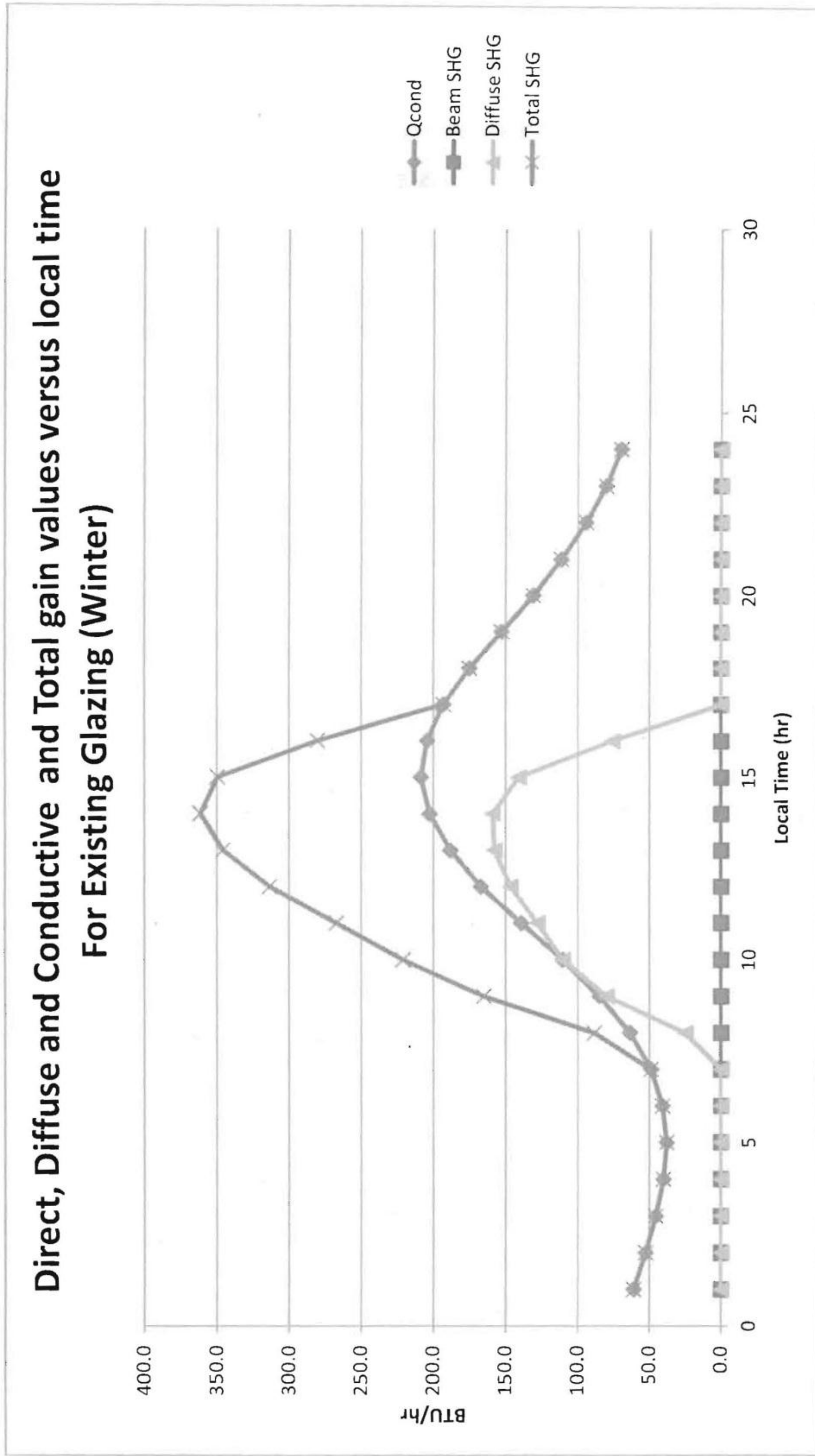
Winter Q_{Total} = 361.5 $\frac{BTU}{hr}$

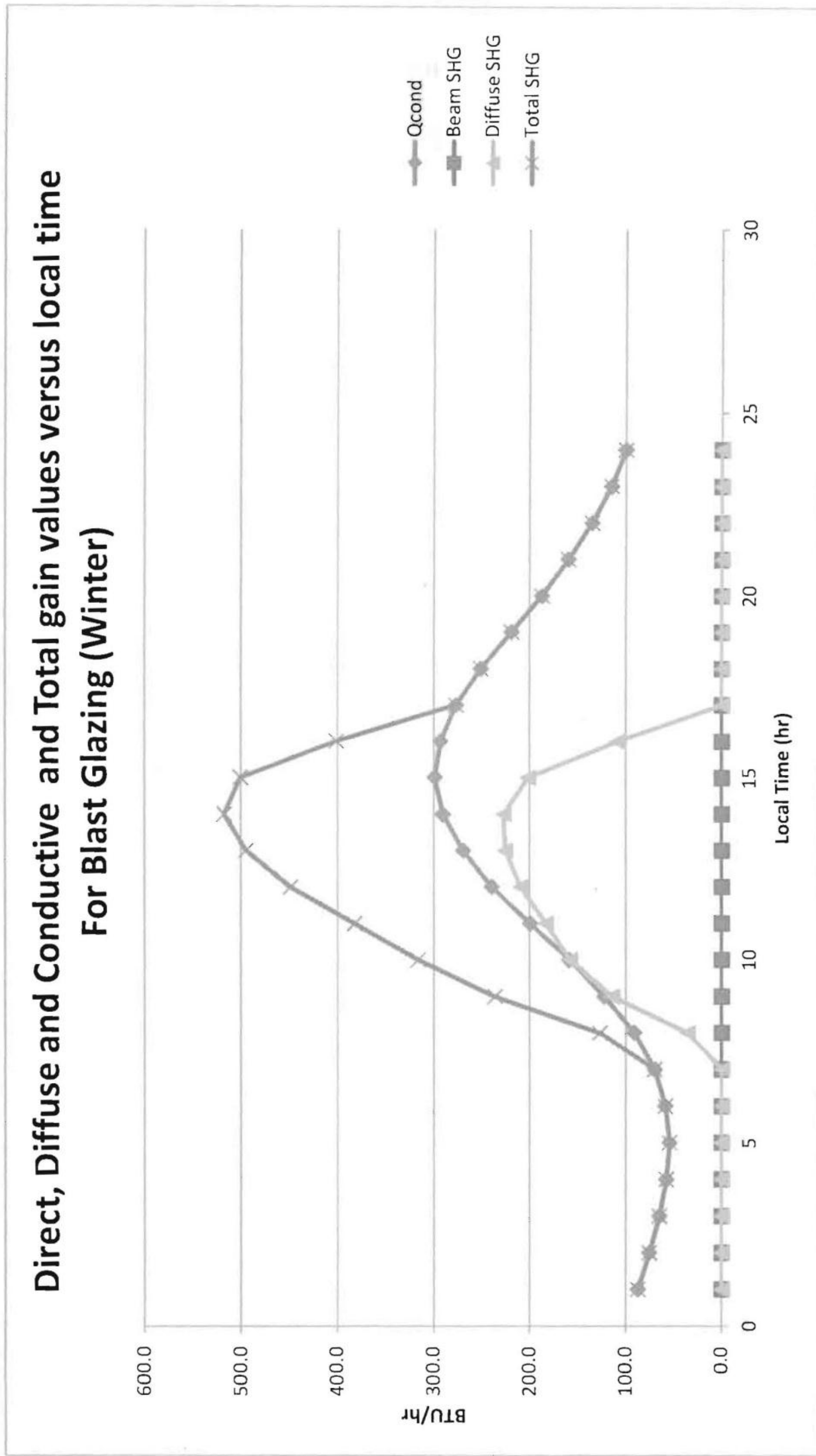
Summer Q_{Total} = 465 $\frac{BTU}{hr}$

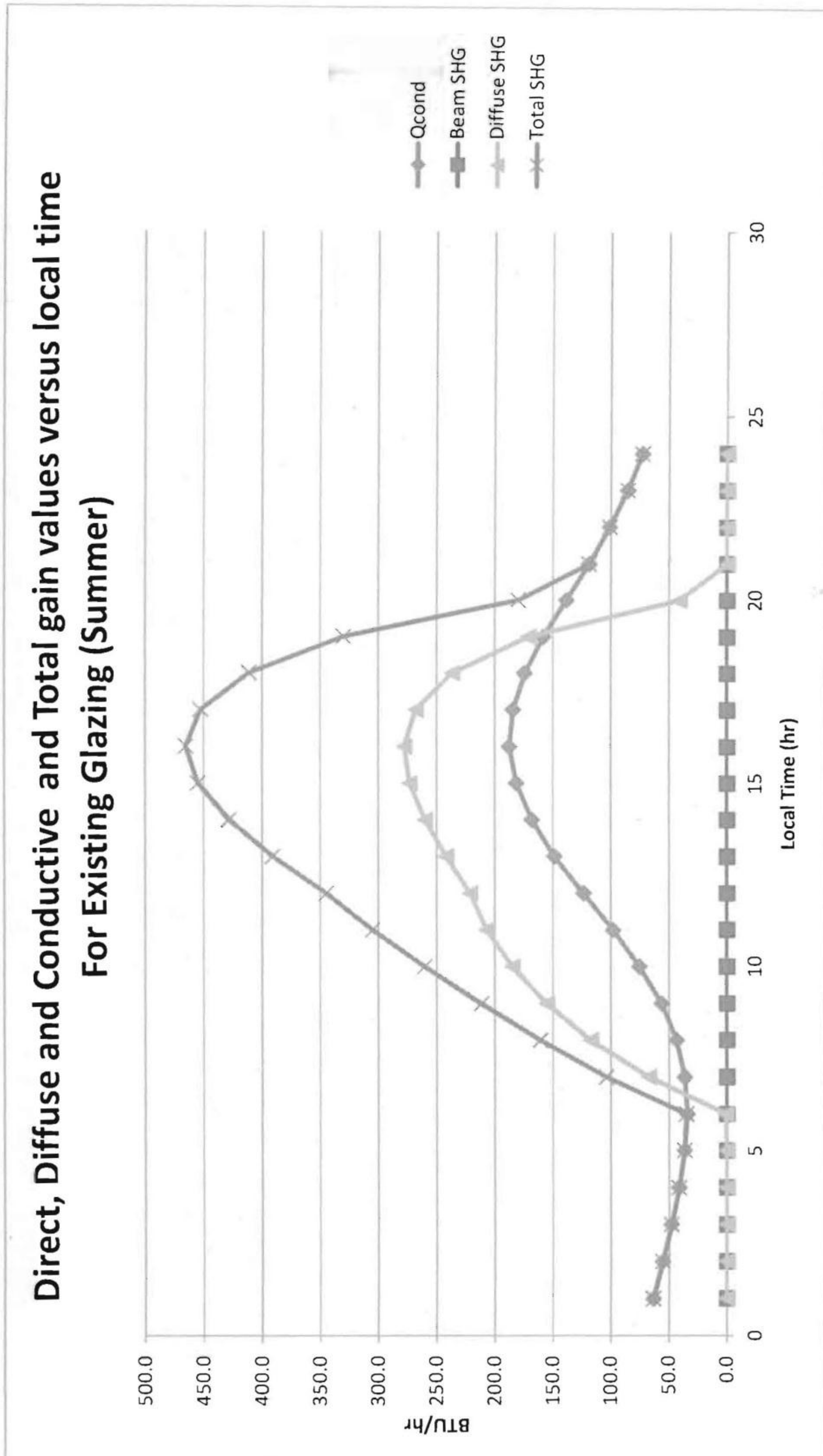
Blast Design Glazing
U Value Winter = ~~0.45~~ 0.43
U Value Summer = ~~0.47~~ 0.45
SHGC = ~~0.55~~ 0.40

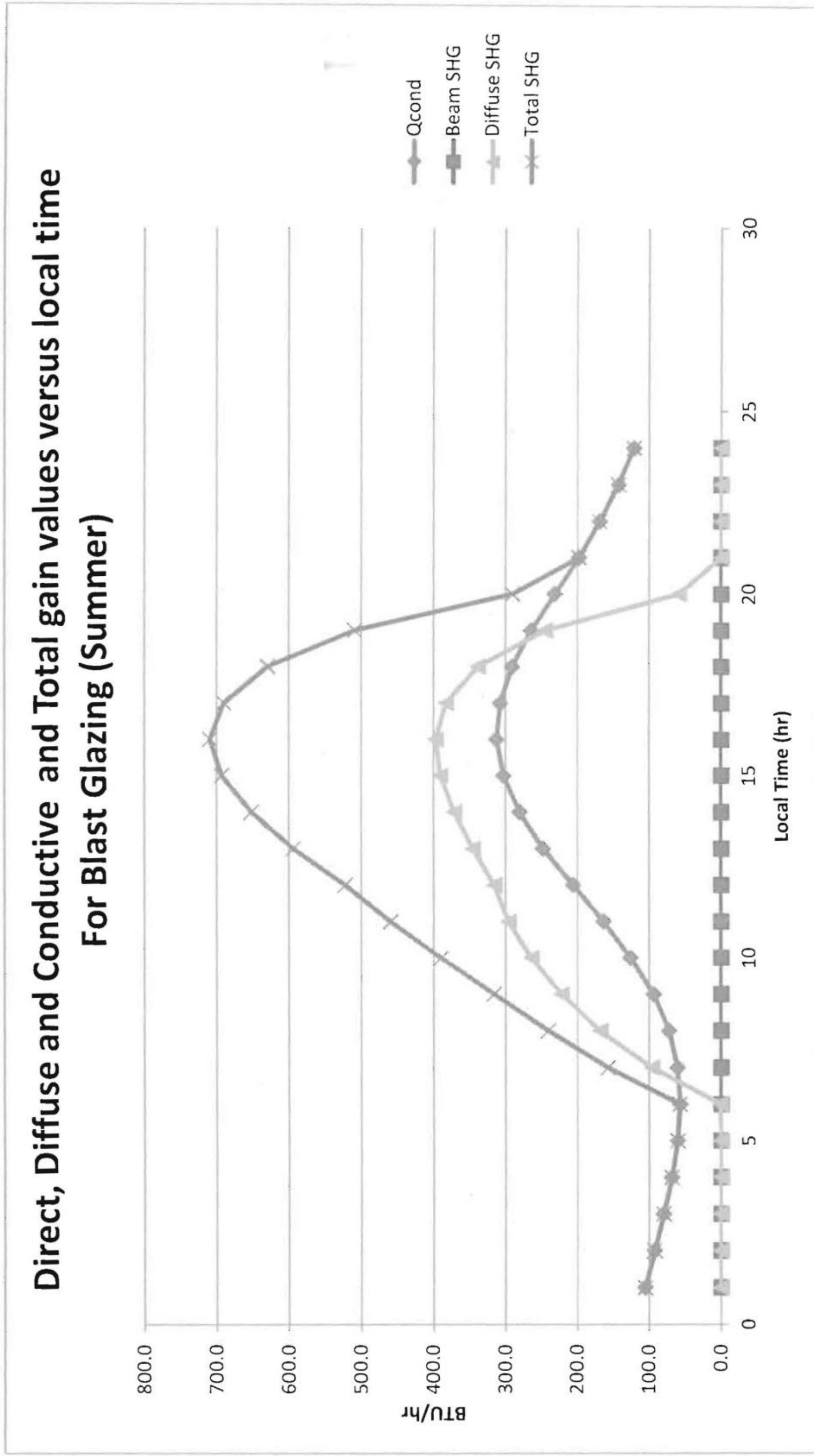
Winter Q_{Total} = 517.4 $\frac{BTU}{hr}$

Summer Q_{Total} = 709.6 $\frac{BTU}{hr}$


Direct, Diffuse and Conductive and Total gain values versus local time
For Existing Glazing (Winter)
BTU/hr
400.0
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0
5
10
15
20
25
30
Local Time (hr)
Qcond
Beam SHG
Diffuse SHG
Total SHG


Direct, Diffuse and Conductive and Total gain values versus local time
For Blast Glazing (Winter)
BTU/hr
600.0
500.0
400.0
300.0
200.0
100.0
0.0
0
5
10
15
20
25
30
Local Time (hr)
Qcond
Beam SHG
Diffuse SHG
Total SHG


Direct, Diffuse and Conductive and Total gain values versus local time
For Existing Glazing (Summer)
Qcond
Beam SHG
Diffuse SHG
Total SHG
BTU/hr
Local Time (hr)
500.0
450.0
400.0
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0
5
10
15
20
25
30


Direct, Diffuse and Conductive and Total gain values versus local time
For Blast Glazing (Summer)
Qcond
Beam SHG
Diffuse SHG
Total SHG
BTU/hr
800.0
700.0
600.0
500.0
400.0
300.0
200.0
100.0
0.0
0
5
10
15
20
25
30
Local Time (hr)

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PPG Industries Performance Glass Calculator Calculated *Center-of-Glass* Thermal and Optical Properties Based on NFRC 100-2001 Environmental Design Conditions

Details for Double Glazing as Specified

Outdoor Glass Lite	5/8" (16mm) Clear
Gas Cavity Dimension	1/2" (13mm)
Gas Fill	90% Argon/10% Air
Indoor Glass Lite	Laminate: 1/4" Solarcool(4) Graylite 14 - 1/4" Clear with 0.060" Clear PVB Interlayer

Calculated Thermal and Optical Properties

Shading Coefficient		0.46	
Solar Heat Gain Coefficient		0.40	
U-Values (K-Values)	**Metric (Kcal/hr/m²/C)**	**Metric (W/m²/C)**	**English (BTU/hr/ft²/F)**
Winter Nighttime	2.08	2.42	0.43
Summer Daytime	2.20	2.56	0.45
Relative Heat Gain	**Metric (Kcal/hr/m²)** 265	**Metric (W/m²)** 308	**English (BTU/hr.ft²)** 98
LSG (Light to Solar Gain Ratio)		0.09	
Transmittance (%)			
Visible		3	
Ultraviolet / Krochman Damage Weighted		0 / 1	
Total Solar Energy		2	
Reflectance (%)			
Visible (Out)		24	
Visible (In)		5	
Total Solar Energy (Out)		13	
Color Properties	**L***	**a***	**b***
Transmittance	21.45	-1.35	4.98
Reflectance	56.50	-8.80	2.73

While PPG believes this calculated performance data to be reasonably accurate, it may not precisely agree with similar performance data calculated using the LBL Window 5.2 program. PPG's published data is based on the LBL Window 5.2 program.

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