
# One and Two City Center Washington D.C.

## Final Report

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Option: Structural

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# One & Two City Center

Washington D.C.

## Project team

*Owner*: Hines | Archstone

*Structural Engineer*: Thorton Thomasetti

*General Contractor*: Clark Construction

Architect: Foster and partners | Shalom Baranes Associates

## Architecture

One and Two City Center are a part of a larger development which includes four residential buildings, public areas, parking and a hotel. The entire development is LEED Gold and LEED-ND (neighborhood development) certified. One and Two City Center are twin buildings with a rectangular shape. Encasing the structures is a glazed aluminum curtain wall with operable louvers. Lots one and two are separated by a pedestrian alleyway. Above the alleyway the buildings are connected by a series of staggered glass bridges. The alleyway creates an introduction to the spaces through between the buildings while the bridges enclose those spaces and allow them to be separate from the rest of the city.

## Structure

- 8.5 inch post tensioned slab
- Concrete Shear Walls
- Shallow and Mat Concrete Foundations
- Wide Flange steel members for Penthouse levels
- Rectangular Concrete Columns

## Mechanical

- Chilled Water air handling units
- Compression tanks power chilled water units
- Three floor Mechanical Penthouse
- Three cooling towers on roof
- Variable air volume and split

## Lighting & Electrical

- 2x4 and 1x4 fluorescent fixtures throughout
- Ceiling mounted motion sensors
- Hallways illuminated by 2x2 Fluorescent fixtures
- 200A and 400 A panels – 400A feeders

# 1. Executive Summary

One and Two City Center are commercial buildings that are a part of a multi-use development located in Washington D.C. Being approximately 312,000 square feet the building is part of a four lot project. Planning and design began as early as April 2007 but due to the economic recession, construction was delayed until April of 2011 and was finished later in 2014.

The twin office buildings now stand 12 stories tall with an average floor to floor height of 11'. The shell of the structures is a glazed aluminum curtain wall with movable louvers. Like many roofs in D.C., there is a rooftop mezzanine on both One and Two City Center with several areas used for grass and plants. Connecting the two buildings on staggered floor are glass coated walkways which span the alleyway separating One and Two City Center. The building has achieved LEED Gold certification and the development has been one of the first to achieve LEED-ND (Neighborhood Development) certification.

The structural floor systems are two way post tensioned concrete slabs supported by typical 24" x 24" concrete columns. These columns run down through the building into the below grade parking garage and come to rest on shallow concrete foundations. Lateral loads are resisted by a series of shear walls which surround the elevators and stairwells. The glazed aluminum curtain wall is fastened to the structure at the concrete slab and supported by HSS sections. The penthouse roof and floor are supported by a series of W10's.

The additional lots feature commercial, residential, parking and public areas. To the north of One and Two City Center (Lot46) is an outside plaza with a captivating reflecting pool. To the east of the site is a four structure commercial and residential development (Lot 47). The two main lots are connected by an alleyway lined with retail stores. At the center of Lot 47 is a small courtyard offering relief from the city. Underneath Lot 46 and 47 is a four story parking garage for public access and the use of delivery trucks.

The proposed structural redesign is a two way flat plate slab. Due to the new gravity loads the shear walls were redesigned as well. Blast design was performed for an exterior and interior explosion. During the analysis interior blast design proved to be too unreasonable and thus a progressive collapse design was performed. The cost of these systems was found and compared. Additionally the time it would take to construct the proposed system was compared to that of the existing. Finally the interior spaces were evaluated for acoustical comfort in terms of reverberation time.

# Table of Contents

# 2. Introduction

## 2.1 Purpose and Scope

This report will detail the structural existing conditions of One and Two City Center. Elements of the structure that shall be discussed are the buildings' general framing system consisting of typical bays and their columns, beams, slabs and how the load transfers through them. Furthermore this preliminary report will also describe structural components such as the lateral systems, foundation systems, building loads, national codes and joint details. The following pages shall provide a general understanding of the building through details and images provided by the owner and design team.

## 2.2 General Building Information

One and Two City Center are type B mixed use buildings located in Washington D.C. that stand 12 stories above grade. Both buildings are similar enough to each other in that they are twins with identical structural systems. Each floor is approximately 26,000 square feet with the total square footage of one building nearing 312,000 square feet. The two buildings are a part of the larger city center development consisting of additional residential and retail complexes. The entire site sits on top of a four story below grade parking garage. City Center has also achieved LEED Gold certification and has become a popular, high end, area of central D.C.

Located in D.C., One and Two City Center has to adhere to the height limitations due to zoning. The most common way to maximize floors and floor height is to use post tensioned slabs. The slabs system for One and Two City Center is 8.5" post tensioned concrete allowing for a greater floor to floor height of 12'. The two twin buildings are connected at every other floor by a concrete on steel deck bridge. Similar to other D.C. structures the City Center offices have a roof top mezzanine with a green roof. Foundations underneath the buildings are isolated concrete footers which support the columns above. Pictures of the buildings are shown in Figure 1 and a site plan is shown in Figure 2.

*Figure 1*: North West exterior view of One and Two City.

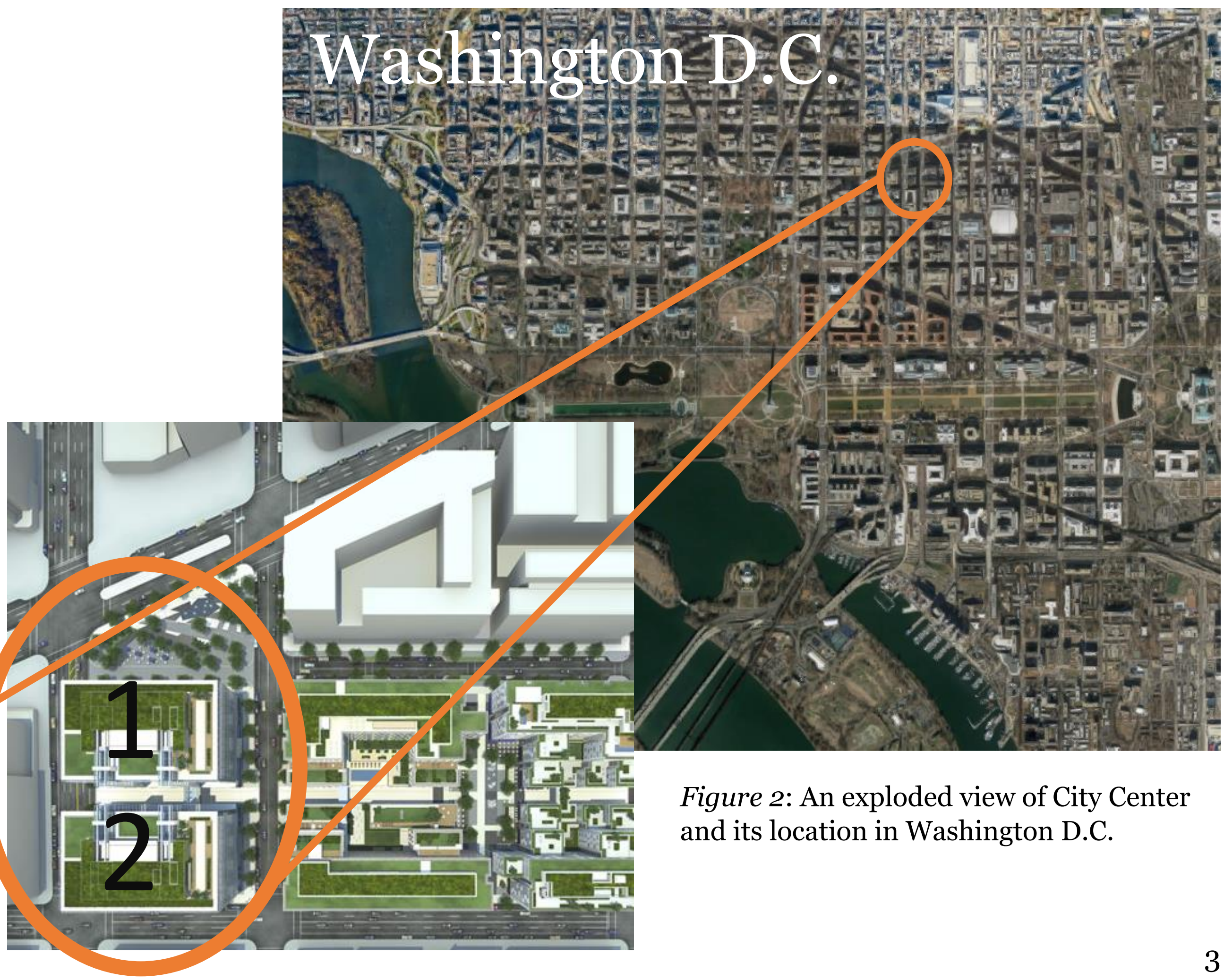



*Figure 2*: An exploded view of City Center and its location in Washington D.C.

## 2.3 Structural Framing System

The bulk of the structural framing for One and Two City Center is concrete. Slabs are 8.5" thick with both post tensioned and conventional steel reinforcement. These slabs are supported by reinforced concrete columns which have drop panels around them. The columns in turn are supported by shallow isolated concrete footings. Resisting the lateral loads on the structure are 16 shear walls, which are located near the main areas of egress. The rooftop mezzanine is supported by steel wide flange sections that are tied into the concrete structure. Bridges span between the two buildings and are connected through a pin and sliding mechanism. Figure 3 below shows the structure of One and Two City Center during construction.


*Figure 3.1*: Construction of One and Two City Center with structure exposed.

# 3. Structural Analysis

## 3.1 Floor Layout

The bays in both buildings are not typical in dimension. They range from the largest being 30' x 30' and the smallest being 25' x 20'. The compressive strength for the concrete used in the slabs is 5000 psi with the exception of the second floor which is 6000 psi. These slabs are typically 8.5" deep and contain 6" drop panels. Several floors are repetitive in structure. For example, the structural floor layout of level 4 is repeated on levels 5-8. Also the structural floor plan of level 11 is the same as level 10. A more detailed sketch of the structural elements throughout the buildings height is shown in Figure 3.2.

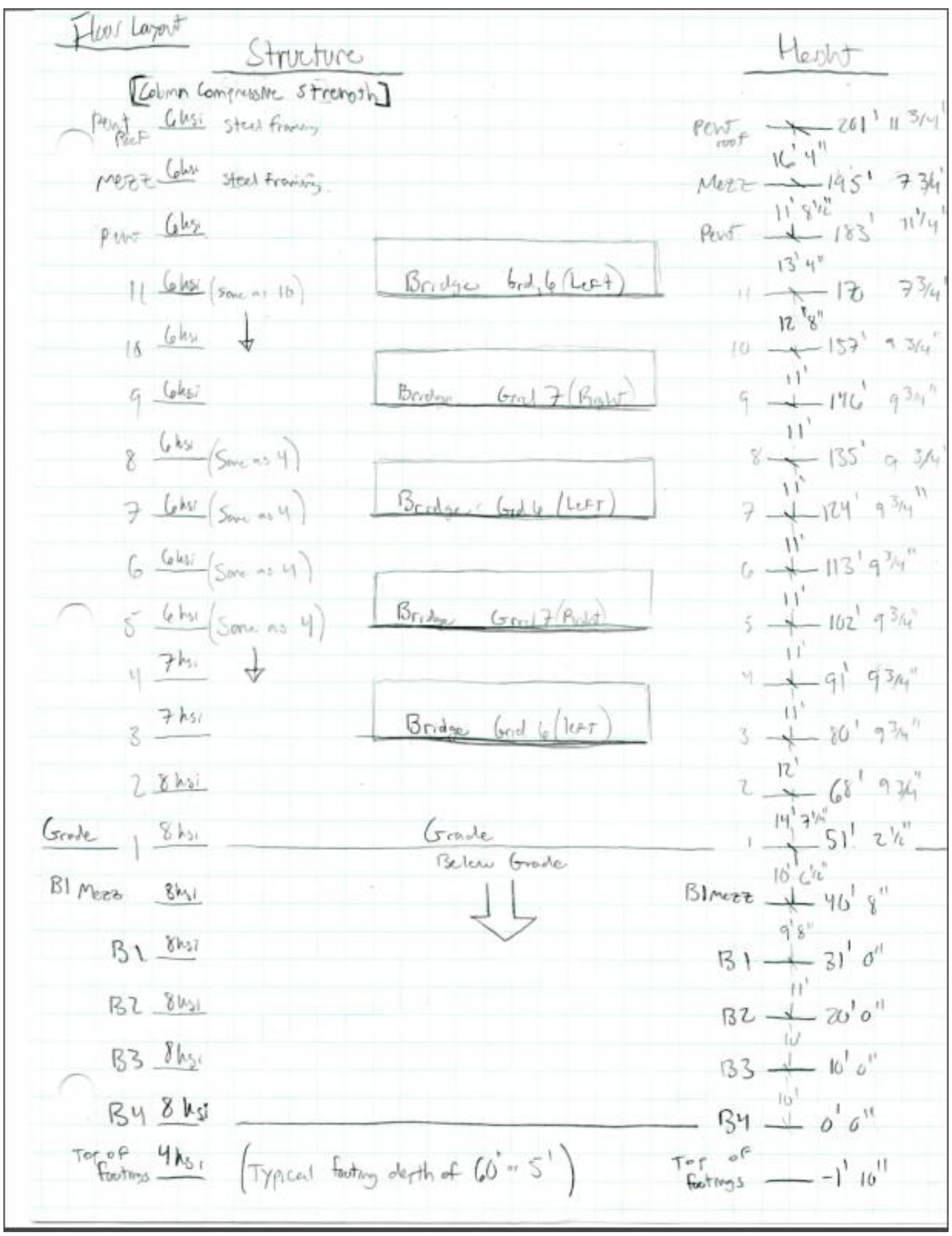



*Figure 3.2:* A hand drawn elevation showing basic structural elements throughout the building.

## 3.2 Post Tensioned Slabs

From level 2 to the penthouse (level 12), the floor system is post tensioned slab. Cables used are ½" diameter, 7-wire strand, grade 270, low relaxation tendons, which run in both directions of the building as seen in Figure 4 below. The tendons that run north to south have been detailed to be spaced 6' apart and stressed to 20 kips/ft. The tendons in the east west direction were detailed to only meet 810 kips. The engineer of record (EOR) most likely let the post tensioning subcontractor decide the layout for these tendons.

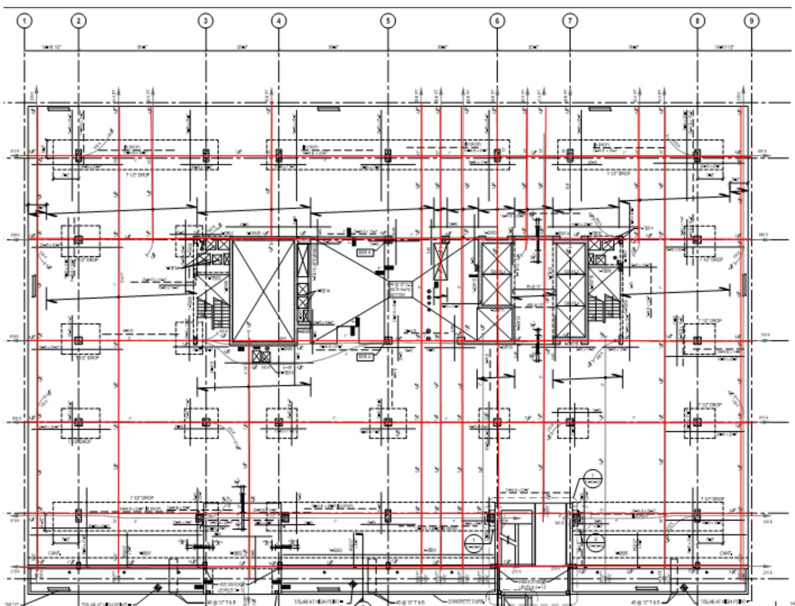


*Figure 4*: A typical floor plan with emphasis on the post tensioned cables.

## 3.3 Openings in the Slab

The main ways to traverse the buildings are through elevators, stairwells and bridges. These areas create openings in the structural slab of the building that have an impact on the load path of the slabs. There are 6 elevators in the same area that are approximately 500 square feet and a service elevator that is approximately 550 square feet. Stairwells can be found next to both the garage intake (service) and public elevators. 42' long bridges span the gap between One and Two City Center. A typical plan of these openings can be seen in Figure 5.

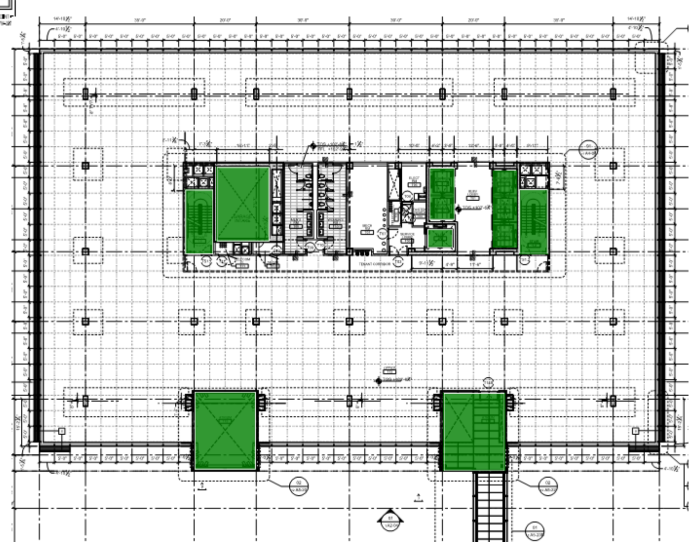


*Figure 5*: A typical floor plan showing the opening in the slab in green.

## 3.4 Stairwells

A sizable amount of the buildings openings comes from stairwells. The stairwells typically have 2 flights in between floors. These stairs are cast with 3000 psi concrete and contain steel reinforcement depending on the stair span and thickness. This reinforcement schedule can be seen in Figure 9. The stairwells are supported by concrete beams, shown in blue on Figure 10. These beams transfer the load from the stairs into the columns. A typical elevation shown in Figure 11 details the structure of the stairs throughout the building. What isn't shown in Figure 11 is that the stairs are also supported by the elevator cores or shear walls.

| STAIR REINFORCING SCHEDULE | | |
|---|---|---|
| STAIR (& LANDING FOR DOUBLE RUN) SPAN 'L' | SLAB THICKNESS 't' | REINFORCING |
| L ≤ 10 | 6" | #4@10" |
| 10 < L ≤ 12 | 7" | #5@12" |
| 12 < L ≤ 14 | 7" | #5@9" |
| 14 < L ≤ 16 | 7" | #5@7" |
| 16 < L ≤ 18 | 8" | #5@6" |
| 18 < L ≤ 20 | 8" | #6@7" |
| 20 < L ≤ 22 | 8" | #6@6" |

*Figure 9*:

6 STAIR THICKNESS AND REINFORCING SCHEDULE

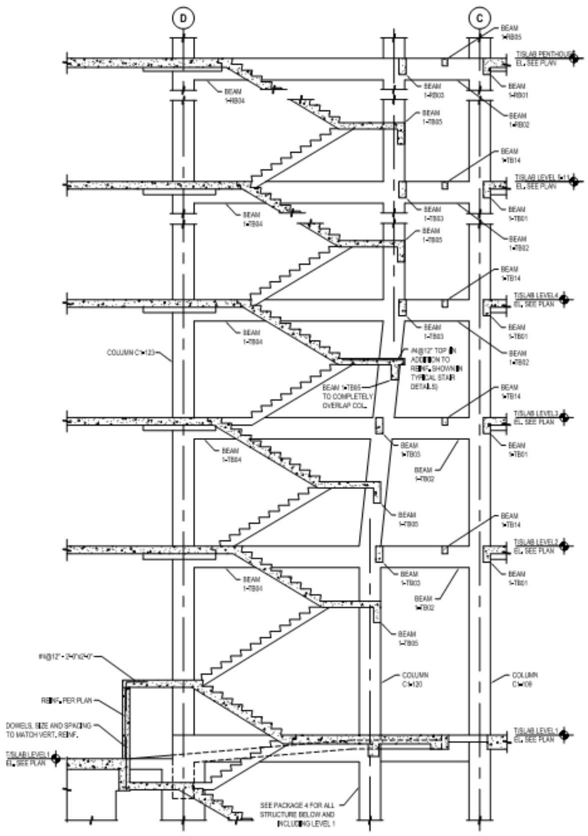


*Figure 11:* A typical stair section that shows the columns and beams that support the stairs.

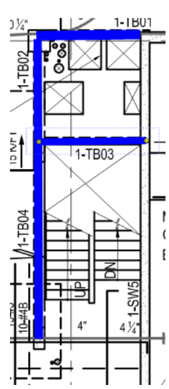
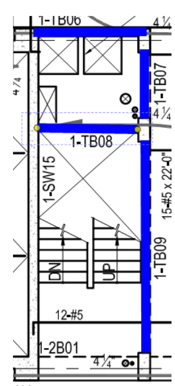



*Figure 10*: Plan view of two stairwells with the beams supporting them shown in blue.



## 3.5 Shear Walls

The elevator cores also act as shear walls taking the lateral loads of the building. The compressive strength of the shear walls follows the compressive strength of the columns. That said it varies by floor, see Figure 3.2 for compressive strength details. The shear walls that surround the elevator shafts have a typical thickness of either 10" or 12" depending on their orientation. This is likely due to the lateral loads from one direction being greater than the other. The reinforcement of the shear walls is either #4 or #5 bars spaced 12" running both horizontally and vertically. Some shear walls change geometry near the bottom floors and become longer as seen in Figure 13. This change in geometry could be due to higher shear at the base of the building. The location of the shear walls are shown on plan in Figure 12.1. Their configuration is most likely to minimize the distance between center of mass and center of rigidity. This will create a smaller eccentricity and make the building more effective at resisting seismic loads.

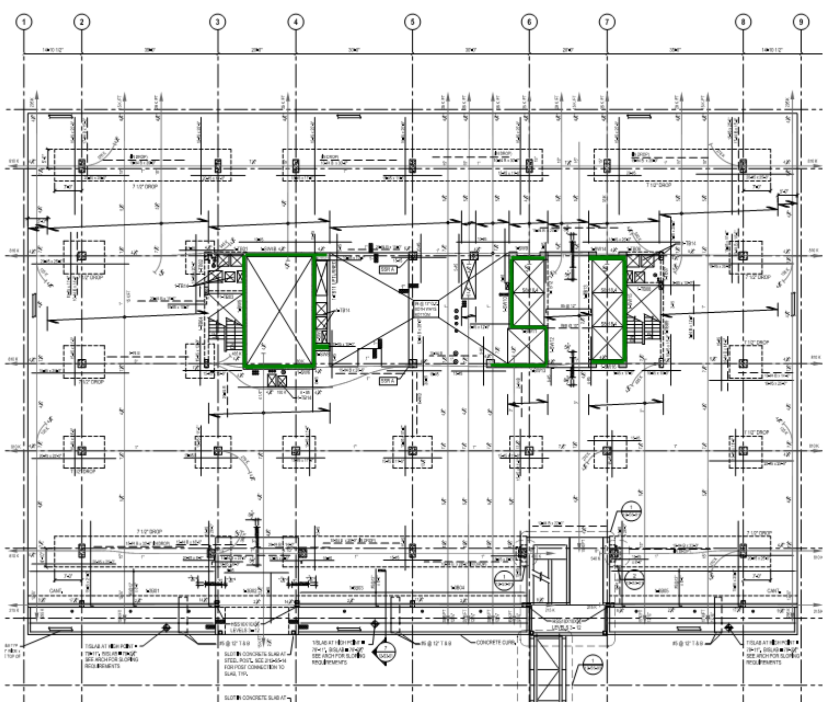


*Figure 12.1*: A plan view of the shear walls shown in green

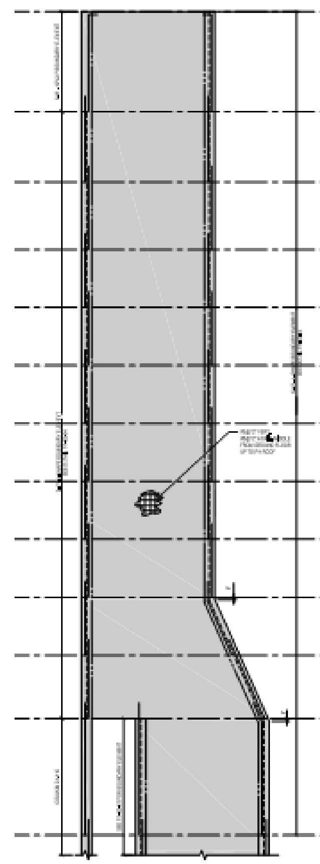


*Figure 13*: An elevation of a shear wall that widens at the lower levels

## 3.6 Drop Panels

On every above grade floor with a post tensioned slab system there are drop panels around several columns. The dimensions of these drop panels are, unless otherwise noted, dependent on the column to column span. It is detailed that the panels will be a length of L/6 where L is the center to center measurement of the span between columns. To ensure that punching shear does not occur the drop panels' length cannot be less than 2' on each side. The minimum extended depth from the slab is the thickness of the slab denoted as h divided by 4 (h/4). The typical reinforcement at each column location is 15 to 25 #5 bars placed each way. Figures 6 and 7 show a typical drop panel detail and location on the plan.

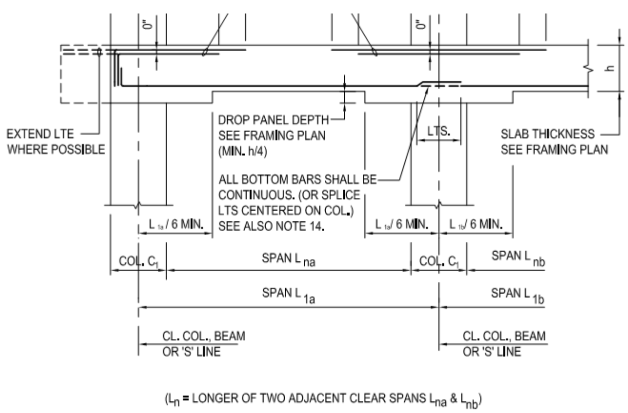



*Figure 6*:
3 TYPICAL DETAIL AT COLUMN STRIP WITH DROP PANELS

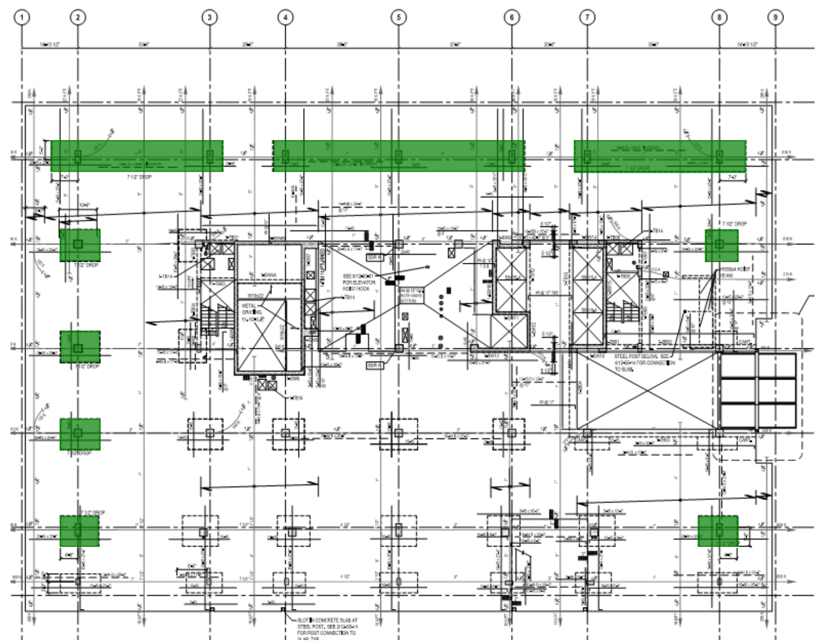


*Figure 7*: A floor plan showing the location and outline of drop panels.

## 3.7 Columns

There are typically 45 columns on each floor. These Columns vary in both compressive strength, size and height throughout the building. Typical steel reinforcing along the length of the column is 8 #8 bars. The compressive strength of the columns decreases at higher levels as seen in Figure 3.2. Column ties are based off of the size of the vertical bars. Figure 14 shows the different spacing of horizontal reinforcement, depending on the vertical reinforcement. Figure 15 details sections of columns and shows the typical layout of reinforcement.

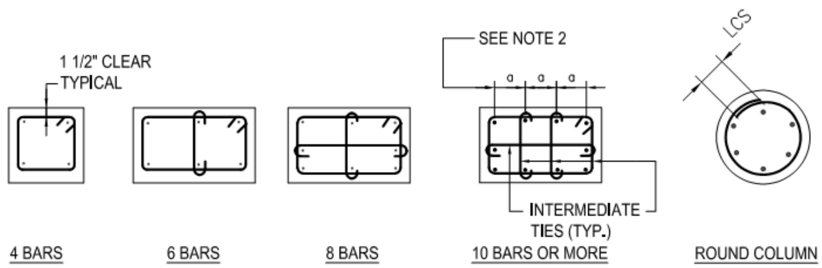



*Figure 14*: Specified column ties based on the vertical reinforcement.

| MAX. SPACING "S" OF COL. TIES | |
|---|---|
| VERTICAL BAR SIZE | TIE SIZE & SPACING* |
| #5 | #3 @ 10"* |
| #6 | #3 @ 12"* |
| #7 | #3 @ 14"* |
| #8 | #3 @ 16"* |
| #9 | #3 @ 18"* |
| #10 | #3 @ 18"* |
| #11 | #4 @ 18"* |
| | |
| | |

* TIE SPACING NOT TO EXCEED LEAST COLUMN DIMENSION

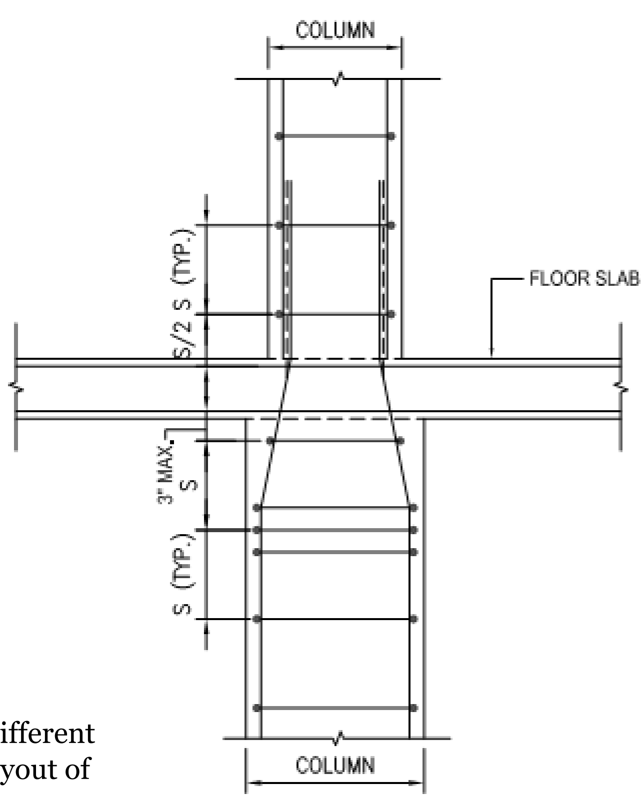



*Figure 15*: Sections of different columns showing the layout of reinforcement.

## 3.8 Foundation/Garage

The columns detailed above come to rest on shallow isolated concrete footings. These footings are several levels below grade. This is because there are five levels of below grade parking underneath One and Two City Center. The decision to include the garage as a part of the building analysis has yet to be made. Therefore the existing structure below grade will be shortly summarized due to how separate the building and garage structures are.

The structure of the parking garage consists of a 10" concrete slab with drop panels extending L/6 distance from the columns and 6" in depth similar to the above grade floors. Normal weight concrete is specified with a compressive strength of 5000 psi. The expansion joints for the garage vary in location and are typically 2" in width. The main reinforcing steel used in the garage slabs are #6 bars both ways with a minimum cover distance of 2" on top and 1" on bottom. The spacing of #6 bars depends on the location and the column.

The columns, at the same location as the columns above grade, have a compressive strength of 8000 psi. These columns are supported by concrete footings and slab foundations. The concrete footings have a compressive strength of 4000 psi and are typically 60" deep. There is no typical size of footing but the average size is 15' x 15. Typically 28 or 30 #10 bars run both ways to support the footing for shear, bending, temperature and shrinkage. Columns are also supported by slab foundations which like the footings have a compressive strength of 4000 psi. Shown in Figure 16 are the locations of the slab foundations underneath One City Center.

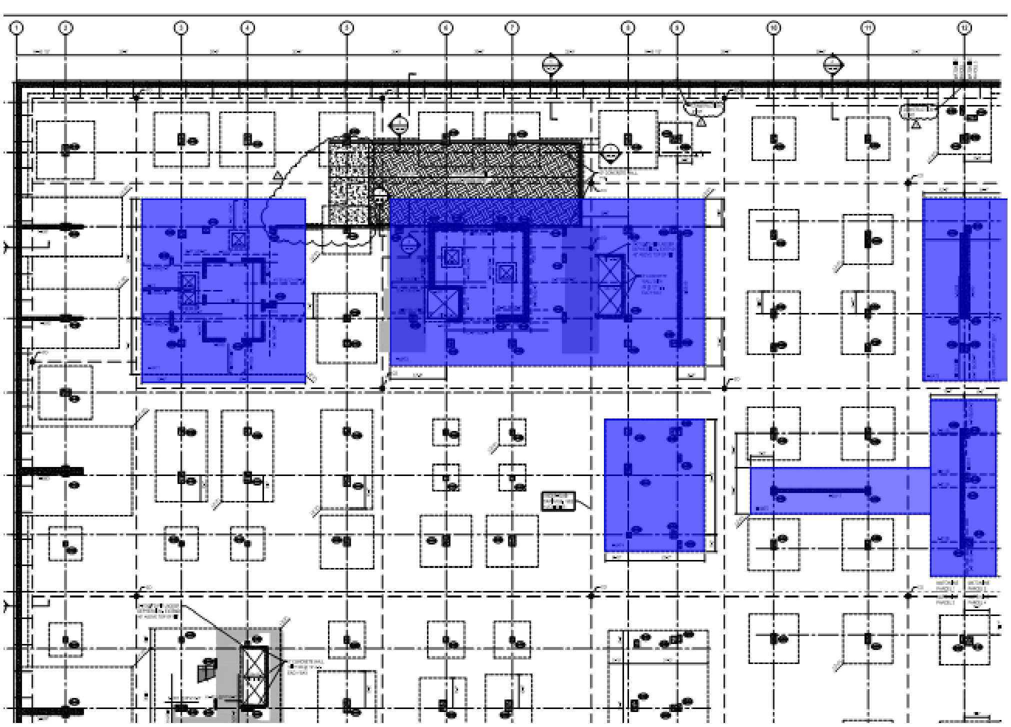


*Figure 16*: A plan view of the slab foundations beneath One City Center.

## 3.9 Roof

Three levels make up the roof component, the penthouse the penthouse mezzanine and the penthouse roof. The penthouse level is similar to the lower 11 floors in that it is post tensioned concrete. Structural steel framing is used for the penthouse mezzanine and the penthouse roof. Both the penthouse mezzanine and roof are approximately 8,000 square feet. W10's are used as beams for the roof while w14's are used as girders to support them. Supporting the glass wall on the roof are a series of HSS members. Shown in Figure 17 are beams drawn in red, girders in blue and HSS members in green. The mechanical penthouse (mezzanine and roof) are permitted to exceed the 130' height limit because it has an occupancy group U.

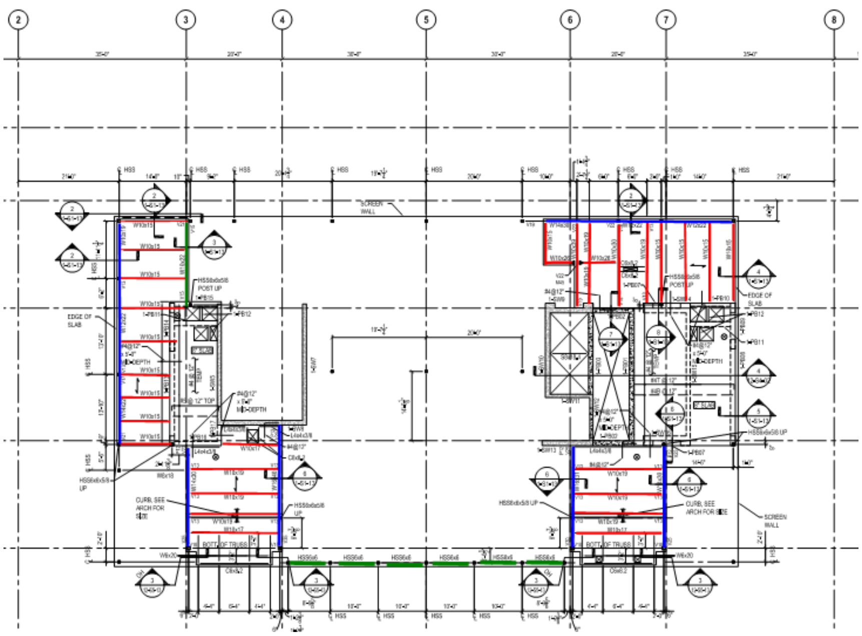
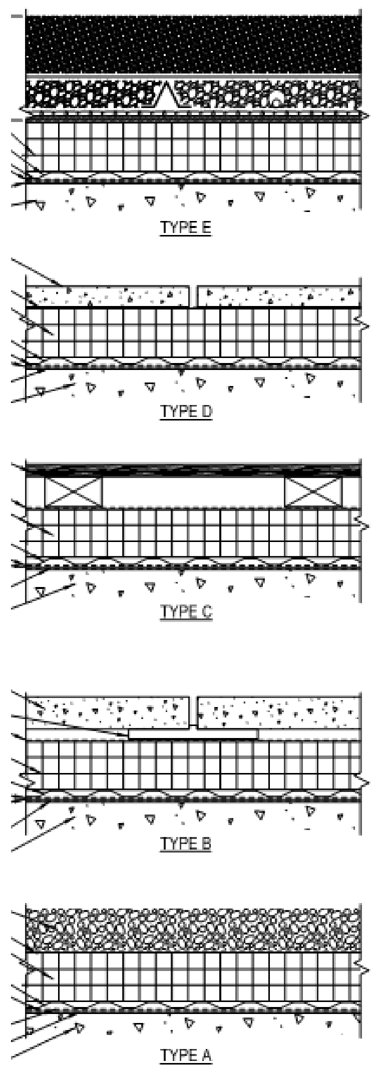


*Figure 17*: A plan view of the structural steel of the penthouse mezzanine.

*Figure 18*: The five different types of roof systems.




The roof membrane consists of five different construction types as shown in Figure 18. All roof construction types have hot fluid applied asphalt as the waterproofing layer with 3" polystyrene insulation board topped with a filter fabric. The discrepancies between the roof types is what lays on top of the filter fabric. The various types of roofing have a topping of either stones, concrete pavers, wood decking or topsoil.

## 3.10 Bridge

One of the noteworthy architectural and structural aspects of One and Two City Center is that there are a series of bridges than span 25' between the two buildings. The bridges occur every other level starting at level 3 and ending at level 11. Every other bridge changes gridlines thus creating a pattern similar to that in Figure 19. The structure of the bridges is 3.25" lightweight concrete on 2"18 gauge composite steel deck. This deck is supported by double steel angles as shown in Figure 20.

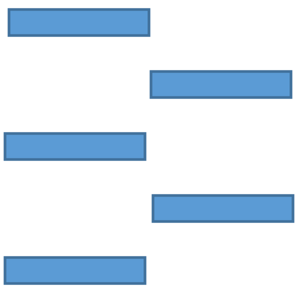


*Figure 19*: Simplified section of what the bridge configuration looks like.

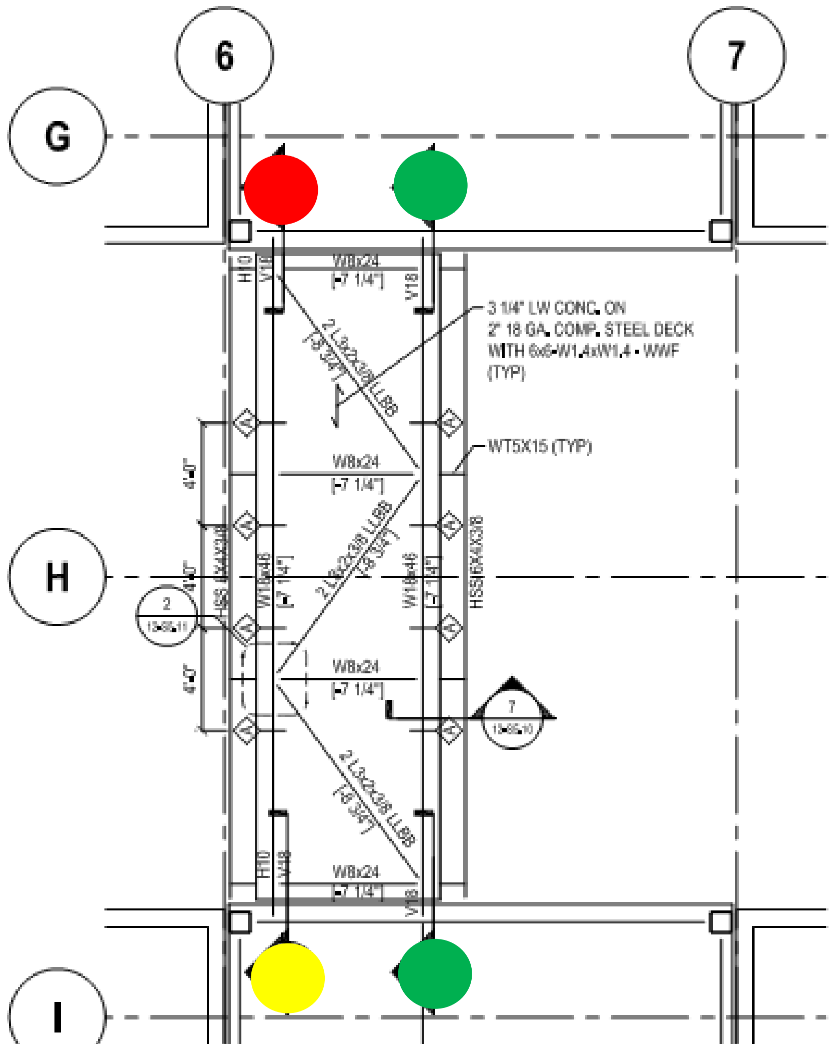



*Figure 20*: Bridge detail where the colored circles represent the type of connection

1 12-S5-10 BRIDGE FLOOR AT LEVELS 3, 7, 11
SCALE: 1/4"=1'-0"

The bridge is connected to each building through pins and sliders. The various connections shown with colored circles in Figure 20 represent different connections. These connections are further detailed in Figure 21. Due to the nature of the connections the two building do not share any loads except for those on the bridges. The bridge does not transfer any lateral loads between the buildings.

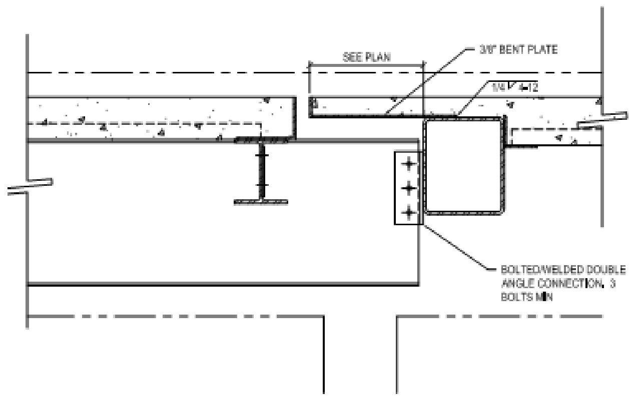
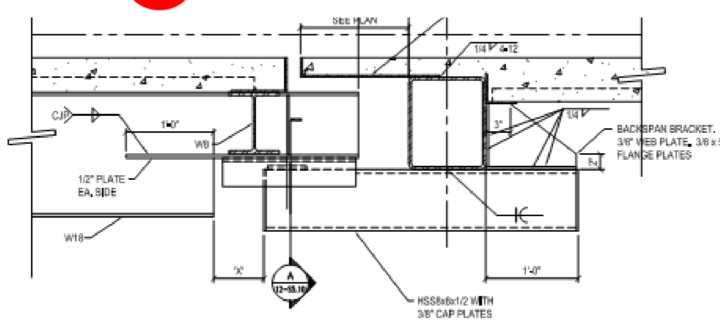


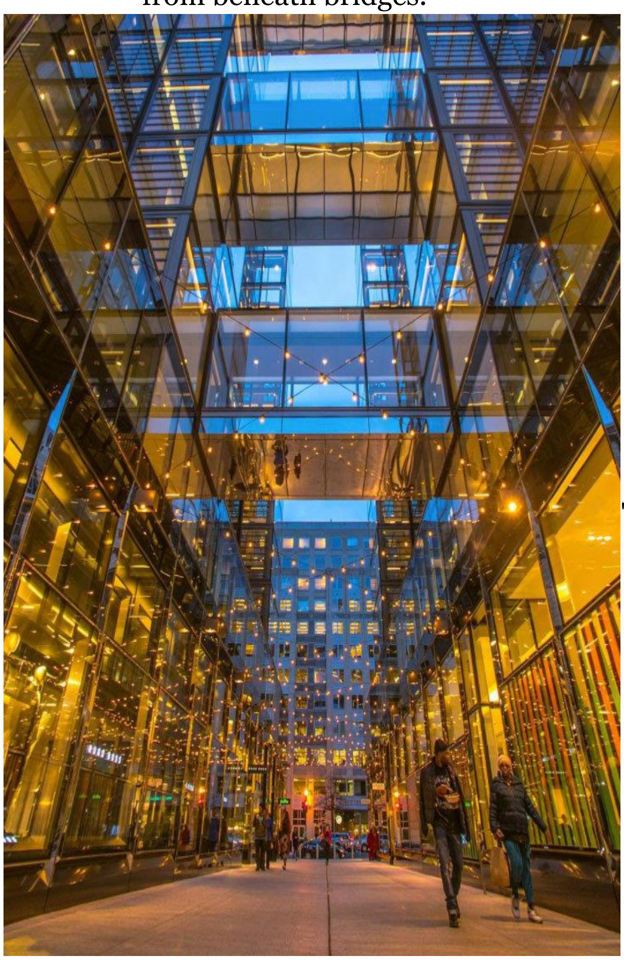

*Figure 21*: Connection details for the bridges at levels 3, 7, and 11.

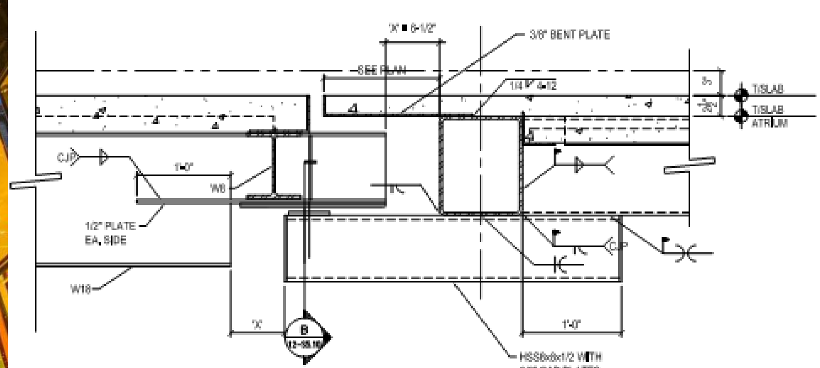
BRIDGE FLOOR GIRDER CONNECTION AT "PIN"
SCALE: 1 1/2" = 1'-0"




BRIDGE FLOOR GIRDER AT GUIDED SLIDING CONNECTION
SCALE: 1 1/2" = 1'-0"

*Figure 22*: Upward view from beneath bridges.






BRIDGE FLOOR GIRDER AT SLIDING CONNECTION
SCALE: 1 1/2" = 1'-0"

## 3.11 Envelope

The façade of the building is made of a glazed aluminum curtain wall. The glass wall encases the building and is supported by connections to the slab. This connection consists of two steel angles bolted into the side of the slab as shown in Figure 23.The envelope of the building also features an atrium space that extends the full height of the building. This can be seen in a general geometric model of the building shown in Figure 24. The glass wall of the atrium space is supported by steel HSS members.

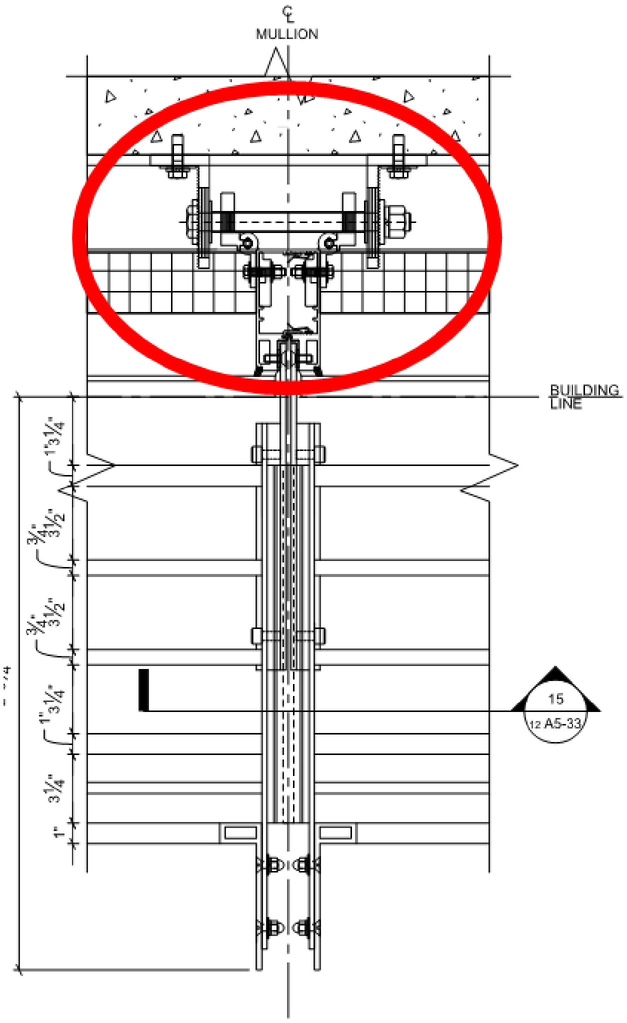



*Figure 23*: Plan detail of curtail wall connection to slab.

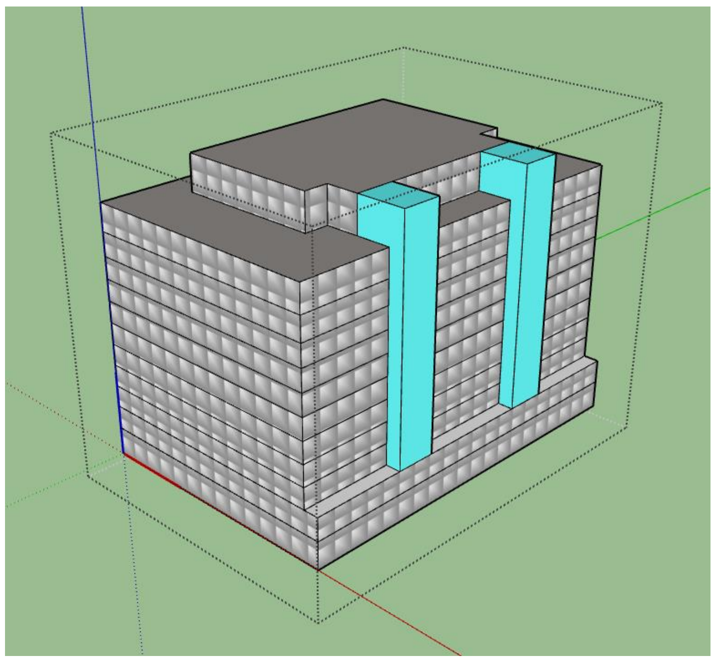


*Figure 24*: Geometric model of One City Center showing the atrium spaces in light blue.

# 4. Loads and Codes

The following load tables consist of the design loads in accordance with the District of Columbia DCRA-12 2003 and International Building Code (IBC) 2000 which in turn references ASCE 7-98. Other design codes used are ACI 318-02 and 530-2000 along with AISC-LRFD second edition. It is important to note that the structural members are not designed for any vibratory loading from equipment as specified by the engineer of record.

## 4.1 Live Loads

These loads are based on the expected occupancy load for given areas of the building. The loading of mechanical rooms was based on assumed weights of the mechanical equipment. Occupancy loads are derived from probabilities and have different safety factors applied for different loading conditions.

| Live Loads | Pounds per square foot (PSF) |
|---|---|
| Office | 80 psf |
| Ground floor, Retail, Lobbies, Stairs | 100 psf |
| Mechanical rooms, Storage | 150 psf |
| Terraces | 100 psf |

## 4.2 Dead Loads

Dead loads are more accurate than live loads because they are based on material weight not presumed occupancy. Mechanical dead loads, same as live loads, are assumed equipment weight. The following table does not include the self-weight of structural members rather the loads that those members would support.

| Dead Loads | Pounds per square foot (PSF) |
|---|---|
| Office Floor/Partitions | 20 psf |
| Mechanical Equipment | 10 psf |
| Green Roof (roof) | 50 psf |

## 4.3 Snow Loads

Loading caused by seasonal snow on buildings is estimated using the specified building code. The code contains maps and minimum standards to abide by for snow loading. The table below shows the variables used in calculating the roof snow load. It is specified that the calculated snow load comes from the variable below plus sliding and drift. It is also noted that the greater of the two loads, from the map (30psf) or from calculations, will govern.

| Snow Factor | Value |
|---|---|
| ground snow load (Pg) | 25 psf |
| exposure factor (Ce) | 1 |
| importance factor (I) | 1 |
| thermal factor (Ct) | 1 |
| roof snow load (Pf) | 17.5 psf |

## 4.4 Wind Loads

Wind loads, like snow loads, have their own map in the code that details a statistical wind speed in miles per hour. The winds pressure on the building is then determined from the wind speed in combination with other factors. The wind load acts on the cladding and in turn the cladding or curtain wall imposes forces on the building. It has been assumed by the engineer of record that theses imposed wind loads from the cladding system create no moments or torsional effects on the structural members. In future reports the wind loads and their torsional effects will be investigated.

| Wind Factor | Value |
|---|---|
| Wind speed | 90 miles per hour |
| Importance factor | 1 |
| Exposure category | B |

## 4.5 Seismic Loads

Seismic loads can place the building under a variety of loading conditions. During an earthquake the building can be pushed up pulled down or be shifted laterally. The most common condition, and thus the design condition, is lateral shift. The seismic resisting system in One and Two City Center are reinforced concrete shear walls which transfer the lateral load throughout the structure to the foundations. It is important to locate the shear walls in such a manner that the buildings center of mass and center of rigidity are not far apart. If they are, then the building will experience torsional twisting effects during high lateral loads. Figure 12.1 below shows the location of the shear walls and an estimated location of the buildings center of mass and center of rigidity. The procedure for determining the seismic forces on the building is the equivalent lateral force procedure. Below is a table showing the factors and results from that analysis.

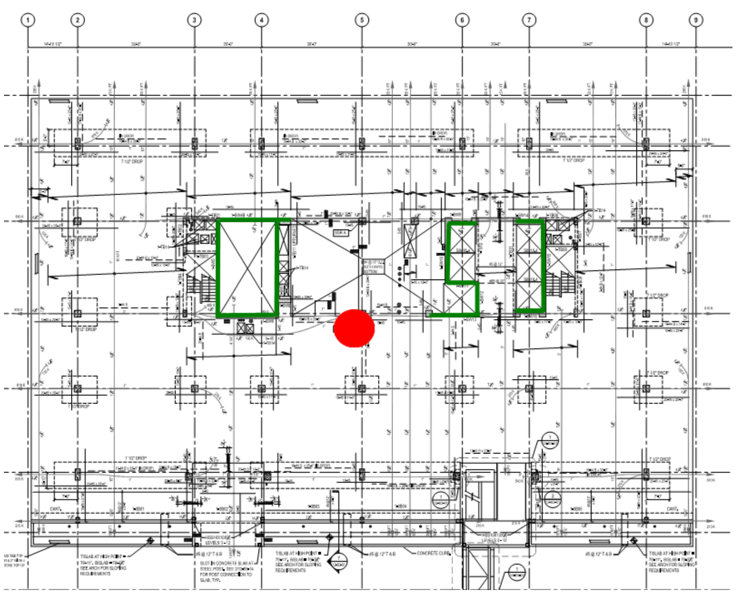


*Figure 12.1*: A plan view of the shear wall locations in green and an estimated location of the center of mass and rigidity in red.

| Seismic Factor | Value |
|---|---|
| Use group | 1 |
| Site Class | C |
| Spectral response coeff | $S_{ds}$=0.143 $S_{d1}$=0.071 |
| Design Category | B |
| Base shear | NS-1082Kips<br>EW-1255Kips |

# 5. Proposal

## 5.1 Background

The existing structure is such that the building can achieve large floor to floor height and be relatively inexpensive. The gravity system is reinforced concrete columns with an 8 ½" post-tensioned slab. The lateral system is reinforced concrete shear walls around the major means of egress. It is fairly common in Washington D.C. to use a post-tensioned system to create a large floor to floor height. Due to the height restrictions in the area the maximum permissible height for occupied space is 130'. In order to achieve the most usable space a small ceiling to floor height must be used. Thus the floor to floor height is 11' due to the ability to have a slim 8 ½" slab. The foundations for the building consist of shallow concrete footers and mat slabs. One and Two City Center are connected in two locations. The first is through a below grade parking garage which supports both buildings. The second is a series of bridges that connect the two buildings above grade. These connections are crucial to the way in which One city center will be analyzed in such that the two buildings do not transfer load between each other. This will be achieved by cutting One City Center at grade level and at the bridge connections. The below grade connection will be treated as a rigid fixed connection while the bridge connection transfers no forces between the buildings due to its pin and roller supports. This figurative "cut" can be seen in Figure 25 below.

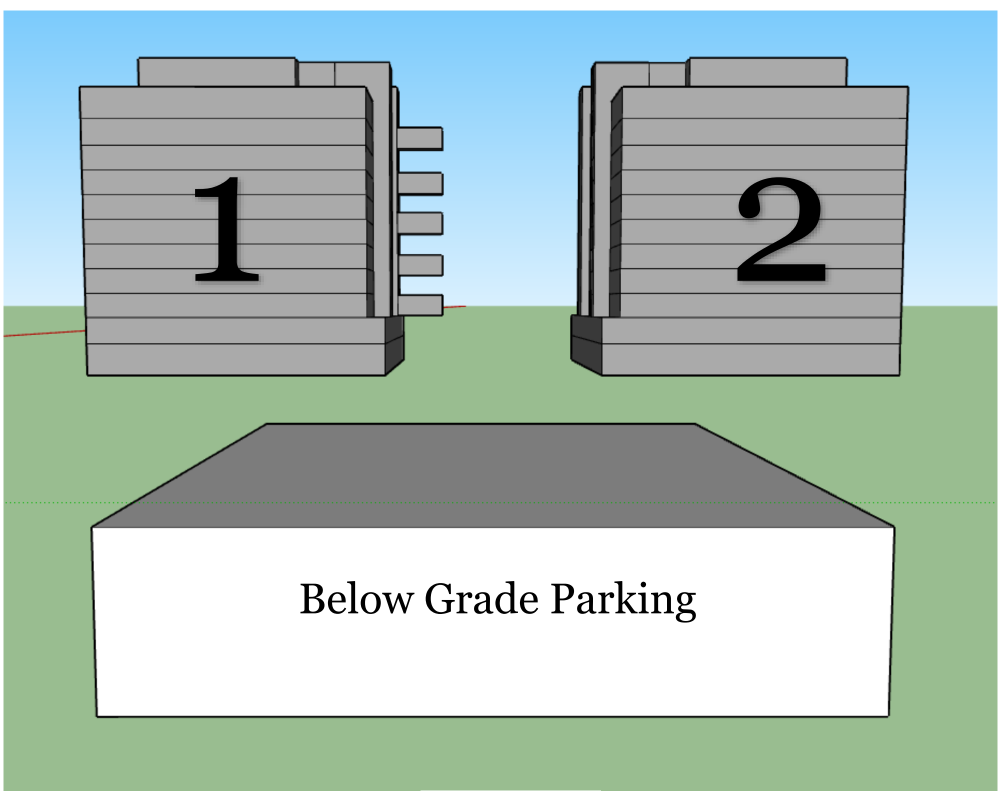





*Figure 25*

## 5.2 Problem Statement

The goal of this proposal is to design a new gravity system for One City Center and additionally design a system to resist blast loading. Such buildings are fairly common in the Washington D.C. area and One City Center shall be redesigned to meet such structural qualifications. If such structural qualifications prove to be too unreasonable a progressive design shall be implemented instead. It is also desired to keep the overall height limit of 130' and ensure that the floor height of 11' only differs by 5%. The new design shall not alter the existing structural grid or architecture unless proven reasonable. Additionally the change in cost for both original and alternative systems will be compared. The proposed structural change for One City Center is from a gravity system of post tensioned concrete to a two way reinforced concrete slab. This redesign will generate new gravity loads in the structure and therefor mandate a redesign of the columns as well. Also as a result of the new loading the lateral system will be redesigned. Both columns and shear walls will not change material properties but will change size if deemed necessary.

| System | Height | Cost(per bay) | Notes |
|---|---|---|---|
| Post Tensioned Slab | -8.5" slab<br>- 7.5" drop panels<br>Total Height = 16" | $29,000 | -complex analysis<br>-involves only concrete subcontractors |
| Composite Metal Deck | -3.5" concrete slab<br>-1.5" metal deck<br>-12" beam<br>-18" girder<br>Total Height = 24" max | $33,000 | -moderate analysis<br>-high level of capacity<br>-best for vibration control |
| One Way Slab | -6" slab<br>-18" beam<br>-24" girder<br>Total Height = 30" | $34,250 | -most expensive system<br>-lowest floor to floor height |
| Two Way Slab | -10.5" reinforced slab<br>-#9, #5 bars both ways top and bottom<br>Total Height = 10.5" | $30,750 | -low level of capacity<br>- heavily dependent on reinforcing steel<br>-best overall height |
| Hollow Core Planks (Design 2) | -8" plank<br>-18" girder<br>Total Height = 26" | $28,100 | -simple analysis<br>-involves multiple contractors of various trades |

## 5.3 Problem Solution

The problem statement shall be met through the use of both hand calculations and structural modeling software. Concrete designs for both gravity and Lateral systems will conform to ACI 318-11 along with IBC 2009 and ASCE 7-10. These codes have been chosen due to the time at which this building was initially designed. It is proposed for this thesis that a two way reinforced concrete flat slab shown in Figure 26 be used for the alternative gravity system. The lateral system will still consist of reinforced concrete shear walls but shall be redesigned with the loads and reactions that come from the new gravity system. This system was chosen due to its feasibility and effective height. The other alternative systems analyzed in notebook B either increase the effective height from ceiling to floor by too much, their cost is too great, or their feasibility will not work as well with the existing architectural grid. Both gravity and lateral systems shall be designed by hand in accordance with the codes previously stated and through software such as RAM Concept, ETABS and spSlab. Loads will be determined from these designs along with the controlling load case. The existing post-tensioned structure will be analyzed through the use of both the equivalent frame method and RAM Concept. Both initial and alternative designs will be compared through their capacities, effective depth, cost and schedule. RS means will be utilized in order to estimate the cost and schedule of each system. The various results of design and construction will be graphed and compared.

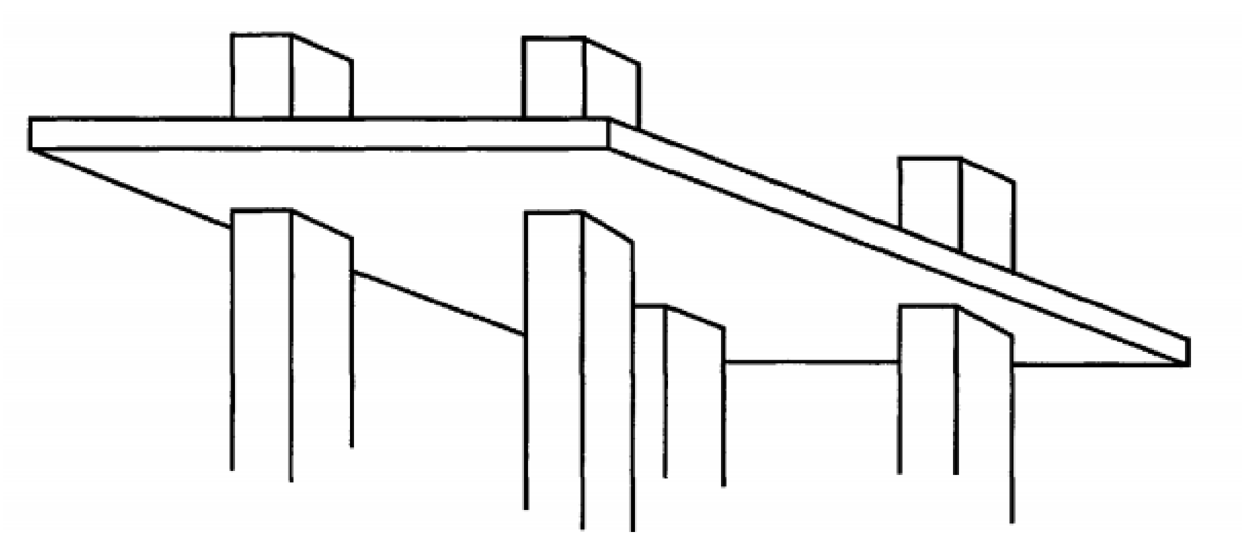


*Figure 26:* Image of a concrete flat slab

## 5.4 Methods

The problem solution will be achieved through the knowledge gained from structural courses previously taken and currently taking. Courses such as AE 402 and 431 will aid in the hand calculations and manual design of the slabs, columns and shear walls. These designs will be checked through the use of software that was learned in AE530 and codes from ACI 318. The existing post tensioned slab will be analyzed through learned methods from AE 597. Construction costs and scheduling will be determined from experience gained in AE 372. Additionally the advice from the AE professors will be available.

## 5.5 Tasks and Milestones

I. Analyze Existing System
   A. Determine slab capacity by hand and RAM concept
   B. Determine column capacity by hand
   C. Determine shear wall capacity
   D. Calculate cost of single floor
   E. Estimate schedule of construction for single floor

II. Design proposed slab system
   A. Determine new loads
   B. Design slab strips in both directions
   C. Check floor system with software
   D. Compare floor cost to existing system

III. Design columns for proposed slab
   A. Determine loads
   B. Design columns
   C. Compare capacities to existing columns

IV. Design proposed lateral system
   A. Determine loads on shear walls
   B. Design walls
   C. Check lateral system with software

V. Construction Management Breadth
   A. Calculate cost per floor then overall cost
   B. Calculate amount of overall material needed
   C. Create schedule for proposed system
   D. Create critical path schedule for proposed system (try to make the schedule close to that of existing building based on dates of construction)
   E. Create graphs and images that compare the two in terms of cost and schedule

VI. Acoustical Breadth

A. Determine typical room finishes and materials
B. Show floor plan of different areas and their sound transmission class
C. Calculate sound transmission between areas
D. Discuss how the changed materials would improve the space

VII. Finalization
A. Create final report draft (periodically)
B. Create Final presentation draft (periodically)
C. Finalize both report and presentation draft.
D. Update CPEP (periodically)

## 5.6 MAE Incorporation

Masters level work shall be thoroughly incorporated in this report. The bulk of this work will consist of computer modeling from software learned throughout AE 530. Additionally MAE work will be shown in the analysis of the existing post tensioned system and design of the post tensioned progressive collapse system learned from AE 597.

## 5.7 Breadth Topics

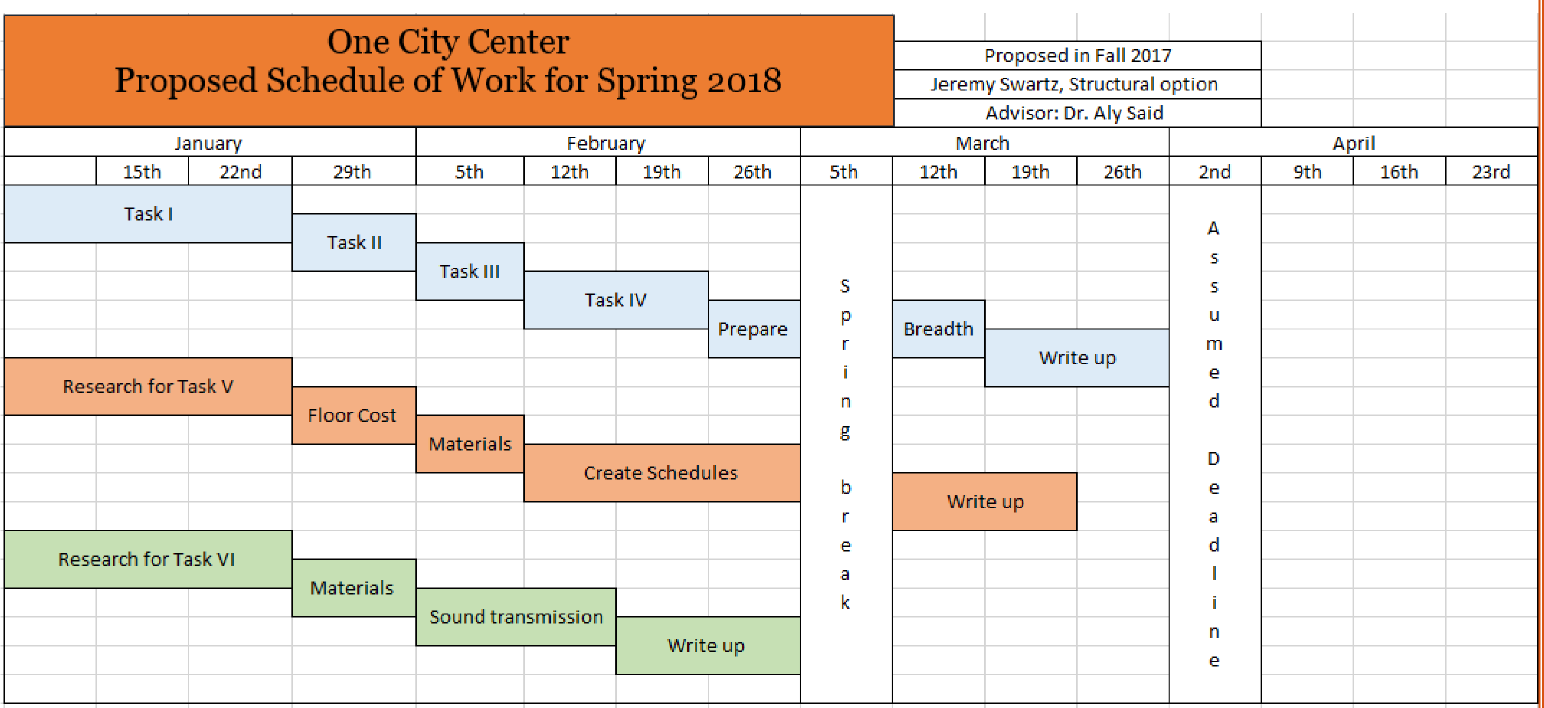
### Construction Management

The construction management aspect of One City Center will be analyzed under the newly proposed structural system. Such an analysis will include a comparison of the old system the new system and the progressive collapse system. Additionally the per floor schedule will be compared for the existing and new systems. This information is key to understanding how the building comes together as a whole and how changes in the structure effect cost and time. Through this comparison greater knowledge will be gained about how changing the structural system has an impact on the constructability of the building.

## Acoustics

The way sound travels throughout a space can very much effect the functions of those inside that space. It is proposed that the architectural acoustics be studied for several typical spaces in One City Center. Furthermore the material properties of each space and their effect on the overall acoustics will be analyzed. This topic is of high interest due to the existing materials inside the building which are smooth and hard. This leads to a potential echo and reverberation in sound. It is proposed that the dimensions and surface materials of public and private areas in the building be examined for auditory comfort. This analysis fits well with the structural depth of blast protection and progressive collapse in that government agency buildings are likely to need little reverberation and high transmission loss in order to keep conversations private.

# 5.8 Schedule



## 5.9 Goals

It is desired that this report accomplish the following

- Design a new structural system for One City Center that do not change the overall floor to floor height by more than 5% or about 6".

- The cost of the new structure is desired not to differ by more than 10%.

- The time it takes to construct this new system will not be greater than the original by a month.

- The structural system shall be designed to resist blast loading for both interior and exterior explosions. The magnitude of which is to be determined.

- The new structural system shall not greatly alter the architecture.

- Interior spaces will be analyzed for acoustical comfort and changed if deemed unsatisfactory.

- Material learned in master's level classes be used at least twice.

# 6. Structural Depth

## 6.1 Introduction

The structural depth features a redesign of the gravity system, lateral system and a new design for both blast protection and progressive collapse. It was desired to explore a two way concrete flab slab as a gravity redesign due to its feasibility which was determined from the systems comparison. From this comparison it was found that the two way concrete flat slab was the cheapest and shallowest system which allowed for more floor to floor height compared to other systems. Due to the redesign of the gravity system the lateral system would see slightly different loading and therefore also needed to be analyzed. Blast protection of members was added to the structural depth due to personal interest. This analysis explores what the structure would need to resist if a bomb were to go off both outside and inside the building. As a result of the loads generated by analyzing blast protection from an interior explosion it was determined that a progressive collapse design is more feasible for an interior detonation. All calculations for this study are located in the appendices.

## 6.2 Gravity Redesign

### *Slab*

The two way flat slab was designed both by hand and by spSlab. From ACI 8.3.1.1 the minimum thickness for the slab is 10” however it was chosen to increase this number to 11” for additional moment capacity. Due to the nature and geometry of the bays and in accordance with ACI 8.10.2 the slab could be designed using the direct design method. The design for the controlling bay, with the longest span and tributary area, was designed for both and interior and exterior. Hand calculations yielded results for interior bays to be reinforced with #9 at 12” both top and bottom within the column strip and #7 at 12” both top and bottom everywhere else. Exterior bays shall be reinforced with #5 at 12” both top and bottom everywhere. Top reinforcement for both interior and exterior shall extend to a minimum of 0.3 x span within the column strip and 0.22 x span within the middle strip as in accordance with ACI 8.7.4.3.1. Figure 27 shows the designed layout for positive moment reinforcement for both an interior and exterior bay.

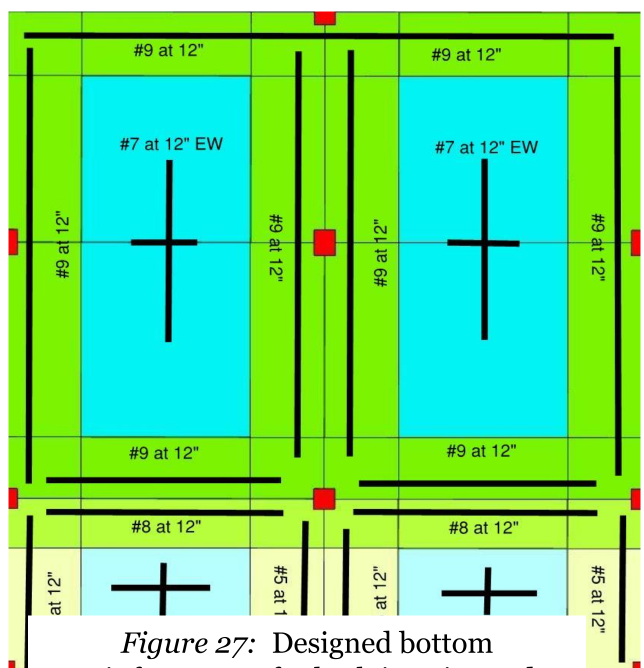



*Figure 27:* Designed bottom reinforcement for both interior and exterior bays

This design was also checked with spSlab and the results for reinforcement matched closely with that designed by hand. As confirmed by Figure 28 which shows that 13 #8 bars spaced evenly across the column strip will work for the design. Figure 28 corresponds to the vertical middle column strip in Figure 27. The results shown below are very similar to what was calculated by hand which was #9 bars spaced at 12" which equated to 13 bars within a column strip width of 13'.

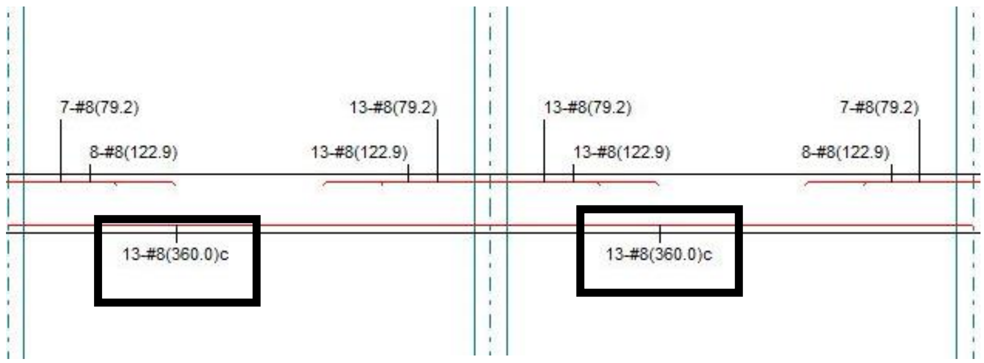



*Figure 28:* Reinforcement detailed from spSlab to be compared with Figure 27.

This software was also checked against the moments that were generated from the direct design method and found to be within 10% of each other and thus the software can be used. Additionally a RAM model was made of the slab system using #8 bars. The results from the model further confirm the feasibility of using #9 bars for if the design works with #8 then it will also work for larger reinforcement. As shown in Figure 29 the design is sound as shown by the green reinforcement. It can however be noted that the design strips were off in the model due to there being reinforcement running through an opening in the slab. However when the reinforcement detailing is further analyzed it is found that neither top nor bottom bars protrude into slab openings as shown in Figure 30.

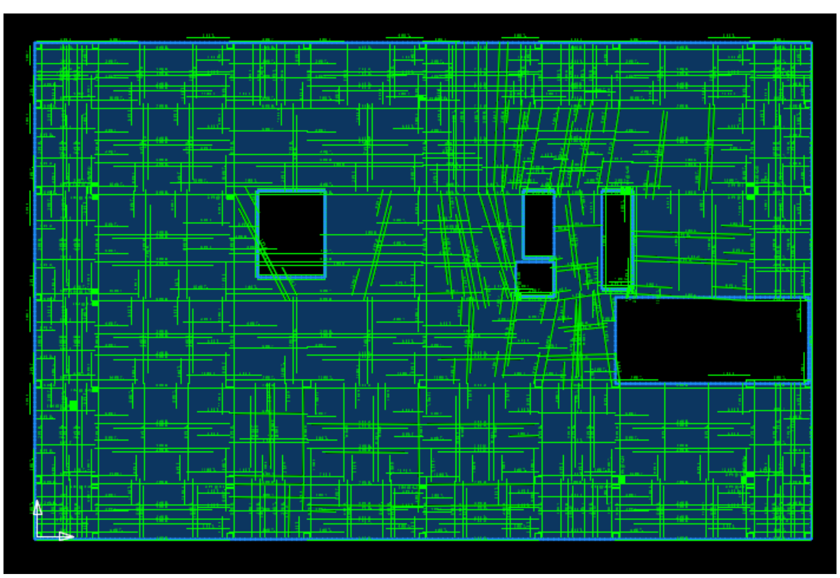
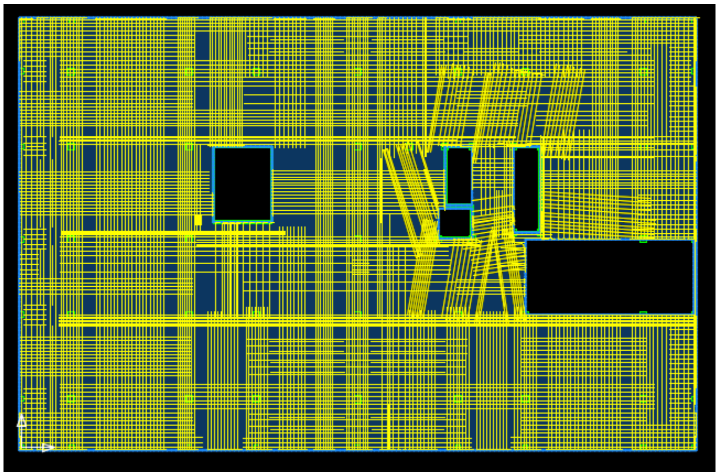
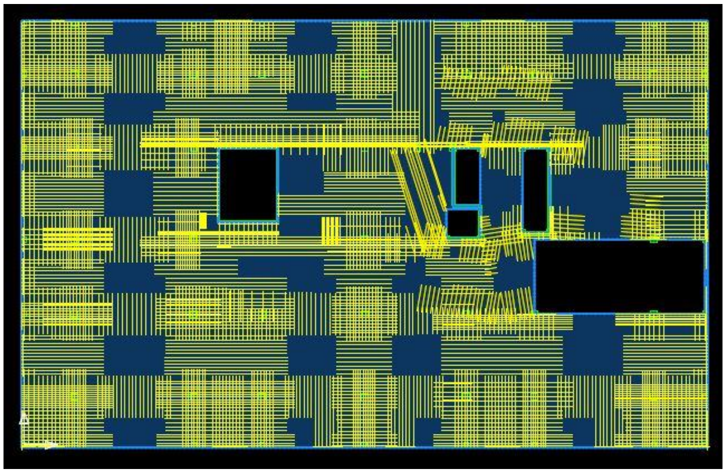


*Figure 29:* RAM concept model of the design status of reinforcement in the modeled two way slab.



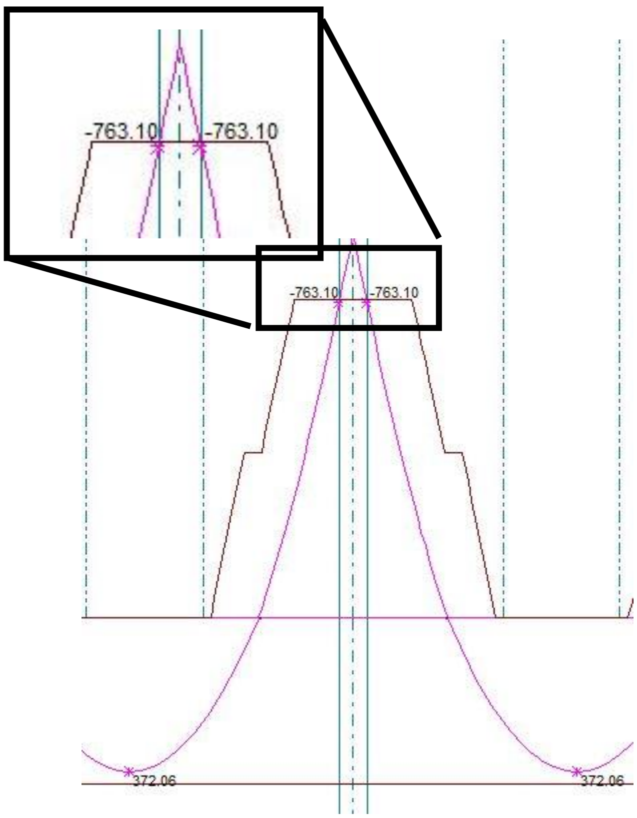
*Bottom*



*Top*

*Figure 30:* RAM concept model of the detailed individual bottom and top bars.

The two systems, spSlab and RAM, are correct and accurate because it can be shown that their values for max moment at the same particular spot are similar and differ by less than 10% as shown in Figure 31 where the max negative moment is -763 kipxft for spSlab and -782 kipxft for RAM. This validates both models and confirms the hand calculations for the gravity system as well. As a result of this new gravity system there are new gravity loads for which the columns and shear walls need to be designed for.



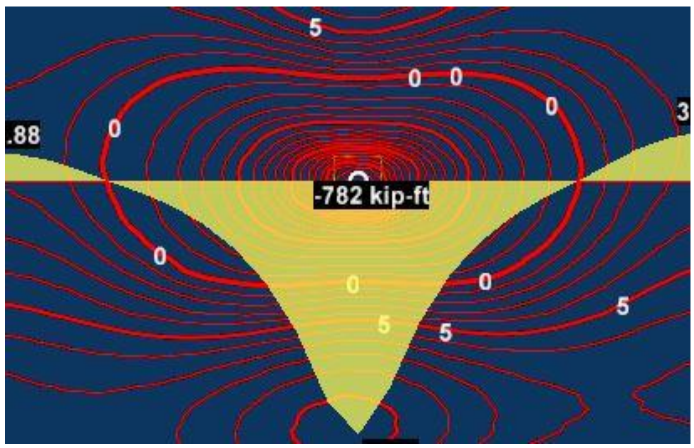
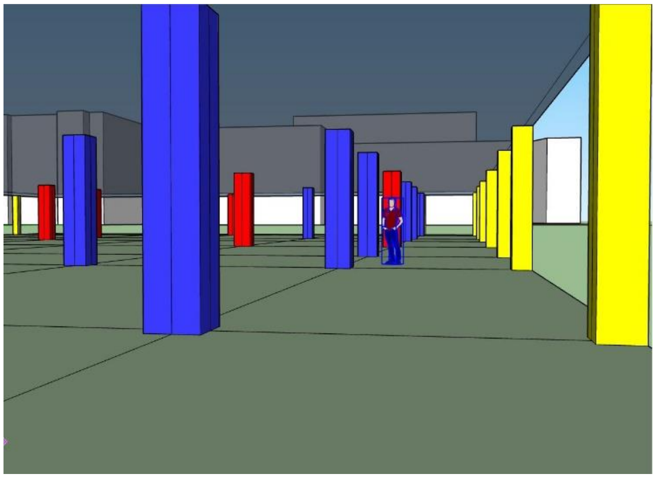




*Figure 31:* Comparison of moments at the same location for both spSlab and RAM models.

### *Column*

From initial inspection of the floor plan it was determined that if a two way flat slab were to be used then either perimeter beams or columns would need to be added to support the 15' cantilevers around the exterior of the building. Due to the overall height restriction and the attempt to not change the average floor to floor height it was chosen to add perimeter columns into the gravity design instead of beams. Both systems are shown below in Figure 32 which shows the special differences between the two systems.



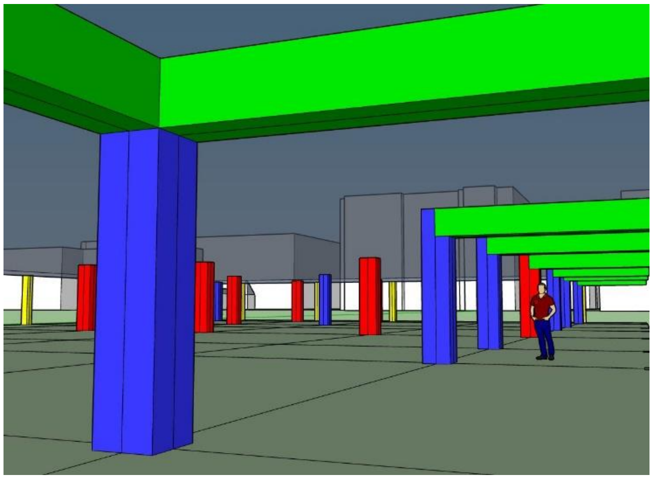


*Figure 32:* Comparison of the two potential gravity systems cantilever beams or exterior columns.

Due to the exterior column system being chosen there are three different types of columns in this design based upon loading which are exterior, regular interior and large interior. These columns change their compressive strength as the building rises in height as shown in the graph display in Figure 33 which compares the original and the new compressive strengths per floor. This idea was from the original gravity system which changed the compressive strength of the columns as the building ascends.

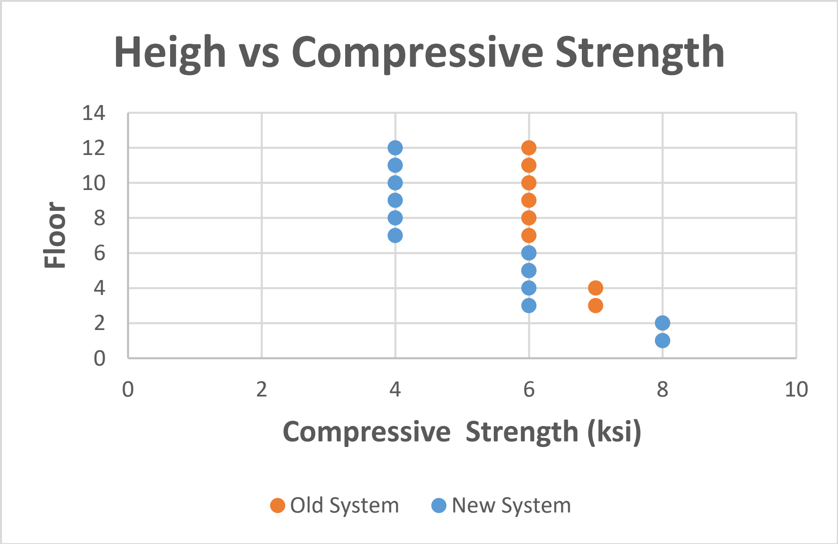



*Figure 33:* Graph representing the compressive strength of the columns per floor for both old and new systems.

Columns were designed both by hand and by the software spColumn. The software was checked against the hand calculated pure axial capacity and if it was within 10% it was deemed acceptable. Designs for exterior columns are 18”x18” with 8#6 bars and #3 stirrups at 12”. The regular interior columns are 24”x24” with 8#8 bars and #4 stirrups while the large interior columns are 24”x30” with 12#8 bars and #4 stirrups. These designs are shown below in Figure 34 which displays details of each column section. The location of these columns are displayed in plan on Figure 35. Slenderness effects were neglected in this design in accordance with ACI 10.12.2. Reinforcement was limited from ACI 10.6.1.1 and as such minimum longitudinal and shear reinforcement were required.

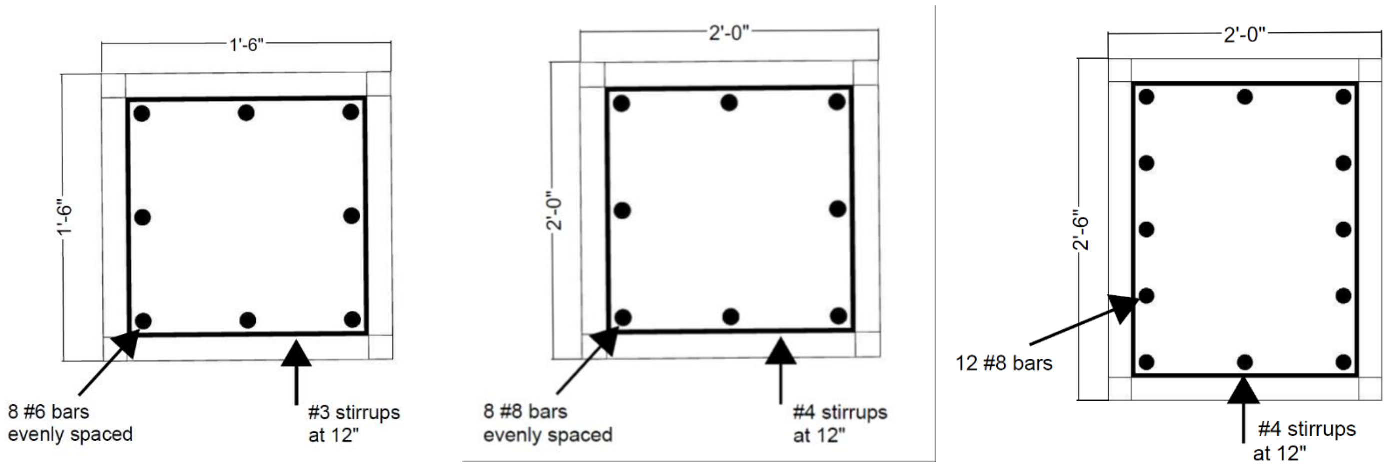



*Figure 34:* Details of the three columns designed. 18x18, 24x24 and 24x30.

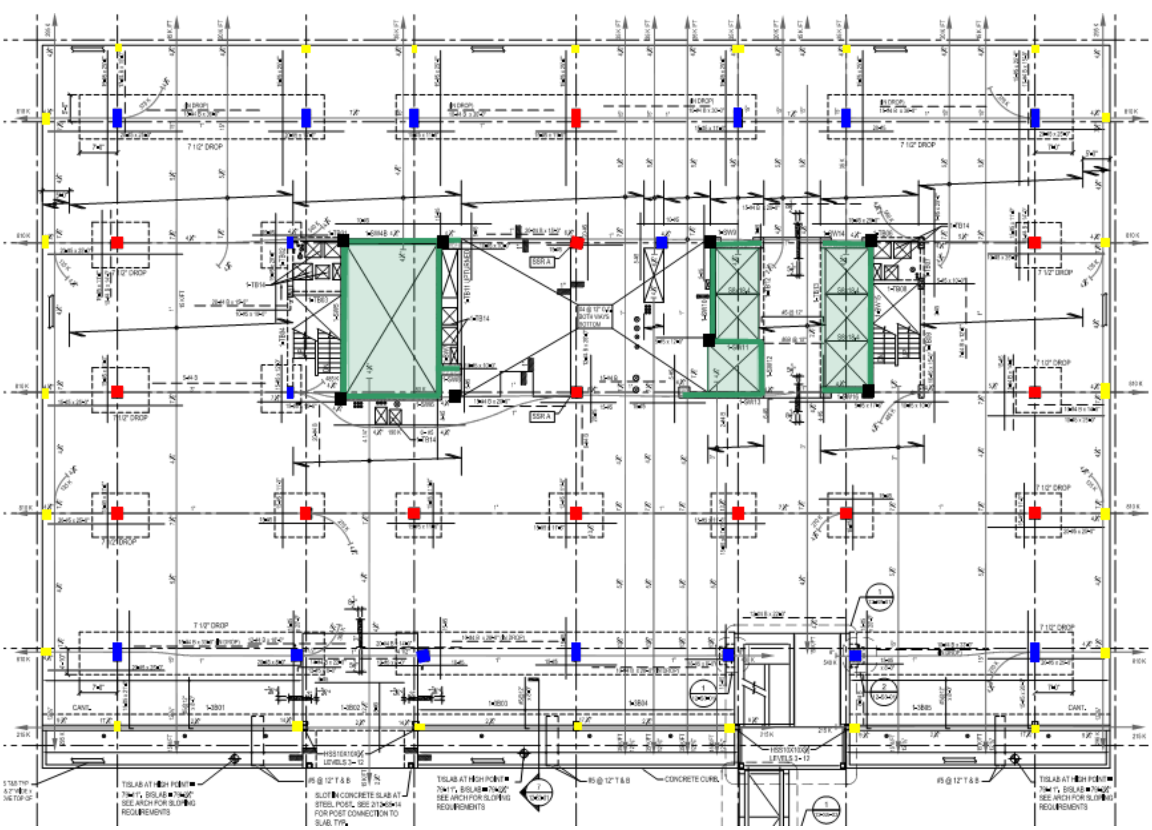


*Figure 35:* Plan view of the location of each type of column.

*Figure 35 key*

18x18 (yellow)

24x24 (blue)

24x30 (red)

## *Shear Walls*

The shear walls were previously analyzed as a part of notebook C. In the aforementioned report the some of the shear walls failed. The controlling lateral case was the flexural capacity due to seismic loads. It is most likely that the dead load on these shear walls was not what the engineer of record had used. On top of the shear walls sits mechanical equipment for the building. It is likely that this load would have caused an increased flexural capacity of the shear walls. The design of shear walls for the new gravity system will not use an approximation of this mechanical equipment and therefor the shear walls need to have an increased capacity. Before the design of the new shear walls it was desired not to alter the architecture or special layout of the existing building and therefor the length and location of each shear wall will not be changed. The other properties that can be altered to increase the capacity of the walls is then to increase the thickness and the longitudinal steel. The design for typical shear walls for the new system consists of a 12" thick section with minimum longitudinal and transverse reinforcement of #5 bars at 24" and with 10 #10 60 ksi bars for shear walls in the N-S direction and 10 #10 80 ksi bars in the E-W direction Figure 36**.** It was assumed that 80 ksi steel is not difficult to get nor is it vastly more expensive than 60 ksi steel. The only issue with constructability is the potential for the 80 and 60 ksi bars to be confused with each other.

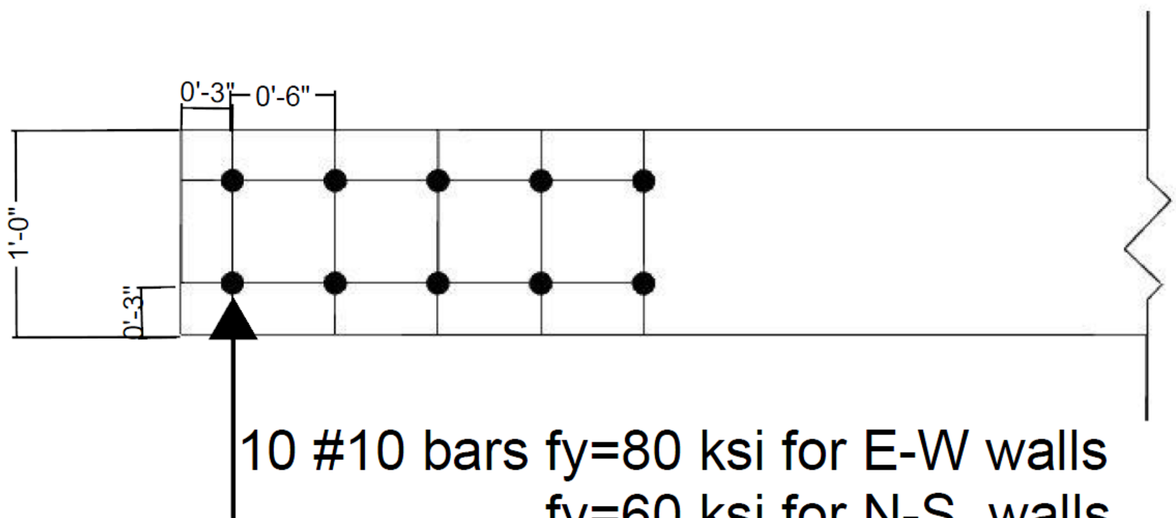



*Figure 36:* Detail of shear walls at the boundary element.

## 6.3 Blast Design

### *Introduction*

The effects of a bomb going off outside or inside the building were investigated as a portion of this structural depth. The main source of information and data for this design was a book called *Blast Resistant Design Of Buildings*. This book heavily references the government document UFC 3-340-02, structures to resist the effects of accidental explosions, created by the Department Of Defense in 2008. Firstly the size of the bomb was approximated from research on typical car bombs and smaller scale bombs. Probable locations that these bombs could be set off at were then determined. Finally the loading effect of these bombs were then calculated and the structure was then designed around such loads.

### *Intensity*

The bomb intensity is related to the equivalent mass of TNT of the explosion. There are a multitude of different types of explosions each with a different equivalent masses of TNT. From research it was decided to choose a relatively small size of bomb to analyze whilst still being practical. Buildings designed for blast protection do not have the same parameters as One City Center. Most buildings would not have an underground parking garage open to the public or have public access to the first floor of the building. Furthermore is presumed that even if this building housed government agencies it would not be under a high degree of threat. It is from these reasons that a smaller bomb has been chosen for analysis. Through internet research it was determined that a typical car bomb has the equivalent TNT mass of 5kg. The 5kg bomb was then analyzed at several locations.

### *Location*

The location of the bomb greatly effects the pressures on the structure. Initially it was desired to analyze several scenarios for a bomb going off on both the exterior of the building and the interior of the building. Originally two locations were thought to be probable for detonation on the exterior of the building as shown in Figure 37.1. After the analysis of these exterior locations and the loads they generated it was determined that the loading would be almost the same as if the bomb were on the inside of the building. Thus it was desired to rethink the location of an exterior detonation to a place more probable and more feasible for design. From this thought process the location shown in Figure 37.2 was determined.

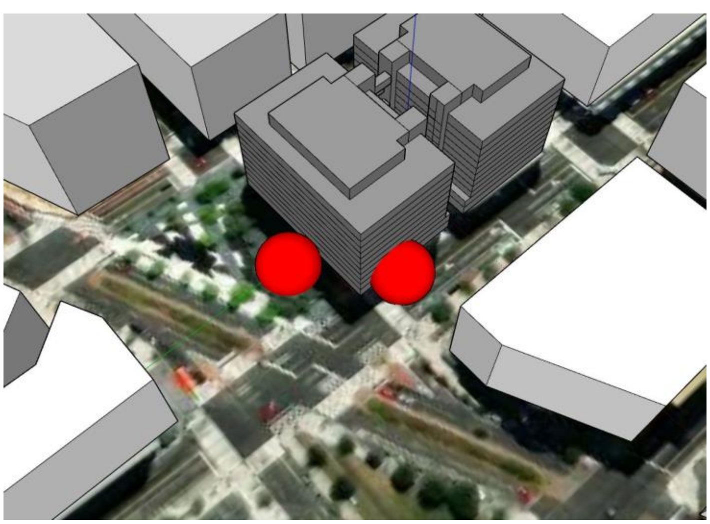


*Figure 37.1:* Original location of bombs detonation outside the building.

This location is at a major intersection in the city and it is adjacent to a small park which makes this location more probable for a bomb to go off.  The interior locations were then analyzed.

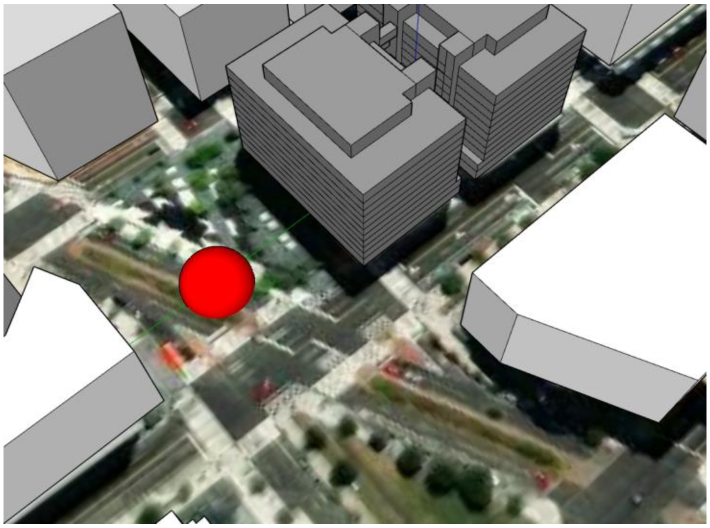


*Figure 37.2:* More probable and feasible location for an exterior detonation.

OtherAn interior detonation was considered for various places inside the building.  It was presumed that the building security would monitor any vertical means of egress such as stair wells and elevator shafts.  This would limit the interior explosion to the first floor.  The initial locations of an interior detonation are shown in Figure 38 below.  These locations were based on both structural parameters and probability parameters.  The structural locations are shown in blue and they would analyze the effect of a bomb on the column or shear wall with the most load.  The probability locations are shown in red and are based on the architecture and functions of the space.

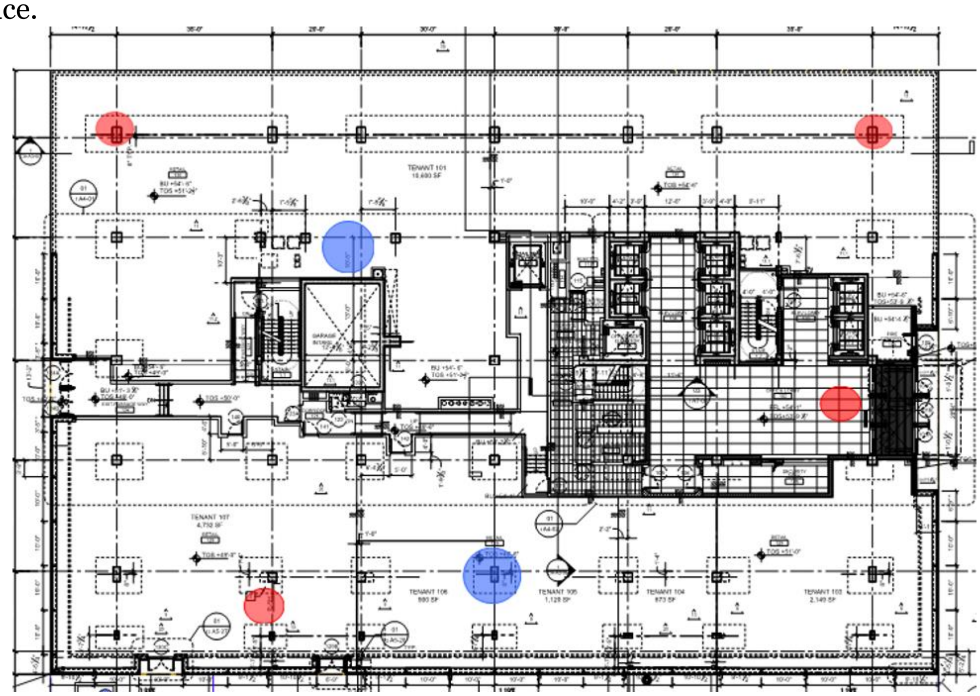


*Figure 38:* Locations for interior detonation based on structure and probability.

## *Effect*

The effects of each bomb is dependent on the weight (intensity) and distance (location) of the explosion. The loading effect is similar to that of a lateral wind load. However it is important to note that the façade of the building will not hold most of the lateral pressures. Furthermore the lateral pressures and their loading on lateral elements such as shear walls cannot be empirically determined. The pressures as a function of weight and distance were determined from the graph shown in Figure 39. This graph works by calculating the scaled distance which is a function of the weight and distance of the bomb and then determining the various parameters from the graph.

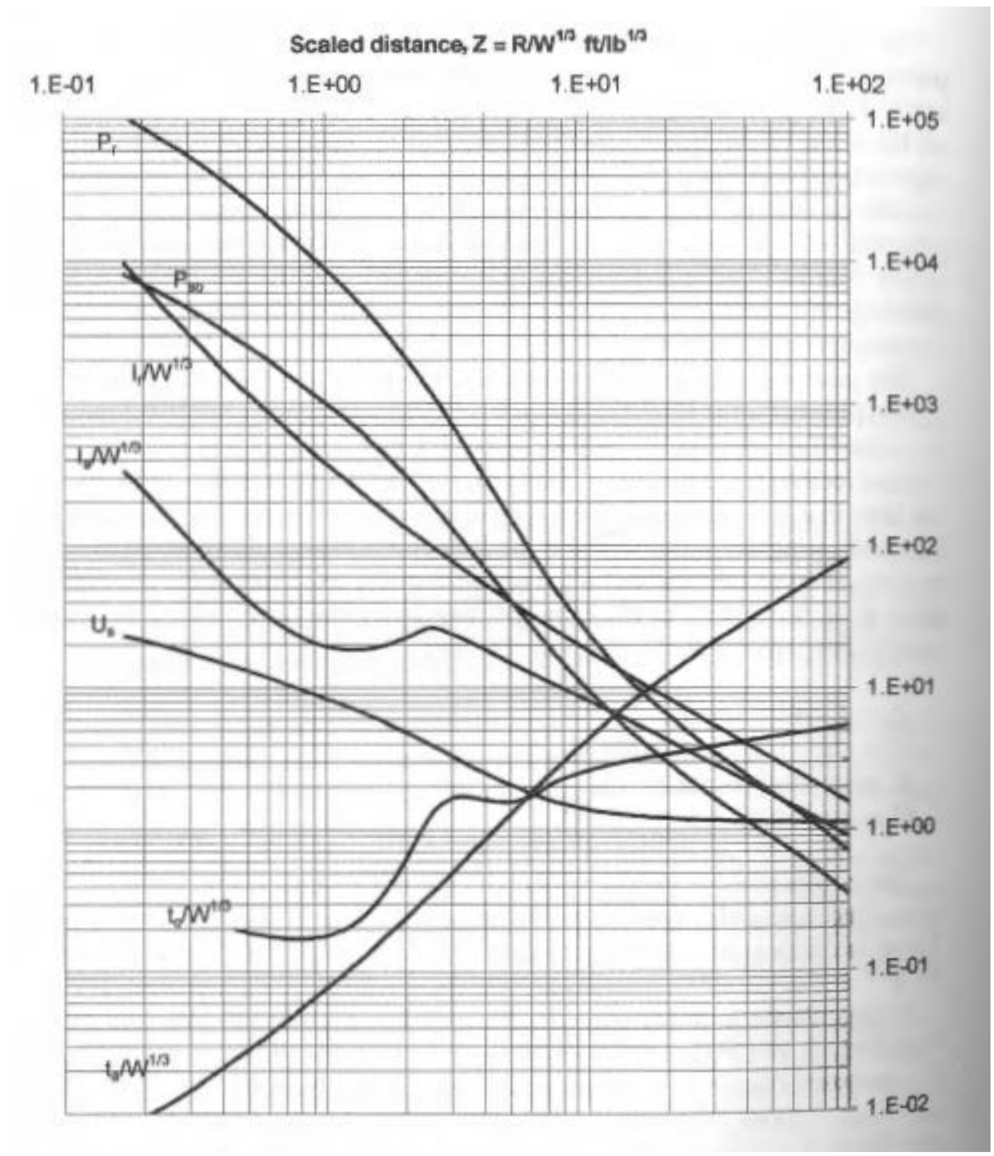



*Figure 39:* Graph of blast parameters such as pressure and time as a function of the scaled distance.

*Figure 39 key:*

$P_r$ = Reflective Pressure

$P_{so}$ = Side-on overpressure

$U_s$ = Shock wave velocity

$t_o/W^{1/3}$ =

Phase duration/bomb weight

$Z = R/W^{1/3}$ =

Distance of bomb/Weight of bomb

It was then determined how pressures from a bomb act on the structure. As shown in Figure 40 the shock wave of a blast creates both positive and negative pressures as a function of time. The chosen method for calculating these pressure is empirical by estimating the positive pressure or impulse as a straight line approximation shown in blue on Figure 40. The negative impulse and pressures are neglected in that these pressures are relatively small compared to the positive pressures and that they are more difficult to approximate and model. It is important to note that the units for typical pressures are in pounds per square inch whilst the units for time are in milliseconds. For exterior pressures the building sees a frontal, roof, side and rear pressures similar to wind. These pressures are shown in Figure 41 below.

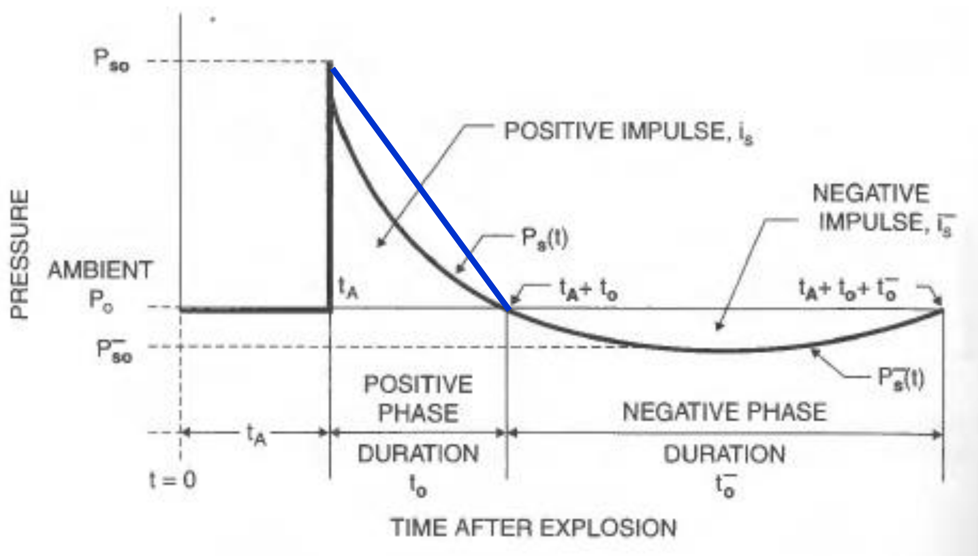



*Figure 40:* Graph of the impulse vs time of a blast. The blue line represents the linear approximation of positive impulse.

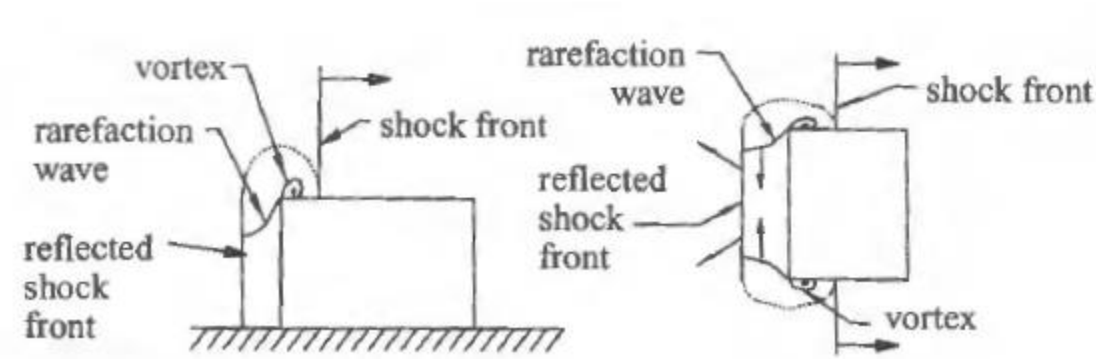



*Figure 41:* Image on how the pressure created by a blast can create additional pressures on the side and roof of a structure.

The pressures created by the blast weight and location outside the structure, as mentioned above, did not generate overly large lateral pressures. As shown in Figure 42 the lateral pressure acting on the first floor exterior column is analyzed with the shown load. The moments created by this loading are not significant and the member itself can handle much more moment on it. Furthermore the distance of the exterior bomb is such that the roof, side and rear pressures are nonexistent. Thus it was found that the main parameter that controls the load created by the bomb is the distance between the bomb and the structure. Therefor buildings that are to be designed for exterior blasts should try and create a perimeter to keep the bomb as far away from the structure as feasibly possible. Unfortunately such a perimeter cannot be created if the bomb were to go off on the inside of the building.

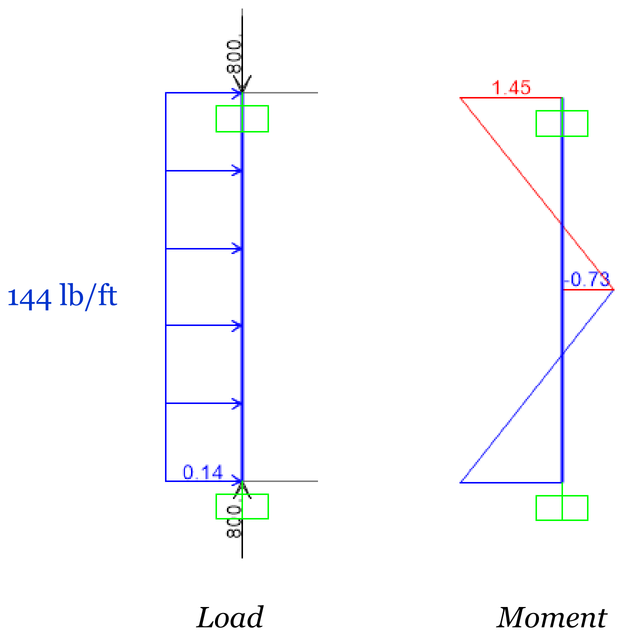



*Figure 42:* Loading and moment diagrams of and exterior column under blast.

Interior pressures can see both impulse and reflective pressures. Reflective pressures are such where if the bomb does not blow open the façade and create a hole for the pressure to escape it can reflect off of surfaces and onto others creating additional loads. It is important to note that reflective pressures were neglected for this analysis due to the inability to predict more than the first reflection. Furthermore it was assumed that the façade would give way thus venting the bomb and allowing there to be little to no reflective pressures. Upon further analysis of an interior detonation it was discovered that members would be unreasonably large and improbable to put in the building. It was from this analysis that it was determined to perform a progressive collapse design instead of blast design for an interior bomb.

## 6.4 Progressive Collapse Design

### *Introduction*

Due to the member sizes and strengths that would be necessary for an interior explosion it was desired to perform a progressive collapse design. This design assumed that there is only ever one bomb going off inside the structure. This design also neglects the deflection criteria for service due to the assumption of the building being evacuated relatively soon after the bomb goes off. From UFC-04-023-03 different progressive collapse design criteria must be followed for different occupancy levels. Due to the unclear nature on how the document defines occupancy the specified minimum was chosen. The design for this minimum is an alternate load path for specific columns and walls upon removal. Overall this method asserts that if a bomb were to go of inside the building the nearest column would be blown away and therefore the columns above it would still have to be supported. For this analysis it was designed such that the range of the bomb would take out the column floor joint on the first levels ceiling. Therefor it is the job of the second floors ceiling to support the columns above. This can be seen in Figure 43 where the second floor is destroyed and the third level needs to support the rest of the structure.

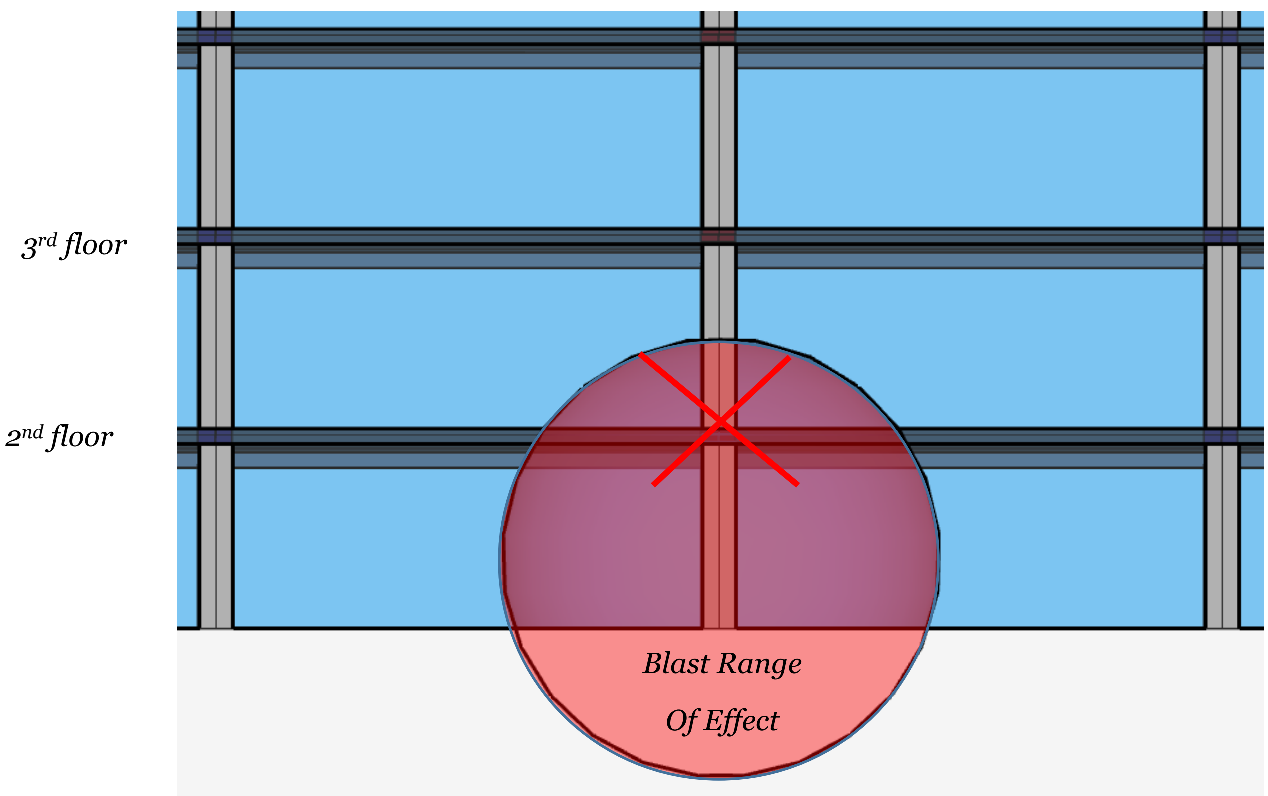



*Figure 43:* Image showing the blast range of an internal explosion and effect on column to slab joint.

It is important to note that the blast range for Figure 43 is for the assumed 5 lbs of TNT assumed earlier. Therefore it is still possible for more than one column to be eliminated if the bomb is great enough. This design does not look at the effects of a variety of different bombs and potential designs for each scenario. Rather this design explores the potential to resist collapse if a column joint were eliminated due to 5 lbs of TNT. The range that a column could survive a 5 lb detonation was then analyzed. By determining the columns moment and shear capacity as an axial member with lateral load it was found that the designed columns could withstand a 5 lb explosion from 9 ft away.

## *Design*

Two system were conceived to best support this scenario. These system were based on how much height the owner and architect would be willing to sacrifice to resist collapse. The first would be Intermediate columns placed halfway between existing supports with a post tensioned beam and girder system to resist the axial load. Intermediate columns would be the same size and have the same reinforcement in them. This design was calculated through analysis of the most stringent bay. That is to say the bay with the largest tributary area, for that would generate the largest spans and column load for the system. This bay is shown below in Figure 44. PT (post tensioned) beams would be 18x36 in dimension with 4 legs of #5 bars spaced 12" apart for shear reinforcement. The tensioning in these beams are 15 half inch cables stressed together at 425 kips. Intermediate beams in this system are 18x24 with no shear reinforcement required stressed with 10 half inch cables at 282 kips. The additionally added intermediate columns can be seen in red on Figure 44. There are 4 intermediate beams and 6 main beams as shown in the figures below.

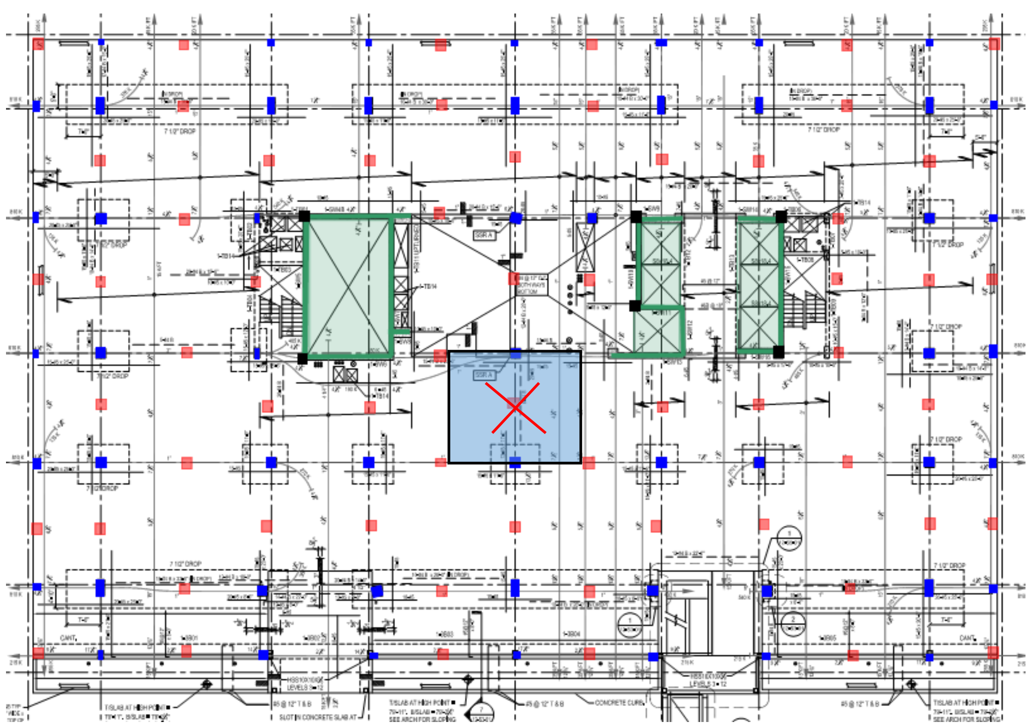
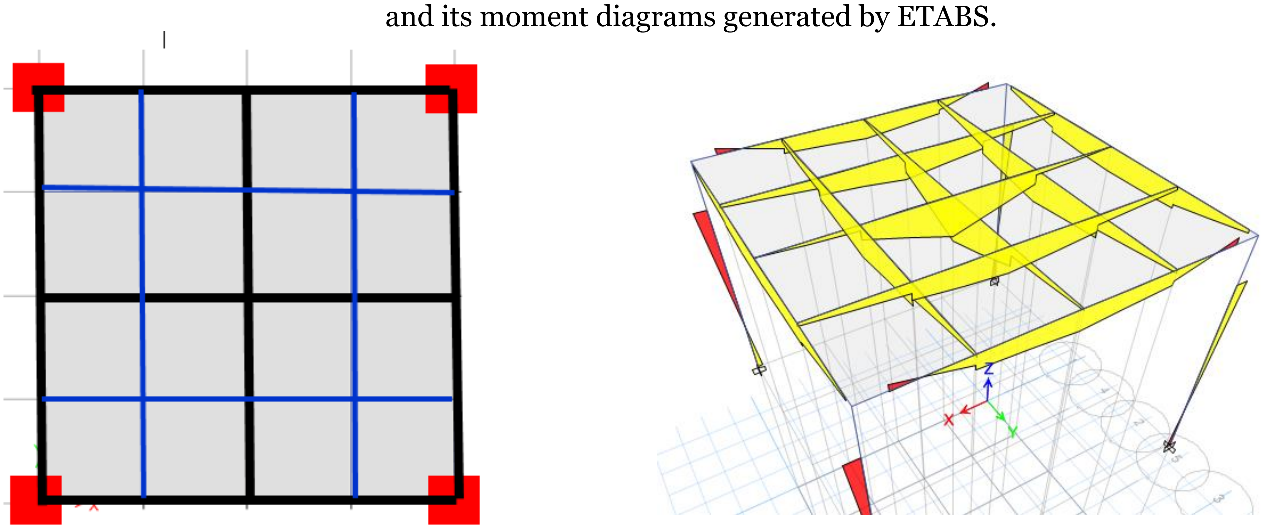


*Figure 44:* Plan view of the additional, intermediate columns added for system 1. These additional columns are shown in red.

The designs for this system was conducted from taking moments from ETABS and designing by hand the PT beams to meet the flexural, shear and stress requirements. The moments from ETABS can be seen in Figure 45 and the design details from these results are shown in Figure 46.

*Figure 45:* Close up view of the bay to be analyzed and its moment diagrams generated by ETABS.

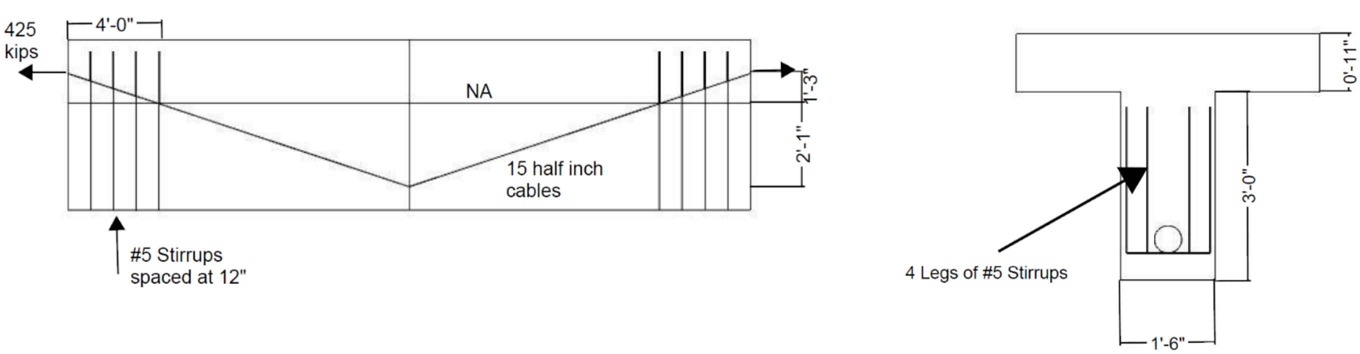


*Main Beams*

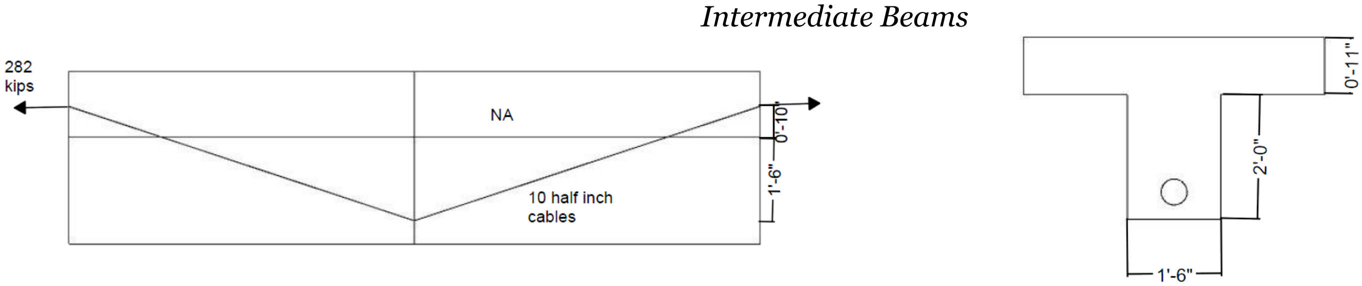
*Intermediate Beams*

*Figure 46:* Details for PT beams for system 1.

*Main Beams*

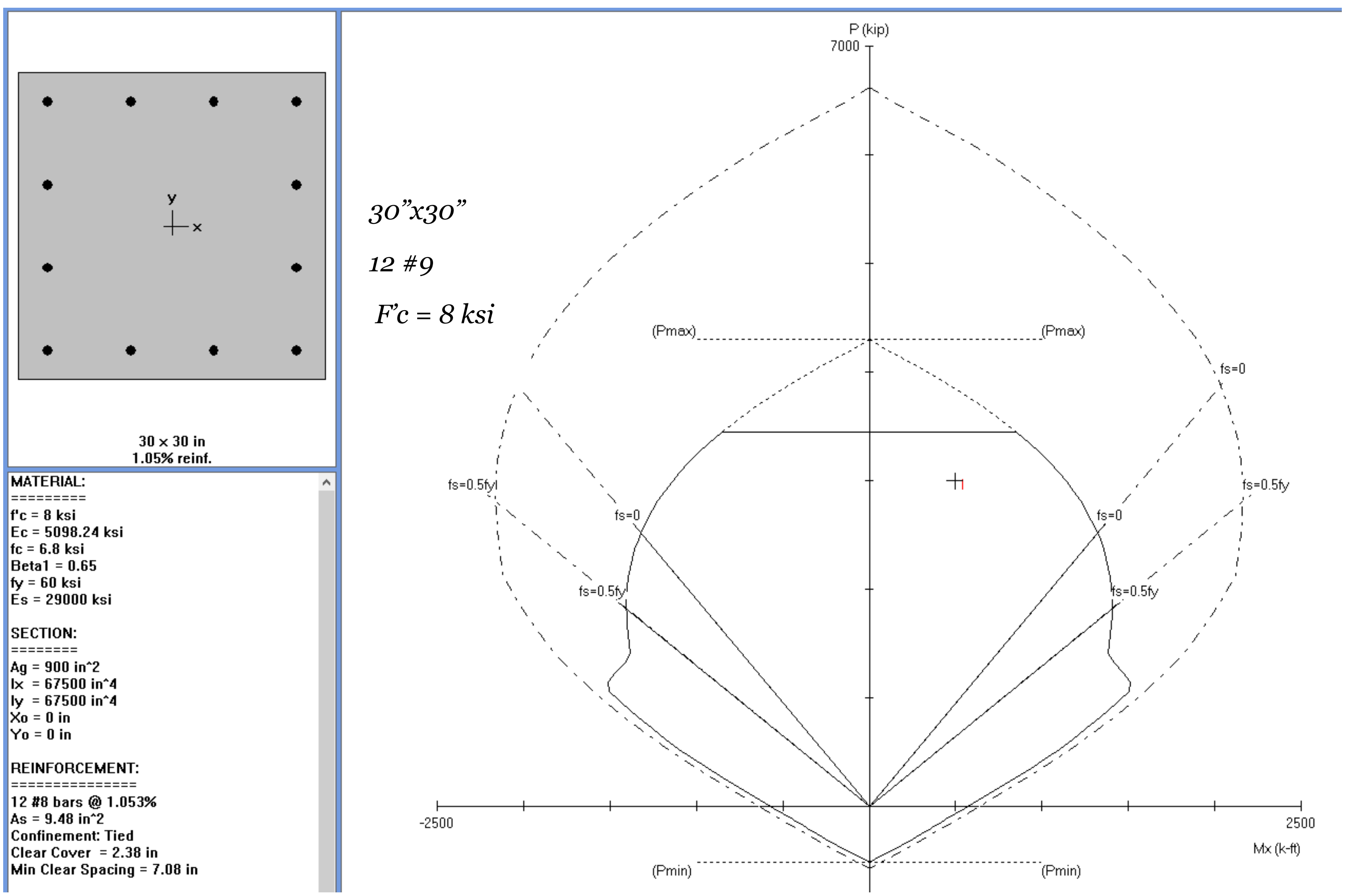



*Intermediate Beams*



This analysis also creates additional load on the columns surrounding the column that is assumed to be lost. Furthermore there would be additional moment in these members as well. As a result of these additional loadings new columns need to be designed. It is important to note that the moment on the column from dead load does help mitigate the moment created by the stand alone column. However the column needs to be designed for this new moment none the less. The design generated from spColumn is shown in Figure 47**.**

*Figure 47:* Interaction diagram of the redesigned column to resist the additional loads caused by a lost column (system1).



The second system would not feature intermediate columns but would however be deeper overall. This system, similar to the first, was determined by analyzing the same bay that had the largest tributary width, spans and column load. A plan view of the bay under consideration is shown in Figure 48. The main beams would be 24x54 in dimension with no necessary shear reinforcement stressed with 20 half inch cables at 565 kips. The intermittent beams are 24x48 in dimension with, similar to the previous beam, no shear reinforcement stressed with 15 half inch cables at 425 kips. The moments from ETABS can be seen in Figure 49 and the design details from these results are shown in Figure 50**.** It is important to note that due to the increase of overall unsupported area, compared to system 1, more intermediate beams are necessary. There are 12 intermittent beams spaced at 5' in this system and 6 main beams also spaced at 5' as shown in the image below.

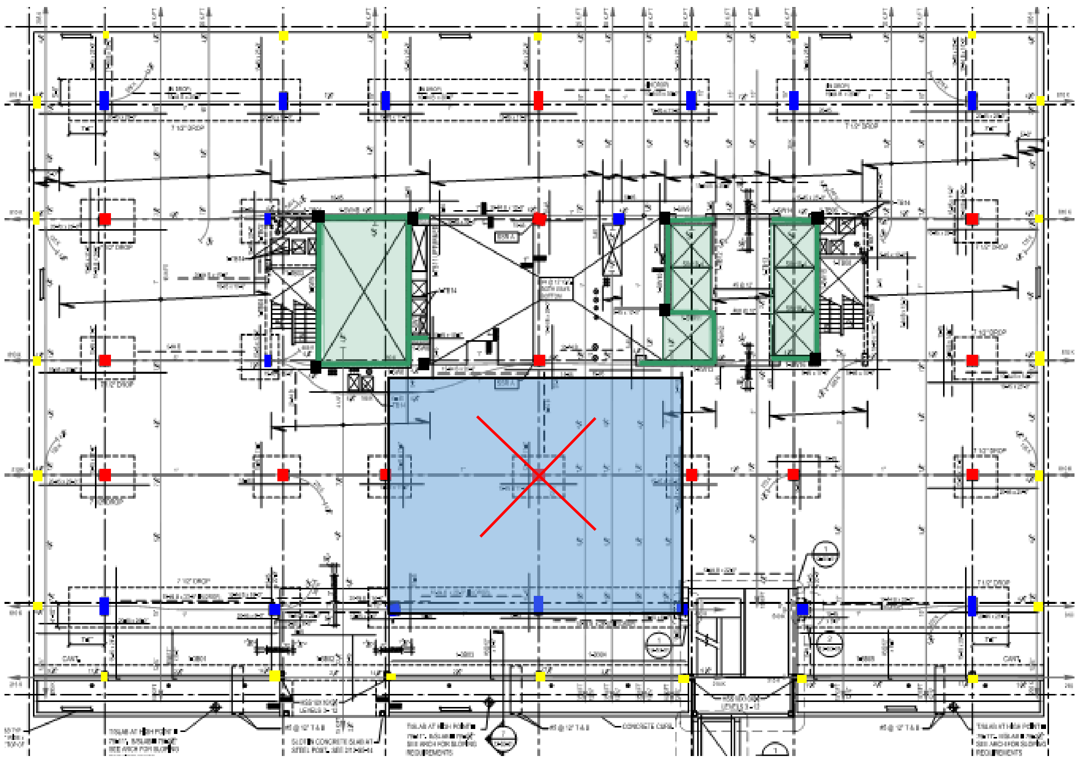
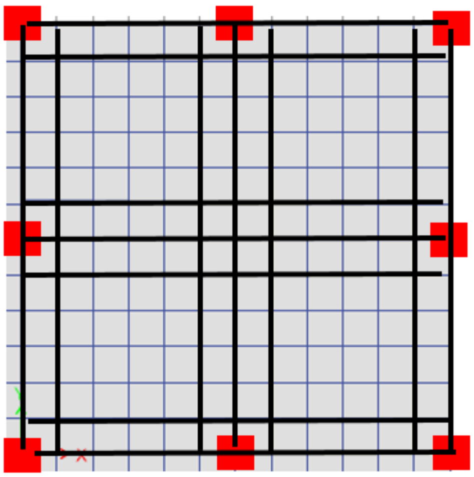
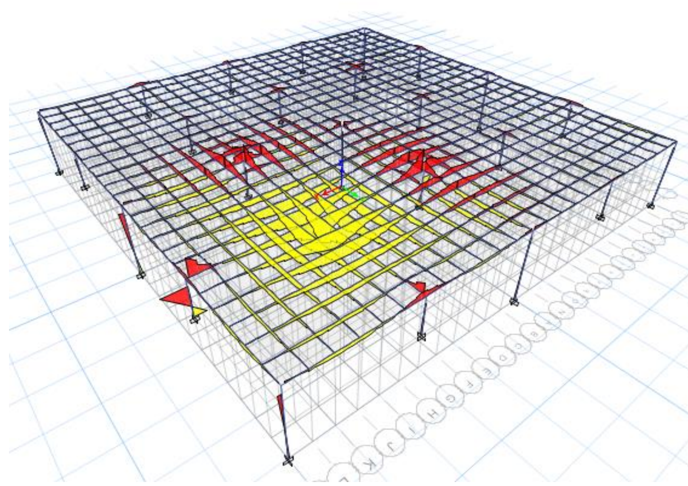


*Figure 48:* Flor plan showing the bay analyzed for system 2.

*Figure 49:* Close up view of the bay to be analyzed and its moment diagrams generated by ETABS.



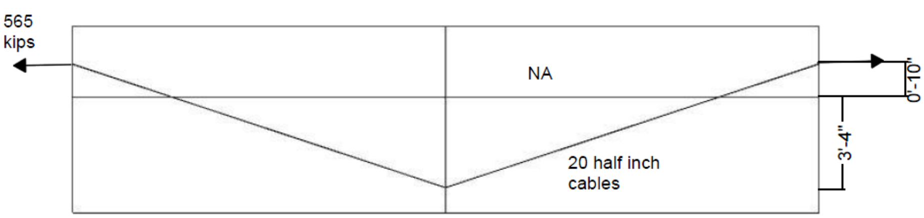


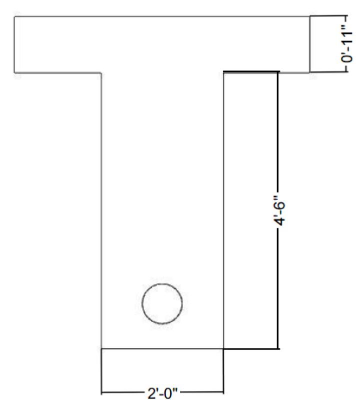
Main Beams

Intermediate Beams

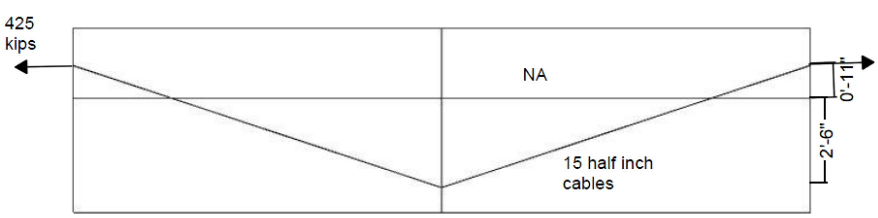
*Figure 50:* Details for PT beams for system 2.

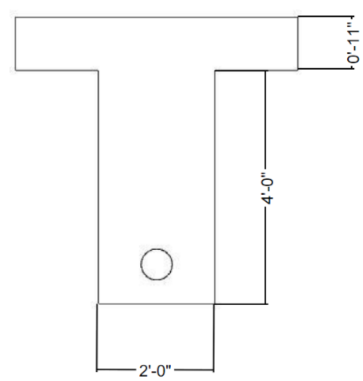
*Main Beams*



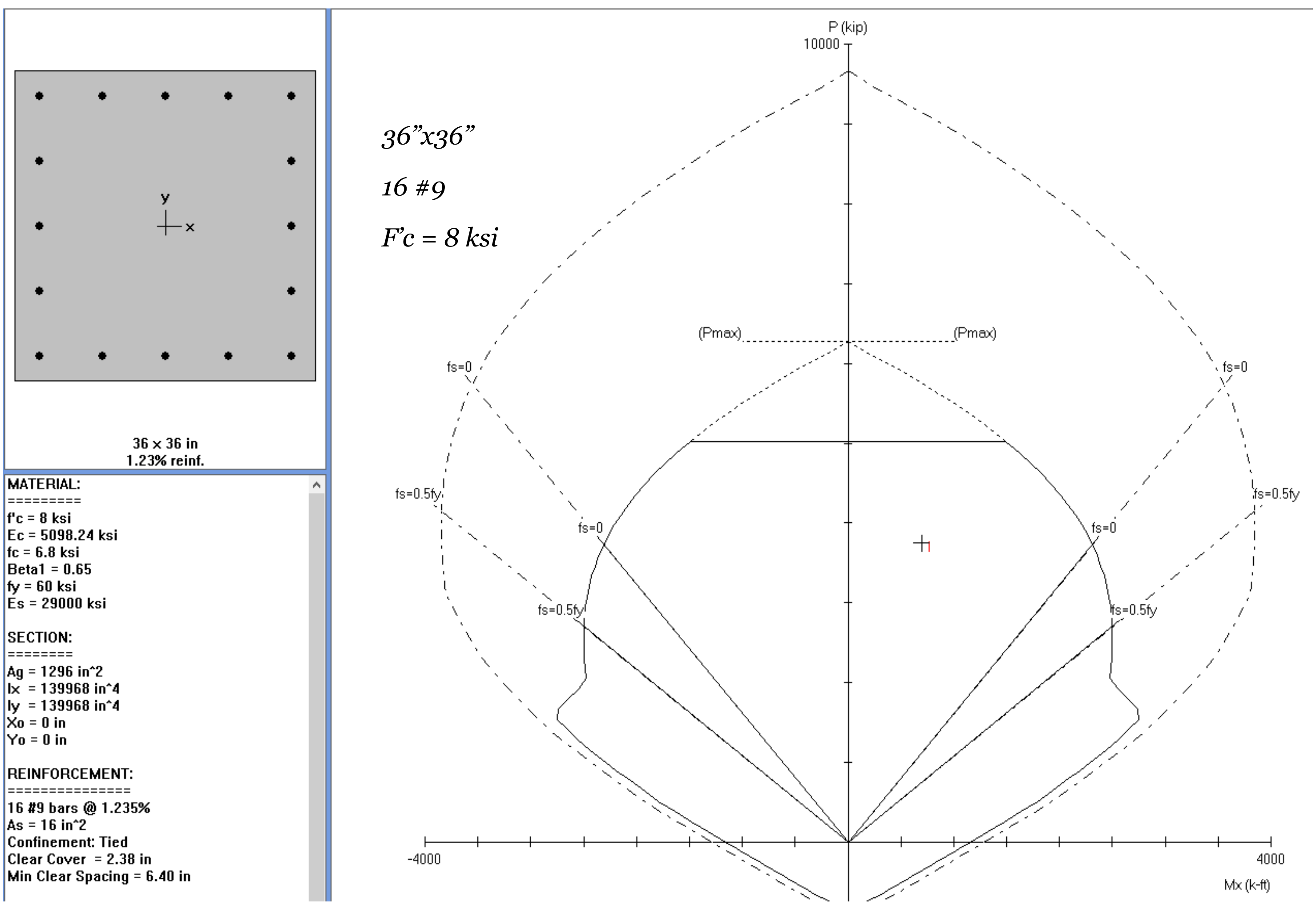

*Intermediate Beams*



Similar to system 1 the gravity loads in system 2 are increased due to the lost column. This puts new moment and axial loads on the interior columns. The effect of adjacent bays and their effect on the moment of interior columns was modeled. While this did lessen the additional moment it did not eliminate the need to redesign the column. This redesign was performed through spColumn and its interaction diagram and detail is shown below in Figure 51.

*Figure 51:* Interaction diagram of the redesigned column to resist the additional loads caused by a lost column (system 2).



While the two systems both work for the progressive collapse scenario they each have their own disadvantages and advantages. The first system does not require as much height to be taken away from the building while it does have intermediated columns thus making the first and second floors more crowded. There is a greater risk for these intermediate columns to collapse if a large enough bomb is detonated due to the range at which these members can survive a 5 lb bomb. The second system requires more depth to be sacrificed however it does not infringe upon existent spaces with more columns but it does require thicker columns at the lower levels. Figure 52 shows the spatial differences between the gravity system and the two progressive collapse systems. Each system first takes away height from the second floor until it is at 11' then additional height is taken away from the bottom floor. This was done because it was assumed that the owner or architect wanted the maximum floor to floor height on the bottom floor. The cost of these systems is analyzed in the construction breadth. From that analysis system 1 is cheaper due to less materials being used. However if an owner or client relied upon the structural engineers advice the chosen system would be 2. This is because even though there are no intermediate columns the system has more redundancy. The intermediate columns are more at risk than the others. This is seen in Figure 53 where the range of a bomb, if sizable enough, could take out multiple columns. As shown in the figure the smallest ring is the range of the analyzed 5 lbs of TNT. It is always possible for there to be a larger bomb with a larger range of effect.

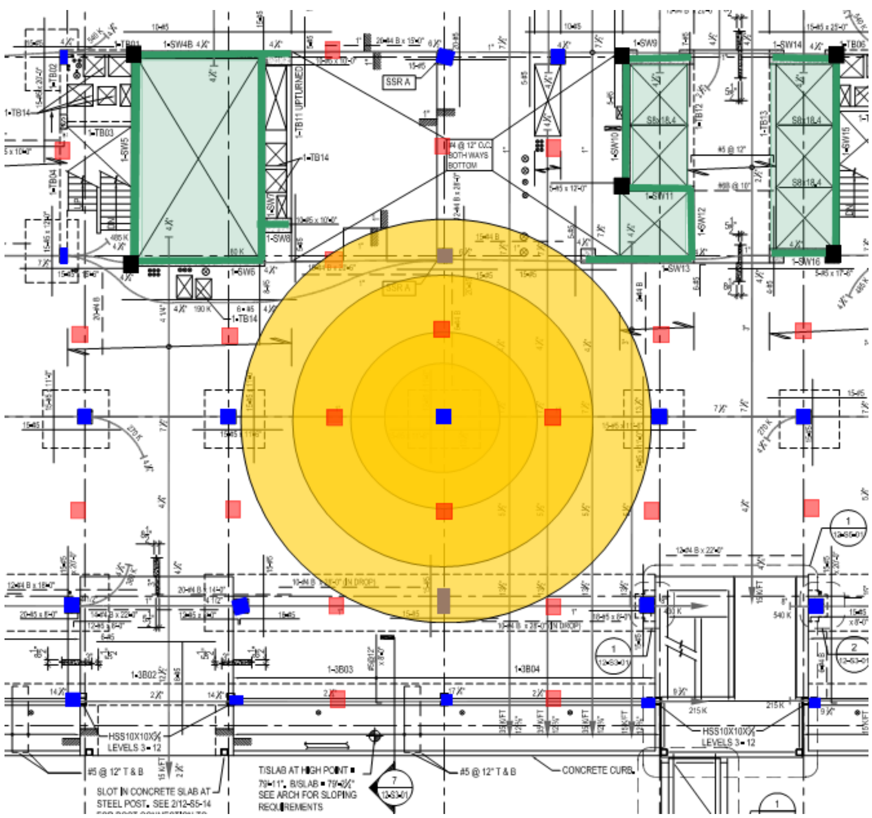
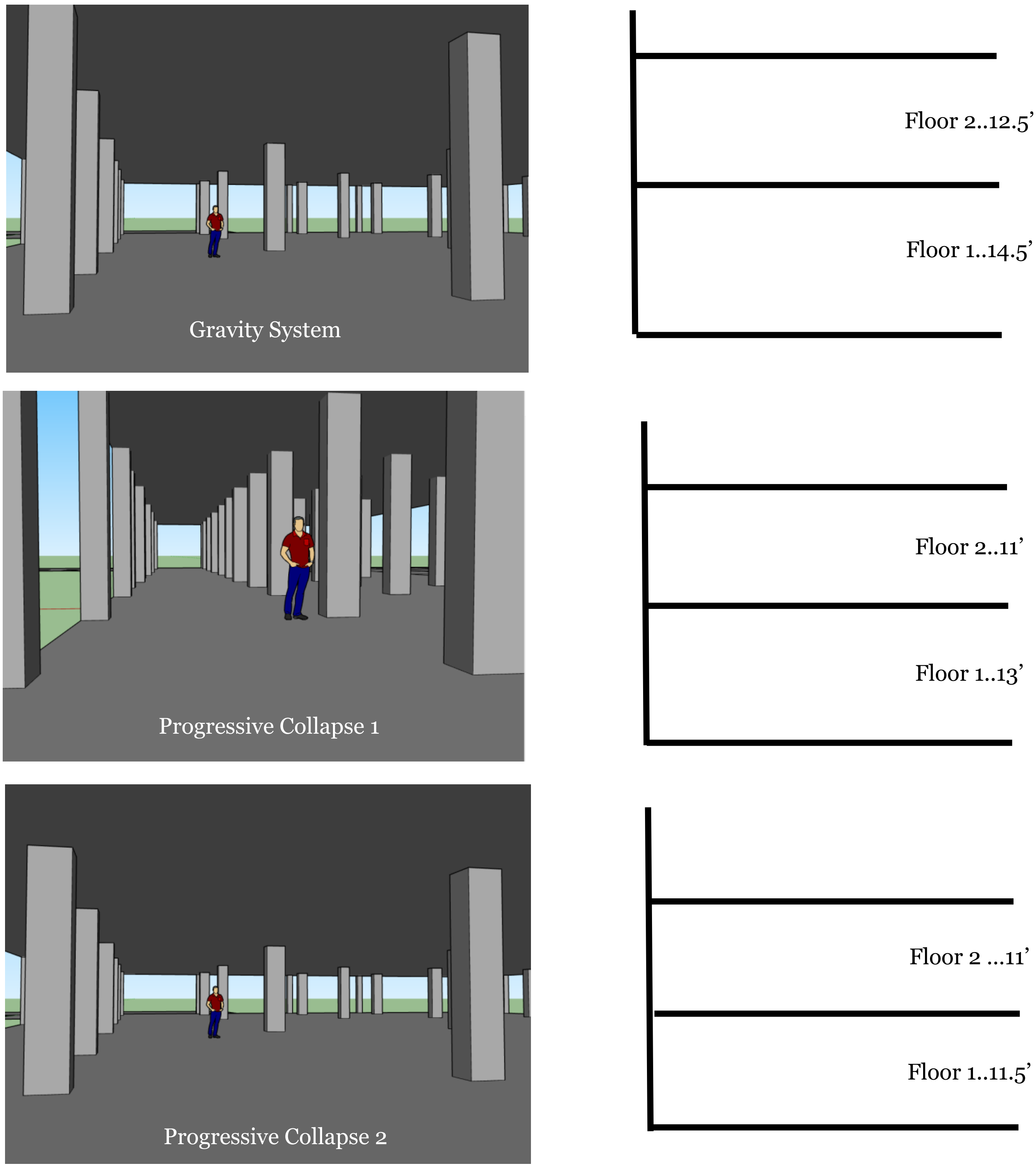


*Figure 53:* Plan view of potential blast radii for an interior explosion. The smallest ring represents the 5 lb TNT blast that was analyzed.

*Figure 52:* Images of special differences between the gravity system and the two progressive collapse systems.



## 6.5 Conclusion

The proposed structural system of a two way flat slab is a practical alternative to the original design. The major differences in the system that would be presented to an owner or architect are the exterior columns, decrease in floor to floor height of 2.5", and increase in shear wall thicknesses.

From the progressive collapse designs and analysis of their costs and spatial differences a favorable system was found. Both systems were designed as potential suggestions to the owner or architect. If they should value first floor height and overall cost whilst being comfortable with the addition of more interior columns then system one would be their preferred option. If the owner or architect wanted more capacity, a greater potential to resist larger threats and no additional interior columns the system 2 would be the preferred option. The last decision would be if the owner or architect relied upon the judgement of the engineer to choose between the systems then the second choice would be preferred. This is due to its higher overall capacity and its ability to resist larger blasts. Furthermore both systems require a sacrifice in height, one more so than the other, therefore if height is already being sacrificed then why not side with the safer system.

# 7. Construction Breadth

## 7.1 Introduction

A study was performed on the both the cost and schedule differences for the existing structural system and the proposed two way flat slab system. This study was done to learn how the change in system can effect such things as cost and construction time. Furthermore it is fairly possible that an owner could ask a structural engineer to design multiple structural systems and estimate the cost of each. This breadth is thus meant to be tied into the structural depth. However the progressive collapse systems were not analyzed for their effect on schedule. This was neglected due to the inaccuracy of the methods used to approximate duration for PT systems. The methods used to determine both cost and duration was quantity take offs of each system to then be multiplied by their per-unit costs from RS means. It was desired to use the 2011 RS means for this was when the building was constructed. However the 2011 RS means was not available and therefor the costs and durations for the systems were computed with the 2018 RS means. This scenario is also possible if this building was designed back in 2011 but was unable to be constructed and thus got tucked away until 2018 due to the market, economy or other financial reasons. All calculations are located in the appendices.

## 7.2 Costs

A detailed cost estimate was performed on the existing, proposed and progressive collapse systems. Initially material quantities were determined from this system such as cubic yards of concrete, tons of structural steel (rebar), necessary formwork and surface area of elements to be cured. The material costs per unit were then found from RS means. The costs for each member were then summed up and compared. This comparison can be seen below in Figure 53. Figure 53 also compares the total cost of each system. It is important to note that progressive collapse system 2 (PC 2) is more than double the cost of progressive collapse system 1 (PC 1). This is due to how many more members were used along with the post tensioning involved. It is also important to note that the new systems total cost is less than the existing systems. This was mostly dominated by the difference between the costs of the floor system. Due to drop panels and post tensioning the existing systems slab was more expensive.

*Figure 53:* Image of the excel chart used to estimate the cost of the various structural systems.

**Total Cost**

| System | Price per floor | Number of Floors | Total Cost ($) | Total Cost |
|---|---|---|---|---|
| Existing | 784686.3384 | 12 | 9416236.061 | 9.4 million |
| New | 753545.3069 | 12 | 9042543.683 | 9 million |
| PC 1 | 480434.265 | 1 | 480434.265 | 0.48 million |
| PC2 | 1166770.267 | 1 | 1166770.267 | 1.1 million |
| | | | | |
| Existing Columns | 18898.50644 | 12 | 226782.0773 | |
| New Columns | 26350.14158 | 12 | 316201.699 | |
| PC1 Columns | 54211.93 | 1 | 54211.93 | |
| PC2 Columns | 54579.72 | 1 | 54579.72 | |
| | | | | |
| Existing Shear Walls | 65678.14867 | 12 | 788137.784 | |
| New Shear Walls | 70383.27634 | 12 | 844599.3161 | |
| | | | | |
| Existing Slab | 700109.6833 | 12 | 8401316.2 | |
| New Slab | 656811.889 | 12 | 7881742.668 | |
| PC1 Slab | 426222.335 | 1 | 426222.335 | |
| PC2 Slab | 1112190.547 | 1 | 1112190.547 | |

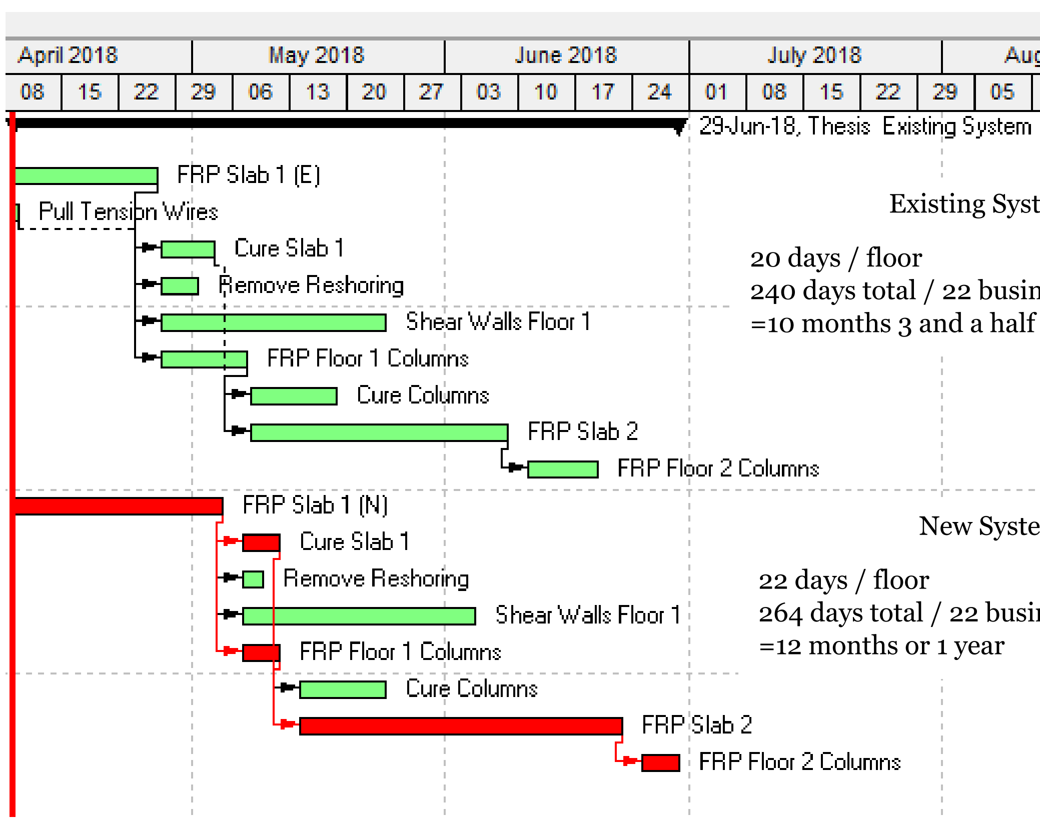
## 7.3 Schedule

The schedule of and duration of any project, especially on such as One City Center, needs to be considered. Schedules for both existing and proposed systems were estimated with similar methods as the cost estimation. The quantities that were found from each system were then multiplied by their daily output from the 2018 RS means. The procession of events for each system was the creation of the formwork followed by the creation of the steel rebar cages which was then proceeded by the placement of the concrete. This procession was detailed as form-rebar-place (FRP). The duration for those events was found and then aligned in the order of slab-shear walls/columns-next slab. The schedule and order of events can be seen in Figure 54. The critical path for both schedules revolves entirely around the slabs and how long it takes to form-rebar-place-cure them and remove the shoring. It is important to note also that Figure 54 is a schedule estimate per floor and not for the entire building. The entire building duration is the summation of time for one floor to be constructed multiplied by the amount of floors plus the additional curing time of the last slab. Another significant note is that the duration to construct the progressive collapse systems is not accounted for. While these system would more than likely change the duration of the project it was not desired to examine these effects. The purpose of this analysis was to compare only the existing and proposed system and only present an owner or client with a cost and design of the progressive collapse systems.

*Figure 54:* Image of the estimated per floor schedule for the existing and new systems.

*Existing System*

*New System*




Existing System

20 days / floor
240 days total / 22 business per month
=10 months 3 and a half weeks

New System

22 days / floor
264 days total / 22 business per month
=12 months or 1 year

## 7.4 Conclusion

From this breadth the existing system and all the proposed systems can be compared. The cost and duration of these systems is what an owner or client would care greatly about. Costs of the original system are greater than that of the newly proposed system by $400,000. However the original system can be constructed 2 months quicker than the proposed system. This is time that the owner of the building could be using to lease out the building and generate potentially more than $400,000 in revenue. It is important to note the limitations of this analysis. The post tensioning for both the existing system and progressive collapse systems were not estimating to a great deal of accuracy. In short RS means applies only certain construction types and methods to be used. Furthermore it is more than likely that a general contractor has their own, more accurate, set of information that allows them to more accurately predict the cost and duration of this type of work. In conclusion it would be up to the owner to choose which system they prefer.

# 8. Acoustical Breadth

## 8.1 Introduction

The acoustical properties of any building should be considered when designing the spaces and choosing surface materials. If this is not done than a space can become unpleasant to be in and to work in. This is even more significant for a government office building which this report is centered around. However the spaces and materials have already been chosen for One City Center and as such this breadth will analyze those spaces and materials and their effect on reverberation time. If their reverberation time is similar to the appropriate reverberation times of like spaces then the materials need not be redesigned. If the reverberation time is outside of the desired range for similar spaces then different material properties will be suggested. It is important to note that the architectural drawings did not define particular boundaries for their tenants and as such, for those spaces, boundaries will be assumed. Furthermore typical office spaces such as a meeting room will be created and analyzed with the proposed materials on the construction documents. Similar to the Structural depth and Construction breadth all calculations are located in the appendices.

## 8.2Reverberation Time Analysis

The analysis for reverberation time began with the determination of what spaces would be desired for analysis. Figure 55 shows in plan what spaces were considered. Note that the floor plan did not specify a particular partition layout for the spaces and as such these boundaries were assumed. The main spaces that were analyzed were the main lobby, atrium, office floor, and a fictitious office. The fictitious office was imagined due to the lack of specification on the design documents. It was desired to determine how a personal office would perform acoustically.

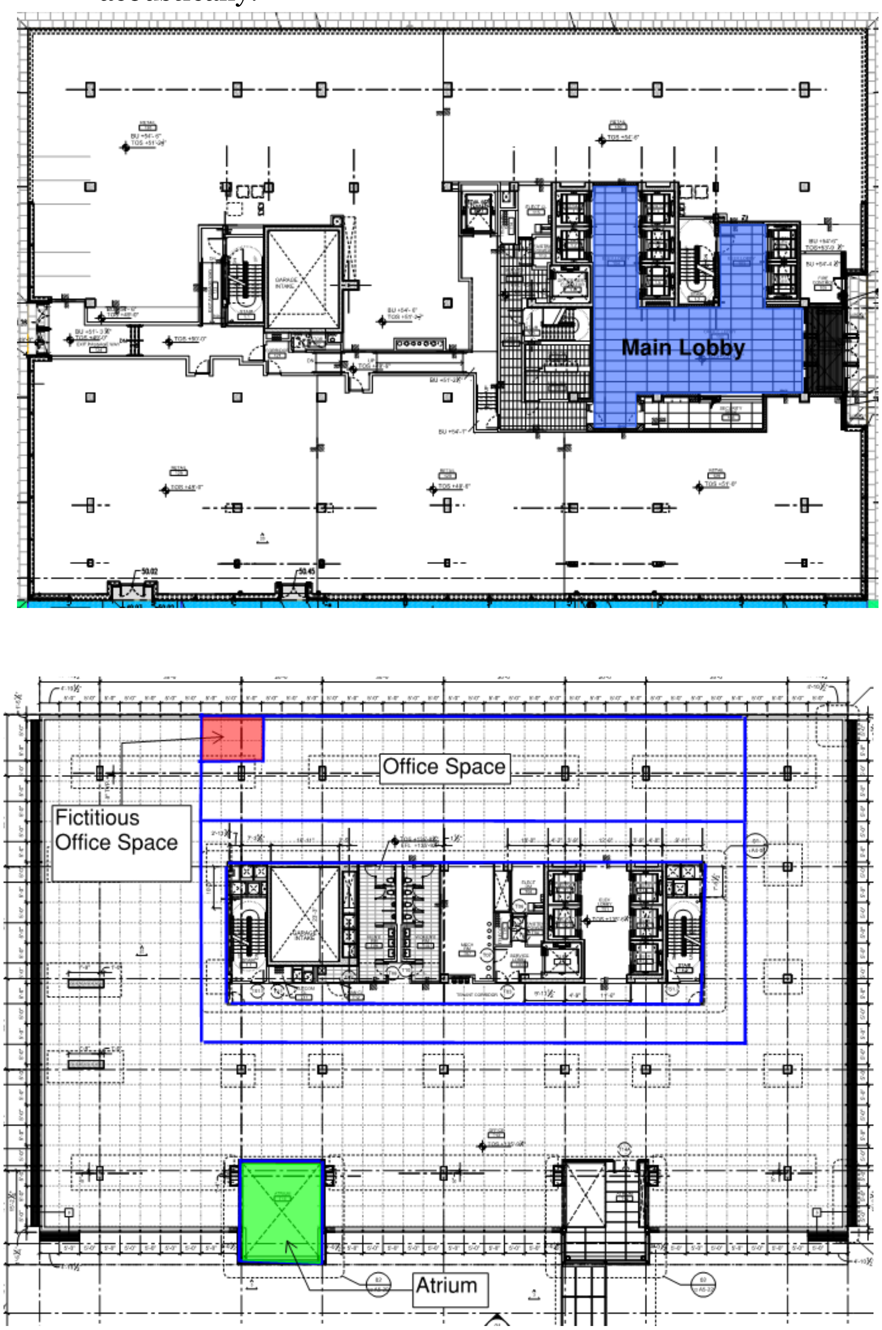



*Figure 55:* Plan view of the spaces that are to be analyzed for reverberation time.

It is assumed that the owner of the building left the spatial layout to the tenant. The reverberation time of the spaces were analyzed by finding the surface materials given by the design documents and then seeing how their combined absorption would affect the reverberation time of the space. Absorption coefficients were determined for 1000Hz 500Hz and 250Hz frequencies because they are the average frequencies of human speech. Using the absorption coefficients and surface area for particular materials the reverberation times were found from either the Norris-Eyring or the Sabine equation. These equations are from Architectural Acoustics by Marshall Long and are shown in Figure 56 below. From these equation it was determined that the spaces were adequate for reverberation time. As it turns out the interior partitions were acoustical in nature with acoustical insulation between the gypsum as seen in Figure 57. However the atrium did have overly large reverberation times for the frequencies studied. Reverberation times for the atrium were 2 – 3 seconds which is too high. This large reverberation time would create lots of echoes and make it difficult to understand conversations. As a result the atrium was redesigned to have a more appropriate reverberation time.

*Figure 56:* Sabine and Norris-Eyring equation for reverberation time.

*$V$ = Volume of space*

*$S_T$ = Total Surface area*

*$\alpha$ = Average absorption coefficient*

*$\alpha_a$ = Air attenuation constant*

For $\alpha \leq 0.2$

$$RT = (0.049V) / (S_T\alpha + 4\alpha_a V)$$

For $\alpha > 0.2$ $$RT = (0.049V) / (-S_T\ln(1 - \alpha) + 4\alpha_a V)$$

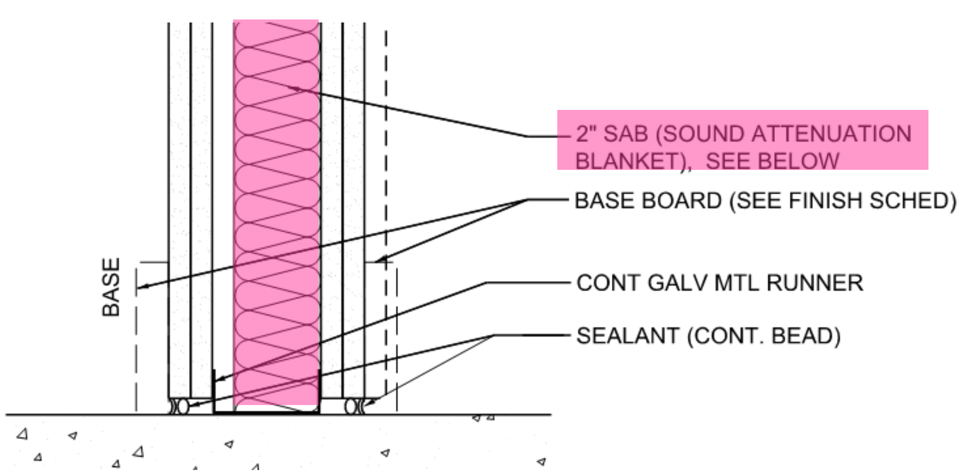
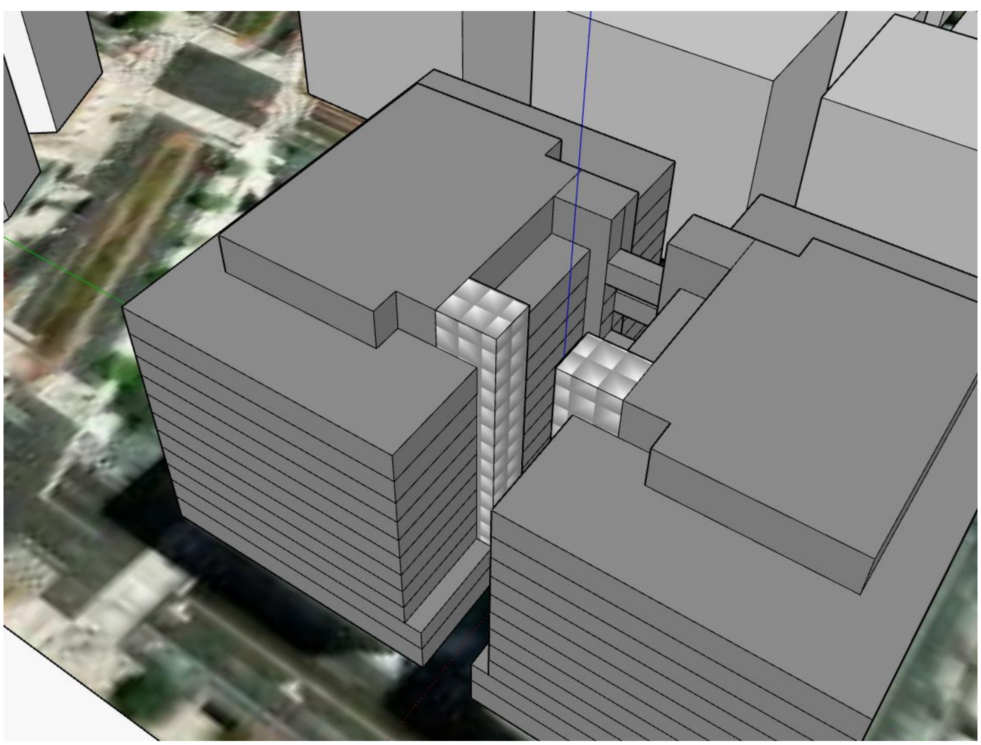



*Figure 57:* Section of an interior partition showing the 2" acoustical blanket used in between the gypsum.

## 8.3 Reverberation Time Design

From the analysis it was determined that the atrium was the only space that needed a different acoustical design. This however would be fairly tricky due to the nature of the atrium. In One City Center the atriums are glass boxes as seen in Figure 58. It was desired not to change the geometry of these spaces nor their architectural effect. Meaning that the dimensions of the atrium would not be changed nor the transparent glass to let in both direct and diffuse light. From a more fundamental analysis on how sound would travel in the atrium, as shown in Figure 59, it was determined that if the sound could be more absorbed and diffused instead of reflected as hard surfaces do then the reverberation would not be as great. Figure 59 shows how a convex ceiling would help diffuse the sound and cause less direct reflections. It was then decided to use a material called honeycomb glass shown in Figure 60. This glass is transparent but also highly absorptive. It is also much lighter than regular glass and can also be molded into shapes such as a convex ceiling.

*Figure 58:* Image showing the glass atriums in One City Center.



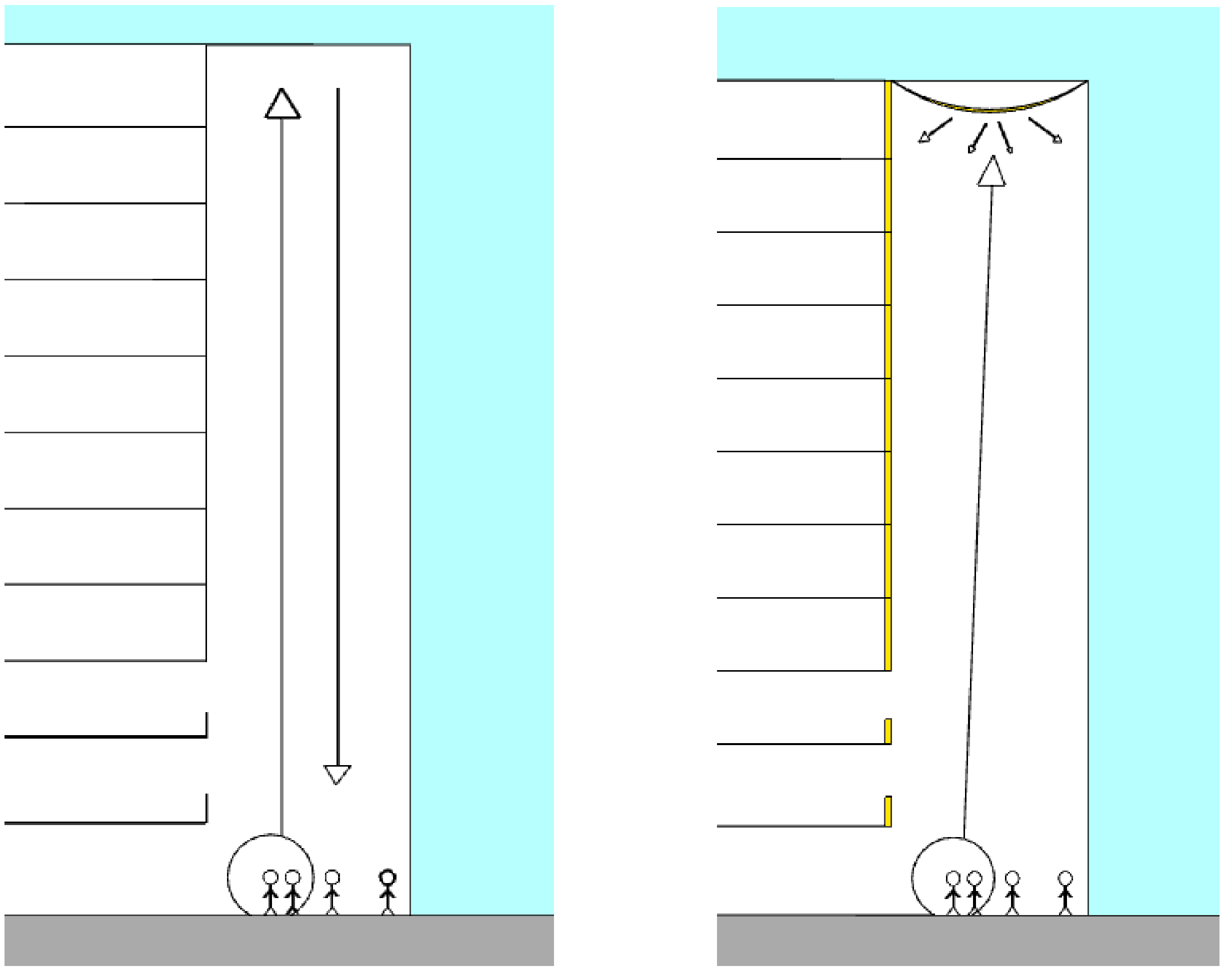
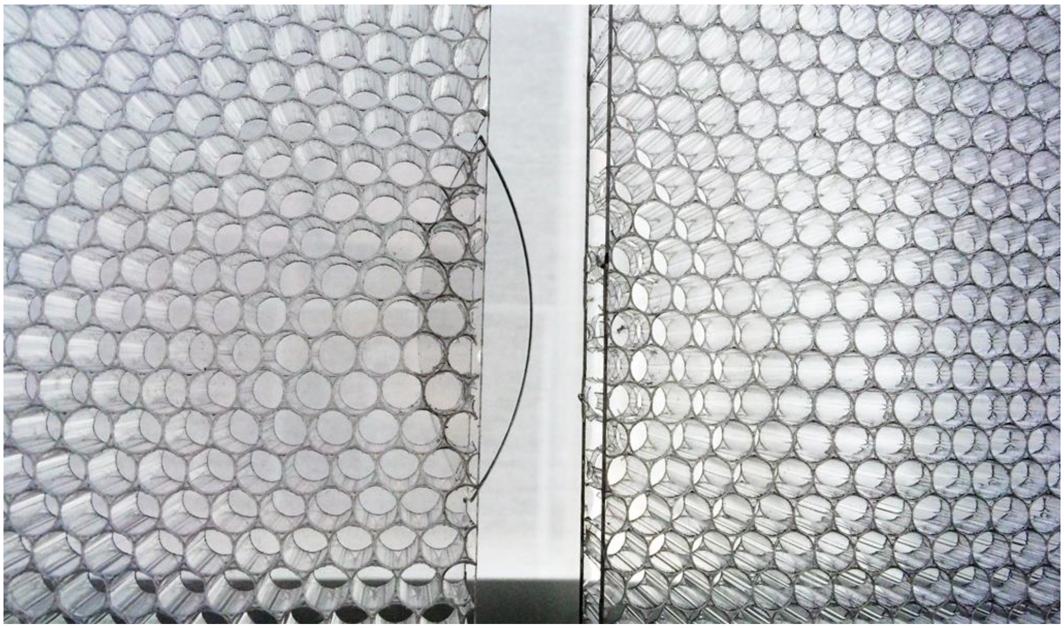


*Figure 59:* Section of the atrium with a flat ceiling and convex ceiling. The proposed material, honeycomb glass, is shown in yellow.

It was then analyzed what the reverberation time would be like if the railings and the ceiling had this honeycomb glass material. The effect was a reduction in the reverberation time of half a second. The effects of the convexity of the ceiling were not included in this calculation due to the absence of an empirical method to calculate convexity effects on sound distribution and reverberation time. The degree of convexity was also not determined due to its complexity. It is again noted that the convex design is based on acoustical fundamentals and not on calculations.

*Figure 60:* Image of the honeycomb glass tiles that are suggested be installed in the atrium to reduce reverberation time.



## 8.4 Conclusion

From the analysis it was concluded that the interior spaces were acoustically adequate in terms of reverberation time. The one space that was not adequate, the atrium, had too much reverberation time due to the large geometry of the space and the surface materials all being glass, a hard reflective surface. As a result of this analysis a proposed design was created utilizing a researched material called honeycomb glass. Not only was this glass lighter and vastly more absorptive than regular glass it was also able to be shaped into a convex shape that could diffuse the sound in the atrium thus decreasing the overall reverberation time. This material was studied if it were to be placed on the ceiling, the railings and the overlooks of the atrium. Reverberation time was decreased to a satisfactory amount. This change in material would have to be approved by the owner and if they wanted even less reverberation time then they would have to change more of the material of the atrium from glass to a material less reflective and more absorptive.

# 9. Evaluation of Goals

Before the Structural depth, Construction breadth and Acoustical Breadth were written goals were established. Below those previously defined goals are evaluated.

- Design a new structural system for One City Center that do not change the overall floor to floor height by more than 5% or about 6". The new flat plate system does change the floor to floor height but only by 2.5 " and thus this goal was achieved.
- The cost of the new structure is desired not to differ by more than 10%. The estimated cost was determined to be less than the existing system by $400,000 and thus this goal was achieved.
- The time it takes to construct this new system will not be greater than the original by a month. The new structural system was estimated to take 2 months longer than the existing system thus this goal was not achieved. It is important to note that some of the cost difference between the two systems could be made more even by increasing the crews and labor for the new system until it took only a month longer.
- The structural system shall be designed to resist blast loading for both interior and exterior explosions. The magnitude of which is to be determined. The exterior of the building was designed to resist a blast however through the analysis it was determined unreasonable to create a blast design of the interior and instead create a progressive collapse design.
- The new structural system shall not greatly alter the architecture. The main architectural difference between the existing and proposed system is the perimeter columns that were added to the new system. Personally I do not think this alters the architecture but I could be mistaken and therefor this goal was only partially achieved.
- Interior spaces will be analyzed for acoustical comfort and changed if deemed unsatisfactory. Interior spaces were analyzed and some were even imagined due to lack of specification on the design documents. The acoustical comfort of all but one space was deemed comfortable in terms of reverberation time and the space that wasn't was redesigned to satisfaction. Thus this goal was achieved.
- Material learned in master's level classes be used at least twice. Master's level courses were used several times in this report. For the analysis of the existing post tensioned system (AE 597), for the design of the progressive collapse post tensioned system (AE 597), for how to implement and create a progressive collapse design (AE 537) and for creating and verifying computer models that were frequently used (AE 530). Thus this goal was achieved.

# 10. Conclusion

In summation the proposed redesign of the structural system for One City Center proved to be practical whilst having slight disadvantages. The two way flat plate system was able to carry the gravity loads with only a small increase in the depth of the slab. The shear walls were able to handle the lateral loads after only a slight increase in the thickness and reinforcing. Exterior columns however were added to the gravity system as well as blast and progressive collapse systems.

Blast design was performed for both an interior and exterior explosion. This study was performed in accordance with UFC-4-023-03 and other related text. The resulting analysis was found feasible for exterior loads acting on the structure but not interior loads. As a result of this a progressive collapse analysis was done. The two designs to resist progressive collapse were additional columns with post tensioned (PT) transfer beams or no additional columns with even larger PT transfer beams. The governing factor on these designs was resisting the large moments that were created when a column was presumed lost. Each system did have its own pros and cons and therefore the owner would have to decide on which one they prefer. If the owner however decided to let the structural engineer chose than the second system would be proposed due to its higher capacity and ability to resist larger explosions. While other systems could've worked similarly or even better it was desired to examine how a PT beam system would support these loads.

The change in construction due to these newly proposed systems was also analyzed. The cost of the flat plate system was cheaper than the existing PT one. However the duration to construct the entire building would take 2 months longer if the flat plate system was chosen. Costs of the flat plat system could've been increased in order to speed up this process. Only the cost of the progressive collapse systems were calculated and the first system (with interior columns) was less than half the cost of the second system.

Additionally acoustical comfort was analyzed through calculation of reverberation time in particular spaces. Due to the design documents not defining the office layout partition locations were imagined. From the analysis only the atrium proved to have too high of reverberation time and as such different materials were studied to lessen this. The proposed material was honeycomb tiles which would be added to the atrium in a way that would absorb and diffuse the sound.

# 11. Acknowledgements

I would like to thank the following people who gave me advice wisdom and support with this report without you this would simply not have been possible.

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- Dr. Aly Said for personally assisting me with this report and answering any and all questions that I had.

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- My fellow fifth years, class of 2018, who I constantly relied upon for both simple and difficult problems whether they be for classes or for personal matters.

# 12. References


- American Concrete Institute. *Building Code Requirements for Structural Concrete*. Vol. 318-14, ACI Committee 318, 2014.

- American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures. *ASCE STANDARD* . Vol. 7-10, ASCE/SEI, 7AD.

- Wight , James K, and James G MacGregor. *Reinforced Concrete Mechanics and Design* . 5th ed., Pearson Education, Inc , 2009.

- Dusenberry, Donald O. *Handbook for Blast Resistant Design of Buildings* . John Wiley & Sons, Inc, 2010.

- MacAlevey, Niall. *Design of Reinforced Concrete Buildings to Resist Blast* .

- Smith , P D, and J G Hetherington. *Blast and Ballistic Loading of Structures*. 1994.

- Department Of Defense United States Of America. *Unified Facilities Criteria (UCF) Design Of Buildings To Resist Progressive Collapse*. UFC 4-023-03, 2010.

# 13. Appendices

The appendices for this document is organized in the following manner

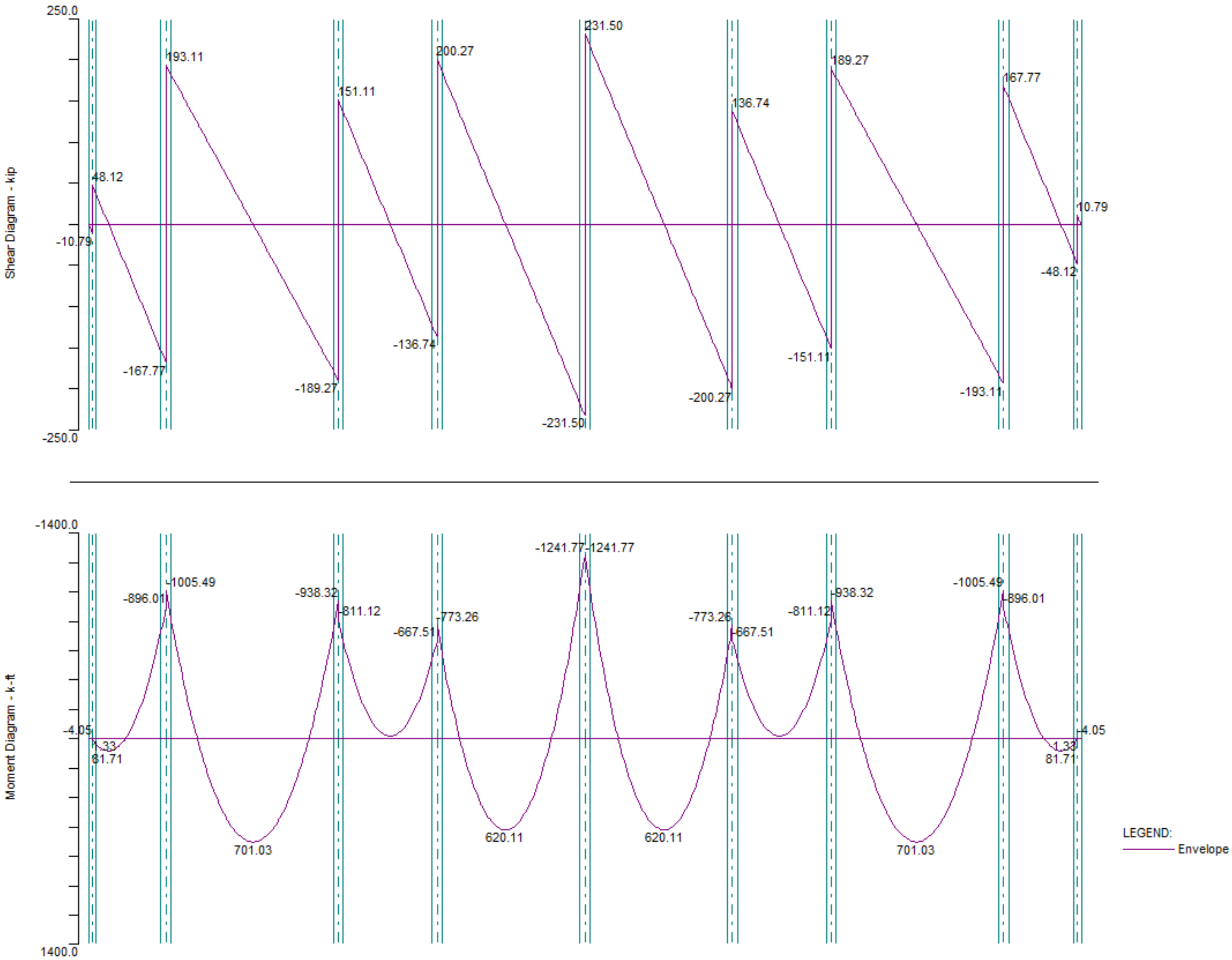
- Model data and results
- Hand Calculations for structural depth and acoustical breadth
- Separately attached excel sheets for Structural Depth, Construction Breadth, Acoustical Breadth

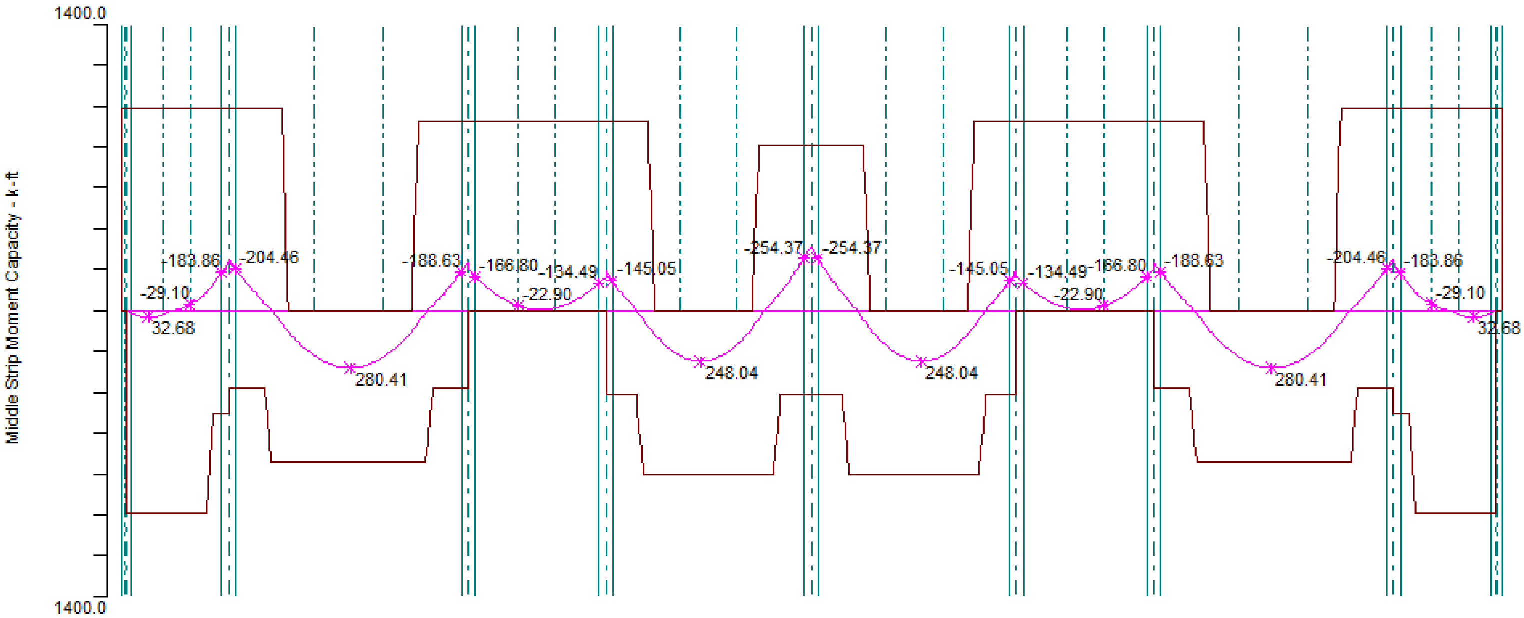
## 13.1 Model Data and Results

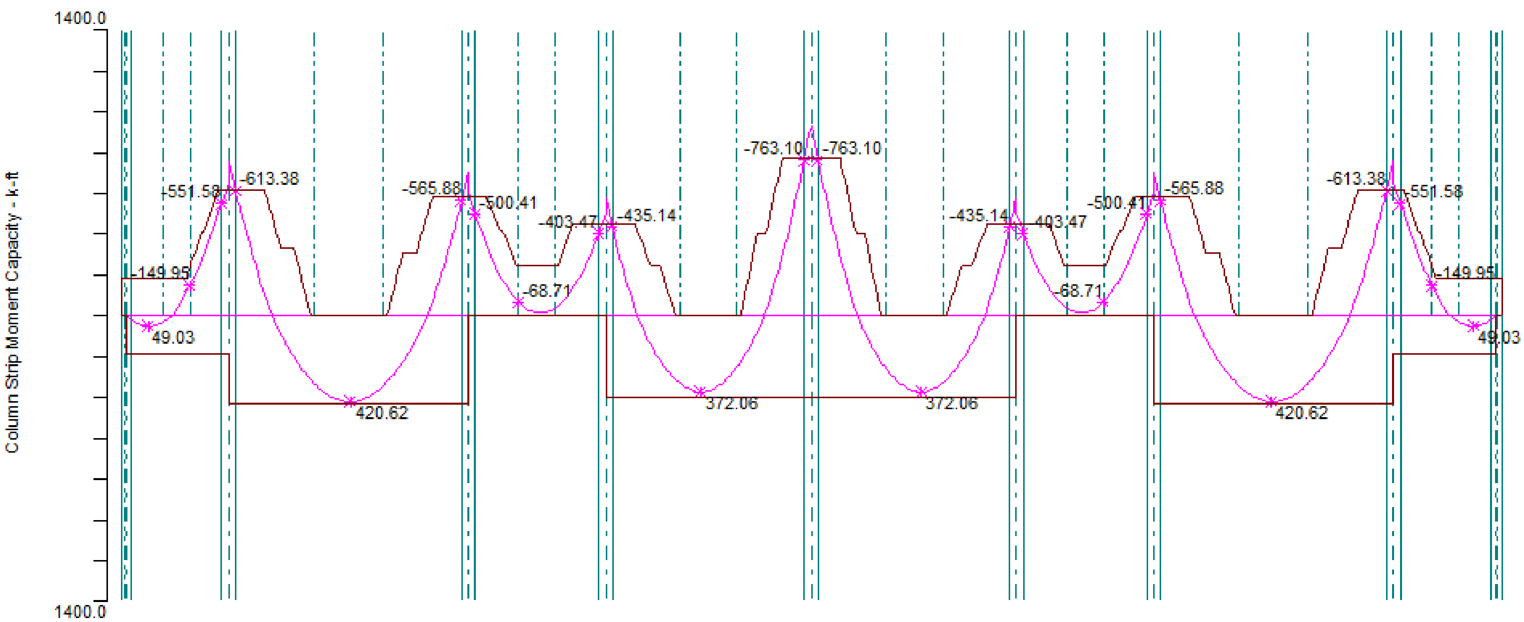
### 13.1.1 Software Used

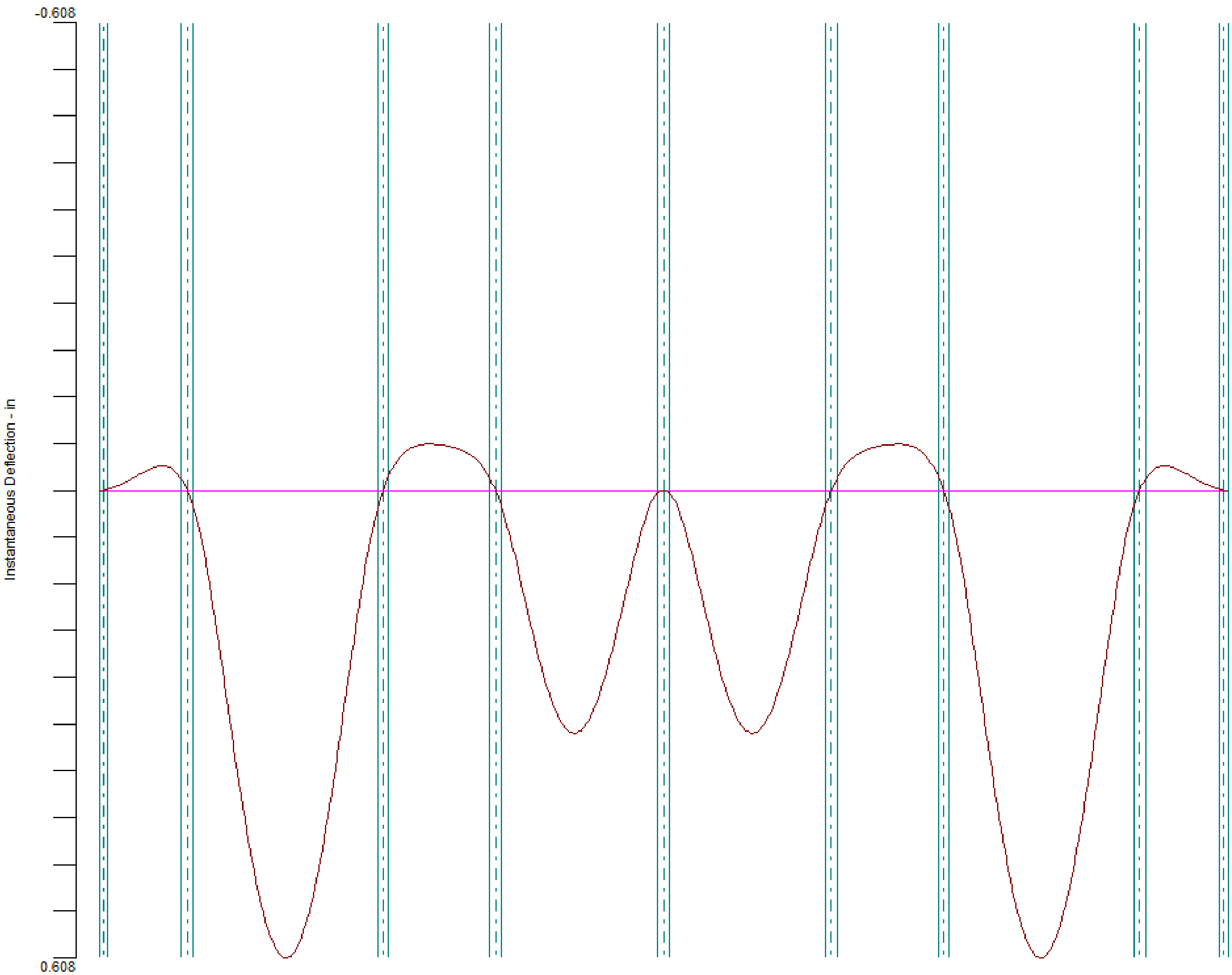
## 13.2 Model Results

### 13.2.1 spSlab

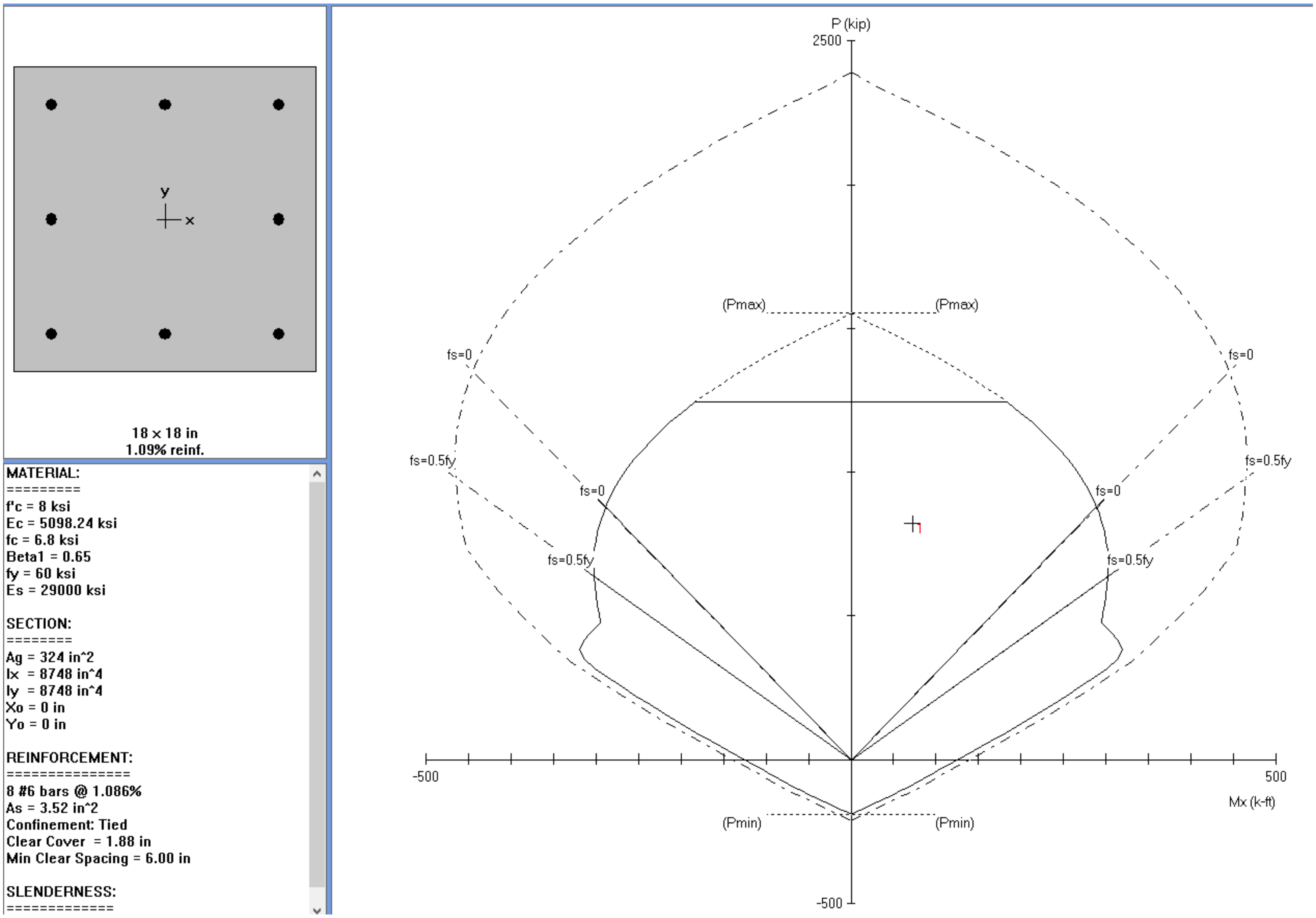


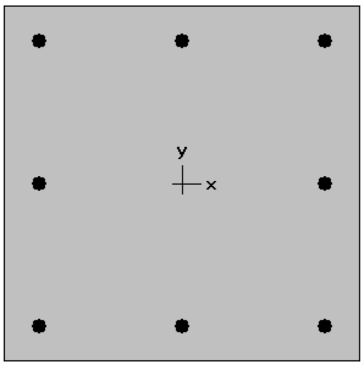

1400.0
Middle Strip Moment Capacity - k-ft
-183.86
-204.46
-29.10
32.68
280.41
-188.63
-166.80
-134.49
-22.90
-145.05
-254.37
-254.37
248.04
248.04
1400.0


1400.0
Column Strip Moment Capacity - k-ft
-551.58
-613.38
-149.95
49.03
420.62
-565.88
-500.41
-403.47
-435.14
-68.71
-763.10
-763.10
372.06
372.06
1400.0


-0.608
Instantaneous Deflection - in
0.608

## 13.2.2 spColumn





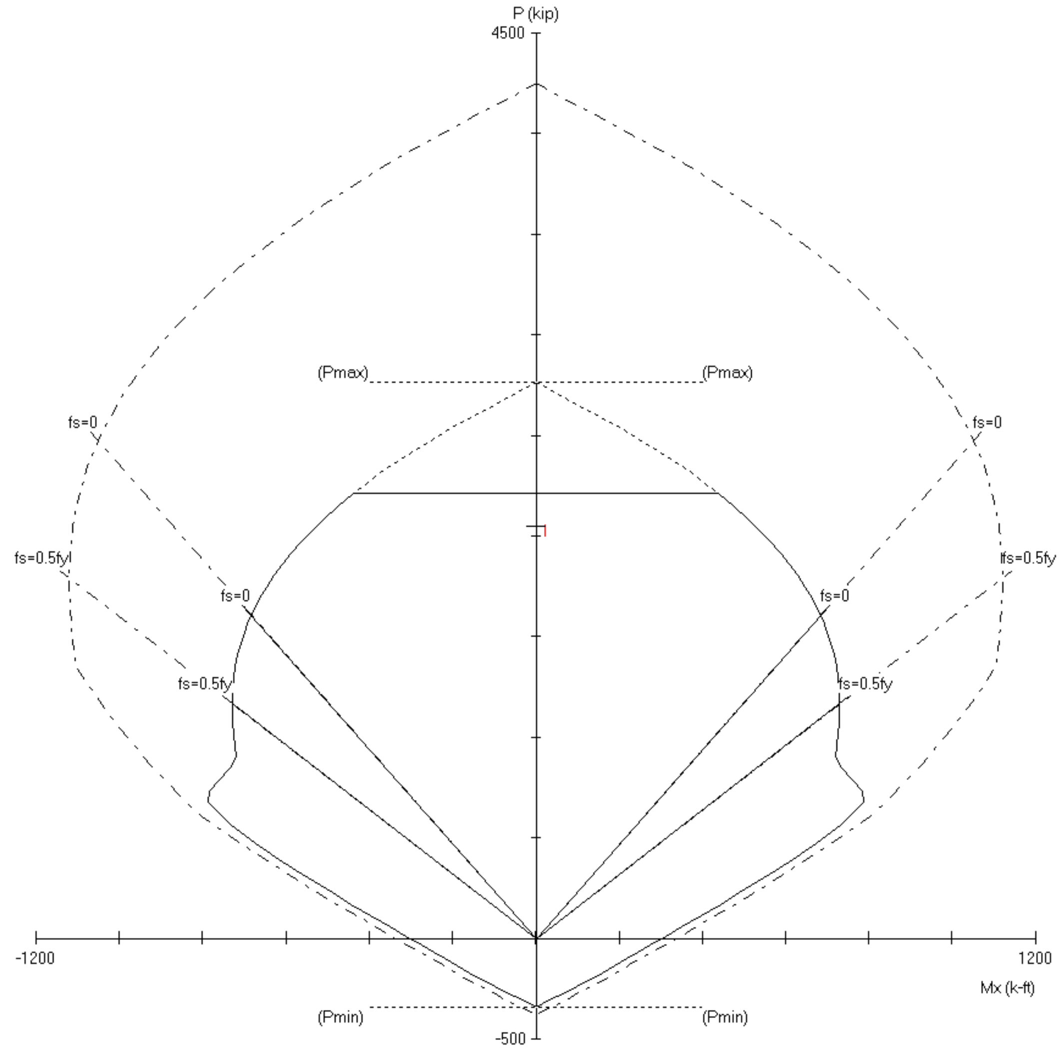

24 x 24 in
1.10% reinf.

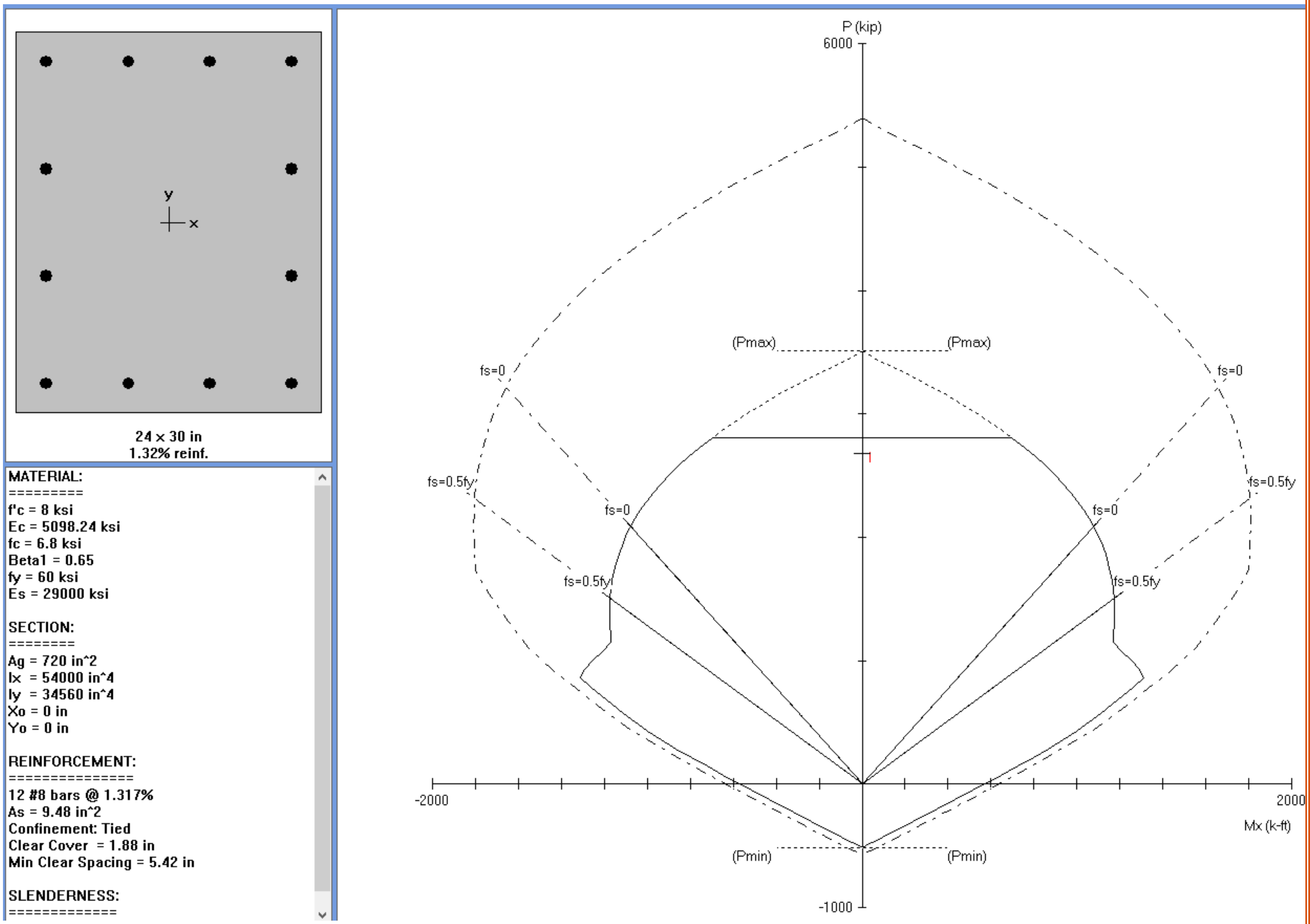
MATERIAL:
=========
f'c = 8 ksi
Ec = 5098.24 ksi
fc = 6.8 ksi
Beta1 = 0.65
fy = 60 ksi
Es = 29000 ksi

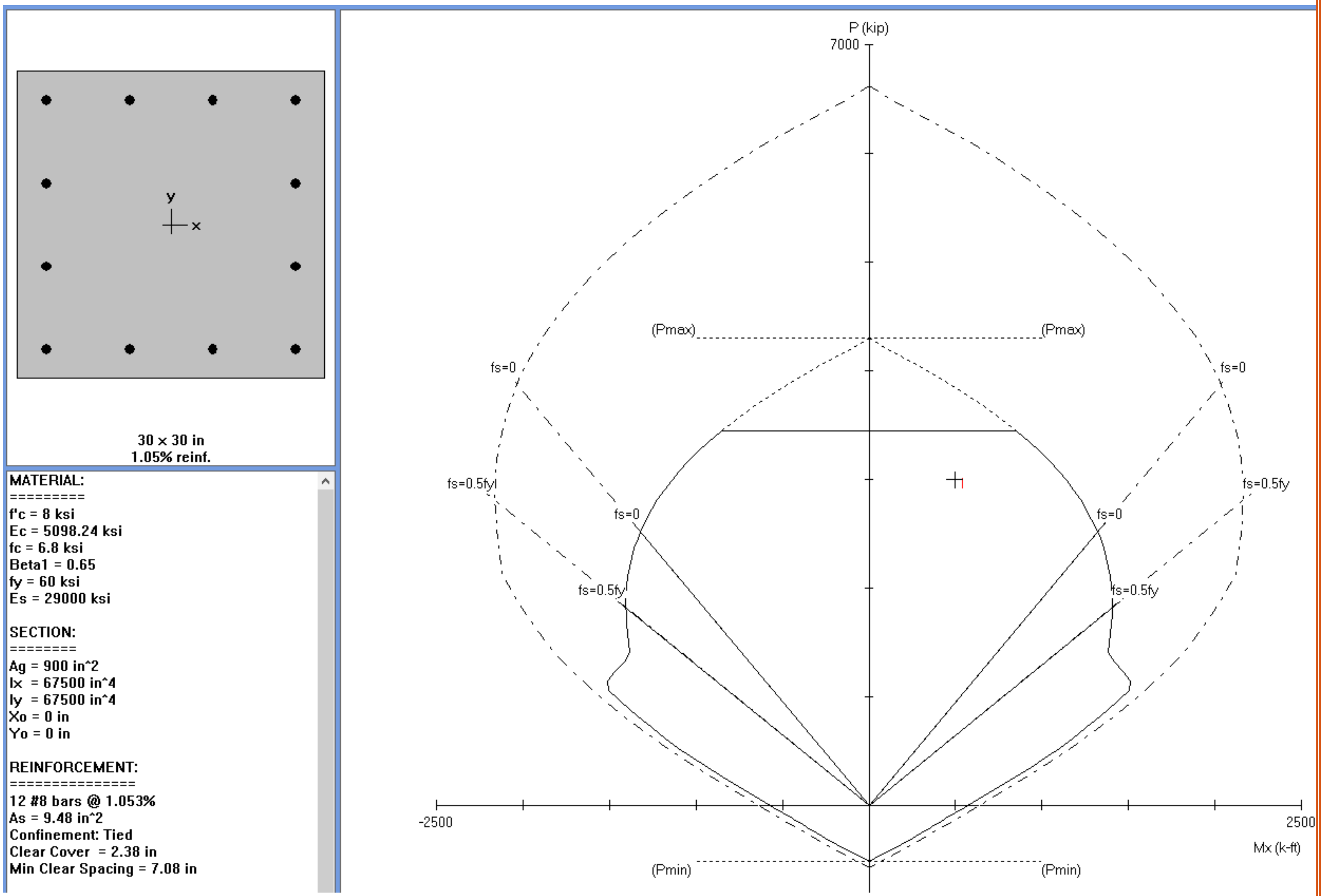
SECTION:
========
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

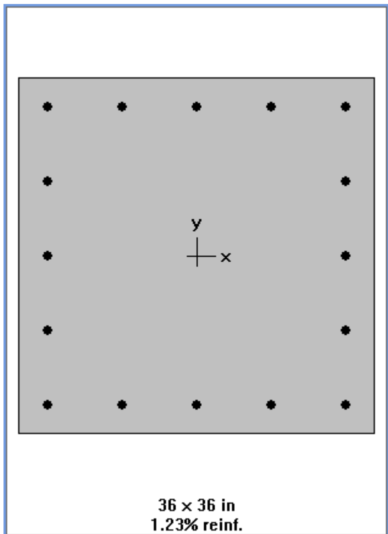
REINFORCEMENT:
===============
8 #8 bars @ 1.097%
As = 6.32 in^2
Confinement: Tied
Clear Cover = 1.88 in
Min Clear Spacing = 8.63 in

SLENDERNESS:
=============




24 x 30 in
1.32% reinf.
MATERIAL:
=========
f'c = 8 ksi
Ec = 5098.24 ksi
fc = 6.8 ksi
Beta1 = 0.65
fy = 60 ksi
Es = 29000 ksi
SECTION:
========
Ag = 720 in^2
Ix = 54000 in^4
Iy = 34560 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
==============
12 #8 bars @ 1.317%
As = 9.48 in^2
Confinement: Tied
Clear Cover = 1.88 in
Min Clear Spacing = 5.42 in
SLENDERNESS:
=============
P (kip)
6000
(Pmax)
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
fs=0
fs=0
fs=0.5fy
fs=0.5fy
1
-2000
2000
Mx (k-ft)
(Pmin)
(Pmin)
-1000

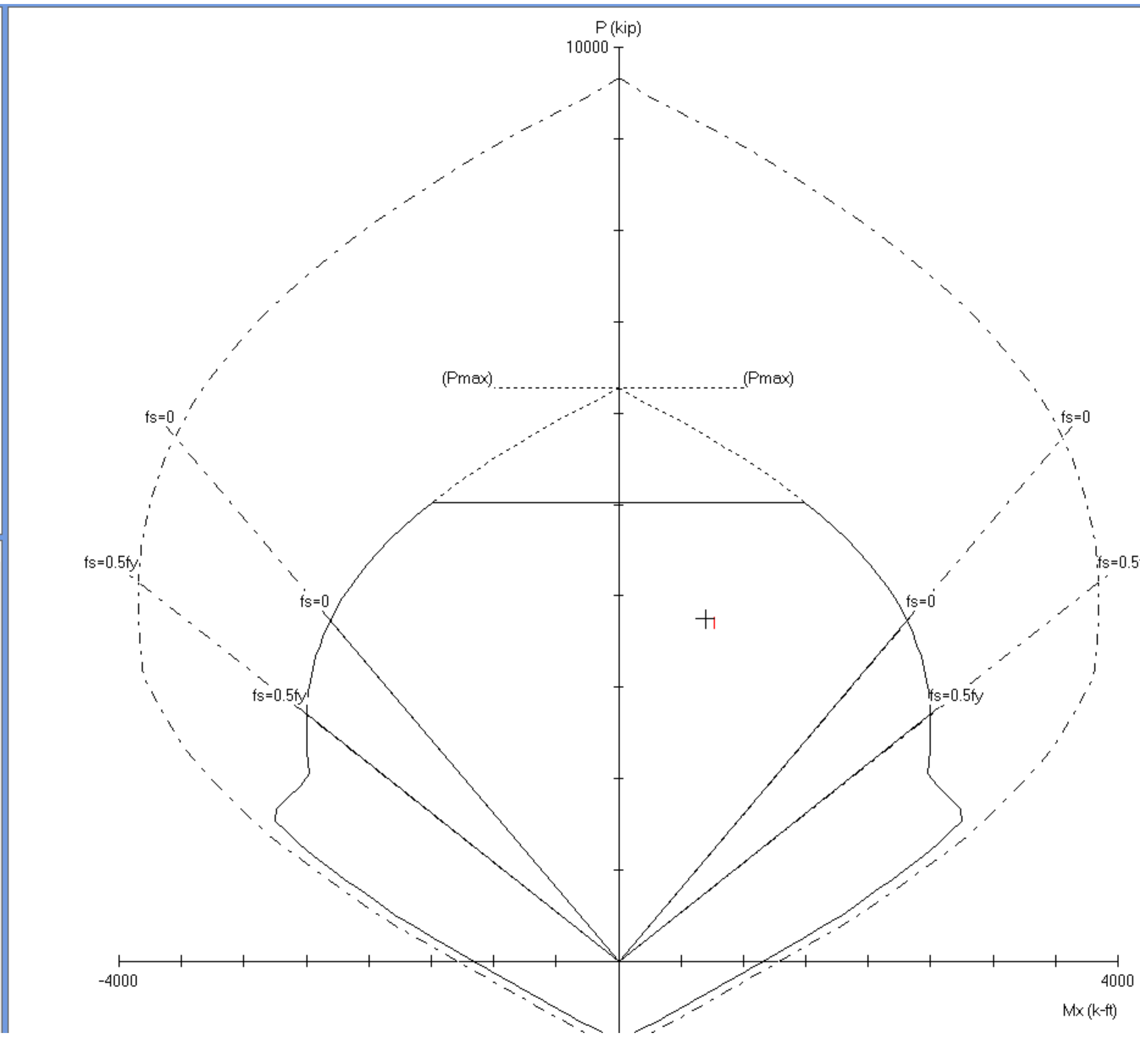

30 x 30 in
1.05% reinf.
MATERIAL:
=========
f'c = 8 ksi
Ec = 5098.24 ksi
fc = 6.8 ksi
Beta1 = 0.65
fy = 60 ksi
Es = 29000 ksi
SECTION:
========
Ag = 900 in^2
Ix = 67500 in^4
Iy = 67500 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
==============
12 #8 bars @ 1.053%
As = 9.48 in^2
Confinement: Tied
Clear Cover = 2.38 in
Min Clear Spacing = 7.08 in
P (kip)
7000
(Pmax)
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-2500
2500
Mx (k-ft)
(Pmin)
(Pmin)



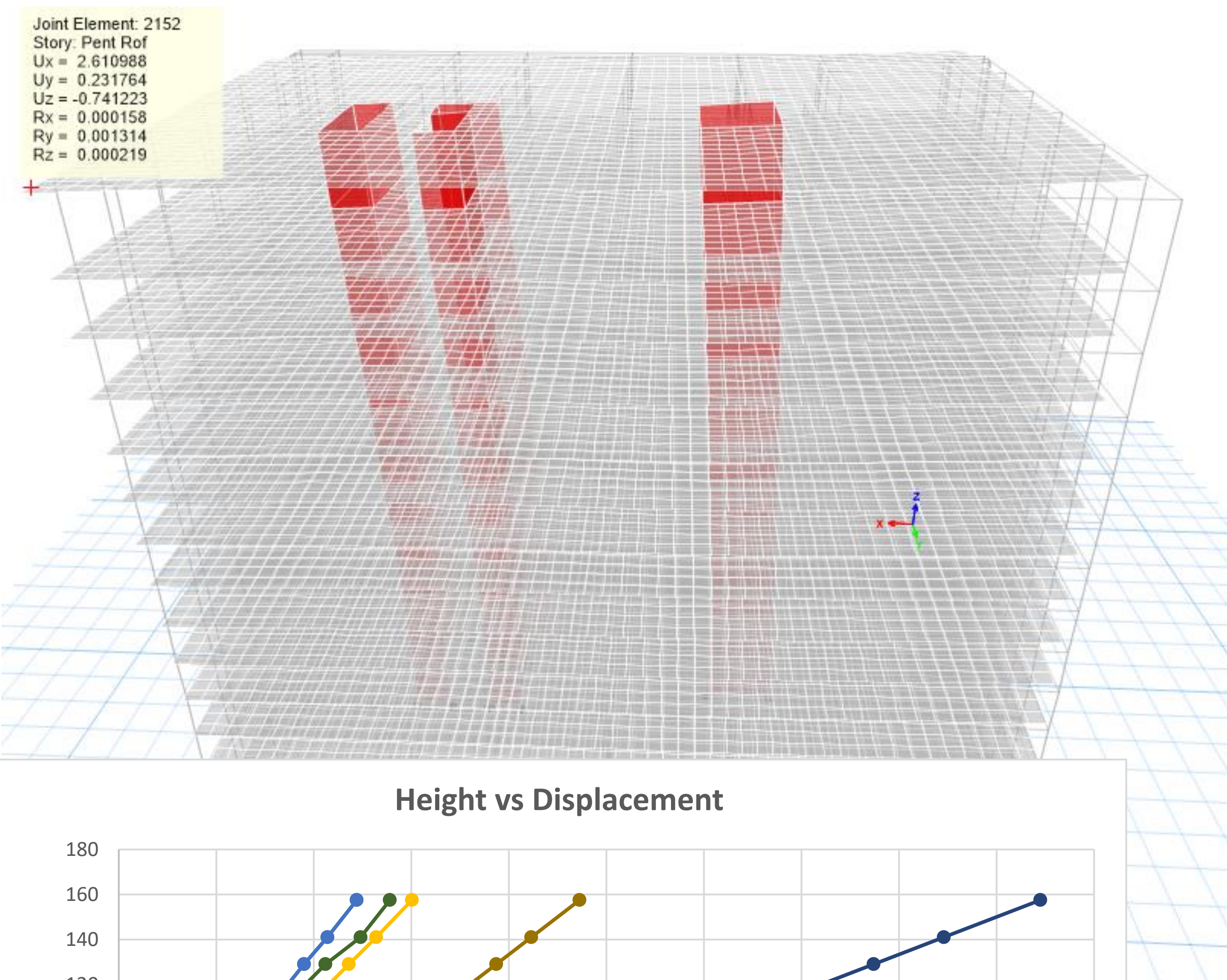

MATERIAL:
=========
f'c = 8 ksi
Ec = 5098.24 ksi
fc = 6.8 ksi
Beta1 = 0.65
fy = 60 ksi
Es = 29000 ksi

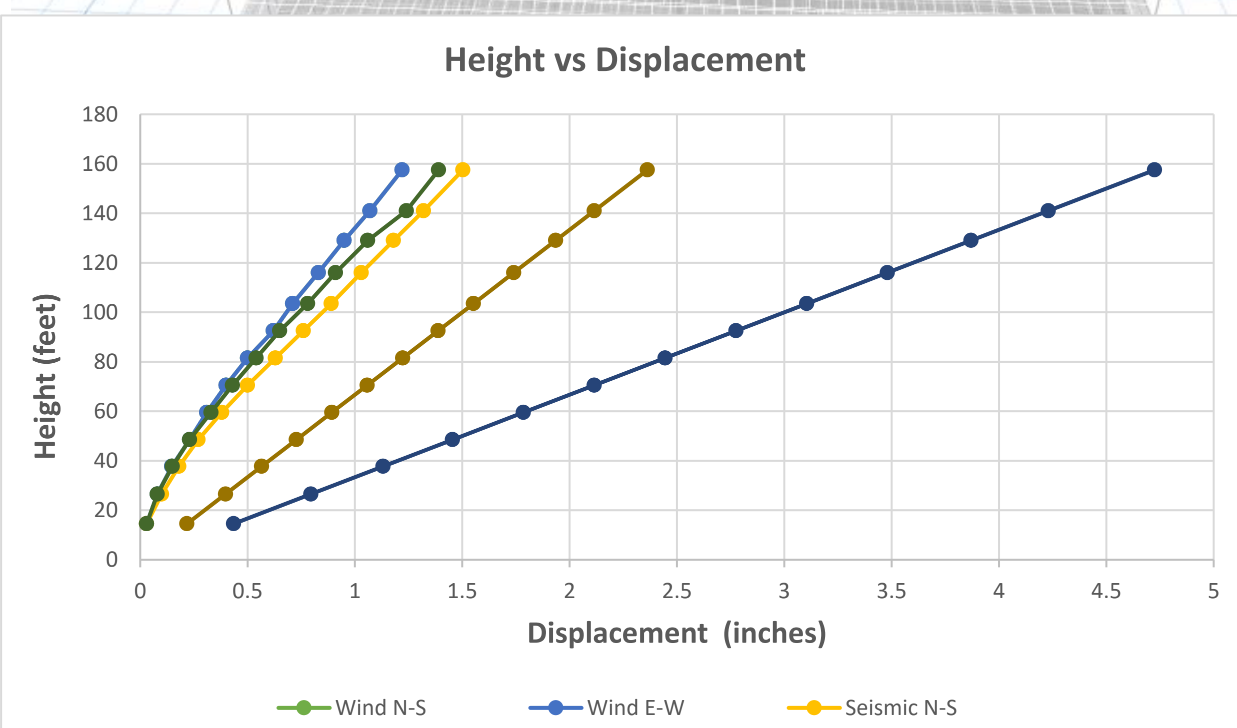
SECTION:
========
Ag = 1296 in^2
Ix = 139968 in^4
Iy = 139968 in^4
Xo = 0 in
Yo = 0 in

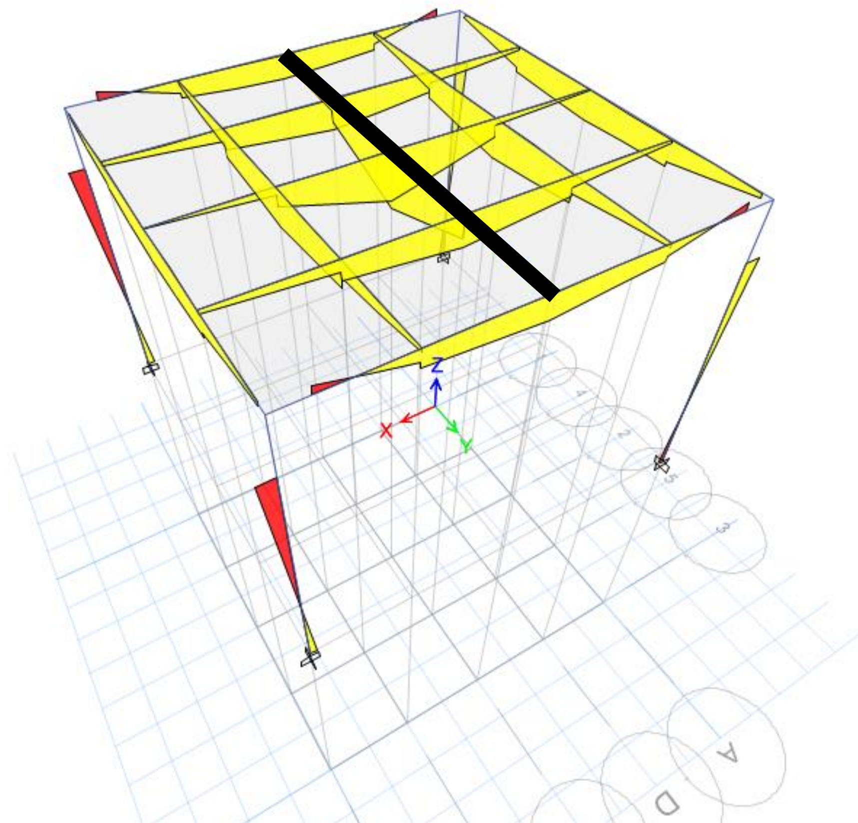
REINFORCEMENT:
==============
16 #9 bars @ 1.235%
As = 16 in^2
Confinement: Tied
Clear Cover = 2.38 in
Min Clear Spacing = 6.40 in

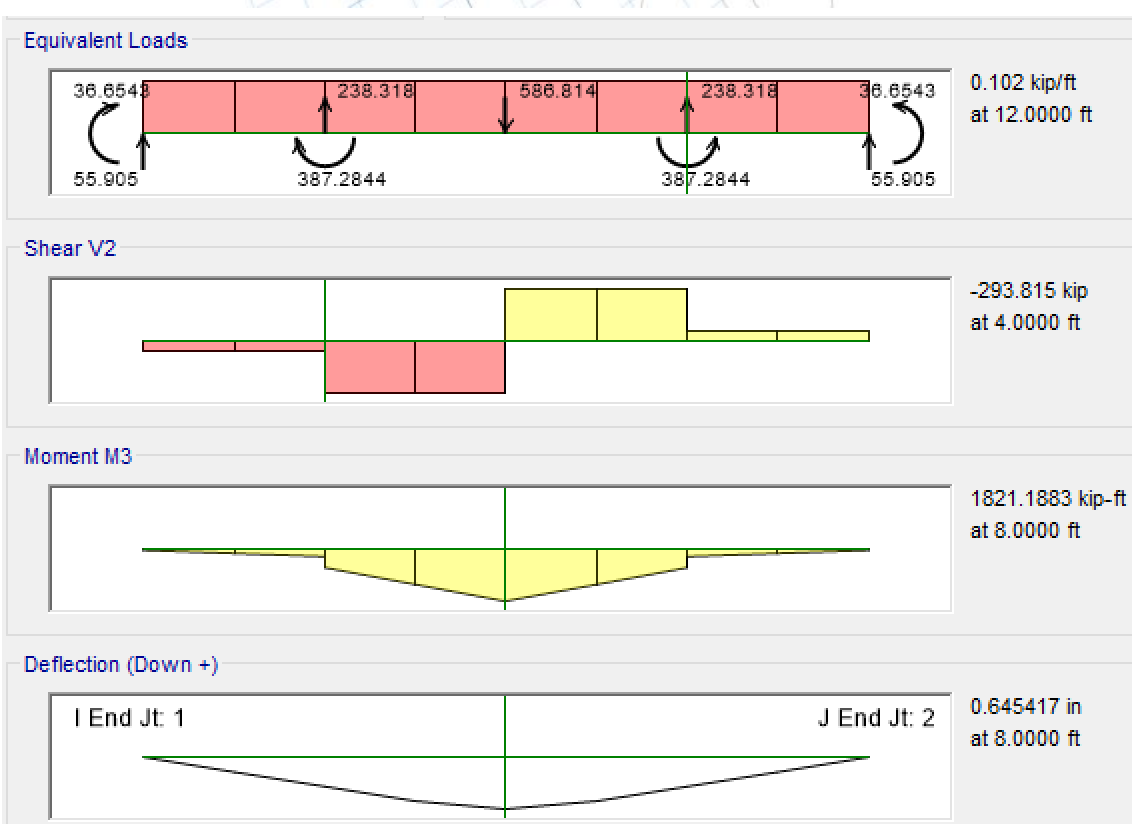


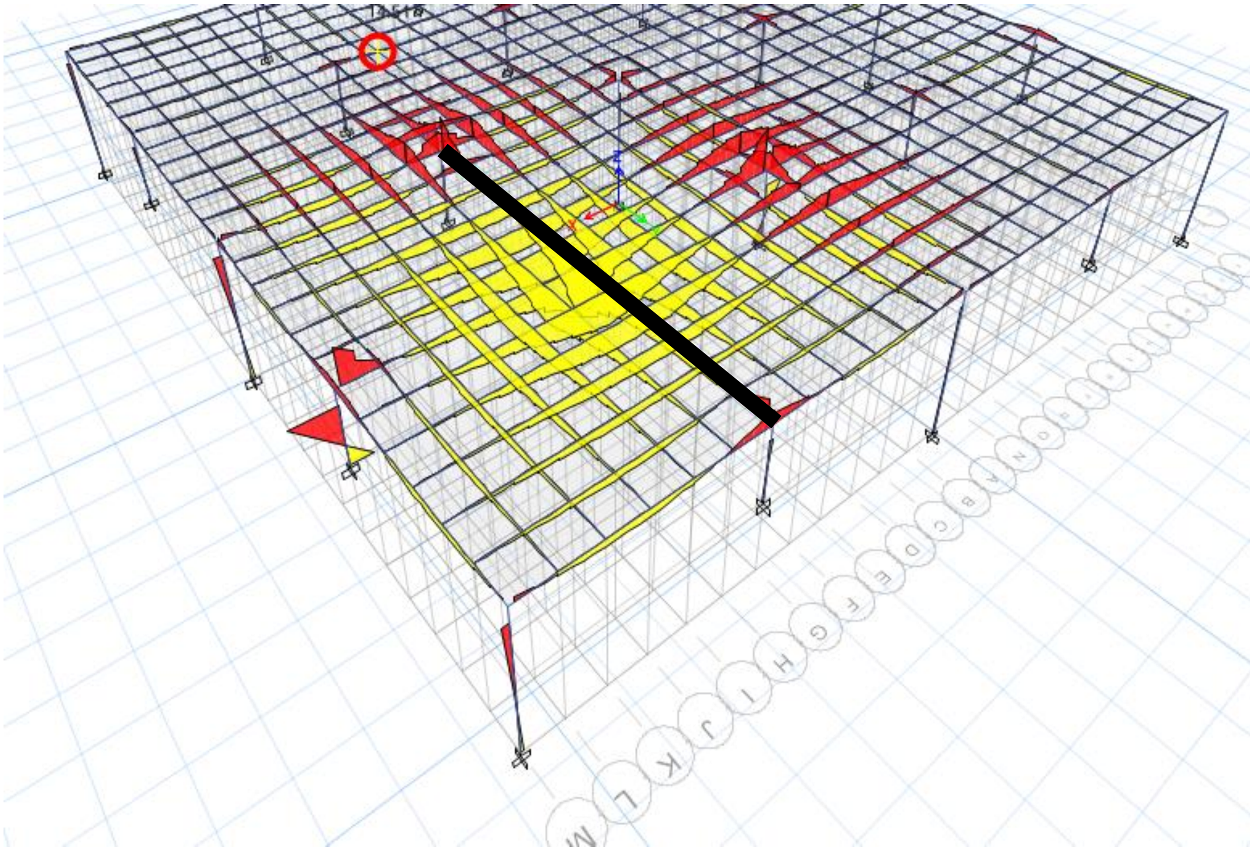
### 13.2.3 ETABS

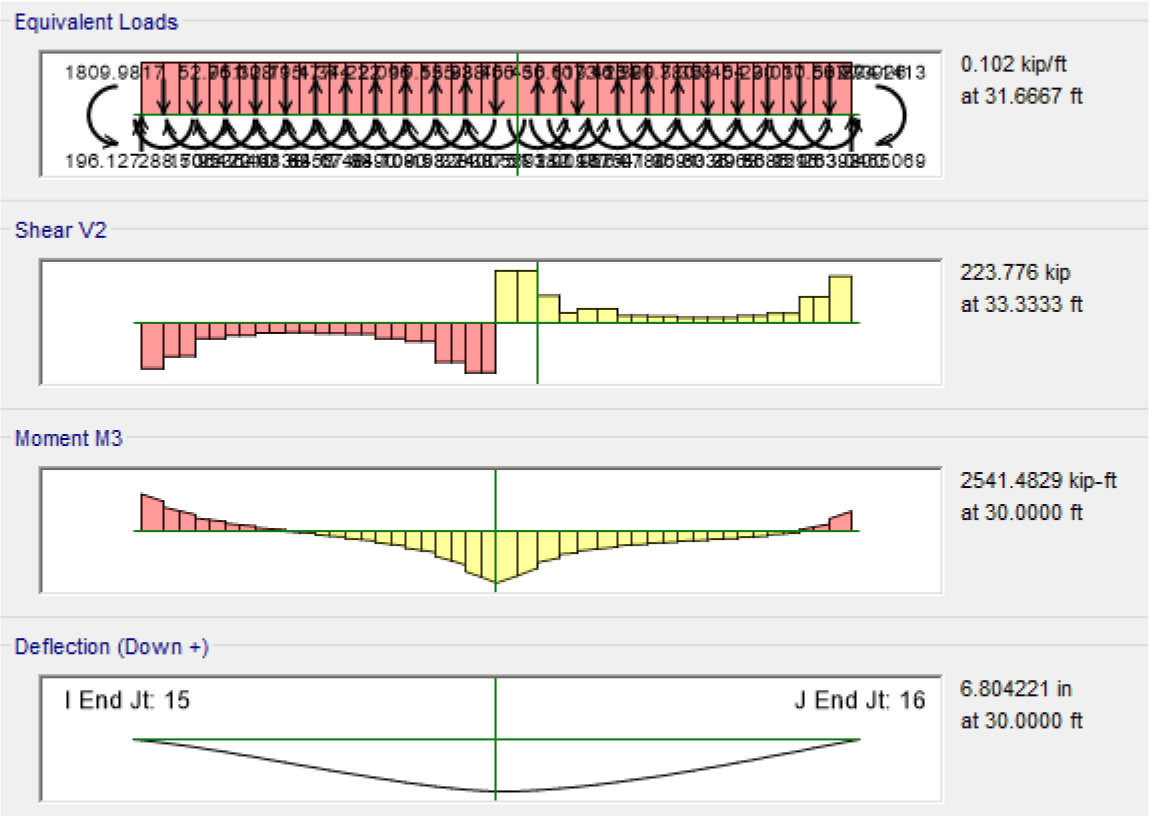

Joint Element: 2152
Story: Pent Rof
Ux = 2.610988
Uy = 0.231764
Uz = -0.741223
Rx = 0.000158
Ry = 0.001314
Rz = 0.000219


Height vs Displacement
Height (feet)
Displacement (inches)
0
20
40
60
80
100
120
140
160
180
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Wind N-S
Wind E-W
Seismic N-S

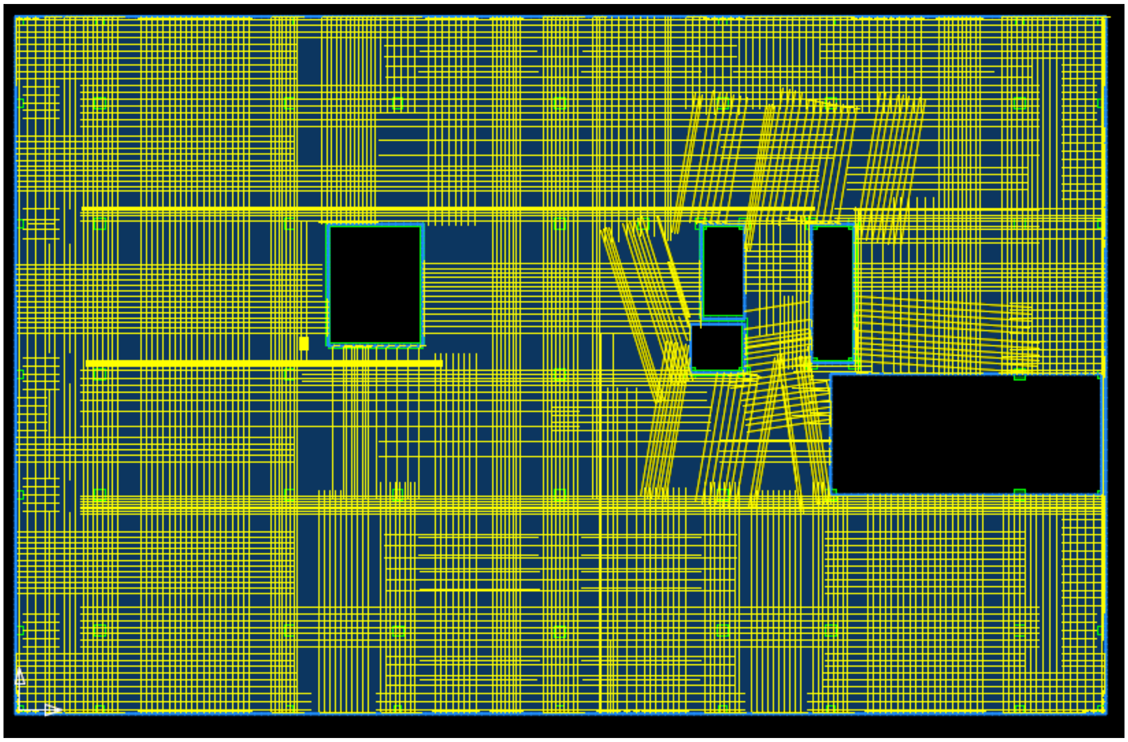

Z
X
Y
A
D

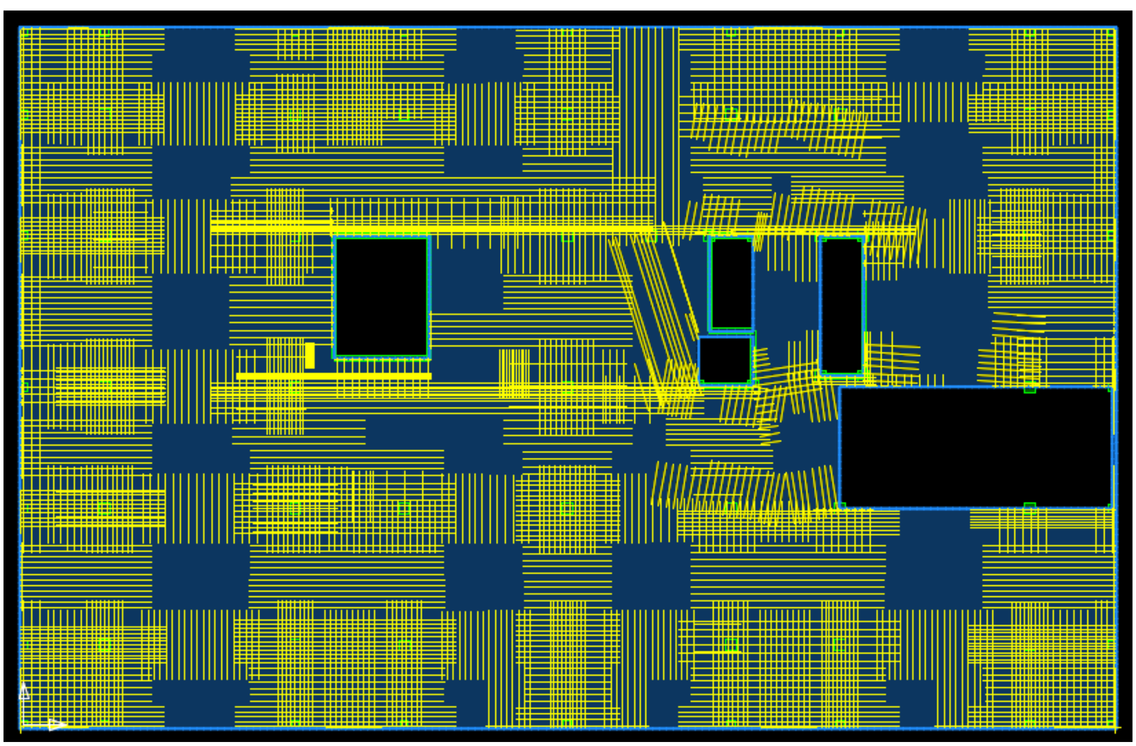

Equivalent Loads
36.6543
238.318
586.814
238.318
36.6543
55.905
387.2844
387.2844
55.905
0.102 kip/ft
at 12.0000 ft
Shear V2
-293.815 kip
at 4.0000 ft
Moment M3
1821.1883 kip-ft
at 8.0000 ft
Deflection (Down +)
I End Jt: 1
J End Jt: 2
0.645417 in
at 8.0000 ft

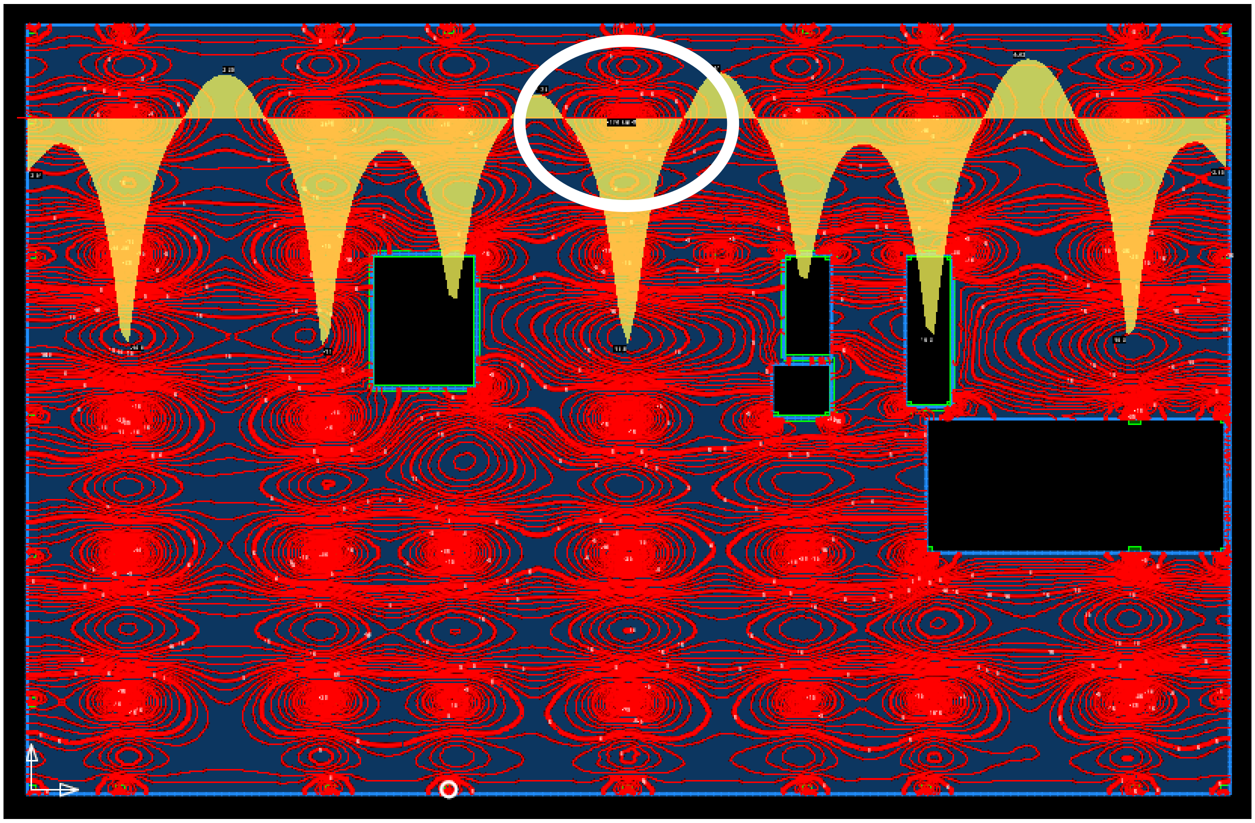


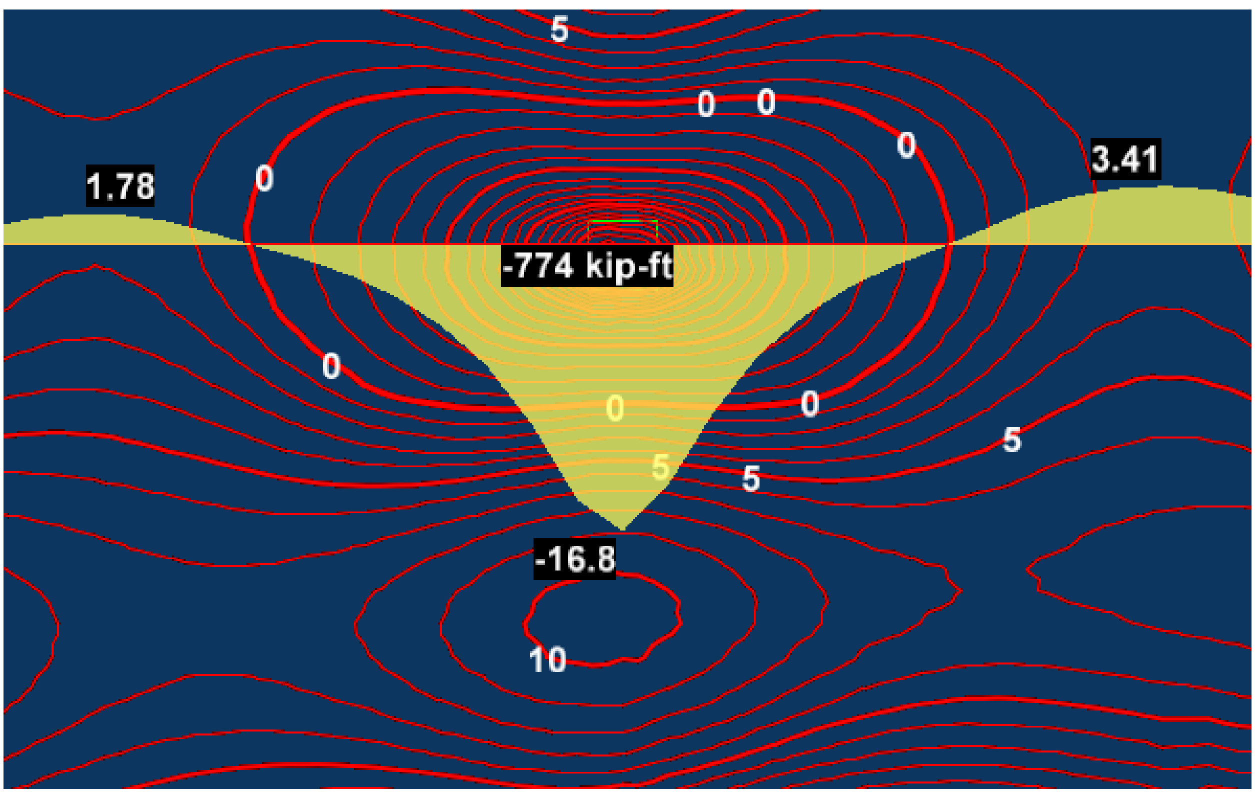

Equivalent Loads
0.102 kip/ft
at 31.6667 ft
Shear V2
223.776 kip
at 33.3333 ft
Moment M3
2541.4829 kip-ft
at 30.0000 ft
Deflection (Down +)
I End Jt: 15
J End Jt: 16
6.804221 in
at 30.0000 ft

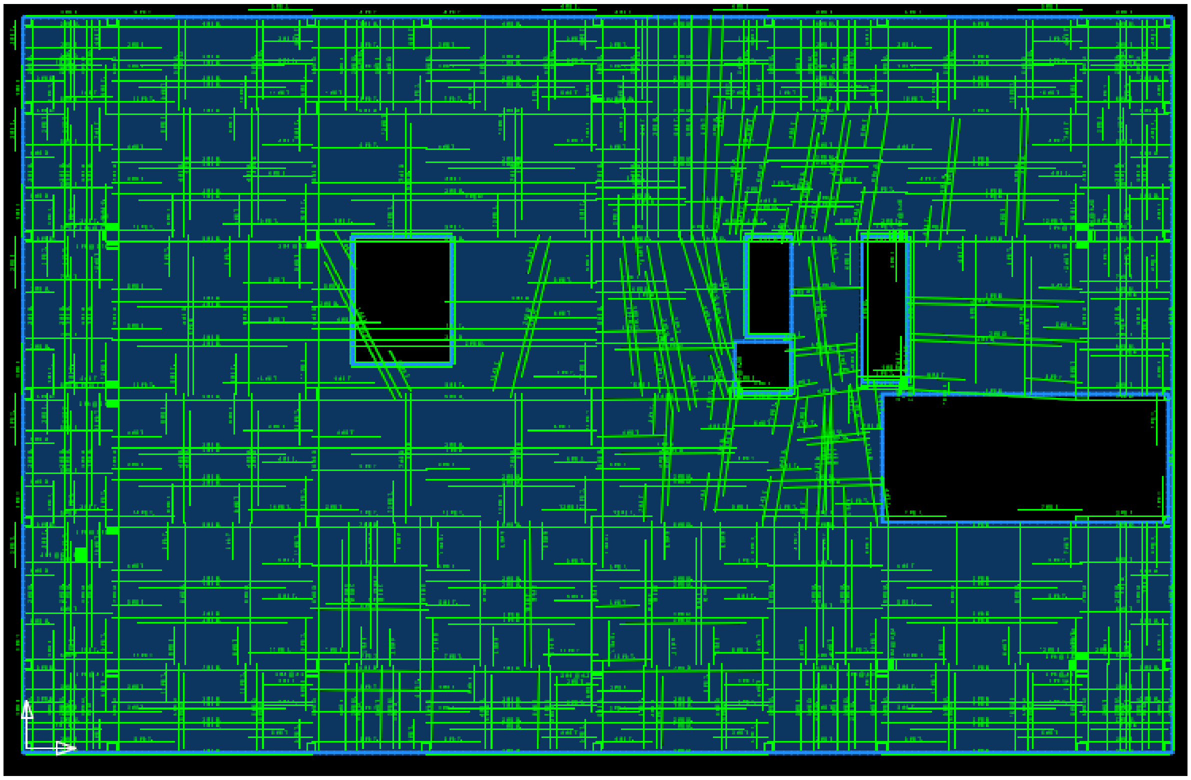
### 13.2.4 RAM Concept

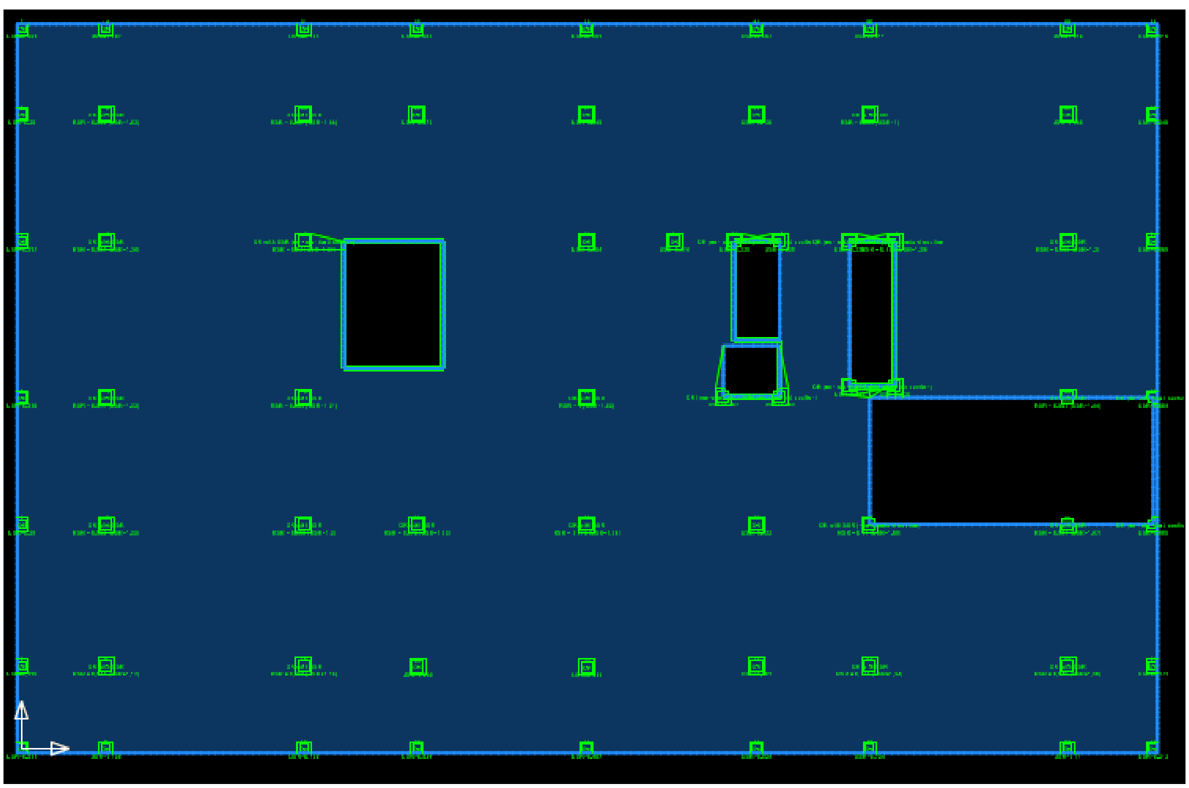







5
0
0
0
1.78
0
3.41
-774 kip-ft
0
0
0
5
5
5
-16.8
10





# 13.3 Hand Calculations

# 13.3.1 Gravity System

# Design Loads

## Live Load

- previously calculated

Choose between the two

$L_{red} = 64$ psf

$L_o = 100$ psf Table 4-1 ASCE

## Dead Load

- Design strip of Typical Bay was 10.5"

Choose 12" for Leeway of later design

$DL = 1\text{ft} \cdot 150 \text{ lbs}/\text{ft}^3$

$DL = 150$ psf

## Snow Load

- Previously calculated as

$SL = 20$ psf

+ 50 psf due to drift 11.5' from central wall.

## Lateral Load

- Previously Calculated
  - for wind and seismic vs story height see previous report.

# Perimeter Column Estimate

- Non Sway
- DL = 170 psf
  LL = 64 psf
  total factored = 306 lb/ft$^2$
- Largest Trib Area
  $= \left(\frac{22.5}{2} + \frac{25}{2}\right)(13/2)$
  $= 155$ ft$^2$
- 11 floors
  155 ft$^2$ × 306 lb/ft$^2$ = 47,430 lbs × 11 floors
  = 521.7 kips ← largest force on perimeter column due to dead + Live
- Assumed eccentricity is half an inch

  $M = P_u \cdot 0.5$
  $= 261$ kips-ft

- estimation of column size

$$A_g \geq \frac{P_u}{0.4(f'c + f_y \cdot \rho_g)}$$

$$A_g \geq \frac{522{,}000}{0.4(8000 + 60{,}000 \cdot 0.015)}$$

$A_g \geq 147$ in$^2$   Need larger than a 12"x12" column

Choose 18"x18" column ← Needs to decrease with height but relish

Note

- f'c would follow stiffness of existing columns which is 8 ksi
- $f_y$ = 60,000 psi
- $\rho$ = 0.015

## Exterior Column Design

General Assumptions / notes

- design is for most stringent column (with largest trib area)
- $f'_c$ is not to change (follows original construction)

- DL = 167.5 psf
- LL = 64 psf

- Load on 1st floor Column

$= 1.2(167.5) + 1.6(64)$

$= 0.303\ k/ft^2 \times$ Trib area

$=$ 68.175 kips · 12 stories above

$=$ 820 kips ← largest gravity load on Ext column

- Trib Area (max)

$= 30 \times 7.5$

$= 225\ ft^2$

Note
- for 7 ksi max $P_u$ = 682 kips
- for 6 ksi, max $P_u$ = 545 kips

- Estimation (Already 18x18)

where $\rho_g = 0.015$

$$A_g \geq \frac{P_u}{0.4(f'_c + f_y \rho_g)}$$

$$> \frac{820}{0.4(8 + 60 \cdot 0.015)} = 230 \text{ so } 18\ in^2 \text{ is ok}$$

**Note:** ~~a 16x16 would work but this column will need to resized for blast anyway~~

16x16 works for blast

$A_g$ for 16x16

$= 256\ in^2$

- slenderness effects ACI 10.12.2

$$k l_u / r \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40$$

$$\frac{0.8(12 \cdot 11)}{0.3 \cdot 18} \qquad (1)$$

$$12.22 \leq 22 \leq 40$$

∴ slenderness effects can be neglected

° Calculating $A_{st}$

$$\phi P_{n\,max} = 0.8\,\phi\left[0.85 f'c\,(A_g - A_{st}) + f_y A_{st}\right]$$

$$820 = 0.8(0.65)\left[0.85 \cdot 8\,([illegible] - A_{st}) + 60 \cdot A_{st}\right]$$

$$0.52\,(1740 + 53.2 A_{st})$$

$$820 = 905.216 + 27.6\,A_{st}$$

$A_{st}$ not needed but chosen to use 4 #8 bars anyway

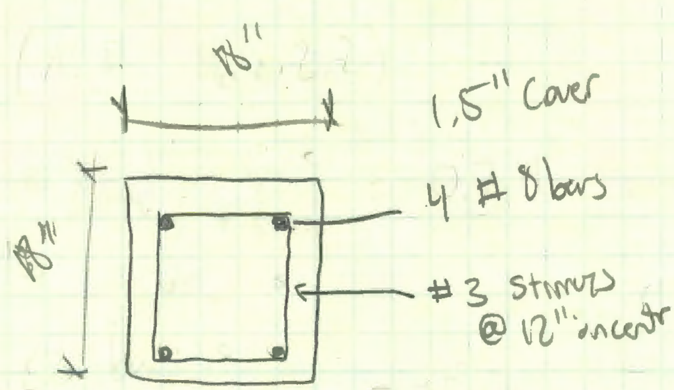
800

$\phi P_n = 990$ kips — fairly close to value from spColumn

• Design of ties    #3 chosen @ 12"    ACI 10.6.2

Spacing min
- 16 dbars = 12.6" — Controls
- 48 dties = 18
- least lateral dimension of column = 16"

Spacing of ties is 12" on center



# Interior Column Design

- DL = 167.5 psf
- LL = 64 psf
- Trib are for typ colum

Largest = 720 ft²

= 600 ft²

largest ≈ second largest

- Load on first floor

$= 0.303 \text{ kip/ft}^2 \cdot 600$ ← factored Load

$= 182 \text{ k} \cdot 12 \text{ stories}$

$= 2181.5 \text{ kips}$

- Estimation

$$A_g \geq \frac{P_u}{0.4(f'c + f_y \rho_g)}$$

$$\geq \frac{2617.9}{0.4(8 + 60 \cdot 0.015)}$$

$$A_g \geq 735.4 \text{ in}^2$$

$24 \times 24 = 576 < 612.7 \text{ in}^2$

for $24 \times 30$ $A_g = 720 \text{ in}^2$

Steel will contribute for the rest

- Columns that can be 24x24

$$576 \geq \frac{P_u}{0.4(f'c + f_y \rho_g)}$$

$$2050.56 \geq P_u$$

$P_u = \text{Load} \cdot \text{Area} \cdot 12 \text{ stories}$

$2050.5 = 0.303 \cdot \text{Area} \cdot 12$

$\text{Area} \leq 563.9 \text{ ft}^2$

- Slenderness effects

$$\frac{k l_u}{r} \leq 34 - 12 \frac{M_1}{M_2} \leq 40$$

$$= \frac{0.5(12 \cdot 11)}{0.3 \cdot} \quad 7.14 \leq 22 \leq 40$$

slenderness can be avoided / neglected

for 1st floor → 12.08

• Calculating $A_{st}$ for 24x24 and 24x30

$$\phi P_{n,max} = 0.8 \cdot \phi \left[0.85 \cdot f'_c (A_g - A_{st}) + f_y \cdot A_{st}\right]$$

24x24

$$2050 = 0.8 \cdot 0.65 \left[0.85 \cdot 8 (576 - A_{st}) + 60 \cdot A_{st}\right]$$

$$2050 = 0.52 \left[3916.8 + 53.2 A_{st}\right]$$

$$2050 = 2036.7 + 27.6 A_{st}$$

$A_{st} = 0.507$ — 8 #8 with — ACI 10.6.1

$\cancel{3.16}$ in$^2$ 6.2

24x30

$$2617.9 = 0.8 \cdot 0.65 \left[0.85 \cdot 8 (720 - A_{st}) + 60 \cdot A_{st}\right]$$

$$2617.9 = 0.52 \left[4896 + 53.2 A_{st}\right]$$

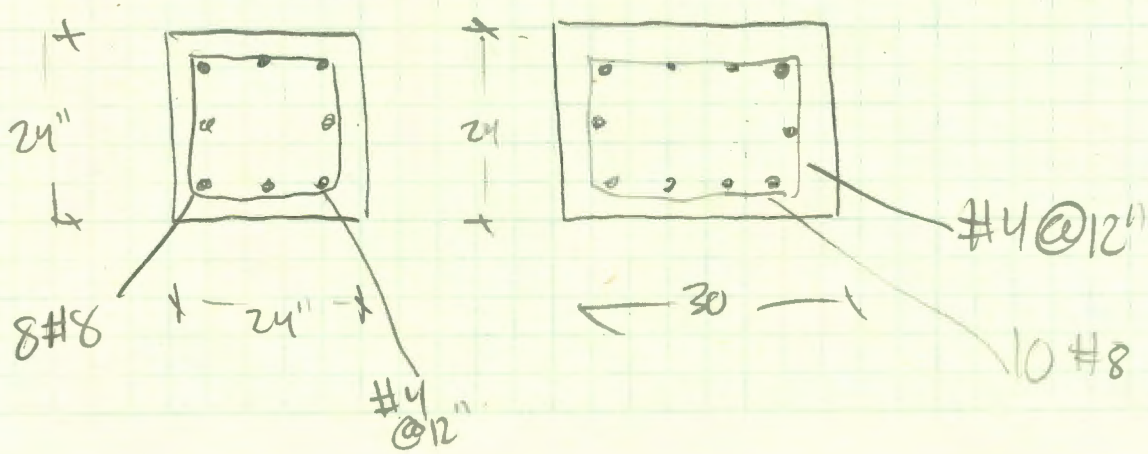
$$2617.9 = 2545.9 + 27.6 A_{st}$$

$A_{st} = 2.608$ — 10 #8    $A_{st} = \cancel{3.16}$ 7.19 in$^2$

10.6.1

ACI 10.6.2

Both 24x24 and 24x30 will have #4 stirrups @ 12"



# Column Design Per Floor

Trib Area Ext = 225 $ft^2$
Trib Area Int = 600 $ft^2$
Load = 0.303 k/ft

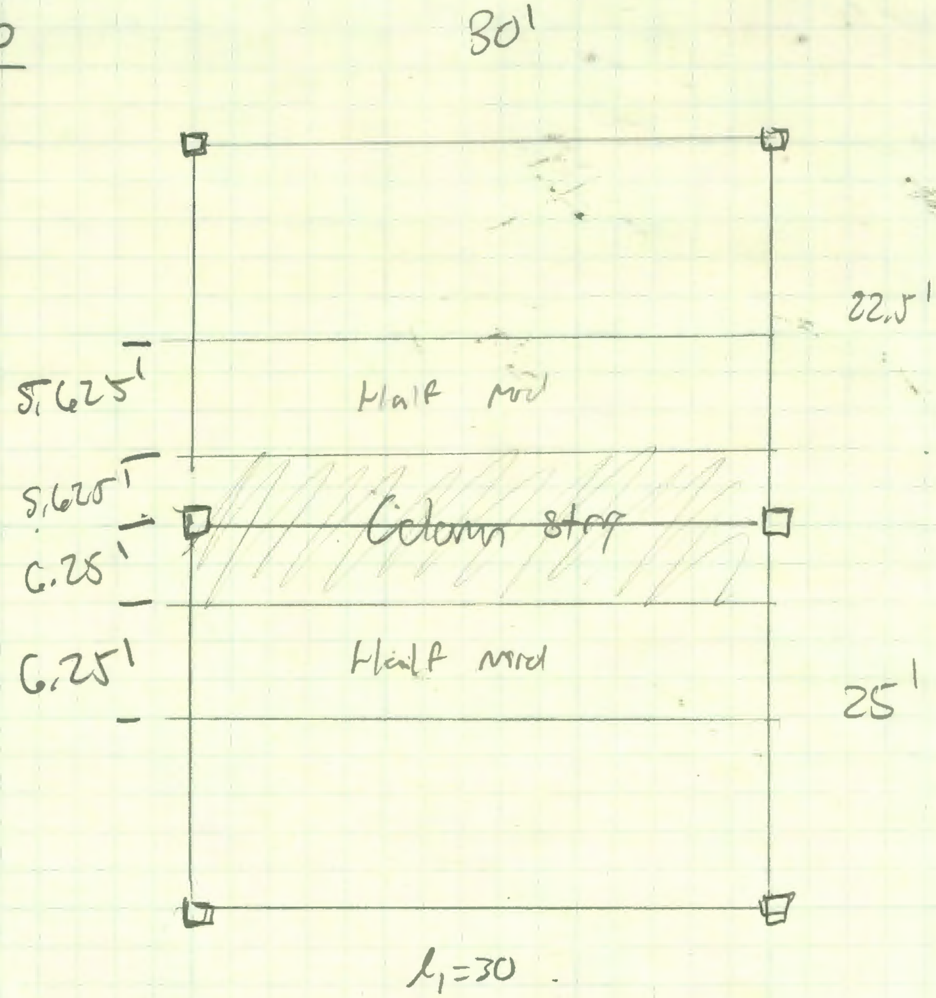
| Floors | INT Load | Ext Load | OG $b_1$ | New INT(1) | Ext | INT(2) |
|---|---|---|---|---|---|---|
| 1 | 2363.5 | 893.15 | 8 | 8 | 6 | 8 |
| 2 | 2181.5 | 824.4 | 8 | 8 | 6 | 8 |
| 3 | 2000 | 755.5 | 7 | 6 | 6 | 6 |
| 4 | 1818 | 686.8 | 7 | 6 | 6 | 6 |
| 5 | 1636 | 618 | 6 | 6 | 6 | 6 |
| 6 | 1454.5 | 550 | 6 | 4 | 4 | 4 |
| 7 | 1272.5 | 480.4 | 6 | 4 | 4 | 4 |
| 8 | 1091 | 411.5 | 6 | 3 | 3 | 3 |
| 9 | 909 | 342.75 | 6 | 3 | 3 | 3 |
| 10 | 727.2 | 274 | 6 | 3 | 3 | 3 |
| 11 | 545.4 | 205.15 | 6 | 3 | 3 | 3 |
| Pen | 363.6 | 136.4 | 6 | 3 | 3 | 3 |
| Mezz | 181.8 | 68.18 | 6 | 3 | 3 | 3 |
| | | | | 30x24 | 18x18 | 24x24 |

Limitations for Direct Design Method ACI 8.10.2

- 3 spans (continuous in each direction) ✓
- Span length (support - support) shall not differ by more than 33% ✓
- Rectangular Panels not to exceed 2 in ratio ✓
- Column offset shall not exceed 10% of span in direction of offset ✓
- Gravity loads uniformly distributed ✓
- Unfactored LL < 2 · Unfactored DL ✓
- Panel with beams on all sides N/A

## Redesign of Slab

- $f_y = 60$ ksi
- Columns are 24"x24" assumed
- $f'_c = 4000$




## Slab Design

- ACI table 8.3.1.1

Interior Panel w/o drop panel

$$\frac{l_n}{36} \qquad \frac{30 \cdot 12}{36} = 10" \quad \text{choose } 11" \text{ for conservativeness}$$

$l_n = 30'$

## Column Strip

$$= \min \begin{cases} l_2/4 = \dfrac{22.5}{4} \text{ or } \dfrac{25}{4} = 6.25 \text{ and } 5.625 \leftarrow \text{controls} \\ l_1/4 = 30/4 = 7.5 \end{cases}$$

## Half middle Strip

$$= 25' - 2(6.25)/2 = 6.25 \quad \text{Same as column}$$

$$22.5' - 2(5.625)/2 = 5.625 \quad \text{Same as column}$$

## Loads

- LL = 64 psf
- DL = Selfweight of slab

$150 \, lb/ft^3 \cdot 11''/12'' = 137.5$

$+ \; 30$ psf – from MEP and Partitions

$= 167.5$ psf

- Factored loads

$= 1.2(DL) + 1.6(LL)$

$= 303.5$ psf

$q_u = 0.303$ ksf

## Design Moments

Controlling

- $M_0 = \dfrac{q_u \cdot l_2 \cdot l_n^2}{8} = \dfrac{0.303 \cdot (25) \cdot (30)^2}{8} = 852.2$ kip·ft

- Dividing $M_0$

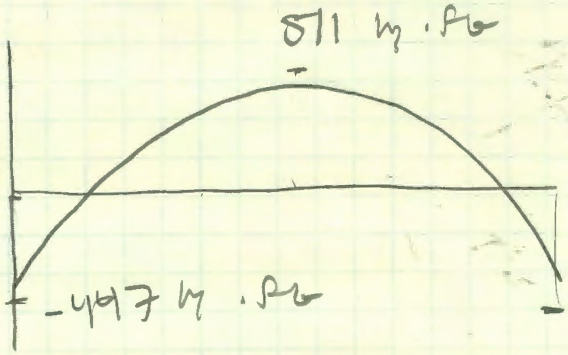
ACI 8.10.4.1

$M_0 = 852.2$

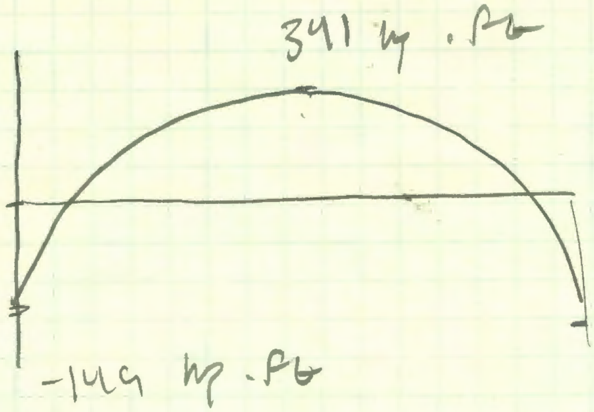
- Int Neg $0.7 = -596$
  - $0.75$ → $-447$ column strip
  - $0.25$ → $-149$ middle strip
- Positive
  - $0.6 = 511$ column strip
  - $0.4 = 341$ middle strip
- Ext Neg $0.3 = -256$
  - No Edge beam
  - (Interior bay

- Moment Diagrams

-Column Strip




-Middle Strip




Mid

$$A_{s,req} = \frac{341 \cdot 12}{0.85(60)12 \cdot 10} = 0.6\ in^2 > A_{s,min}$$

Choose

#7 @ 12"

$T_{op+Bott}$ EE

## Reinforcement

- ACI 8.6.1.1

1' section

$$A_{s,min} = 0.0018 \cdot b \cdot h$$
$$= 0.0018(12)(11)$$
$$= 0.2376\ in^2/ft$$

- ACI 8.7.2.2

$$S_{max} = 2 \cdot \text{slab} = 22''$$

or 18" ← controls

∴ ~~Choose #5 @ 12"~~

slab

$d = 11''$
$-0.75''$
$-0.5\,d_b$

$d = 10''$

- $A_{s,req} = \dfrac{M_u}{\phi f_y \cdot jd} = \dfrac{511 \leftarrow \max \cdot 12''/ft}{0.85(60) \cdot 12 \cdot 10}$

$$= 0.896\ in^2 > \#5 \text{ bars}$$

- Use #9 @ 12"

## Sheer

- $V_u = q_{DU} \cdot (30 \times 25) + q_{LU}(30 \times 25/2)$
  $= 150750 + 38400$
  $= 189150$ lbs

- $M_u = M_{slab} = 0.07\left[(q_{DU} + 0.5\, q_{LU}) \cdot l_2 \cdot l_n^2 - q_{DU}' \cdot l' \cdot l_n'^2\right]$

  $= 0.07\left[(201 + 0.5(1024)) \cdot 25 \cdot 30^2 - 201 \cdot 25 \cdot 30^2\right]$

  $5674500 - 4522500$

  $= 80.6$ k·ft

- Max $V_u$

$$V_{umax} = \frac{V_u}{b_o d} + \frac{\gamma_v \cdot M_u \cdot c}{J_c} = \frac{189150}{136 \cdot 10} + \frac{0.4 \cdot 806000 \cdot 17}{267693.3}$$

$= 142$ psi

- $\gamma_v = 1 - \gamma_f = 0.4$

- $b_o = 2(b_1 + b_2)$
  $= 136''$

- $\gamma_f = \dfrac{1}{1 + \frac{2}{3}\sqrt{b_1/b_2}} = 0.6$

- $b_1 = d + x = 10 + 24 = 34''$
  $b_1 = b_2 = 34''$ $\quad b_1/b_2 = 1$

- $c = b_1/2 = 17''$

- $J_c = 2\left(\dfrac{b_1 \cdot d^3}{12} + \dfrac{d b_1^3}{12}\right) + 2(b_2 d)(b_1/2)^2$

  $= 2\left(\dfrac{34 \cdot 10^3}{12} + \dfrac{10 \cdot 34^3}{12}\right) + 2(10 \cdot 34)(34/2)^2$

  $= 267693.3$

$$\phi V_c = \min \begin{cases} \phi \cdot 4 \cdot \sqrt{f'_c} \cdot b_o \cdot d = 0.65 \cdot 4 \cdot \sqrt{4000} \cdot 136 \cdot 10 = 223.6 \text{ psi} \\ \left(2 + \frac{4}{1}\right)\sqrt{f'_c} \cdot b_o \cdot d = 516 \\ \left(\frac{40 \cdot d}{b_o} + 2\right)\sqrt{f'_c} \cdot b_o \cdot d = 428 \end{cases}$$

↑ governs

ACI 22.6.5.2

$\phi V_c = 223.6$ psi $> v_{u,max} = 141$ psi

OK ✓

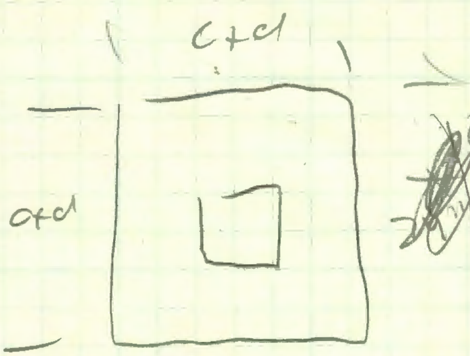
• One way shear

$V_u = q_u \cdot A_{influence}$

$= 0.303\,(c+d)\,2$

$= 0.303\,(24+10)\,2$

$= 20.6$ kips




$\phi V_c = 0.75 \cdot 2\sqrt{f'_c} \cdot b \cdot d$

$= 0.75 \cdot 2\sqrt{4000} \cdot 136 \cdot 10$

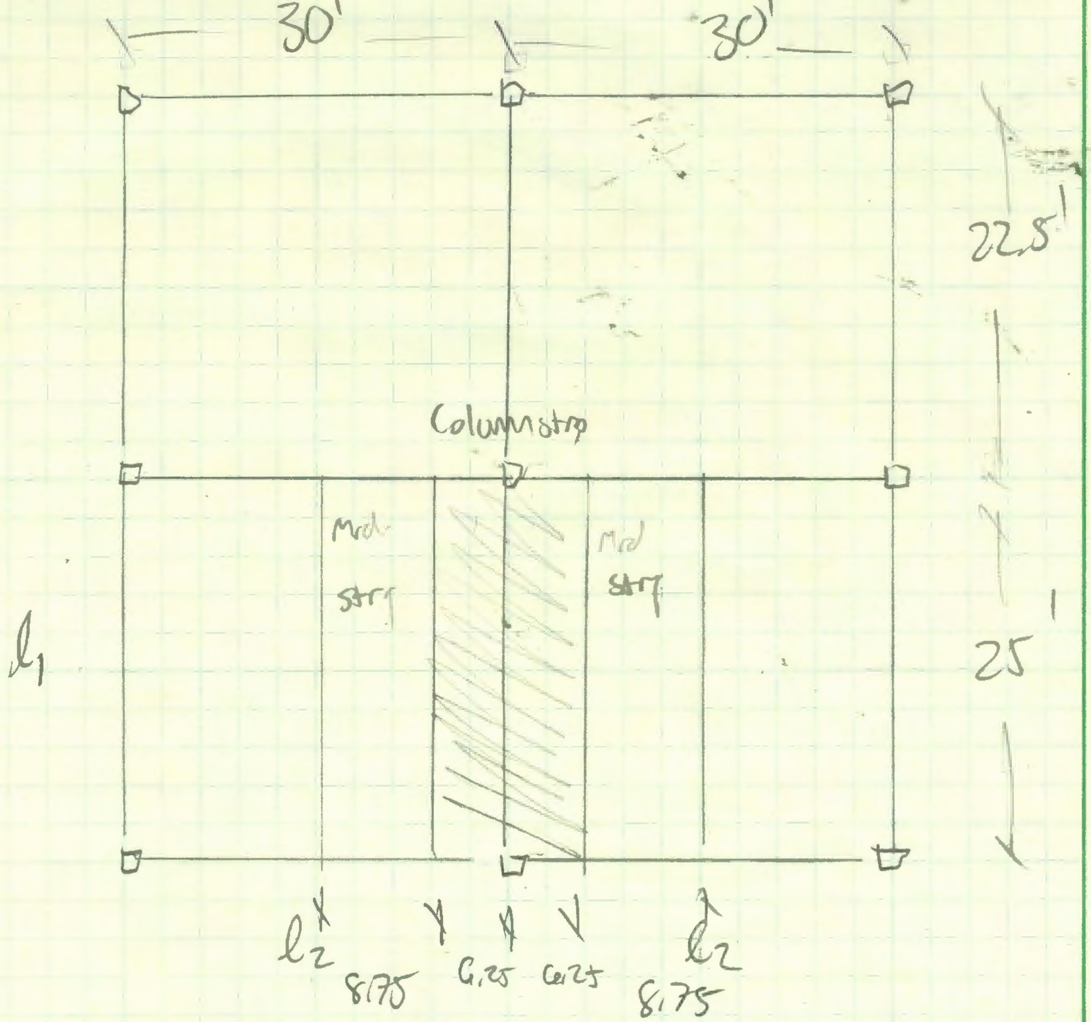
$\phi V_c = 129$ kips $> V_u$ ∴ OK

Other Direction

$l_1 = 28'$

$l_2 = 30'$

$l_n = 25'$




Column Strip

$= \text{Min} \quad l_2/4 = 30/4$

$l_1/4 = 25/4 \leftarrow 6.25$

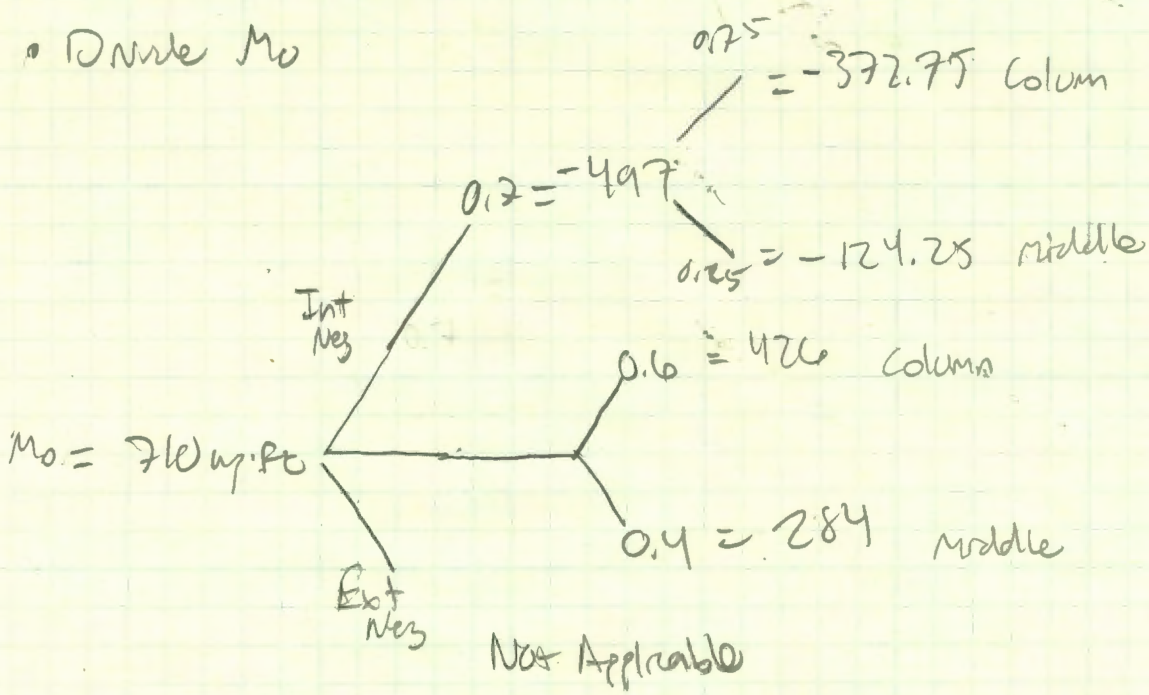
Middle Strip

$$= \frac{30 - 2(6.25)}{2} = 8.75''$$

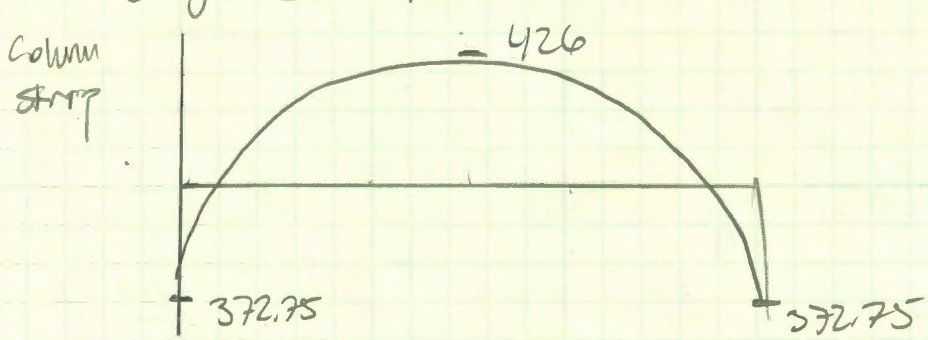
## Design Moments

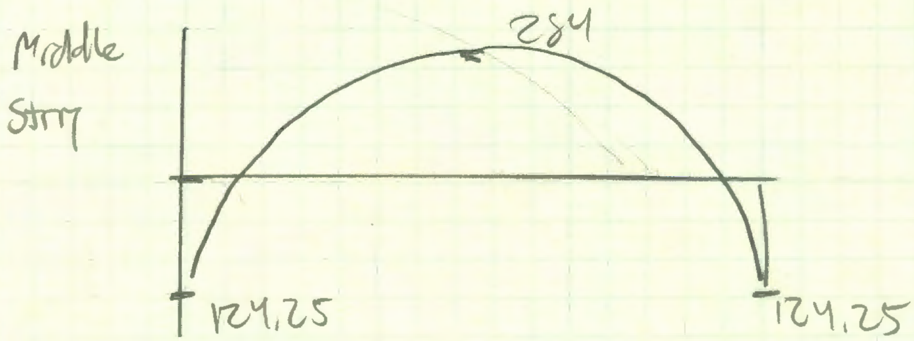
- $M_o = \frac{q_u \cdot l_2 \cdot l_n^2}{8} = \frac{0.303(30)(25)^2}{8} = 710\ k \cdot ft$

- Divide $M_o$




- Diagrams k·ft



## Design Moments

• $M_o = \frac{q_u \cdot l_2 \cdot l_n^2}{8} = \frac{0.303(30)(25)^2}{8} = 710\ k \cdot ft$

• Divide $M_o$

$M_o = 710\ k \cdot ft$

Int Neg $0.7 = -497$

$0.75 = -372.75$ Column

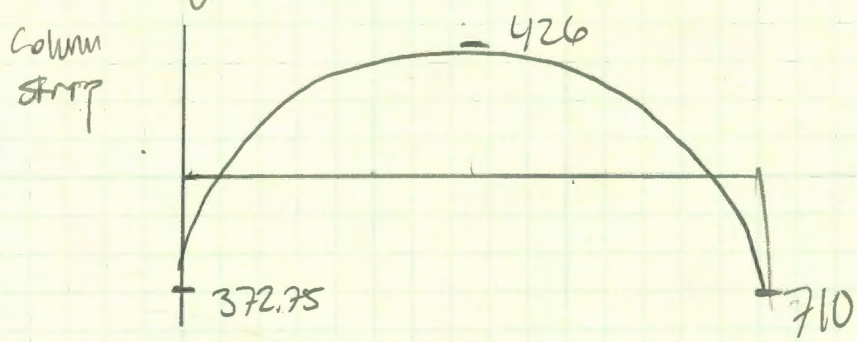
$0.25 = -124.25$ middle

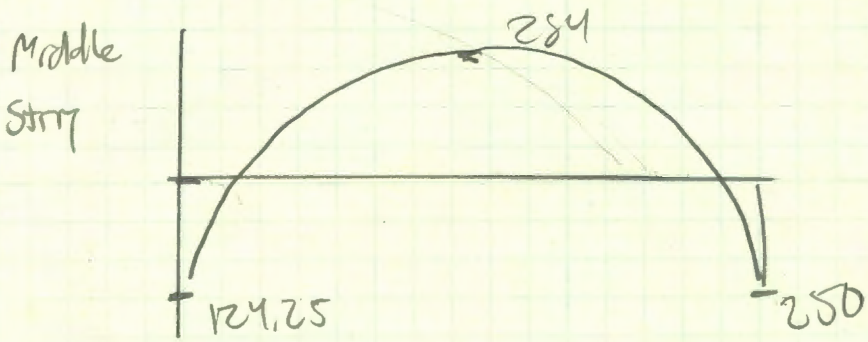
$0.6 = 426$ Column

$0.4 = 284$ Middle

Ext Neg Not Applicable

• Diagrams $k \cdot ft$



## Reinforcement

- $As_{min} = 0.0018\, b \cdot h$

  $= 0.2376\ in^2/ft$

- $S_{max} = 2$ slab thickness $= 22$

  or

  $18''$ $\;= 18 \leftarrow$ governs

- $As_{req} = \dfrac{Mu}{\phi f_y \cdot jd} = \dfrac{426 \cdot 12}{0.9(60) \cdot 12 \cdot 0.9 \cdot 10} = 0.876\ in^2/ft$

  column

$As_{req} = \dfrac{Mu}{\phi f_y \cdot jd} = \dfrac{284}{0.9(60) \cdot 12 \cdot 0.9 \cdot 10} = 0.584\ in^2/ft$

middles

Choose #9@12" for Column strip $As = 1\ in^2/ft$

Both ways

Choose #7 @ 12" for Middle strip $As = 0.79\ in^2/ft$

both ways

# Shear

- $V_u = 189150$ lbs — same parameters as previous calc only backwards
  $30 \times 25 = 25 \times 30$

- $M_u = M_{slab} = 0.07\left[(q_{DL} + 0.5 q_{LL})(\ell_2)(\ell_n)^2 - q_{DL}' \cdot \ell' \cdot \ell'^2\right]$

$= 0.07\left[(201 + 0.5(102.4))(30)(25)^2 - 201(30)(25)^2\right]$

$= \{\, 4728750 - 3768750 \,\}$
960000

$= 67200$ lb·ft
$= 67.2$ kip·ft

- Max shear stress

$$V_{u,max} = \frac{V_u}{b_o d} + \frac{\gamma_v \cdot M_u \cdot c}{J_c}$$

$$= \frac{189150}{136 \cdot 10} + \frac{0.4(67200)(17)}{267693.3}$$

$= 139 + 1.7$
$\approx 142$ psi

$b_o = 136$
$b_1 = b_2 = 34$
$d = 10''$
$c = 17$
$J = 267693.3$

Same as previous calculation

- $\phi V_c = \min \begin{cases} \phi \cdot 4 \cdot \sqrt{f'c} \cdot b_o \cdot d = 223.6 \text{ governs} \\ \left(2 + \frac{4}{1}\right)\sqrt{f'c} \cdot b_o \cdot d = 516 \\ \left(\frac{40 \cdot d}{b_o} + 2\right)\left(\sqrt{f'c} \cdot b_o \cdot d\right) = 425 \end{cases}$

$\phi V_c \geq V_{u,max}$

$\underset{psi}{223.6} \geq \underset{psi}{142}$ ∴ ok ✓

- One way Shear

$V_u = q_u \cdot A_{influence}$

$= 20.6 \text{ kips}$ ← same as before

$\phi V_c = 0.75 \cdot 2 \cdot \sqrt{f'c} \cdot b \cdot d$

$= 129$

$\phi V_c > V_u \quad 129 > 20.6$ ∴ ok

## Design of Negative Moment Reinforcement

- Max Neg Moment = 400 k · ft
  @ column face

- $A_{s\,min} = 0.0018\, b \cdot h$
  $= 0.2376\ in^2/ft$

- $S_{max} = \underline{18''}$ or $22''$

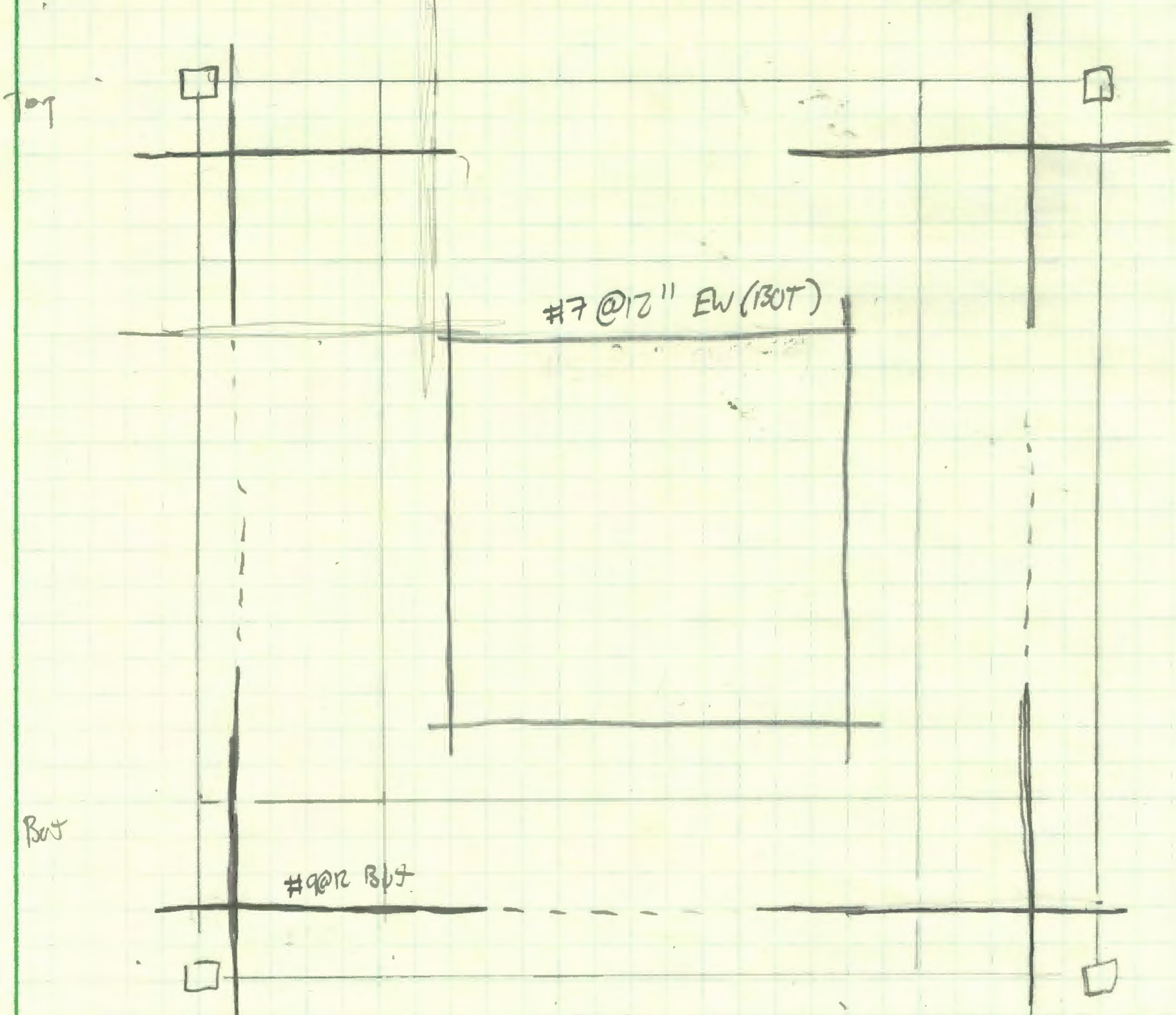
- $A_{s\,req}$ column strip $= \dfrac{M_u}{\phi f_y \cdot j \cdot d} = \dfrac{492 \cdot 12}{0.9(60)(12)(0.9)(10)} = 1.02\ in^2/ft$

Choose #9 @ 12" for Top Reinforcement

- $A_{s\,req}$ middle strip $= \dfrac{180 \cdot 12}{0.6 \cdot 60 \cdot 0.9 \cdot 12 \cdot 10} = 0.463\ in^2/ft$

choose #7 @ 12" for Top Reinforcement

# Interior Bay Detail



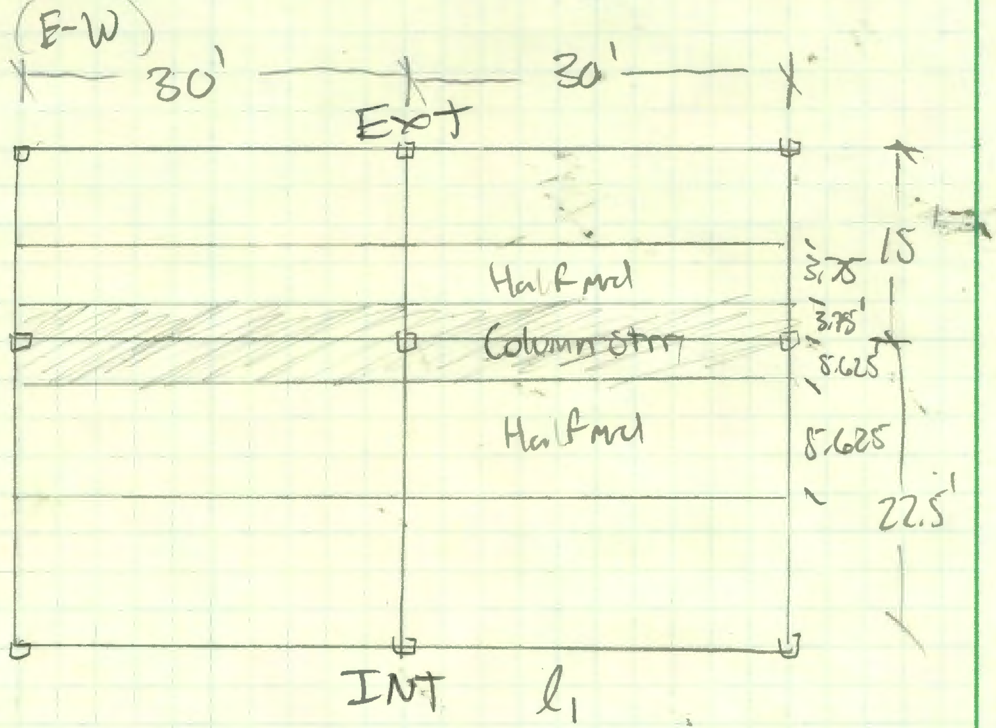

## Notes

- within 6.5' of column reinforcement shall be
  Bottom #9@12" EW

within 9' of column
Top #9 @ 12" EW
(Length is 9')

- outside 6.5' of column line
  Bottom #7@12" EW
  Top #7@12" EW (length is 6.5')

## Exterior Span (E-W)




- $f_y = 60$ ksi
- $f'_c = 4$ ksi
- Ext columns are 18"x18"
  Int columns are 24"x24"
- Slab thickness = 11"
- $l_n = 30'$

**Column strip**

$$= \min \begin{cases} l_2/4 = \begin{cases} 15/4 \\ 22.5/4 \end{cases} \quad 3.75' \text{ and } 5.625 \\ l_1/4 = \end{cases}$$

**Half middle strip**

$$= \frac{22.5 - 2(5.625)}{2} = 5.625'$$

$$= \frac{15 - 3.75(2)}{2} = 3.75'$$

**Loads**

- $LL = 64$ psf
- $DL = 167.5$ psf
- factored $q_u = 0.303$ ksf
- $M_o = \dfrac{q_u \cdot l_2 \cdot l_n^2}{8} = \dfrac{0.303(22.5)(30)^2}{8} = 767$ kip·ft

$$= \frac{0.303(15)(30)^2}{8} = 511 \text{ kip·ft}$$

22.5 span

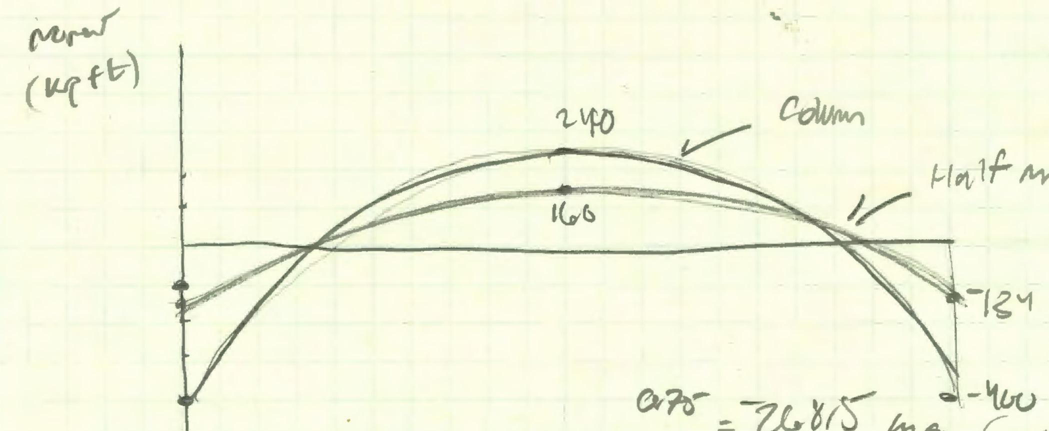
Int Neg — 0.7 = $-537$ k ft

- 0.75 = $-402.75$ (Column)
- 0.25 = $-134.25$ (middle)

$M_0$ = 767 k ft — Int Pos — 0.52 = 400 k ft

- 0.6 = 240 (Column)
- 0.4 = 160 (middle)

~~No ext Neg~~




15' span

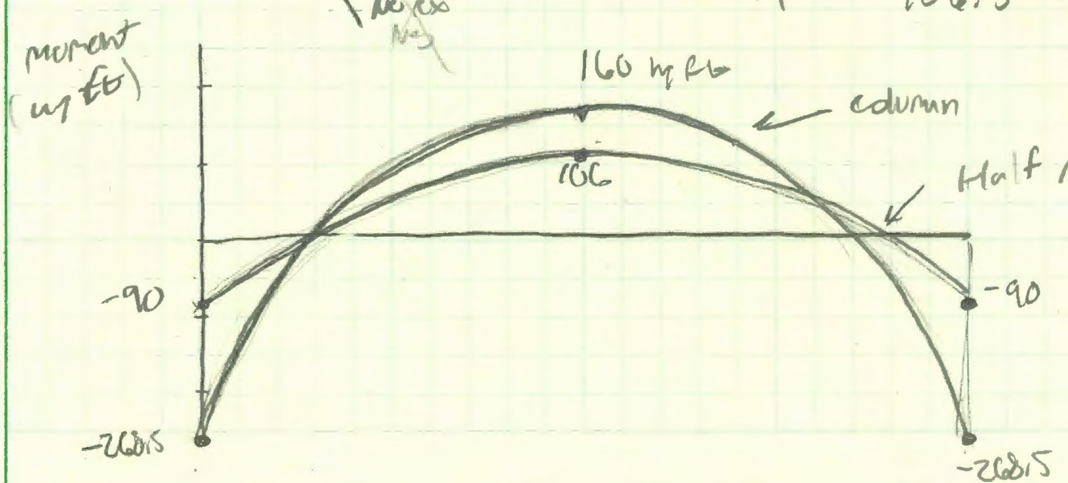
Int neg — 0.7 = $-358$

- 0.75 = $-268.5$ k ft (Column)
- 0.25 = $-90$ k ft (middle)

$M_0$ = 511 k ft — Int pos — 0.52 = 265.7

- 0.6 = 159.5 k ft (Column)
- 0.4 = 106.5 k ft (middle)

~~No ext Neg~~



## Reinforcement

• $As_{min} = 0.0018\, b \cdot h$
$= 0.0018(12)(11)$
$= 0.2376\ in^2/ft$

• $S_{max} = 2 \cdot slab = 22''$
or $18''$ ← controls

• $As_{req} = \dfrac{Mu \cdot 12}{\phi f_y \cdot j\, \underset{=10''}{d}} = \dfrac{400 \cdot 12}{0.45(60)(12)(10)} = 0.701\ in^2$

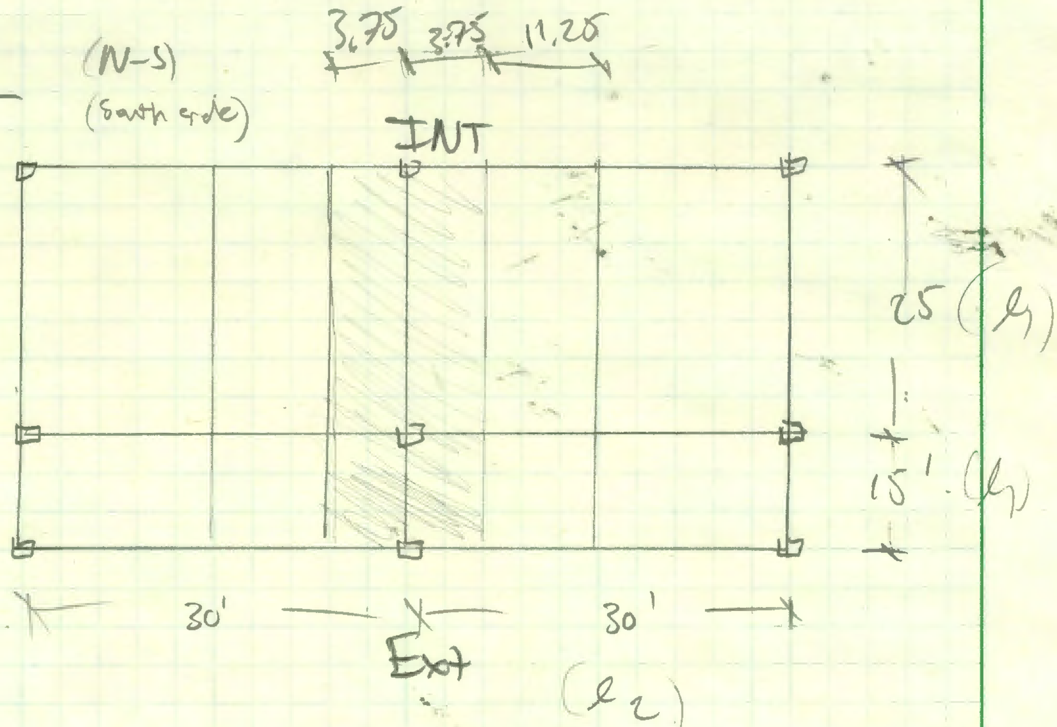
$j = 0.9$ (column strip)

$= \dfrac{Mu \cdot 12}{\phi f_y \cdot jd} = \dfrac{160 \cdot 12}{0.45(60)(12)(10)} = 0.28\ in^2$
(half mid)

• Choose
#8 @ 12" for column strip
#5 @ 12" for half middle strip

## Exterior Span

- $f_y = 60$ ksi
- $f'_c = 4$ ksi
- Ext columns 18"x18"
- Int columns 24"x24"
- Slab thickness = 11"
- $l_n = 25'$




**Column Strip**

$$= \min \begin{cases} l_2/4 = \cancel{30/4} \\ \text{or} \\ l_1/4 \begin{cases} 15/4 = 3.75 \leftarrow \text{controls} \\ 25/4 = 6.25 \end{cases} \end{cases}$$

**Half middle strip**

$$= \frac{30 - (3.75)(2)}{2} = 11.25'$$

$$l_n = \frac{25+15}{2} = 20'$$

$$M_o = q_u \cdot l_2 \cdot l_n^2 / 8$$

$$= \frac{0.303 \cdot 30 \cdot (25)^2}{8} = 710 \text{ k ft} \quad \text{INT}$$

$$= \frac{0.303 \cdot 30\,(15)^2}{8} = 255.6 \text{ k ft} \quad \text{EXT}$$

18' span

INT (−) 0.7 = −179
- 0.75 — −134.25 k-ft (Column)
- 0.25 — −44.75 k-ft (Mid)

$M_0$ = 256 k-ft — (+) 0.52 = 133
- 0.6 — 80 k-ft (Column)
- 0.4 — 83. k-ft (Mid)

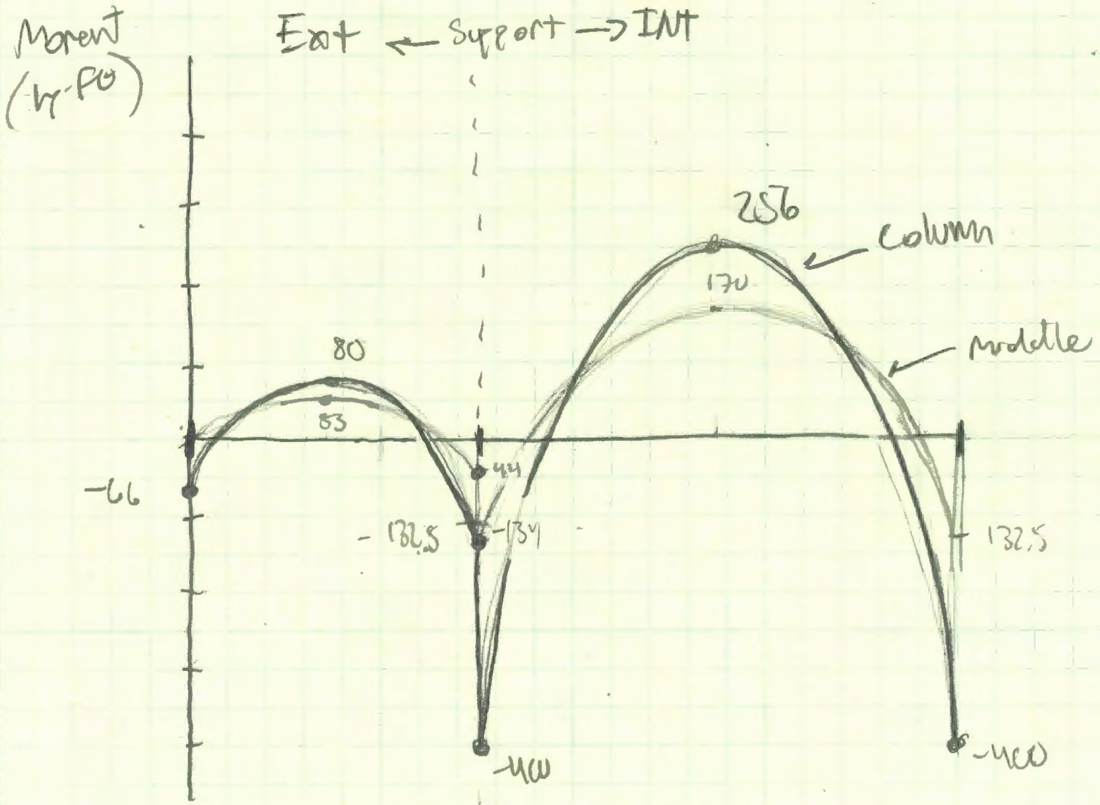
Ext (−) 0.26 = −66.5
- — −66.5 (Column)
- — 0 (Mid)

25' span

0.7 = −532.5 k-ft
- 0.75 = −400 k-ft (Column)
- 0.25 = −132.5 k-ft (middle)

$M_0$ = 710 k-ft — 0.6 = 426 k-ft
- 0.6 = 256 k-ft (Column)
- 0.4 = 170 k-ft (Middle)

No Ext Neg



Reinforcement

- $As_{min} = 0.0018\ b \cdot h$
  $= 0.2376\ in^2/ft$

- $S_{max} = 2 \cdot slab = 22''$
  or $18''$ ← Controls

- $As\ req = \dfrac{Mu \cdot 12}{\phi fy \cdot jd} = \dfrac{86 \cdot 12}{0.95(60)(12)(10)} = 0.14\ in^2$

Asmin Controls

For both column and middle strip

- Choose

#5 @ 12'' for Both column and middle strip

Design of Top Reinforcement

- Max Mu = 270 column E-W

  = 90 middle

  Max Mu = 133 column N-S

  = 44 middle

- $A_{s\,min} = 0.2376\ in^2/ft$

- $A_{s\,req}$ column E-W $= \dfrac{270 \cdot 12}{0.9(60)(0.9)(12)(10)} = 0.55\ in^2/ft$

  5832

- $A_{s\,req}$ middle E-W $= \dfrac{90 \cdot 12}{0.9 \cdot 60 \cdot 0.9 \cdot 12 \cdot 10} = 0.185\ in^2/ft < A_{s\,min}$

- $A_{s\,req}$ column N-S $= \dfrac{133 \cdot 12}{0.9 \cdot 60 \cdot 0.9 \cdot 12 \cdot 10} = 0.273\ in^2/ft$

- $A_{s\,req}$ middle N-S $= \dfrac{44 \cdot 12}{0.9 \cdot 60 \cdot 0.9 \cdot 12 \cdot 10} = < A_{s\,min}$

Designs

Column E-W choose #7 @ 12"

Midd E-W choose #5 @ 12"

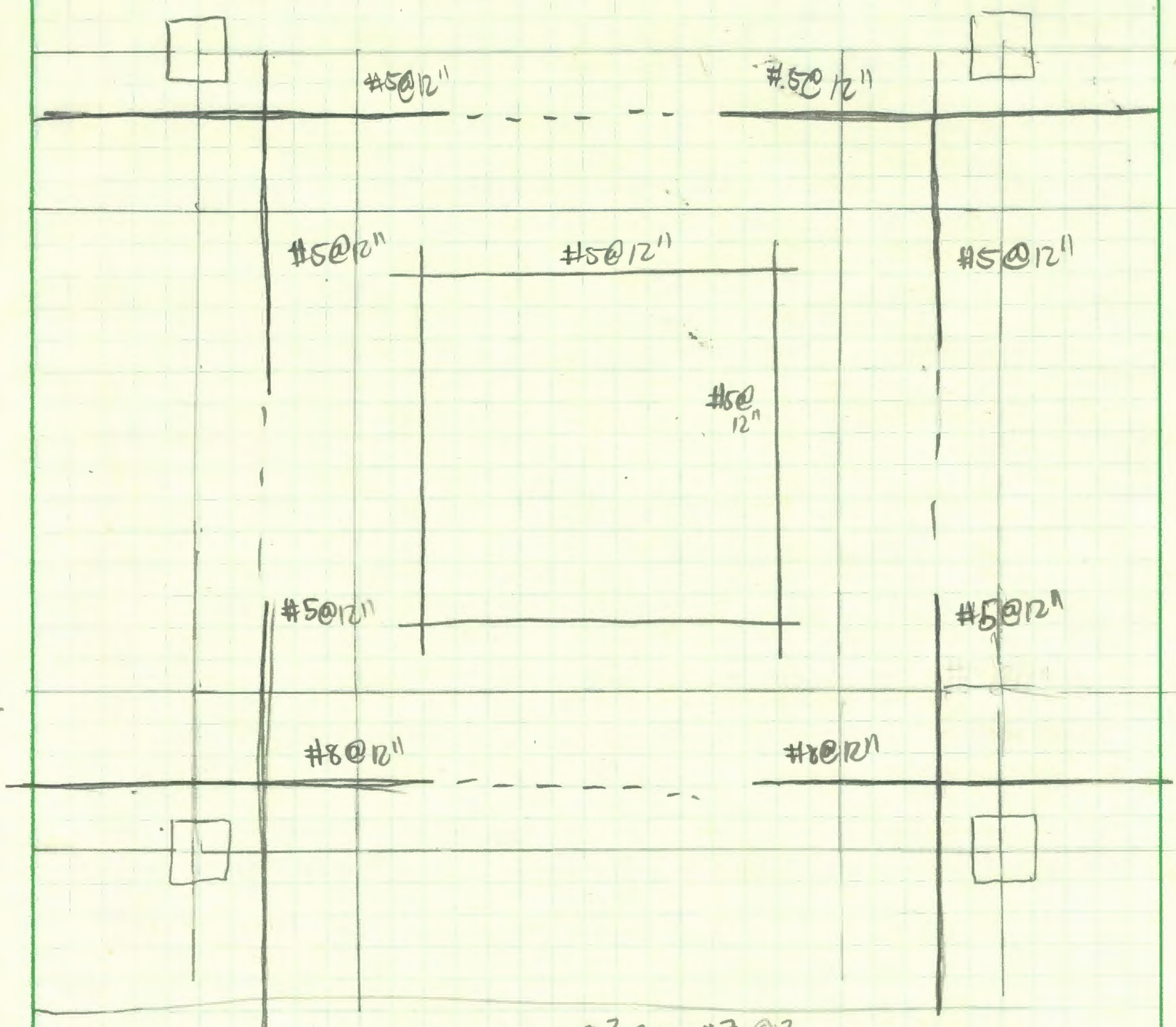
Column N-S choose #5 @ 12"

Midd N-S choose #5 @ 12"

# Reinforcement Detail for Ext Bays

Ext

Ext Bay




1st Interior Bay

Notes

- Top

| | | | |
|---|---|---|---|
| E-W | Extruding | 0.3 span | #7 @12 |
| | | 0.22 span | #5 @12 |
| N-S | Extruding | 0.3 span | #5 @12 |
| | | 0.22 | #5 @ 12 |

- Within 4' of column strip use #8@12" For 1st INT — outside of 4' use #5@12"
- Within 4' of column strip use #5@12" for Ext Bay — outside of 4' use #5 @ 12"

- Design will hold for Ext edge bay

# 13.3.2 Lateral System

# Shear Wall Redesign

Note

- Shear walls re-analyzed for new gravity load
- Shear walls in existing analysis failed - likely due to how DL on shear walls was being calculated

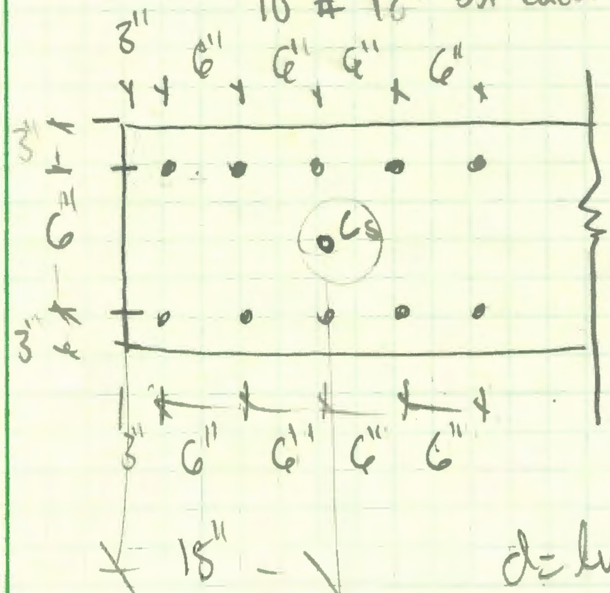
  ↑ Controlling condition was always E-W walls (more severe than wind)

  ↑ most likely difference from EOR DL calc is that I didn't know the Mechanical load on the roof.

- For new design, increased DL - not by a lot. Still have SW failing in E-W direction. ∴ make thicker change from 10" - 12", doesn't help Nu that much
- Can't add load where it doesn't exist, don't want to change SW geometry (length) ∴ change the reinforcement.
- Choose 80ksi bc it would of saving what would've been

  14 #10's on each side -28
  to
  10 #10's on each side -20

  saved 8 bars but used higher strength steel




ACI 17.7.2

Max spacing

$$\min = \begin{cases} 5 \cdot t = 60" \\ \text{or} \\ 18" \quad \text{or } 18" \end{cases}$$

↑ governs

$d = l_w - 15"$

Shear walls that need the Reinforcement previously detailed

3, 10, 12, 8, 6

New $A_s f_y = 10 \cdot 1.27 \cdot 80 = 1016$ kips

Together #10 $f_y$

Causes the Listed SW above to pass for flexure

New SW design [Seismic Controlled]

N-S SW

10 #10 for Flexural Reinforcement
60ksi

E-W SW

10 #10 for Flexural Reinforcement
80ksi

- Transverse and Longitudinal Reinforcement per ACI 11.6.1
because $V_u \le 0.5\, \phi V_c$
for all shear walls

Using NO.5 $f_y = 60$ksi

$\rho_t = 0.0012$ $\rho_l = 0.002$

Max spacing $= \min \begin{cases} 5h \\ \underline{\underline{30''}} \end{cases}$

Max spacing $= \min \begin{cases} 5 \cdot 12 \\ \text{or} \\ \underline{\underline{30''}} \end{cases}$

Max Spacing from ACI 11.7.2

$\rho_t = \dfrac{A_v \text{ horiz}}{h \text{ spacing}}$

thick    24"

$A_v = 0.0012(12)(24)/2$

$= 0.17 \text{ in}^2$

choose #5 @ 24" each face

$\rho_\ell = \dfrac{A_v}{h \cdot s_1}$

$A_v = 0.0012(12)(24)/2$

$= 0.1288$

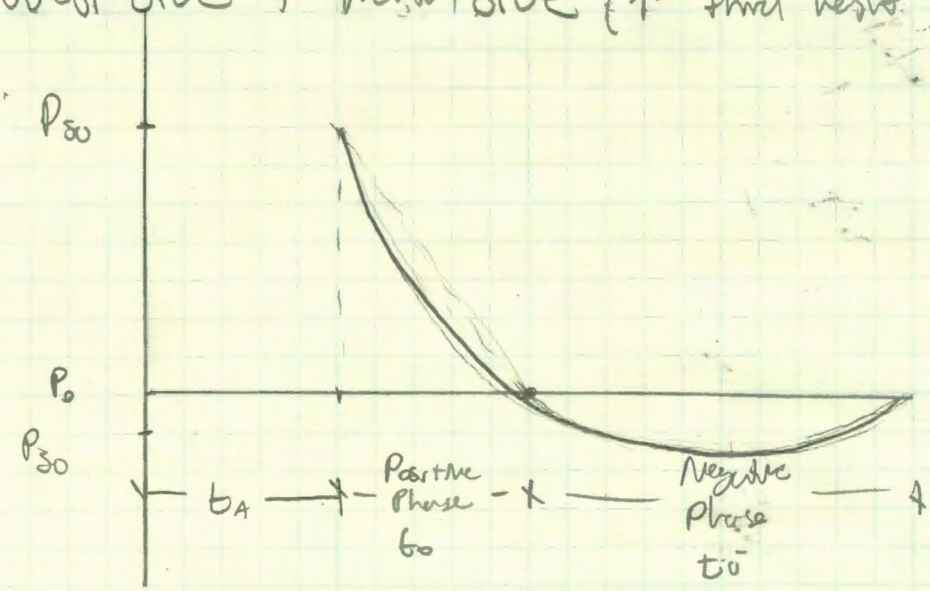
Choose #5 @ 24" Each face

Note: this As does not need to be so high but same reinforcement will be use to avoid construction error

# 13.3.3 Blast Design

## Blast Pressures:

West Side + North side (1st third height) — Empirical Method




Graph of Shock wave as a function of time

Pressure vs time

- Assumed Bomb weight 5 kg TNT ≈ 11 lbs = W
  ↓
  Bus or car bomb

- Distance from source to building 6' = R

- $Z = R / W^{1/3} = 2.7$
- $P_r = 1 \times 10^3 = 1000$ psi

$P_{so} = 1.8 \times 10^2 = 180$ psi

$U_s = 3.5 = 3.5$

$t_o / W^{1/3} = 1.6 = 1.6$

~~$q_o = 0.022 (P_{so})^2 =$~~ psi

$q_o = 300$ psi

- Positive phase duration

$$t_o^+/W^{1/3} = 1.6 \text{ ms}/lb^{1/3}$$

$$t_o = \left(t_o/W^{1/3}\right) \cdot W^{1/3} = 1.6 \cdot (11)^{1/3} =$$

$$= 3.55 \text{ ms}$$

- Positive phase wavelength

$$L_w = U_s \cdot t_o = 3.5 \cdot 3.55$$

$$= 12.425 \text{ ft}$$

- Clearing time

$$t_c = 4S / [1 + S/G] C_r$$

$$S = \min \begin{cases} \text{Building height} = 115' \text{ avg} \leftarrow & S = 115' \\ \text{Building width} = 134 \end{cases}$$

$$G = \max \begin{cases} \text{Building height} \\ \text{Building width} = 134 \leftarrow & G = 134' \end{cases}$$

$C_r$ = Velocity of sound

Using linear extrapolation

$$y_2 = 1.65 + \frac{180 - 40}{(50-40)} (1.75 - 1.65)$$

$$C_r = 3.05 \text{ ft/ms}$$

$$t_c = 4(115) \Big/ \left(1 + {}^{115}/_{184}\right)(3.08)$$

$$= 81.16$$

- Stagnation Pressure $P_s$

$$P_s = P_{so} + \underset{\substack{\uparrow \\ =1}}{C_d}(q_o)$$

$$= 180 + 300$$

$$P_s = 480 \text{ psi}$$

- Impulse

$$I_s = 0.5(P_r - P_s)t_c + 0.5 P_s \cdot t_o$$

$$= 0.5(1000 - 480) \cdot 81.16 = 22319$$

$$+ 0.5(450)(3.55) = 798.75$$

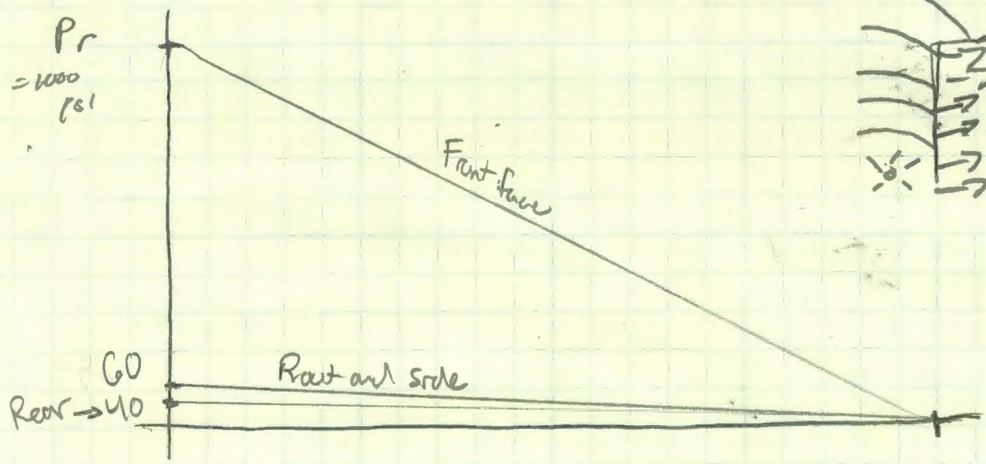
$$I_s = 23117 \text{ psi/ms}$$

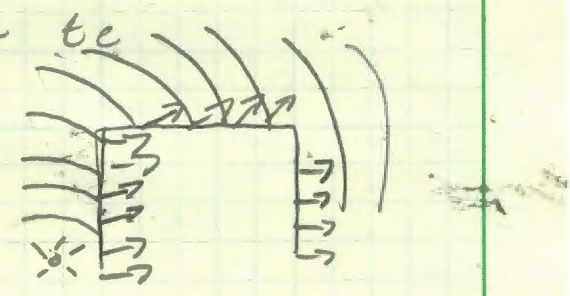
- Effective Duration

$$t_e = 2I_s / P_r = 2(23117) / 1000$$

$$= 46.23 \text{ ms}$$

• Equivalent Loading is $P_r$ over time $t_e$



Positive pressures = → on building

$t_e = 46.23$ ms

Pressure vs Time for West side

• Pressure on Roof and side wall

$$P_a = C_E \cdot P_{SO} + C_d \cdot q_0$$

for Roof and sidewall $\quad L_{wf}/L = \dfrac{\text{wave length}}{\text{length of roof}} = \dfrac{12.425}{1 \text{ section}} = 12.425$

$q_{so} = 300$

$C_d = -0.2$

$$P_a = C_E \cdot P_{SO} + C_d \cdot q_0$$
$$= 180 - 60$$
$$= +60 \text{ psi}$$ over 46.23 ms

• Pressure on Rear wall

$$P_a = C_E \cdot P_{SO} + C_d \cdot q_0$$

$q_0 = 300$

$cd = -0.4$

$$= 1 \cdot 180 - 120$$
$$= 40 \text{ psi}$$

$t_r = 0$

# Northside Pressures

- **Notes/Parameters**
  - North side is more public
  - higher risk
  - susceptible to coordinated assault
  - 2 bombs or more ↑

- Height = 130'
- width = 200'
- length = 134'

- 15 kg of TNT = 33 lbs = W
- Distance from Blast to building ≈ 25' = R
- $Z = R/W^{1/3} = 25/33^{1/3} = 7.8$
- $P_{so} = 15$ psi
- $P_r = 42$ psi
- $U_s = 1.6$ ms
- $t_o/W^{1/3} = 2.3$ ms
- $q_o = 4$ psi
- $t_o = 2.3 \cdot (33)^{1/3} = 7.4$ ms
- $L_w = U_s \cdot t_o = 1.6 \cdot 7.4 = 12$ ft
- $t_c = 4S/\{1+S/G\}\, C_r$

$$= \frac{4(130)}{\left(1+130/200\right)1.36}$$

$$= 232 \text{ ms}$$

$S = \min \begin{vmatrix} \text{Height} - 130 \\ \text{width} \end{vmatrix}$

$G = \max \,|\, \uparrow \; = 200$

$C_r = 1.36$

- $P_s = P_{so} + C_d(q_o)$
  $= 15 + 1(4)$
  $= 19$

- $I_s = 0.5(42-19)(232) + 0.5(19)\,7.4$
  $= 2738.3$

- $t_e = 2I_s/P_r = 130.4$

• Side wall pressure

assumed length unit 1'

$q_o = 4$ psi

$C_d = -0.4$

$\frac{L_w}{L} = \frac{12}{1} = 12$

$P_a = C_E \cdot P_{SO} + C_d \cdot q_o$

$= 15 + (-0.4)(4)$

$P_a = 13.6$ psi

• Rear wall loads

$\frac{L_w}{L} = \frac{12}{130} = 0.09$

$CE = 0$

$P_a = 0 + C_d \cdot q_o$

$= -1.6$ psi, deemed negligible

• Roof pressure

use whole section

$L = 134$

$L_w/L = 12/134 = 0.01$

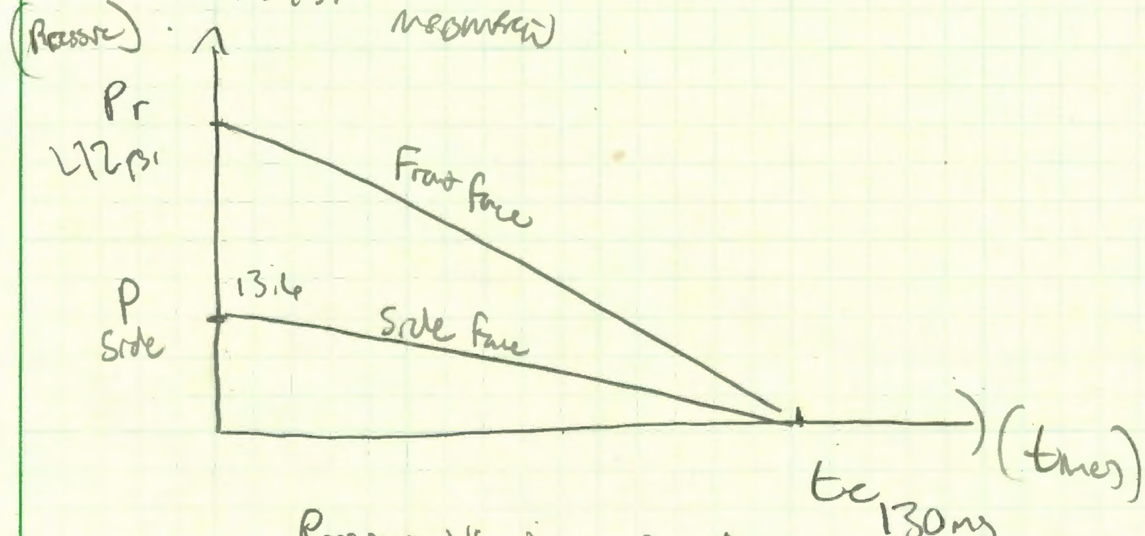
$CE \approx 0$

$P_a = CE \cdot P_{SO} + C_d \cdot q_{SO}$

$= 0 + (-0.4)(4)$

$= -1.6$ psi

deemed negligible



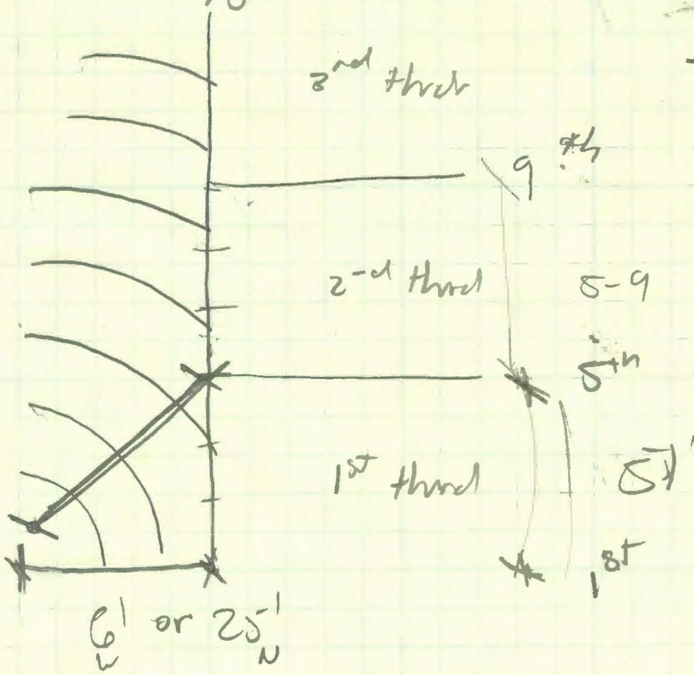

Pressure VS time for North side

## Blast Pressures

West side + North side ( $2^{nd}$ third of height )

- Assumed bomb weight on west side is 11 lbs of TNT

- Distance to 2/3 of height is calculated below




Note: For this approximation only use the $P_r$ & arrival time

$\theta = \arctan\left(\frac{5'1}{6}\right)$

$= 83.3$

$\sin(83.3) = 51/x$

$x = 51.35$

so $51.5' = R\ (W)$

$11 lb = w\ (W)$

$56' = R\ (N)$

$33 = w\ (N)$

- $Z = R/W^{1/3} = 51.5/(11)^{1/3} = 23.2$

- $I_r/W^{1/3} = 8$ $\quad q_0 = 0.1$ psi

- $\underline{P_r = 5 \text{ psi}}$

- $I_s/W^{1/3} = 4.5$

- $P_{so} = 2.5$ psi

- $U_s = 1.2$

- $t_0/W^{1/3} = 2.3 \text{ ms/lb}^{1/3}$ $\quad t_0 = 2.3 \cdot W^{1/3} = 5.1$ ms

- $L_w = U_s \cdot t_0 = 6.12$ ft

- Pressure on Rear Wall

$-C_d = -0.4$

$-C_E = 1$

$$P_a = C_E \cdot P_{so} + C_d \cdot q_{so}$$
$$= 1 \cdot 2.5 - 0.4 \cdot 0.75$$
$$P_a = 2.8 \text{ psi}$$

West 2/3

Front = 5 psi

Rear = 2.25 psi

North Parameters

- $W = 33$ lbs
- $R = 56'$
- $Z = R/W^{1/3} = 17.5$
- $P_r = 8.5$ psi
- $q_0 = 0.1$ psi
- $P_{so} = 4$ psi

North 2/3

Front = 8.5 psi

Rear = 4 psi

Rear Pressure

$-C_d = -0.4$

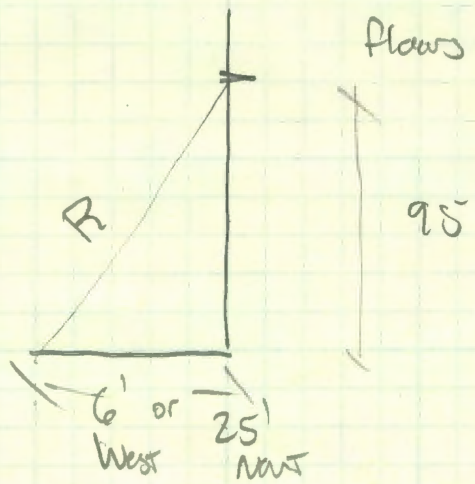
$-C_E = 1$

$$P_a = 4 - 0.4(0.1)$$
$$= 4 \text{ psi}$$

# Blast Pressures

West + North Side (3rd third of height)

floors 9 - Penthouse




| West | North |
|---|---|
| W=11 | W=33 |
| R=98' | R=98' |

West side Parameters

- $Z = R/W^{1/3} = 43.18$
- $P_r = 1.5$ psi
- $P_{so} = 1$ psi
- $\rho_{so} = 0.022$ psi
- $P_{rear} = P_{so} - (0.4)(\rho_o)$
  $= 1 - 0.4(0.022)$
  $= 1$

North side Parameters

- $Z = R/W^{1/3} = 30.6$
- $P_r = 3.6$ psi
- $P_{so} = 1.8$ psi
- $\rho_{so} = 0.07128$
- $P_{rear} = P_{so} - (0.4)(\rho_o)$
  $= 1.8 - 0.4(0.07128)$
  $= 1.8$

West Side 3rd height

Front = 1.5 psi

Rear = 1 psi

North side 3rd height

Front = 3.6 psi

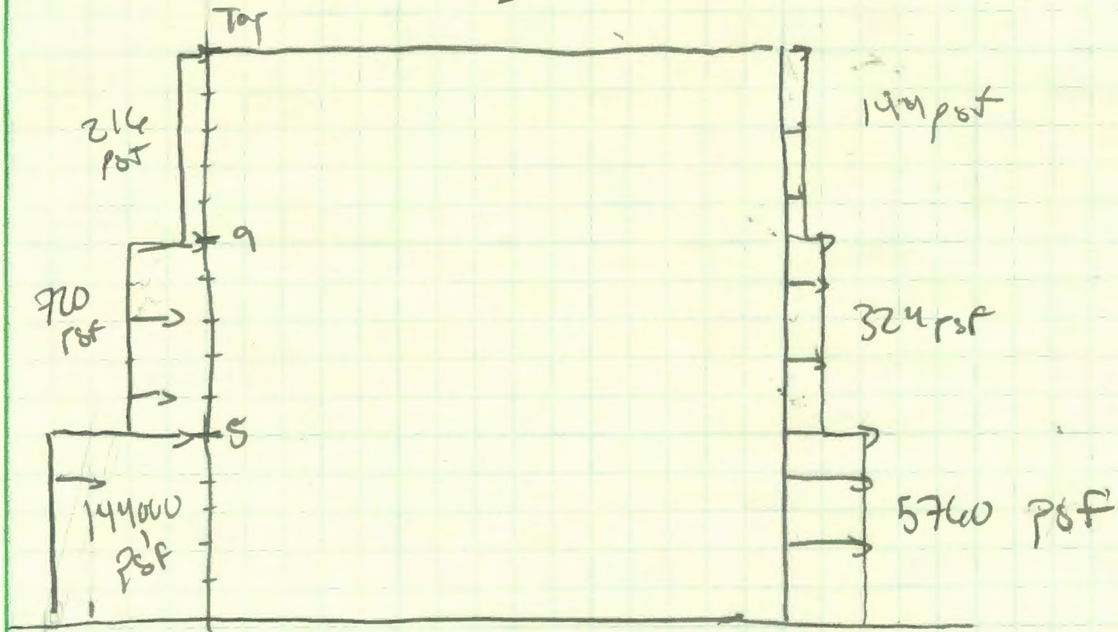
Rear = 1.8 psi

# Pressure Vs Heigh Diagram

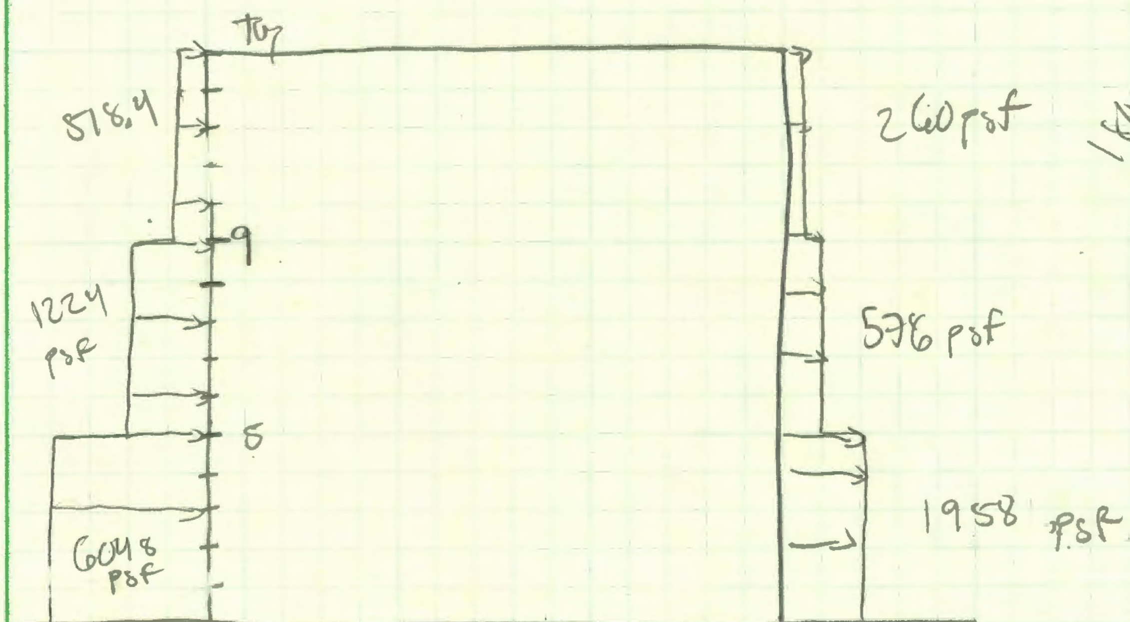
- West Side (Converted to Psf, psi · 144 = psf

$lbs/in^2 \cdot 144\ in^2/ft^2 = lbs/ft^2$

W → E




- Nort Side N → S




$144\ in^2/ft^2$

180 psi

$180\ lb/in^2$

$= 25920$ psf

Note: Pressures on the page before were analyzed and it was found infeasible to design such a building to resist pressures at such a close distance. There fore a more feasible location will be chosen and the pressures generated from such a location will be analyzed

# External Pressures For Feasibility

- New assumed location is along NY Ave North side of The Park at City Center DC
- Distance $R = 190'$
- Weight of bomb = 11 lbs / 5 kg of TNT

**Parameters**

- $Z = R/W^{1/3} = 190/(11)^{(1/3)} = 85.2$
- $P_r = 1$ psi
- $P_{so} = 0.5$ psi
- $I_r/W^{(1/3)} = 2$
- $I_s/W^{(1/3)} = 1$
- $U_s = 1.2$ ft/ms
- $t_o/W^{(1/3)} = 5$
- $t_s/W^{1/3} = 70$
- $P_r/P_{so} = \left[2 + 0.05(P_{so})\right]$
  $= 2.025$

- $q_o = 0.022(P_{so})^2$
  $= 0.0055$ psi
- $t_o = t_o/W^{(1/3)} \cdot W^{1/3} = 5_{ms} \cdot (11)^{1/3} = 11.12$ ms
- $L_w = U_s \cdot t_o = 1.2 \cdot 11.12 = 13.34$ ft
- Dimensional Properties of Building

  Height = 130'
  width = 200'
  depth = 127'

- $t_c = 4S / [1 + S/G] C_r = \dfrac{4(130)}{[1 + 130/200]\,1.13} = 280$ ms

  clearing time

$S = \min \begin{cases} \text{building height} \\ \text{building width} \end{cases}$ $G = \max \begin{cases} \text{building height} \\ \text{building width} \end{cases}$ $C_r = 1.13$

Load to be used for front wall analysis is

$P_{SO} = 0.5$ psi

Loading on Rear or side wall

$P_a = C_E \cdot P_{SO} + C_d \cdot q$

Lwave length / span length

↓

$= 13.34 / 127$

$= 0.1$

$t_o / W^{1/3} = 20$

$-C_d = -0.4$

$-C_E = 0.21$

$P_a = 0.21\ 0.5 - 0.4(0.0055)$

$= 0.09$ psi ← same pressure for the roof

Note: be the $L_w$ < distance from wave to the building

$13.34 < 190'$ — This pressure is not used i.e. No pressure on roof or side walls

# Column Design for Ext Blast

**Note:** - This design only analyzes the first floor, this is because the forces of gravity and blast will be greatest at the base level.

- Blast forces only act on Ext column
- Tributary area (largest) for Ext column is

$$30 \times 7.5 = 225\ ft^2$$

- $DL = 167.5$ psf $\rightarrow$ 11 floors $= 1842.5$
- $LL = 64$ psf $\rightarrow$ 11 floors $= 704$

$2550\ lbs/ft^2 \cdot 225$

$= 573$ kips

largest Gravity force on Ext column 1st floor

- Lateral force on 1st floor

$$= \underset{P_{so}}{0.5\ lb/in^2} \cdot 144\ \frac{in^2}{ft^2} = 72\ lb/ft^2$$

only acts on column face

assume a 2 ft column width

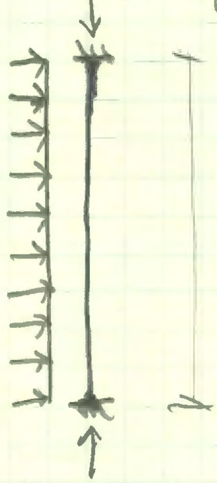
$\therefore$ the Lateral load on the column

$$= \underset{wide}{2'} \cdot 72\ lb/ft^2 = 144\ lb/ft$$

- Load on 1st floor Column

576 kips

144 lb/ft



14.5'

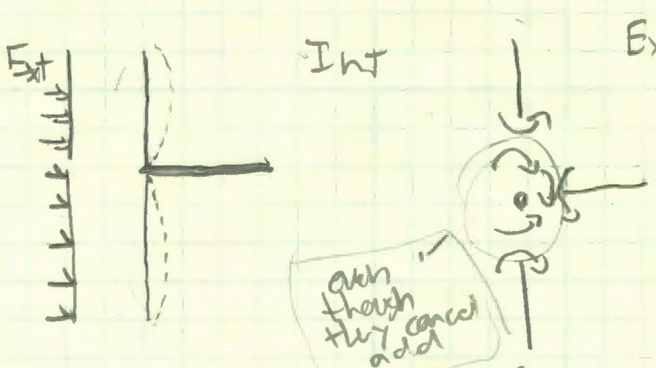
Material Properties

- $f'c = 8000$ psi • flexural increase of 1.19 for blast
  $= 9520$
- Assumed dimensions $= 2' \times 2'$
- Height $= 14.5'$

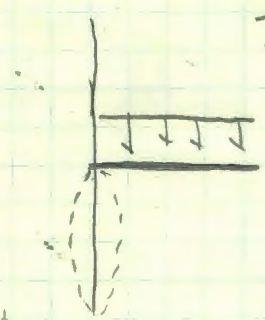
– Moment from Blast and Moment from Gravity could be additive ∴ it is conservative to analyze the column for moment created by Blast and Gravity

Visual Justification

Blast




Gravity




moment =

$$\frac{w\ell^2}{11} = \frac{144(14.5)^2}{11}$$

$$= 2752^{\,lb\cdot ft}$$

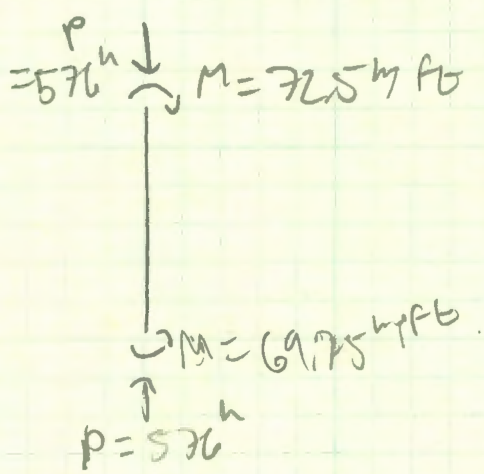
$$= 2.75 \text{ kip}\cdot\text{ft} \times 2 \text{ from Column above} = 5.5$$

From Gravity design
Ext moment
$= 67 \text{ kip}\cdot\text{ft}$

largest moment $= 72.5^{\,kip\cdot ft}$ ~ at top

Total Axial $= 576$ kips

Smallest Moment $= 67 + 2.75 = 69.75^{\,kips}$
only 1



# Design of Ext Column

- Is it slender (no)

ACI

$$\frac{k l_u}{r} < 34 - 12\, M_1/M_2$$

$$< 34 - 12\left(72.5/69.75\right)$$

K=0.5 for fixed fixed

$$\frac{0.5(14.5)(12''/ft)}{5.4} < 21.5$$

$$16.11 < 21.5 < 40$$

ok

$l_u = 14.5'$

$r = 0.3(h)$ ← dimension of column try 18"

$= 5.4$

∴ Column is considered short

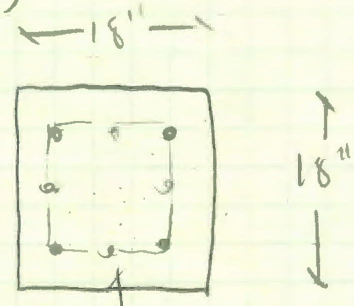
- Design Check for SP column

Using point of Pure axial for a reference to compare to the point of Pure axial from SP —

$$P_o = 0.85 \cdot f'_c\,(A_g - A_{st}) + A_{st} \cdot f_y$$

$$= 0.85(8)(324 - 6.32) + 6.32 \cdot 60$$

$$= 2540$$




8#8 bars

$f'_c = 8000\,psi$

$A_g = 324\,in^2$

$A_{st} = 6.32$

$\phi P_n = 1650.5$ kips

Prom from Interaction Diagram ≈ 1650 kips

∴ SpColumn Interaction Diagram is ok ✓

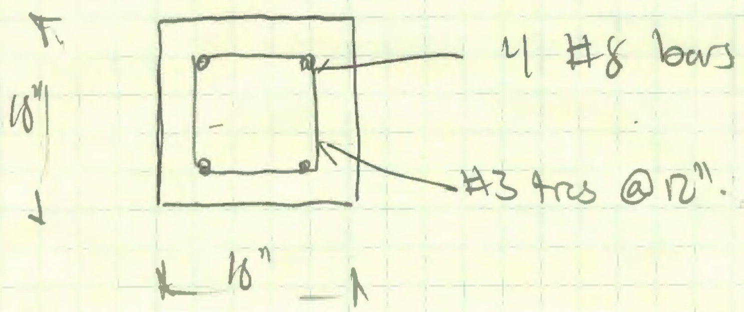
$\phi M_n = 300$ kip ft ∴ Design is ok

• Design of Ties

— as per previous design

Note: Different than design on previous page but that was a software check not actual design

#3 Stirrups @ 12" on center




$\phi P_n = 1200$ kips $> 576$ kips

$\phi M_n = 175$ k·ft $> 35$ k·ft

@rows

## Interior Blast Pressures v Time

### Lobby

- Assumptions
  - 11lbs of OR TNT — outside estimation moved inside
  - vent area $\approx 1000\,ft^2$
  - 10' vent distance

~~Where $P_{so}$ = 180psi — From previous outside blast calcs~~

~~$P_r/P_{so} = 6 \cdot P_{so}$~~

~~$P_r = 1080$ psi~~

- $Z = 10 / 16^{1/3} = 4$

- $P_{so} = 70$ psi

- $P_r = 300$ psi

- $t_0 / W^{1/3} = 1.6$ $\quad t_0/W^{1/3} \cdot W^{1/3} = t_0$ $\quad t_0 = 4\,ms$

$$1.6 \cdot 16^{1/3} =$$

- $L_w = U_s \cdot t_0 = 2.5 \cdot 4 = 10\,ft$

- $U_s = 2.5\,ft/ms$

- $q_0 = 0.022(P_{so})^2 = 107.8$ psi

- loading density $W/V_f = 16/11000 = 0.00145\,lbs/ft^3$

$V_f = 11000\,ft^3$

- $P_g = 26$ psi

peak gas pressure

- Shock impulse

$i_r / W^{1/3} = 40$

$A / V_f^{2/3} = \frac{1000}{11000^{2/3}} = 2$

- $W / V_f = 0.0015$

- higher of

graph with $i_r / W^{1/3} = 100$ $\quad i_g / W^{1/3} = 700$

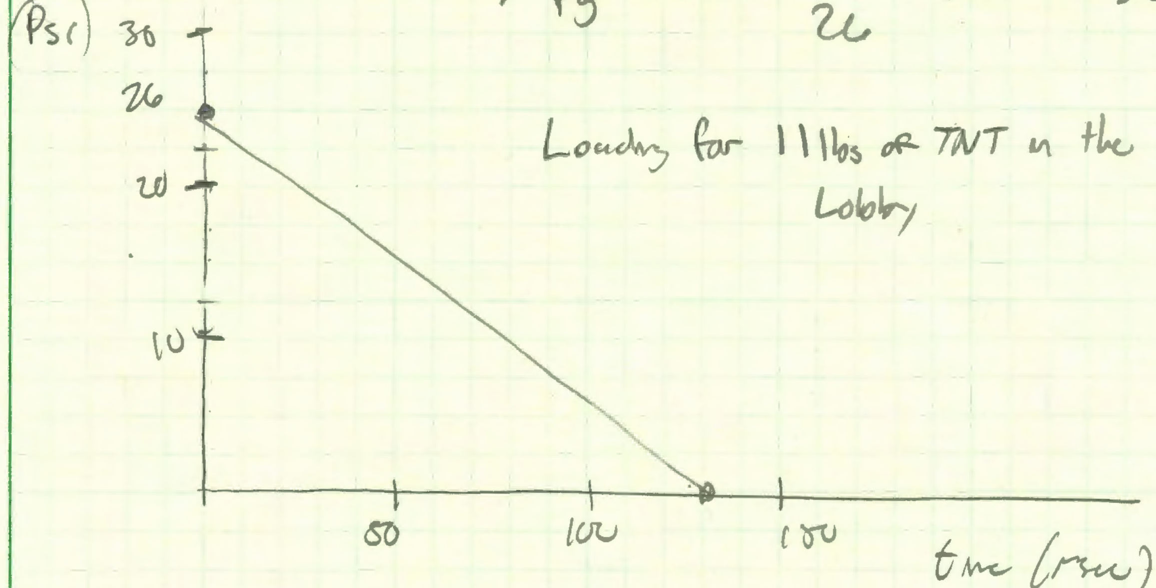
and $i_r / W^{1/3} = 20$ $\quad i_g / W^{1/3} = 780 \text{ psi} \times \text{msec} / lb^{1/3}$

$i_g = 780 \cdot 11^{1/3}$

$= 1668 \text{ psi} \cdot \text{msec}$

- Duration

$= 2 \times i_g / p_g = \frac{2 \cdot 1668}{26} = 128 \text{ msec}$




Loading for 11 lbs of TNT in the Lobby

26 psi over 128 msec

## Retail Space

- Assumptions
  - 11 lbs of TNT = W
  - Vent Area ≈ 4700 sf    Vent Volume = 4700 sf · 14' = 65,800 ft$^3$
  - Minimum vent distance ≈ 25' ≈ 6' for ceiling/slab
  - $Z = \frac{\text{Vent dist}}{\text{Weight}^{1/3}} = 11.25$ (columns/walls) = 2.7

| Columns / walls | Ceilings / slabs |
|---|---|
| $P_r = 5$ psi | $P_r = 100$ psi |
| $P_{so} = 2.2$ psi | $P_{so} = 18$ psi |
| $t_o/W^{1/3} = 3.6$ | $t_o/W^{1/3} = 1.8$ |
| $t_o = 1.6$ ms | $t_o = 0.81$ ms |
| $L_w = 2.02$ ft | $L_w = 3.24$ ft |
| $U_s = 1.25$ | $U_s = 4$ |
| $q_o = 0.022(P_{so})^2 = 0.106$ | $q_o = 0.022(P_{so})^2 = 7.12$ |
| Loading density $= W/V_f = 1.67 \times 10^{-4}$ | Loading density $= 1.67 \times 10^{-4}$ |
| $P_g = 20$ psi | $P_g = 20$ psi |
| $i_r/W^{1/3} = 8$ psi · msec/lb$^{1/3}$ | $i_r/W^{1/3} = 90$ |

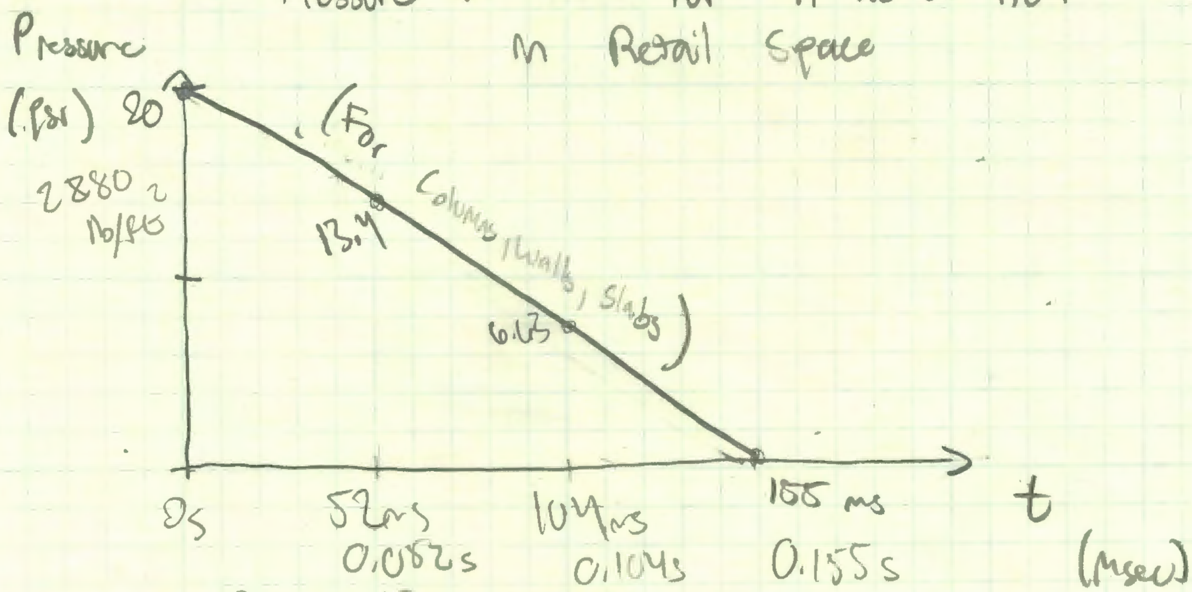
Columns / walls

- $i_g / W^{1/3} = 700$
- $A / V_f^{2/3} = 2.88$
- $i_g = i_g / W^{1/3} \cdot W^{1/3} = 1554$ psi $\cdot$ Msec
- $t_g = 2 \cdot i_g / P_g = 155$ Msec

Columns / slabs

- $i_g / W^{1/3} = 700$
- $A / V_f^{2/3} = 2.88$
- $i_g = 1554$ psi $\cdot$ Msec
- $t_g = 155$ Msec

Pressure v time for 11 lbs of TNT in Retail Space




$x = \frac{20}{155} = 0.13$

$y - y_1 = m(x - x_1)$

$0 - y_1 = 0.13(155 - 52)$

$(155 - 104)$

$= -0.13($

# Material Properties of RC under Dynamic loads

From the Handbook for
Blast Resistant Design of Buildings

- $F'_c$ for      $f'_t \approx 8\%$ of $F'_c$

Columns/walls = 8 ksi      = 640 psi

Slabs = 4 ks      = 320 psi

or Table 4.2 / UFC 3-340-02
Pg 137
Has Dynamic Increase Factors

| $F'_c$ | | Flexure | Comp | Shear (Direct) | |
|---|---|---|---|---|---|
| Colums/walls | = 8 ksi | 1.19 | 1.12 | 1.0 | – Conc |
| Slab | = 4 ksi | 1.17 Flexo | 1.10 | 1 | – Steel |

Colums/walls $f'_c$ = 9.52 ksi
Slabs $f'_c$ = 4.76 ksi

Colums/walls $f_y$ = 70.2 ksi
slabs $f_y$

# 13.3.4 Progressive Collapse Design

# Progressive Collapse Design

<u>Notes</u>: - This was chosen over blast design for the interior due to the feasibility of blast design. It is more simple and conservative to assume that the members will not survive blast and therefor the structure must be designed for progressive collapse.

- Design will be based on 2 criteria

Criteria 1
- Probable Location of Bomb

Within criteria 2 grounds of probability →
- Most Extreme of each member ( Column with most load; Shear wall with most load; Largest Slab span )

- For Progressive collapse of a shear wall what are the chances that design wind forces will hit the building while a bomb goes off and eliminates a SW, very small.

∴ only combination for Progressive Collapse is 1.2D + 1.6L ← controlling for gravity

(No analysis for Lateral from above assumption)

- Determination of area of effect from a bomb, existing system can withstand

slab - twice the DL already on it

$\overset{DL}{\downarrow}\uparrow\uparrow$ = $\uparrow$ which is $\downarrow$ DL
DL DL      DL      what I already have

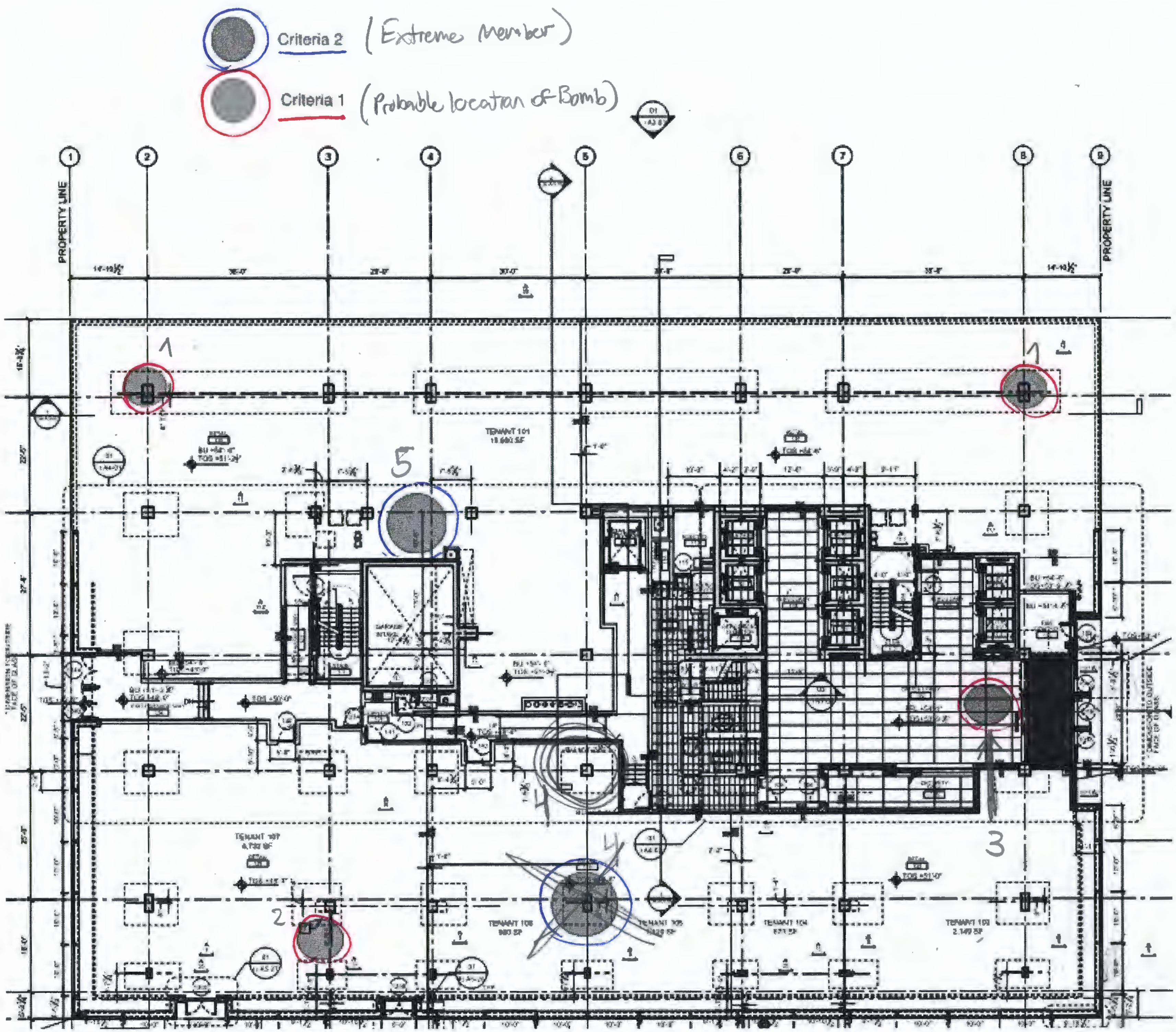
Max Load for Slab → $= 335\ lb/ft^2$
$= 335\ lb/ft^2$
$= 2.32\ psi$

Column - From known column capacities

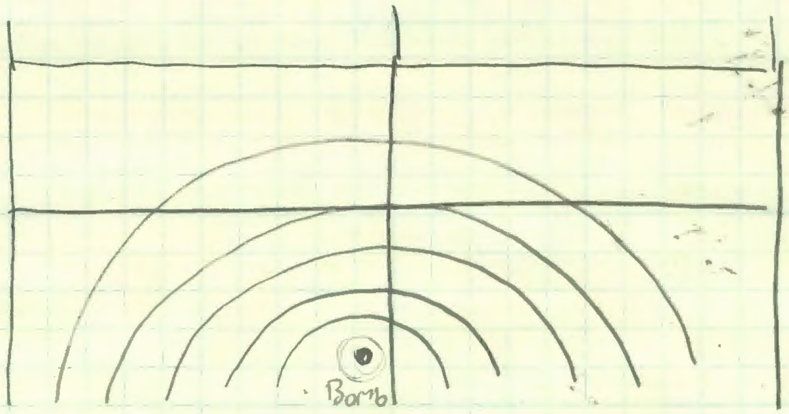
$M_u \approx 0$ from DL+LL

$\phi M_n$ for 24x24 is ≈ 400 k/ft for bending and axial

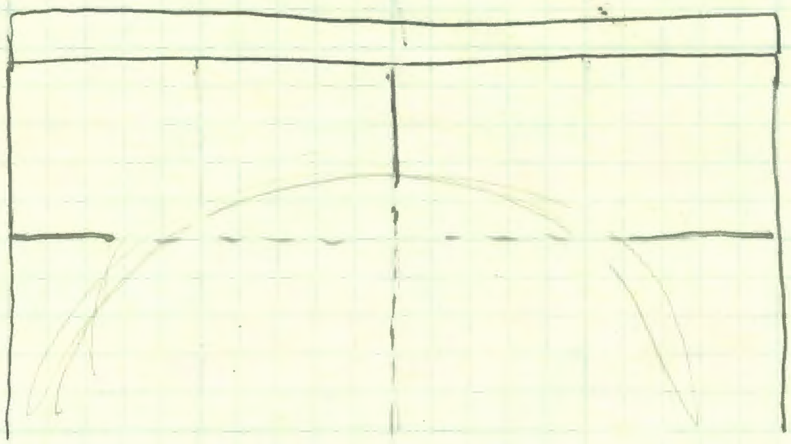
max load for column →
pressure × dimension = load k/ft


Criteria 2 (Extreme Member)
Criteria 1 (Probable location of Bomb)
PROPERTY LINE
PROPERTY LINE
1
2
3
4
5
6
7
8
9
TENANT 101
TENANT 107
TENANT 106
TENANT 105
TENANT 104
TENANT 103

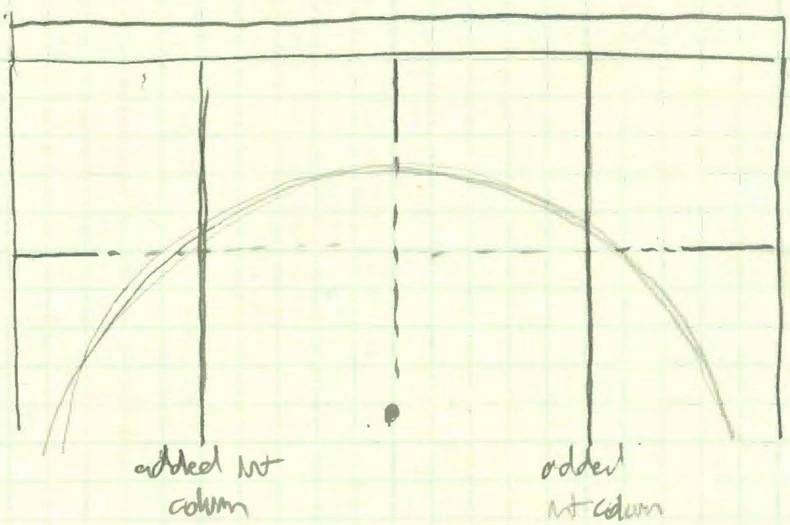
# Basis for Design of Progressive Collapse (cases 1, 2, 4)




- Bomb will take out the slab
- Idea is to prevent structure above from collapsing



- Solution 1
  add trasfer Girder



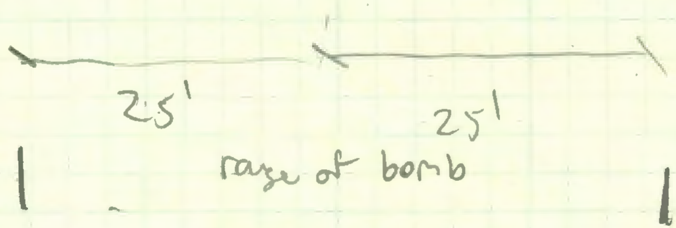

- Solution 2
  add interior columns with a transfer girder
  ↑
  Note added interior columns need to withstand the blast load

# Design of slab for Progressive Collapse

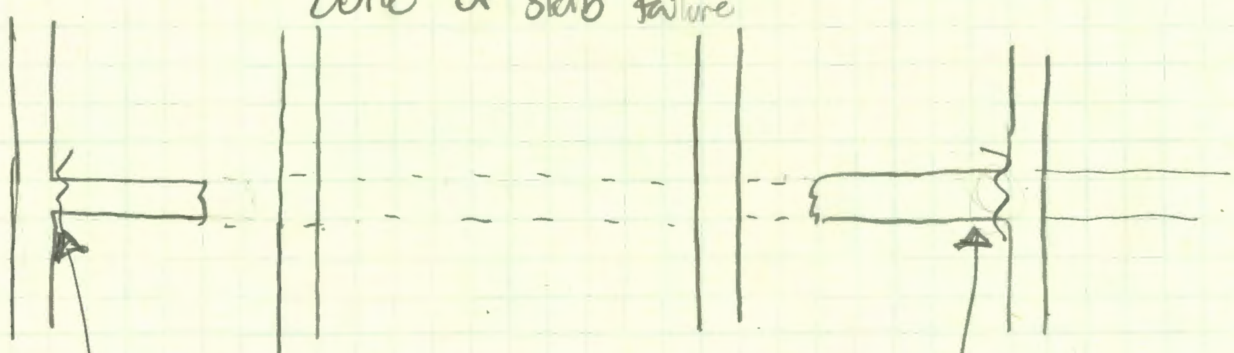
**Notes:** • The slab designed for gravity will survive a range of about 25' away from a bomb : half a bay

• Purpose of this design is such that the bomb can be isolated, a free standing slab doesn't cause the rest of the slab to collapse


consecutive small bays




Zone of slab failure



failure mode is now here but not from bomb

• slab will not continue to progressively collapse after bomb

Analysis of Range of Blast

(which members will be effected)

- For $W = 5.5$ lbs of TNT
  find R such that $Z = 12.5$

  $Z = R/W^{1/3}$

  $12.5(5.5)^{1/3} = R$

  $R = 22.06$ – slab inside 22' of a 5.5 bomb will be destroyed

To shorten or eliminate this distance either add interior beams or make slab thicker

---

- Max pressure on 24"x24" Column

$\phi M_n \approx 800$ k-ft

for the increased f'c from 8000 → 9520

$\frac{wl^2}{10} = 800$ $\quad w = \frac{800 \cdot 10}{14.5^2}$

$w_{max} = 38$ k/ft
$= 38000$ lbs/ft
$\div$ column width $= 2'$
$= 19000$ lb/ft$^2$

$\approx 130$ lb/in$^2$

∴ column can survive fairly close to bomb

Close enough to

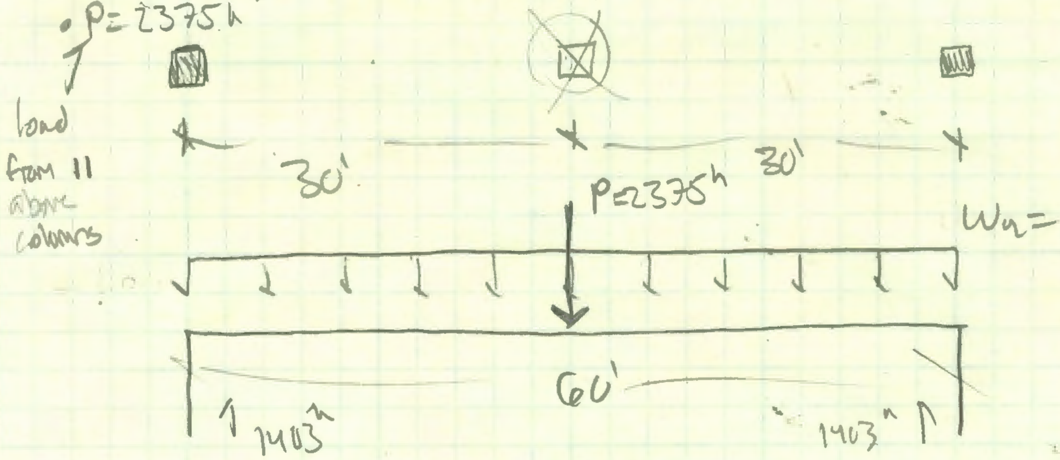
Largest Interior moment from Gravity = 0 k-ft

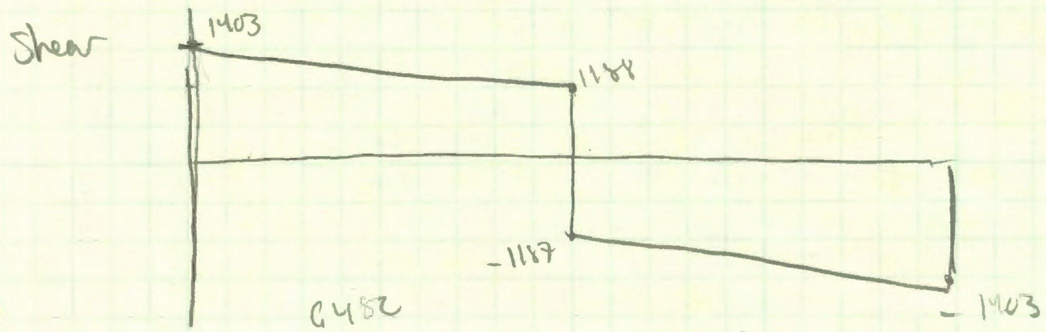
Needs to resist ~~2~~ 100 k-ft to survive a blast from a 5.5 lb bomb at 12.5' away

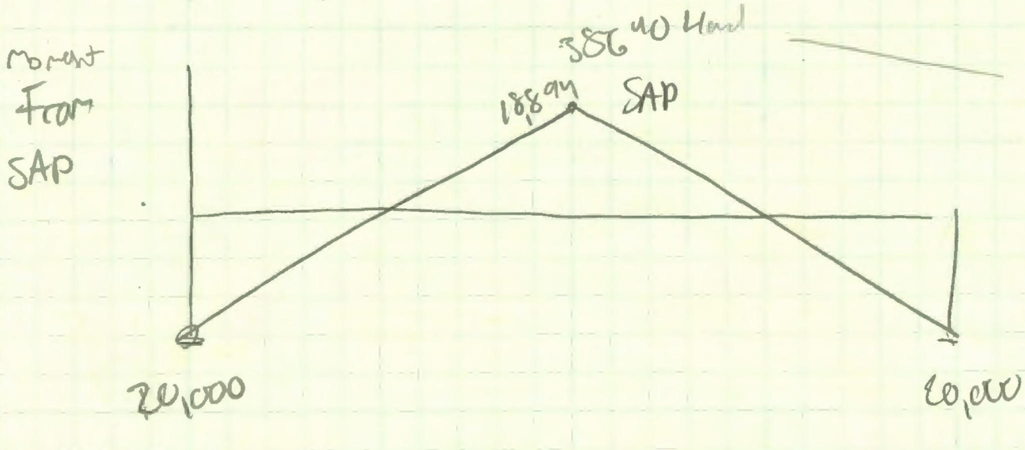
- If columns are designed to take this additional moment then it can be said that if a member is outside of the 12' range of the blast then it does not need to be eliminated in the progressive collapse analysis.

# Progressive Collapse Design @ Location 4

- Too be designed / solved using a T-Beam – slab + Beam
- $w = 0.303$ k/sf • Trib width of 23.75 $\approx 7.2$ kips/ft
- $P = 2375$ k




SAP = Hand / 2

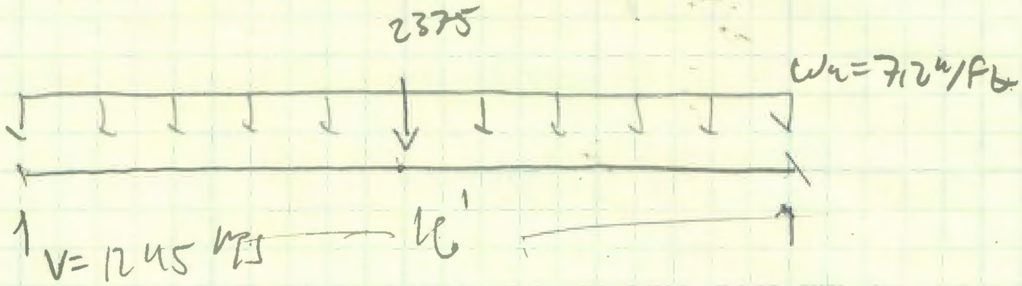
divd to fixed connection

design. Moment is 20,000 kip·ft

Too large for a feasible design ∴ add interior columns

Progressive Collapse Design @ Location 4

- To be designed with Intermediate columns and Beam
- w = 0.303 k/sf · Trib width = 7.2 k/ft
- P = 2375 k

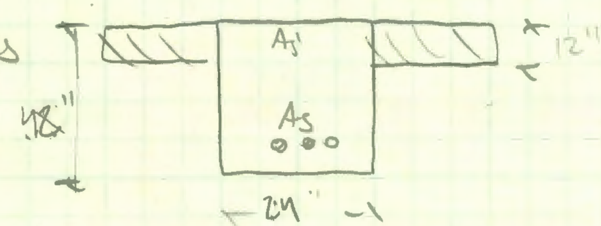



From SAP model

Max Moment = 4900 k·ft

Shear / support reaction = 1245 k

Design of beam to support above loads




$d = 33''$

$b = 18''$

$$\rho_{req} = \frac{0.85 f'c}{f_y}\left(1 - \sqrt{1 - \frac{2R}{0.85 f'c}}\right)$$

$$R = \frac{M_u \cdot 12}{0.9 \cdot b \cdot d^2} = \frac{4900 \cdot 12}{0.9 \cdot 24 \cdot 45^2} = 1.34$$

$$= \frac{0.85 \cdot 8}{60}\left(1 - \sqrt{1 - \frac{2(1.34)}{0.85(8)}}\right)$$

$$= (0.113)(0.14)$$

$$\rho_{req} = 0.027$$

$$\rho_{0.005} = \frac{0.32 (f'c)(\beta_1)}{f_y} = \frac{0.32(8)(0.65)}{60} = 0.027$$

$\rho_{0.005} > \rho_{req}$ ∴ Don't Need Comp Steel → But include anyway

$$\beta_1 = 0.85 - 0.05 \cdot \left[\frac{f'c - 4000}{1000}\right] \geq 0.65 = 0.65$$

• $A_{s1} = \rho_{0.005} \cdot b \cdot d$

$= 0.021 \cdot 24 \cdot 45$

$= 22.68 \text{ in}^2$

Choose 2 layers of 5 #14 = 22.5 in2

Further Confirmed that Section doesn't act as T-Beam

• $M_{n1}$

- $A_{s1} \cdot f_y = 0.85 \cdot f'_c \cdot a \cdot b$

$a = \frac{22.5 \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{22.5 \cdot 60}{0.85 \cdot 8 \cdot 18} = 11.03''$ ← in flange

$c = a/B_1 = 11.03/0.65 = 16.96$ ← outside of flange

$\varepsilon_s = \varepsilon_{cu} \frac{(d-c)}{c} = 0.003 \frac{(45 - 16.96)}{16.96} = 0.004981$ ← Close enough to 0.005

Tensile Steel Yields

$\phi = 0.9$

$M_{n1} = A_s f_y (d - a/2)$

$= 22.5 \cdot 60 (45 - 11.03/2)$

$M_{n1} = 4,442$ kip ft

• Moment still needed

$M_u \leq (M_{n1} + M_{n2})\phi$

$\frac{4900}{0.9} \leq 4442 + M_{n2}$

$M_{n2} \geq 1002.4$ kip·ft

• Calculate $A_s$ needed for compressive steel (steel in slab).

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

$d' = 8''$

$$= 0.003\frac{(16.96-8)}{16.96}$$

$$= 0.00158 < 0.207$$

Comp steel doesn't yield

$$- f_s' = \varepsilon_s' \cdot E_s$$
$$= 0.00158 \cdot 29000$$
$$f_s' = 45.96\,ks$$

$$- A_s' = \frac{M_{n2} \cdot 12}{45.96(45-8)} = \frac{1002.9 \cdot 12}{45.96(45-8)} = 7.07\,in^2$$

$$- A_{s2} = \frac{A_s' \cdot f_s'}{f_y} = \frac{7.07 \cdot 45.96}{60} = 5.418$$

- Total Area of Steel, Tensile

$$A_{s1} + A_{s2}$$
$$= 22.68 + 5.418$$
$$= 28\,in^2$$

Choose 3 rows of 6 with #11

$$= 18 \cdot 1.56 = 28.08\,in^2$$

ok

$- A_{smin}$

$$= \begin{cases} \frac{3\sqrt{f'c} \cdot b_w \cdot d}{f_y} = 4.8\,in^2 \leftarrow < \\ \frac{200 \cdot b_w \cdot d}{f_y} = 3.6 \end{cases}$$

$\therefore$ ok

• Shear Reinforcement

- $V_u = 1245$ kips

- $\phi V_c = \phi \cdot 2\sqrt{f'c} \cdot b_w \cdot d$

$= 0.75 \cdot 2\sqrt{8000} \cdot 24 \cdot 45 \,/\, 1000$

$= 144.89^k < V_u \therefore$ need stirrups

- $V_s = \dfrac{V_u}{\phi} - V_c$ ← $\phi V_c/\phi$

$= 1660 - 193.18$

$= 1467^k$

Choose #14 - 2 of them with $f_{yt} = 60$ ksi

$V_s = \dfrac{A_v \cdot f_{yt} \cdot d}{V_s}$

$V_s = \dfrac{2(2.25)(80)(45)}{s}$

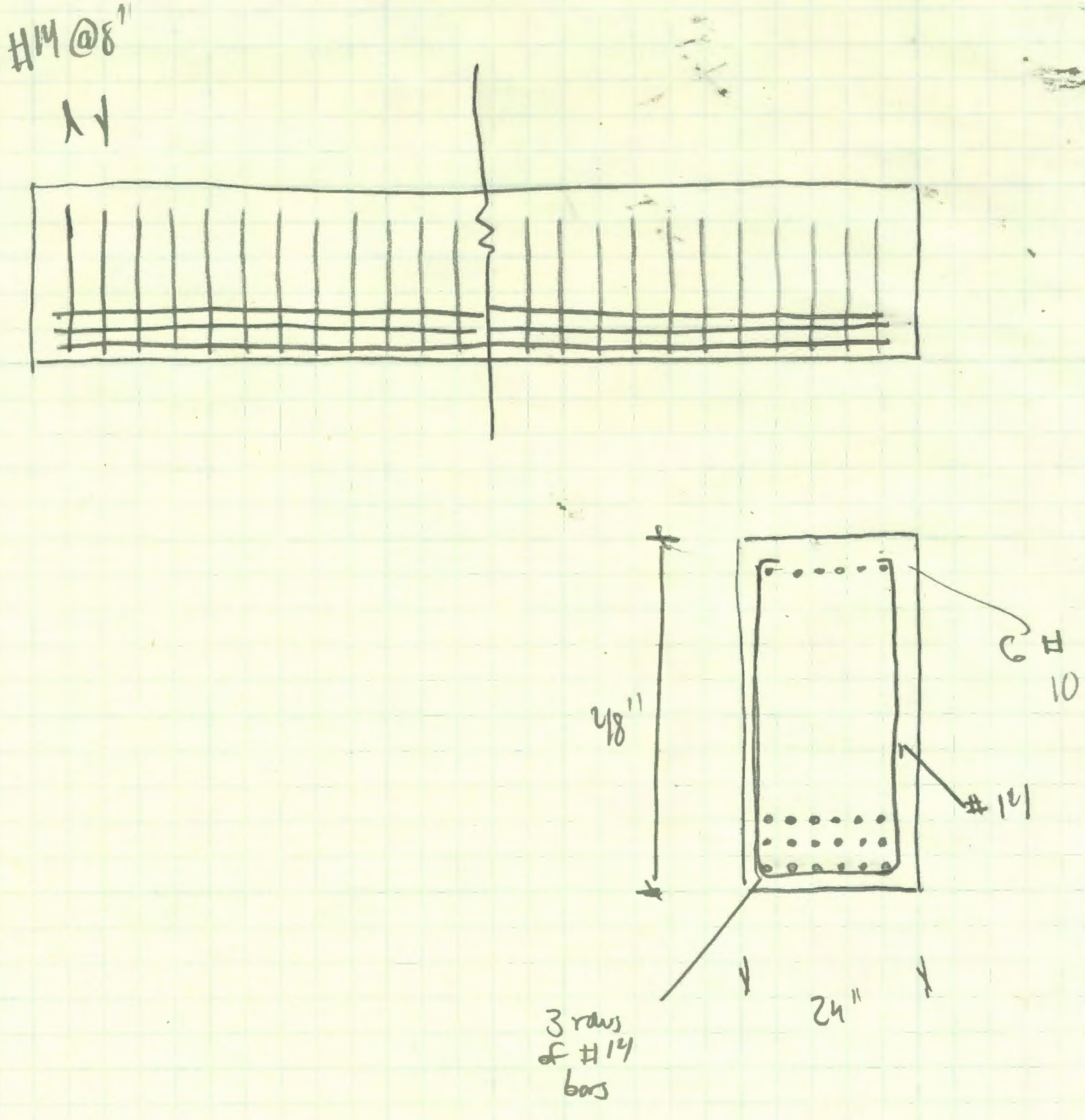
Spacing = 8.4" so 8"

- $A_{v\,min} =$

$$\begin{cases} 0.75\sqrt{f'c} \cdot b_w \cdot s / f_{yt} = 0.21 \text{ in}^2 & \text{ok} \checkmark \\ \dfrac{50 \cdot b_w \cdot s}{f_{yt}} = 0.16 \text{ in}^2 & \text{ok} \checkmark \end{cases}$$

$s_{max}$ = least of $\begin{cases} d/2 \\ \text{or } 24'' \end{cases}$ — 8" less than both

Detailing for Transfer Girder



Amount / bay (2)

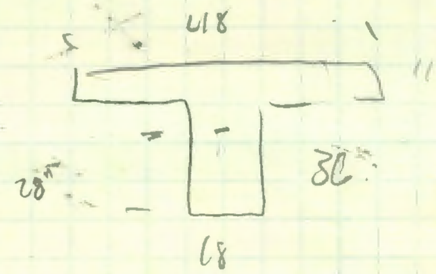
# Design of PT Transfer Girder for Added Column Solution

$M_{max} = $ 1880 @ mid span

~~907 @ ends~~

$V_{max} = 597^{k}$

Using a 18x38




$A = 1176 \text{ in}^2$

$S = 8265.16 \text{ in}^3$

$f_{pe} = 184$

$P = 184 \cdot 15(0.153)$

$= 422.3$

15, ½" Cables

• Stress @ ends

$$-f_{top} = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S}$$

$$= \frac{-907(12)}{8265} - \frac{422.3}{1176} + \frac{422.3 \cdot 15}{8265}$$

$$= -1.316 - 0.36 + 0.766$$

$$= 0.91 \text{ ksi Compression}$$

$$-f_{bot} = M/S - P/A - \frac{Pe}{S}$$

$$= \frac{907(12)}{8265} - \frac{422.3}{1176} - \frac{422.3(15)}{8265}$$

$$= 1.316 - 0.36 - 0.764$$

$$= 0.19 \text{ ksi Tension}$$

– Stress Limits

Comp = $0.6 f'c$ = $4.8 > 0.91$ ✓

Tens = $12\sqrt{f'c} = \frac{12\sqrt{8000}}{1000} = 1.07 > 0.19$ ✓

• Stresses @ Midspan

$$-f_{top} = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S}$$

$$= -\frac{\cancel{1880}\ 1880 \cdot 12}{8265.16} - \frac{422.3}{1176} + \frac{422.3 \cdot 25}{8265.16}$$

$$\cancel{= -1.814 - 0.36 + 1.27}$$

$$-2.685 - 0.36 + 1.27$$

$$= -1.44 \text{ ksi Comp.}$$

$$-f_{bot} = M/S - P/A - \frac{Pe}{S}$$

$$= \frac{1200 \cdot 12}{8265.16} - \frac{422.3}{1176} - \frac{422.3 \cdot 25}{8265.16}$$

$$\cancel{= +1.814 - 0.136 - 1.27}$$

$$+2.685 - 0.36 - 1.27$$

$$= 1.055 \text{ ksi Tension}$$

- Stress Limits

Comp = 4.8 > 1.44 ✓

Tens = 1.07 > 1.055 ✓

Shear Capacity

$V_u = 597$

$$V_c = \min \begin{cases} \left(0.6\sqrt{f'_c} + 700 \cdot \dfrac{V_u \cdot d_p}{M_u}\right) \cdot b_w \cdot d_p & = 600 \\ \left(0.6\sqrt{f'_c} + 700\right)\left(b_w \cdot d_p\right) & = 600 \\ 5\sqrt{f'_c} \cdot b_w \cdot d_p & = 384^k \end{cases}$$

$$\underbrace{\left(0.6\sqrt{8000} + 700 \cdot \frac{600 \cdot 44}{1250 \cdot 12}\right)}_{1520} \cdot 18 \cdot 44$$

$V_s = V_u - V_c$

$= 213^k$

4 legs #5

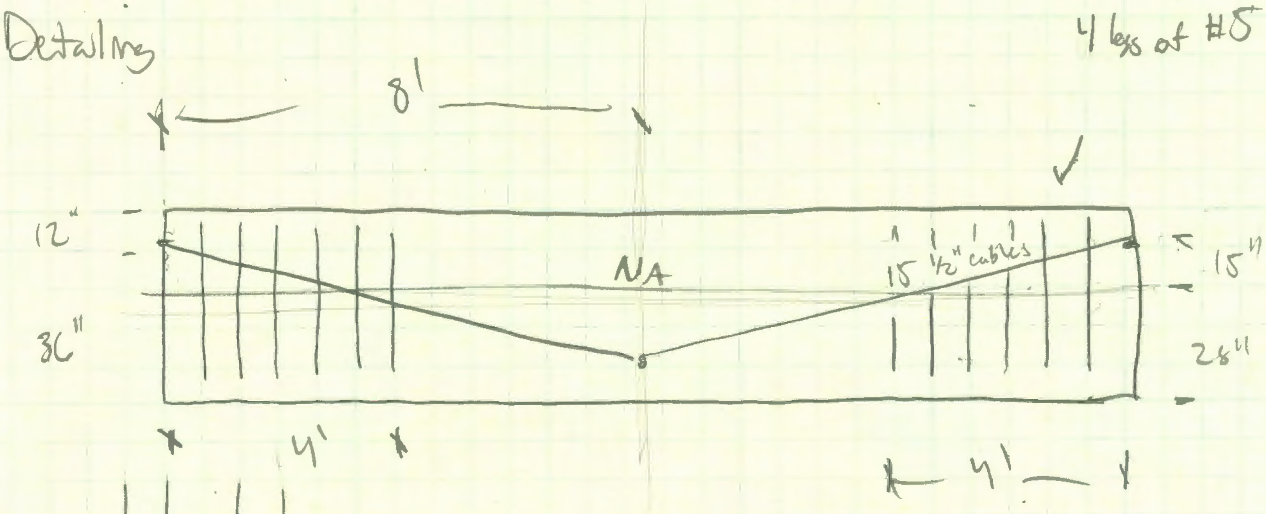
$$S_{req} = \frac{A_s \cdot f_y \cdot d}{V_s} = \frac{4(0.31)(44) \cdot 60}{213^k}$$

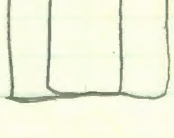
$A_v = 1.36 \text{ in}^2 > A_{v,min}$ ✓

$= 15''$

Choose 12''

Detailing



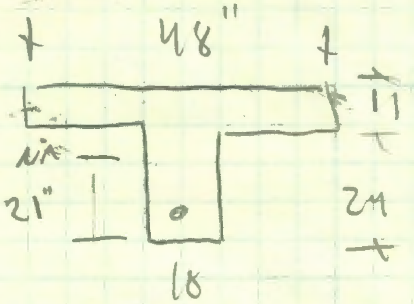

170" of steel for 1 stirrup

113.3 ft of #5 bars

Intermittent Beam (2) Amount/Bay

$M_{max} = 480$ @ ends $\qquad V_{max} = 180^{kips}$

$= 425$ @ middle




• Stress @ ends / middle

$-f_{top} = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S}$

$= -\frac{480 \cdot 12}{4570} - \frac{281.5}{960} + \frac{281.5(10)}{4570}$

$= -1.18 - 0.293 + 0.616$

$= -0.857$ ksi Comp $< f_{c\,max}$

$d_p = 32$ @ middle

$A = 960$

$S = 4570$,

$f_{pe} = 184$

$P = 184 \cdot 10(0.153)$

$= 281.5$

$e = 18$ mid

$= 10$ ends

$-f_{bot} = \frac{M}{S} - \frac{P}{A} - \frac{Pe}{S}$

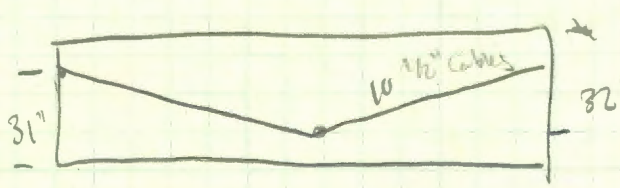
$= 1.18 - 0.293 - 0.616$

$= 0.271$ ksi Tens $< f_{t\,max}$

Shear Capacity =

$V_c = \begin{cases} \cdot\ \cancel{388.9} \\ \cdot\ \\ \cdot\ 257.5 \leftarrow \text{controls} \end{cases}$

Detail




$V_c > V_u \therefore$ No Shear reinforcement

# Design of PT Transfer Girder Post Progressive Collapse

- Max Moment = 2470 k ft — @ midspan and
  800 — @ 5' from support

Max Depth can be 4.5' deep

- Max Shear = 400 k — @ center
  200 — @ ends

- $f_{pu} = 270$ ksi

  $f_{py} = 0.9 f_{pu}$

  $f_{pi} = 0.82 f_{py}$ ← 199 ksi gains

  $f_{pe} = f_{pi} - \text{losses}$ assumed 15 ksi

  $= 184$ ksi

- $A_{ps}$ = 20 1/2" Tendons

  $= 3.06\ in^2$

- $P_i$ and $P_e$

Not checking @ Transfer

→ ~~$P_i = F_{pi} \cdot A_{ps}$~~

~~$= 199 \cdot 3.06$~~

~~$= 307.47$ kips~~

$P_e = f_{pe} \cdot A_{ps}$

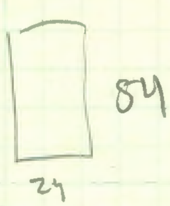
$= 184 \cdot 3.06$

$= 563$ kips

| 20 | 15 |
|---|---|
| 563 kips | 2,295 |
| | 422 kips |

- Section Modulus

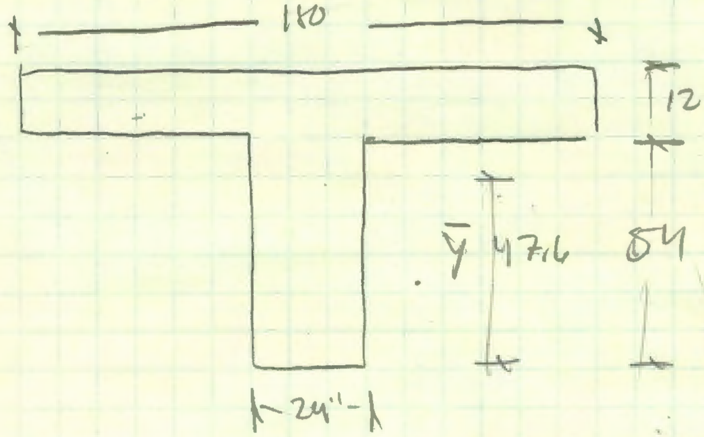
  24 x 48




Note: Tendon Profile should follow the moment diagram

$$bh^2/6 = 24(84)^2/6 = 11664\ in^3$$

• Calculation of NA for T-Beam




$$\bar{y} = \frac{\Sigma y'A}{\Sigma A} = \frac{24(84)(27) + 180(12)(60)}{(24)(54) + (180)(12)}$$

$$= \frac{164592}{3456}$$

$$= 47.6'' \text{ — NA}$$

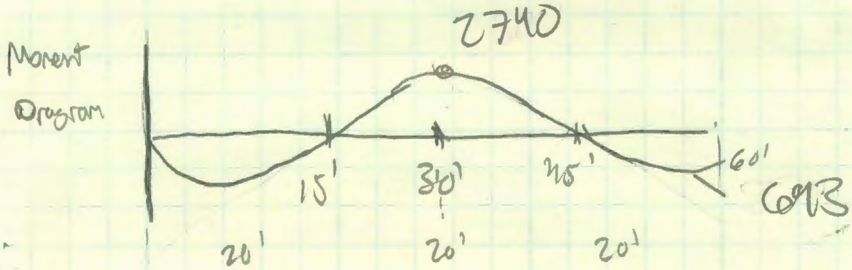
$\therefore$ e can be $\underline{\underline{40}}$ "

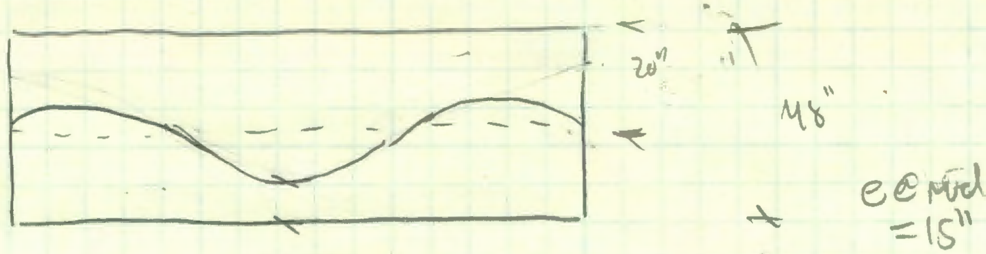
$10 in^2$ is 20 $1/2$" tendons

$A = \pi r^2$

$10 = \pi r^2$

$r = 1.78''$ — Not very Large $\therefore$ able to fit 20 ½ in tendons in the member




• Stress @ ends

$$-f_{top} = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S}$$

$\overset{||}{0}$ $\quad -\dfrac{543}{24 \times 84} \quad + \quad \cancel{3(\ldots)}$ @ ends = 0

$= -0.43$ ksi Compression

$$-f_{bot} = \underset{\overset{||}{0}}{M/S} - P/A - \underset{\overset{||}{0}}{Pe/S}$$

$= -0.43$ ksi Compression

– Compressive limit

$= 0.45 f'_c$

$= 0.45(8000)$

$= 3.6$ ksi $> 0.43$ ∴ ok ✓

• Stresses @ Midspan

$-3.56 + 0.43 + 2.44$

$$-f_{top} = -\frac{M}{S} - \frac{P_i}{A} + \frac{Pe}{S}$$

$$= \frac{-2740 \cdot 12}{11664} - \frac{563}{24 \times 54} + \frac{563 \cdot 40}{11664}$$

$$= -2.81 - 0.43 + 1.93$$

$$= -1.31 \text{ ksi Comp}$$

$f'_c = 8000\,psi$

$$-f_{bot} = \frac{M}{S} - \frac{P}{A} - \frac{P \cdot e}{S}$$

$$= \frac{3500 \cdot 12}{9216} - \frac{281.5}{24 \cdot 48} - \frac{281.5 \cdot 15}{9216}$$

$$= 4.55 - 0.244 - 0.458$$

$= 3.84$ ksi Tension ← too much Tensile stress

(add more Tensioning steel and increase section to $(24 \times 54)$)

with 15 $\frac{1}{2}$"

$$\frac{2740 \cdot 12}{11664} - \frac{563}{24 \cdot 48} - \frac{563 \cdot 40}{11664}$$

$$= 2.818 - 0.43 - 1.93$$

$= 1.053$ ksi Tens

Compressive limit $0.6 f'c = 4.8 > 1.31$ ✓

Tensile limit $12\sqrt{f'c} = 1.073 > 1.053$ ✓

• Stress @ 5' from supports

e @ 5' from support $\approx 15"$ max

$$f_{top} = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S}$$

$$= -\frac{693(12)}{11664} - \frac{422}{24 \times 54} + \frac{422 \cdot 15}{11664}$$

$$= -0.713 - 0.325 + 0.542$$

$$= 0.496 \text{ ksi} \quad \text{Tension}$$

$$f_{bot} = \frac{M}{S} - \frac{P}{A} - \frac{Pe}{S}$$

$$= \frac{693(12)}{11664} - \frac{422}{24 \times 54} - \frac{422 \cdot 18}{11664}$$

$$= 0.713 - 0.325 - 0.542$$

$$= -0.154 \text{ ksi Comp}$$

Comp Limit $= 0.6 f'_c = 4.8\text{ ksi} > 0.154\text{ ksi}$ ✓

Tension Limit $= 12\sqrt{f'_c} = 1.073\text{ ksi} > 0.496\text{ ksi}$ ✓

## Shear Capacity

$V_u = 425$ kips

$M_u = 2740$ kip ft

$$V_c = \min \begin{cases} 1 & \left(0.6 \cdot \sqrt{f'c} + 700 \cdot \dfrac{V_u \cdot d_p}{M_u}\right) \cdot b_w \cdot d_p \\ 2 & \left(0.6\sqrt{f'c} + 700\right) \cdot b_w \cdot d_p \\ 3 & 5 \cdot \sqrt{f'c} \cdot b_w \cdot d_p \end{cases}$$

d of tendon @ midspan

$$1_1 = \left(0.6 \cdot \sqrt{8000} + 700 \cdot \frac{425 \cdot (0.8)(58.5)}{(2740)(12)}\right) 24 \cdot 58.5$$

$$= (53.6 + 423 \ldots)$$

$= 670^k$ ← doesn't govern

$$2 = \left(0.6 \cdot \sqrt{8000} + 700\right) 24 \cdot 58.5$$

$= 1008^k$ ← doesn't govern

$$3 = \left(5 \cdot \sqrt{8000}\right) \cdot 24 \cdot 58.5$$

$= 627^k$ ← Controls

$V_c > V_u \therefore V_s$ not needed

Design for Spandrel Beams @ 5'

First 3 away from middle

$M_{max} = 1650$ k-ft

$V_{max} = 19.4$ kips

Last 3 away from middle

$M_{max} = 502$ k-ft

$V_{max} = 30$ kips @ 3.78' from support

Intermitent Beams
2 of them

$M_{max} = 975$ k-ft

$V_{max} = 41$ kips @ 4' from support

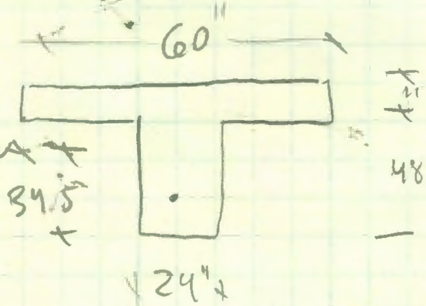
Amount (6)

Design of others from Middle

@ mid  @ supports

$M_{max} = 1650$ k-ft   1640.7

$V_{max} = 144$




• Stresses @ ends / middle

$-f_{top} = -\frac{M}{S} = \frac{P}{A} + \frac{Pe}{S}$

$= \frac{-1640\cdot 12}{8443} - \frac{422.3}{1812} + \frac{422.3(20)}{8443.2}$

$= -2.34 - 0.233 + 1$

$= -1.57$ ksi comp $\angle$ 4.8 Limit ✓

$d_p = 54.5$

$A = 1812$

$S = 8443.2$

$e_{ends} = 26.5$

$e_{mid} = 30$

$A_{ps} = 18.4$

$P = 18.4 \cdot 15 \cdot (0.153)$
$= 422.3$

$-f_{bot} = \frac{M}{S} - P/A - Pe/S$

$= 2.33 - 0.233 - 1.075$

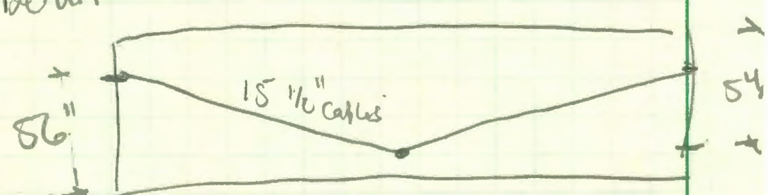
$= 1.022$ ksi Tens $\angle$ 1.07 Limit ✓

Shear

$$V_c = \begin{cases} \left(0.6\sqrt{8000} + 700 \cdot \frac{144 \cdot 54.5}{1650 \cdot 12}\right)(24 \cdot 54.5) = 432.2^k \\ \end{cases}$$

controls

Detail




$V_c > V_u$ ∴ No Shear Reinforcement

Column Design for Progressive Collapse

| | System 1 | System 2 |
|---|---|---|
| Additional Load | – 2375 | – 2375 kips |
| Existing Load | – 630 | – 1380 kips |
| Additional Moment | – 800 k ft | – 663 k ft |
| Existing Moment | – 0 | – 0 |

Note: Assumed to be an interior column not exterior

System 1 colums

$P_y = 3000^{kips}$

$M_x = 800^{k\,ft}$

30" x 30"

12 #8 evenly spaced

System 2 colums withstand

$P_x = 3725^{kips}$

$M_x = 700^{k\,ft}$

36"x36"

12 #9 evenly spaced

# 13.3.5 Acoustical Analysis

Approach for Acoustical Depth

- Spaces to be analyzed
  - Main Lobby (has opening in floor slab above) Arch LvL2 FLvL1
  - Atrium
  - Imagine office dimension/space — open plan offices
  - STC of the exterior wall (to block sound from outside)
- Mention how the floor plan is already laid out well for acoustics, every major system is in the middle of the building, also on roof

- Reverberation time, Based on Graph in Notes from Figure 17.10 of Architectural Acoustics.

- Material Properties of each space and their absorption coefficient $\alpha$

– Main lobby

Floors OG-5 / ST-1

Walls GL-15 $1/4" \to 1/2"$

Ceiling Painted concrete

– Office

typ walls — See partition schedule

Calculation of Reverberation time

freq ≈ 1000

(table 7.1)

$V = 10000\ ft^3$

- Main lobby

| Surface | Material | $\alpha$ | SA | $SA \cdot \alpha$ |
|---|---|---|---|---|
| - Floor | Glazed tile | 0.01 | 1066 | 10.66 |
| - Ceiling | Painted Conc | 0.02 | 350 | 7 |
| - Hole in Ceiling | open | 0.02 | 367 | 7.34 |
| - Walls | 5/8" Gyp with 2" Sound Blanket | 0.07 | 805 | 56.35 |
| - Storefront | 1/2" glazed glass | 0.06 | 210 | 12.6 |

$\sum SA = 3000$ $\quad \sum SA \cdot \alpha = 94$

Use Sabine Eqn $\longrightarrow \bar{\alpha} = 0.034$

$$RT = \frac{0.049 \cdot V}{\sum SA \cdot \bar{\alpha} + 4 m_a \cdot V} = \frac{0.049 \cdot 10{,}000}{3165 \cdot 0.027 + 4(10{,}000)(1.85 \times 10^{-4})}$$

$$= \frac{490}{100} \approx 5 \text{ Seconds}$$

$\boxed{RT = 5 \text{ Seconds for } 1000\ Hz}$ — TOO High

Specs 474 084413

- Atrium

A = 500 sf

Levels

$V = 500 \cdot 110$
$= 55000\ ft^3$

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| Surface | Material | | | | SA |
|---|---|---|---|---|---|
| - Floor | glazed tile | | | | 500 |
| - Walls Ext | Double IGU | 1/4" Ext panel | 7/16" Spacer | 5/16" Int | 2700 |
| - Walls Int | Single ordinary | | | | 7700 |

- Office Floor

$V = 35606\ ft^3$

| Surface | Material | SA |
|---|---|---|
| - Floor | Carpet 8mm Needle Felt on Concr | 3560 |
| - Walls | GWB (1/2") 2x4 studs | 5000 |
| - Ext Walls | Double IGU | 1500 |
| - Ceiling | ACT Gypsum plaster tiles 12"x24" | 3560 |

Analysis of RT

Recomended RT
from asona acoustics

- Lobby

Recomended 0.5 – 1

need to change 250 Hz RT

- Atrium

Recomended 1 – 2

(Add Acoustical floor tile)

need to change RT for every freq

- Office floor

Recomended 0.1 – 0.4

Change RT for 250 Hz freq

- Office / meeting Room

Recomended 0.4 – 0.6

No need to change