706 Madison Avenue | New York, USA

Structural Redesign Final Report

Submitted to: Dr. Linda Hanagan, Advisor

Prepared by: Yong Yue [Structural Option]

Prepared on: April 3rd, 2017

706 Madison Avenue

New York

YONG YUE | STRUCTURAL

ADVISOR | DR. LINDA HANAGAN

http://allenyueyong.wixsite.com/thesis

GENERAL INFORMATION

Building Height:	65 feet
Occupancy Types:	High-end Retails
Size of the Building:	48491 sq. ft.
Number of Stories:	5 above grade
Cost of Building:	$1000/ft^2 (Estimate)
Date of Construction:	Mar. 2015 - 2017
Project Delivery Method:	Design-bid-build

PROJECT TEAM

Owner: Friedland Properties

Construction Manager:

Architect: Page Ayres Cowley Architects, LLC

Structural Engineer: Simpson Gumpertz & Heger, PC

MEP Engineer: Ettinger Engineering Associates, PC

STRUCTURE SYSTEMS

Framing.........Structural Steel - Beams, girders and columns; composite Metal Deck with Concrete Slab

Foundation.....2'-6" thick mat slab reinforced with steel reinforcement at the top and bottom

Lateral.........Existing Building - Exterior Masonry Walls/Cores; Addition - Steel Moment Frames

MECHANICAL SYSTEMS

- A heat pump on the sub-cellar level with cooling capacity of 30 MBH and a heating capacity of 32 MBH, coupled with an air cooled condensing unit on the roof
- A general supply fan, with a 15 KW duct heating coil, and general return fan located on the sub-cellar level that provide heating and 600 CFM of ventilation
- Multiple unit heaters of three different size, 5 KW, 7.5 KW, 10 KW, are provided on every level
- Duct risers and roof-top areas reserved for the mechanical systems of future fit-outs.

LIGHTING AND ELECTRICAL SYSTEMS

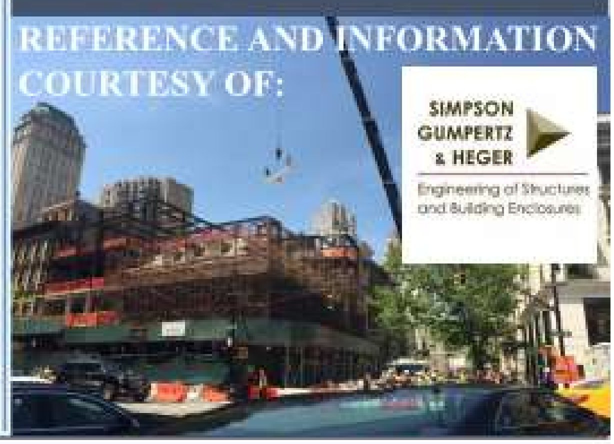
- 6 fixtures for the whole building; a 1'x4' fluorescent surface mounted fixture is the most common fixture
- A 3 phase 1200 kVA transformer bumps up the voltage to 480/277V for the mechanical equipment
- Skylights tends to improve the scales rate of the store that have access to that daylight
- Each tenants is switched separately

CONSTRUCTION

- Excavation adjacent to historical townhouse
- Challenges including tangent-pile wall misalignment, below-grade protrusions at the adjacent buildings, and high ground water

ARCHITECTURE

- 3-story landmark with a 5-story enlargement
- The façade of the addition needed to match the existing to meet the historical requirements
- Elevators, stairs, corridors and MEP rooms are placed as close as possible to provide more space for retail use.
- A roof garden at the top of the existing building provides a



Executive Summary

706 Madison Avenue is a 48,500 square-foot, high-end retail building located on the southwest corner of Madison Avenue and 63rd Street in the upper east side of Manhattan, New York. The building consists of a 3-story existing landmarked building and a five-story horizontal extension on two sides.

The existing landmarked building was built in 1920 and was initially constructed with masonry walls, steel columns, cinder concrete slabs, and marble and brick façades. Back in the 1920s, building codes didn't require seismic design for structures. So the old building wasn't designed to resist seismic load. However, the masonry walls and core stairwells in the building have been designed for wind.

Construction began on March 2015. It is still under construction and scheduled to be done in January 2016. The structural system of the addition consists of steel columns, concrete slab with composite metal deck, a mat foundation and moment frames for the lateral load resisting system. Due to the difficulty of rebuilding the old building with the new building codes, the lateral system of existing building is kept. The addition's lateral load resisting system is designed independently from the existing building.

The building's design was based on the 2008 New York City Building Code. Additionally, the exterior of building needed to meet the historical requirements, which are regulated by Landmark Preservation Commission (LPC).

The proposed thesis will include an investigation of a concrete rigid frame structure, a two-way concrete slab system and reinforced concrete moment frames. The redesign will also propose a reconstruction of the whole building as opposed to an addition of two separate buildings. The façade of the existing building will be preserved in order to meet the historical requirements and Landmark Preservation Law.

In addition to an in-depth structural analysis, the historic facade preservation and the indoor air quality will be studied with redesigning the building in the spring 2016 semester.

Table of Contents

[1] Introduction

1.1 Purpose

This report has been written to develop a new design of structural system in the 706 Madison Avenue. The building's technical information and challenges have been indicated so that readers can learn the fundamental information of building quickly through the introductory narratives. More detailed structural information of the building has been introduced in subsequent sections, which provide a deeper understanding of the existing structural system of the building. Upon the analysis of the existing structure system, a new design has been proposed and achieved by the end 2017 spring semester.

1.2 Scope

In order to fulfill the objective indicated above, the content of this report will focus on four major sections: structural aspects, proposal, new design, and breadth topics. Structural aspects include an overview of the building, structural framing systems, lateral resisting systems, foundation systems, joint connections, load determination, load paths, building enclosure/façade, code requirements and the historical requirements. The structural proposal contains a problem statement, proposed solution, tasks and tools, two breadth topics, MAE Coursework and a schedule chart. The new design is comprised of gravity depth, lateral depth and cost estimation. The breadth topic consists of façade preservation and indoor air quality.

1.3 General Building Description

706 Madison Avenue consists of a three-story existing landmark and a five-story new addition on two sides. The existing building is protected under the Landmark Law and a very important part of the City's Heritage. Therefore, LPC (Landmark Preservation Commission) must approve in advance any alternation, reconstructions, demolition, or new construction. The total area of the building is 48,500 square feet. In the Figure 1, the existing building is rectangular shape and about 72' x 40'. The new enlargement is L shape and dimensions of all perimeters are shown in the figure. The building was converted from a bank to high-end retail use, which includes a sub-cellar floor with storage and mechanical spaces, multiple floors of retail clothing stores, and an outdoor-café roof terrace as shown in the Figure 2 below.

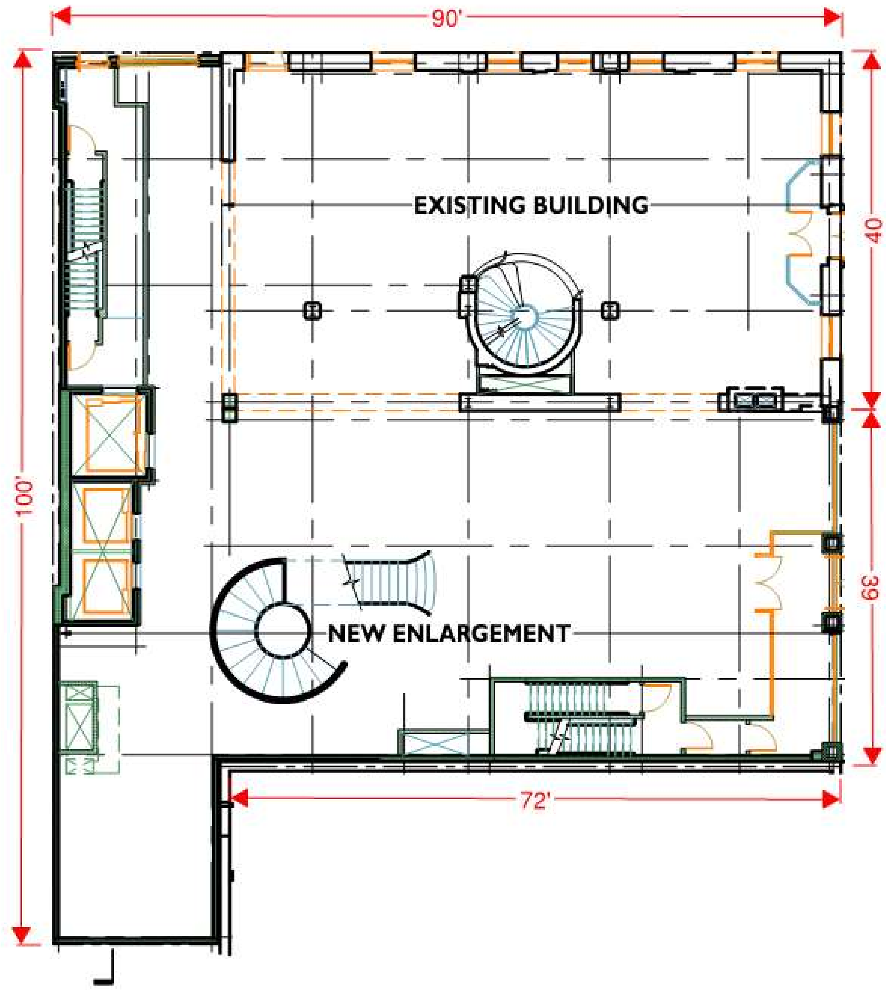


Figure 1 – Building Floor Plan

Figure 2 - Building Section from 63 Street

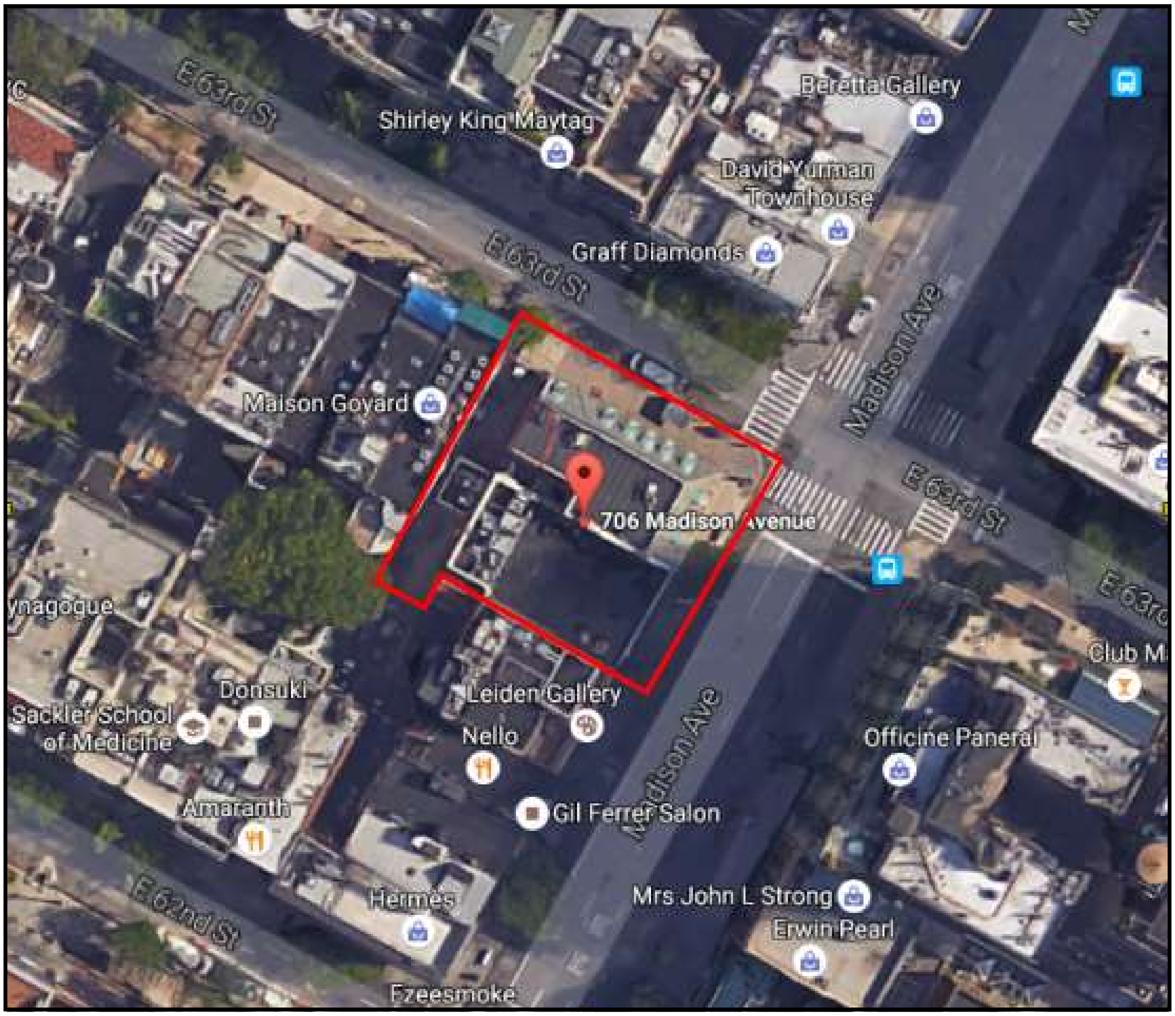


Figure 2 - Building Site [Courtesy of Google Maps]

As shown in the Figure 2 above, 706 Madison Avenue is located at the southwest corner of Madison Avenue and East 63rd Street, which is in an historical district at the upper east side of Manhattan, New York. The district preserves and reinforces the unique retail and residential character of Madisonn Avenue an the surrounding area. Since the building is in the historical district, the historical requirements for buildings influence the design of this building, especailly in building façade design.

This building started design development in March 2015 and will be finished in January 2016. The project delivery method is design-bid-build and the cost of the project is estimated to be $1000 per square foot without design fees. The building is designed by Page Ayres Cowley Architects and the structural consultant is Simpson Gumpertz & Heger (SGH). JRM construction has been chosen to be the construction management team cooperating with the designers and individual contractors to construct the building on site.

1.4 Structural Framing System Overview

In the 1920s, the existing landmarked building was built to be a steel frame structure with a structural assembly including beams, columns, cinder concrete slabs, masonry walls and a masonry core. Cinder concrete slab construction became one of the most dominant structural slab systems used from the 1920s to the 1940s. However, the cinder concrete slab cannot span longer since the steel draped wire mesh in the slab is not able to provide enough tension force. Therefore, it's replaced by the composite deck with concrete slab. The columns in the center of the building are able to be tear out to have a more open space for retail use. Considering the lateral system in the existing building, the exterior masonry walls and interior stairwells were designed to resist wind loads.

The addition is structurally independent from the existing building. The structure of the addition is comprised of composite metal deck with concrete slab, moment frames as a lateral load resisting system and mat-slab foundation. The doorways are adjoin two building which are separated by four inches spacing. . The addition will be analyzed in the following reports due to its height, complexity and accessibility.

The design of the new addition was challenging due to the constrained site conditions. The building has two below-grade stories where the new excavation is adjacent to historic townhouses. Because of multiple unforeseen conditions, including a tangent-pile wall misalignment, below-grade protrusions at the adjacent buildings, and high ground water, the team needed to re-design the foundations and lateral system of the addition several times as the construction proceeded.

[2] <u>Structural Framing System</u>

In this section, the detailed structural framing systems within the building will be introduced and discussed, including typical bay framing, floor and roof framing, foundation system, columns, lateral load resisting system, and load paths.

2.1 Typical Bay Framing

The typical bay framing in this building is classified into two bay categories: ordinary bay framing in the addition and the renovated bay framing in the existing building as shown in the Figures 4 and 5 below. The dimensions of the bay in the addition is approximately 29'-0" x 16'-0" and the dimension of the renovated bay is approximately 17'-7" x 19'-7". Two bays are both framed by steel beams, steel columns, and a composite metal deck with concrete slab.

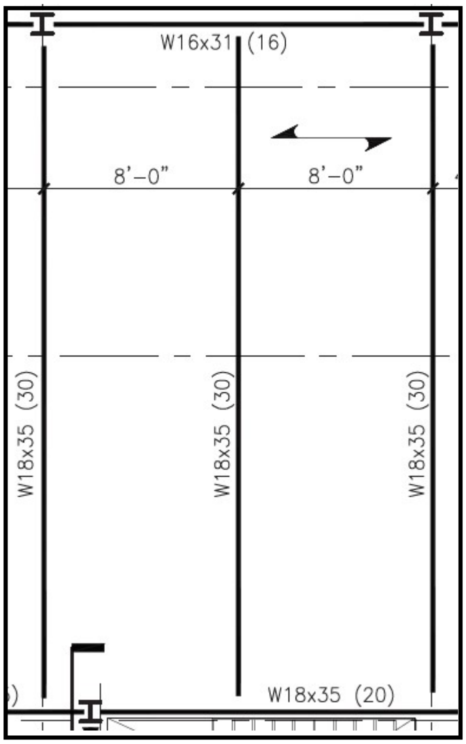


Figure 4 - Bay Framing in the Addition

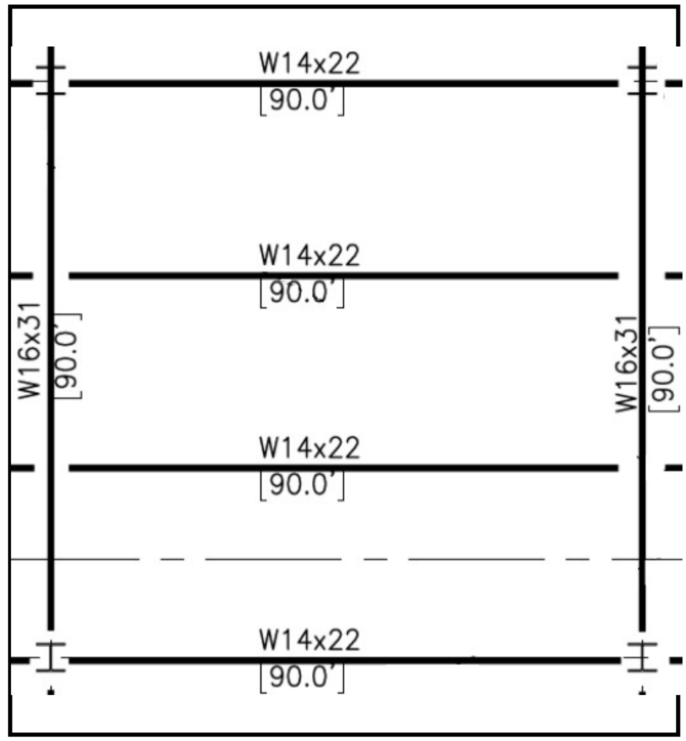


Figure 5 - Bay Framing in Existing

2.2 Floor and Roof Framing

The cinder concrete slab construction in the existing building is replaced by a concrete slab on composite metal deck, which is made of by 3 ¼" lightweight concrete over 1 ½"-18GA metal deck reinforced with welded wire fabric (WWF4x4). The addition adopts similar slab/deck system; however, it uses 3"-16 GA. metal deck to accommodate longer slab spans. Furthermore, headed shear studs are arranged to be 1 stud per foot in order to provide a composite construction for the slab and the steel beams. A typical detail for the reinforcement of the concrete slab is shown below in Figure 6.

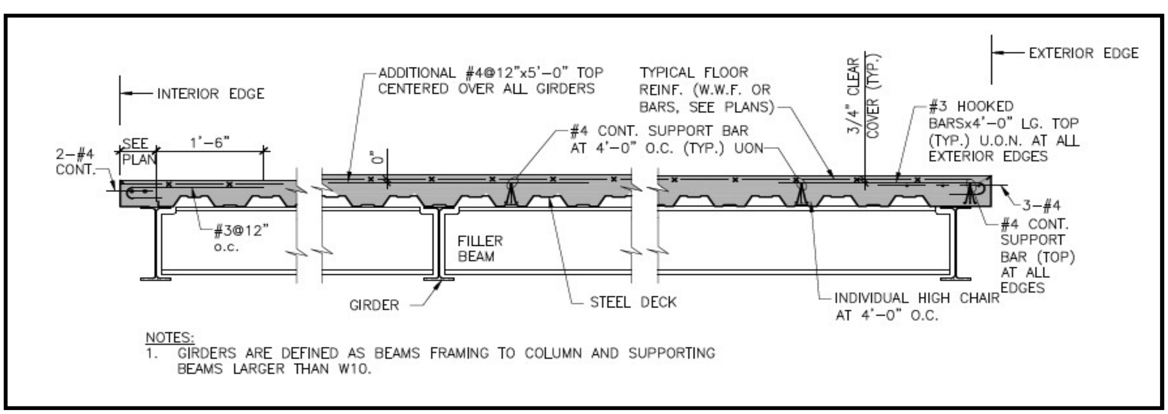


Figure 6 - Typical Floor section

2.3 Foundation System

The foundation system of 706 Madison Avenue addition is comprised of 2'-6" thick mat slab reinforced with steel reinforcements at the top and bottom. Shear reinforcement is also provided around the steel column and base plate to prevent the foundation from cracking due to shear. Figure 7 shows a detailed diagram of the reinforcement of the foundation slab as well as the 24" concrete pile wall along the slab step. The minimum reinforcement in slab is at least 0.0018 times the area of the concrete in each direction. The minimum concrete clear cover is 3" at the bottom of the mat slab and 2" at the top of the mat slab.

The concrete slab that runs horizontally and vertically through the foundation has a compressive strength of 5000 psi. According to a recommendation given by geotechnical engineers, the mat slab is designed for a maximum allowable bearing of 2.5 KSF typically and 4 KSF in the southwest corner of new addition.

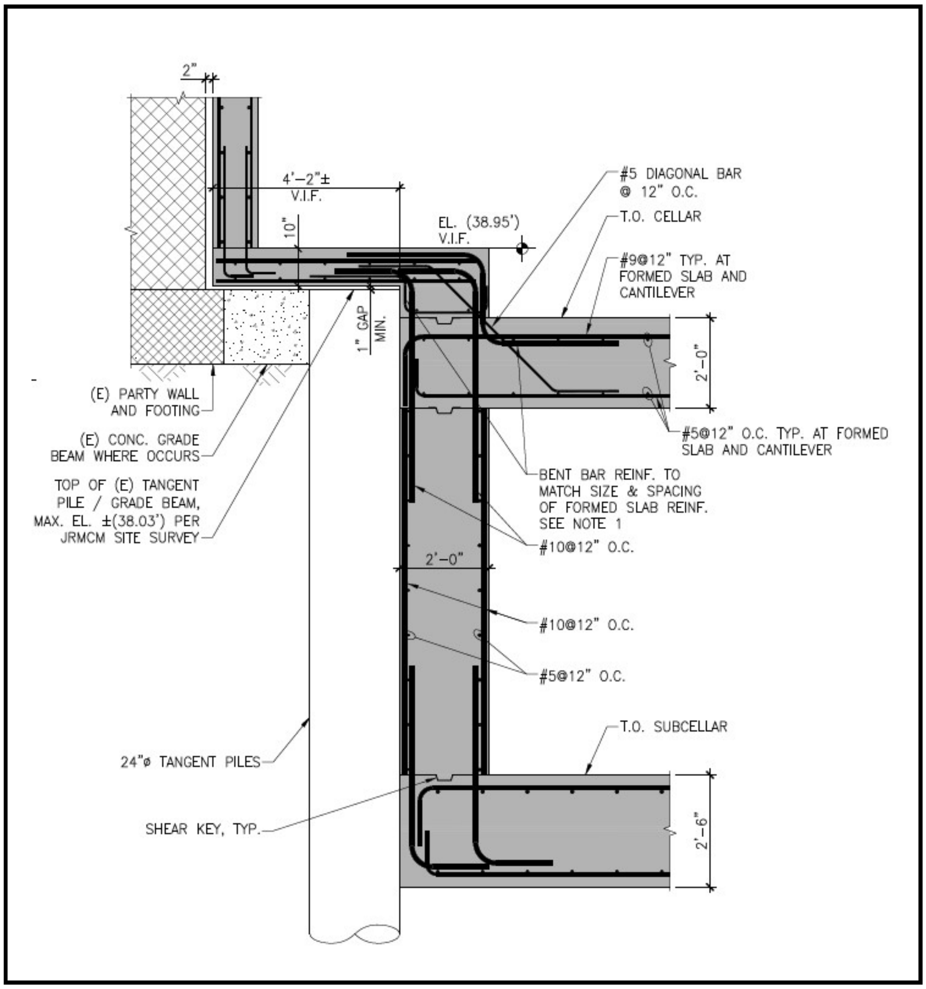


Figure 7 - Slab Step Section

2.4 Columns

The structural columns specified in structural framing design of the 706 Madison Ave are mostly W-Shape ASTM A992/A992M steel columns and a small group of HSS ASTM A500 hollow steel columns in the corridor or around stairwells. Most columns from sub-cellar level to cellar level are W10. Column size is in a range between W10 to W14 and the largest size of the column utilized in this building is W14x176.

The columns are typically spliced at the interfaces between the 1st and 2nd floors, 3rd and 4th floors, and 5th floors and roof. Three different splices utilized in column connection design are gravity column splice, gravity column splice with changed nominal depth, and moment frame column splice as shown in Figure 8. Most of column splice constructions are welded. Moreover, at the foundation all columns will be welded onto ASTM A36 steel base plates and be connected to the foundation mat by anchor bolts.

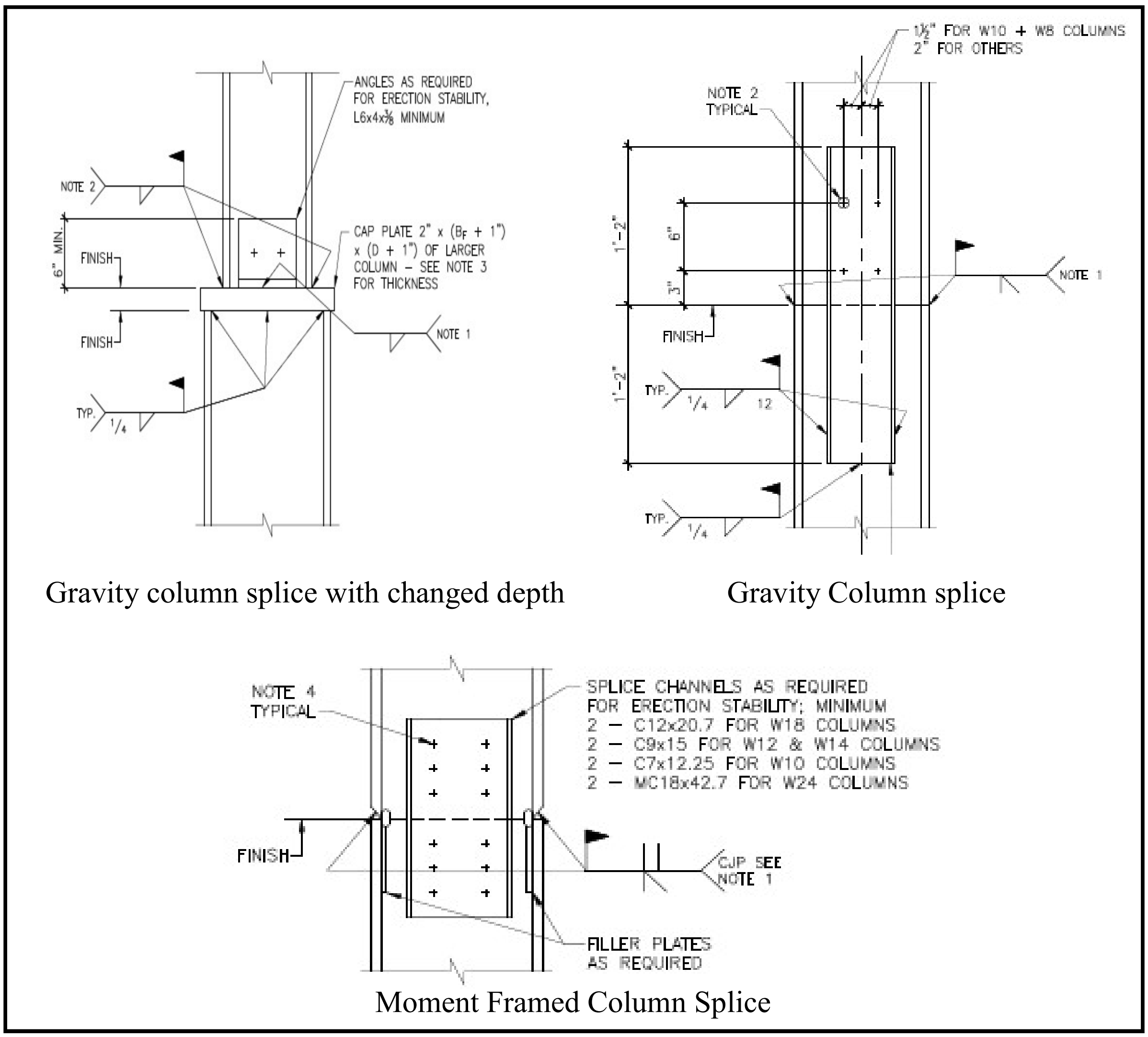


Figure 8 - Typical Column Splices

2.5 Lateral Load Resisting System

As mentioned in Section 1.4, the exterior masonry walls and the interior masonry stairwells both serve as the lateral load resisting system for the existing building. All lateral load resistance and stability of the new addition is provided by steel moment frames that are shown in Figure 10. The new addition is seismically independent from the existing building as a result of 4" seismic gap between the addition and the existing building. This is provided in order to accommodate an expansion joint assembly. The moment frames are detailed to include designed lateral connection at the surface of the column and beam as shown in Figure 9.

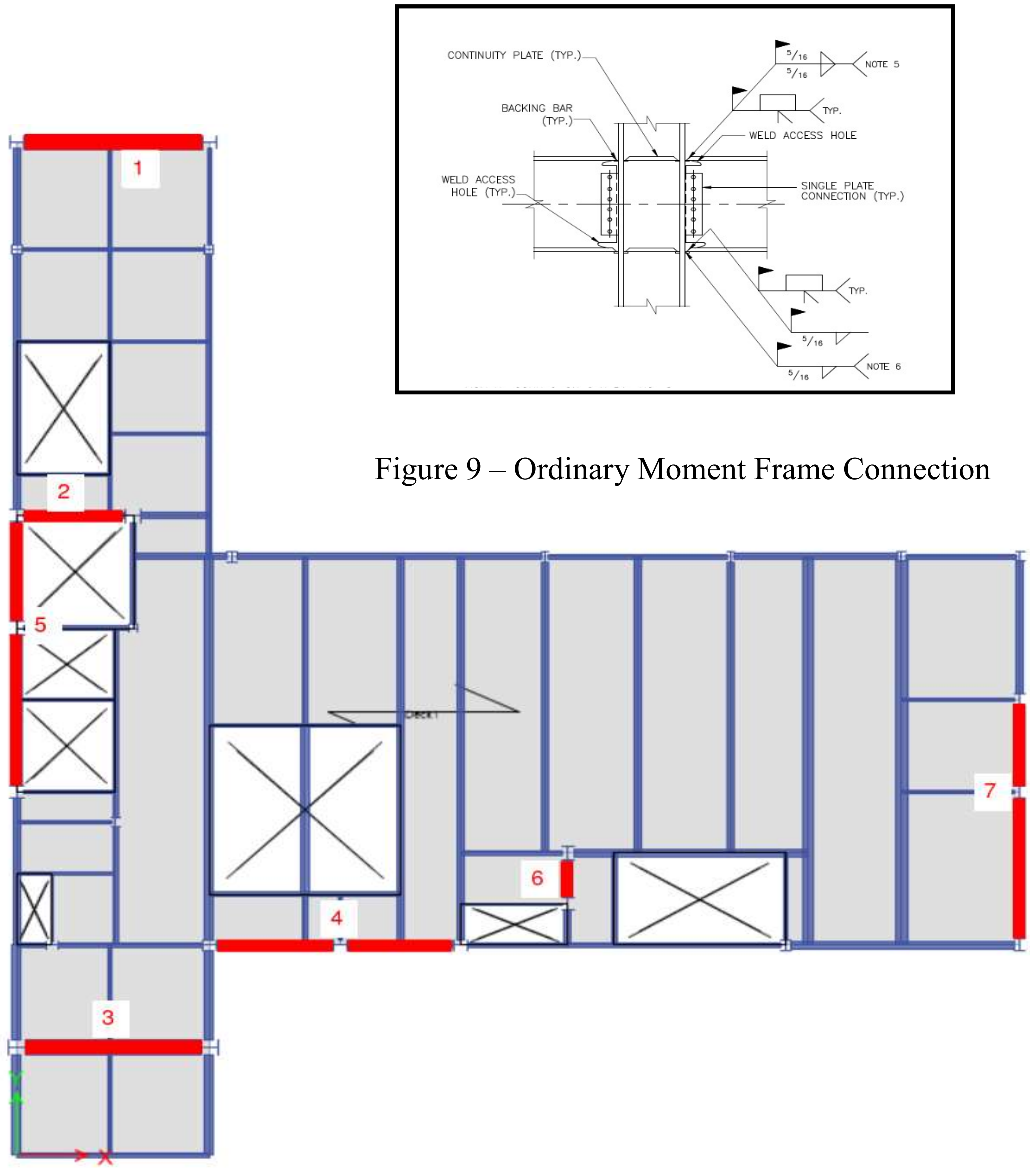


Figure 9 – Ordinary Moment Frame Connection

Figure 10 – Ordinary Moment Frame Connection

2.6 Load Paths

In order to determine the load path of the structural design, two load classifications must be considered: gravity and lateral.

The typical gravity loads, including dead, live, snow, and rain will be resisted by the roof or floor slabs transmitted to the steel girders through the beams, and transferred to the mat slab through the steel columns. The mat slab then spreads the gravity load out into the ground. The foundation is designed to prevent the slab from cracking and to prevent differential settlement caused by gravity loads.

Gravity loads are not only type of load that is considered when designing a structure. Lateral loads including wind and seismic loads must also have a complete load path to transfer them to the ground. Unlike gravity loads, which act in a downward direction, lateral loads can act in a horizontal direction or even cause an uplift effect. Wind loads act on the exterior facade of the building directly. Seismic loads are caused by an earthquake. When an earthquake occurs, the earth accelerates and it causes structure to move. The seismic loads are caused by acceleration and mass of the building and horizontally distributed on the structure. Because wind loads and seismic loads hit the building horizontally, they accumulate within the diaphragm, or floor, of a building. As they accumulate, they follow their load path according to stiffness within the structure. In this building, the composite concrete on metal deck floors serves as a horizontal diaphragm that distributes the lateral wind and seismic forces from exterior facades to the lateral elements, which are moment frames. Moment frames then carry the applied lateral loads to the building foundation. The foundation is designed to resist uplift resulting from the overturning moments caused by lateral loads.

[3] <u>Loads</u>

This section focuses on a description of loads that have been used to design 706 Madison Ave and how they were determined per the national codes, standards and design codes.

3.1 Building Codes and Reference Standards

All the building codes, standards and structural design codes used to design 706 Madison Avenue have been listed in the table below (Table 1).

Table 1 – Applicable Codes

Category	Building Codes/Reference Standards
Building Codes	New York City Building Code (NYCBC) 2008
Load Determination	American Institute of Civil Engineers (ASCE) 7-02
Concrete Design	American Concrete Institute (ACI) 301-306, 315, 347
Steel Design	American Institute of Steel Construction (AISC) 360-05
Seismic Design	American Institute of Steel Construction (AISC) 341-05
Welding Design	American Welding Society (AWS)
Composite Deck	Steel Deck Institute (SDI)

3.2 Dead Load

The design dead loads were determined based on the materials' characteristics and manufacturer's data. The structural drawings describes dead load as "All permanent stationary construction". Therefore, dead loads are determined by the self-weight of the building components.

3.3 Live Load

The following design live loads were determined on the basis of the reference standard ASCE 7-02. The primary design live loads haven been found in structural drawings and listed in the table below (Table 2).

Table 2 – Live Loads

Live Load	Load value
1. Retail - 1st Floor	105 psf
2. Retail - Upper Floors (2nd , 3rd, and 5th floors)	75 psf
3. Public Assembly space (4th floor, including setback roof terrace)	100 psf
4. Stairs and Exits	125 psf
5. Storage (Sub-cellar and Cellar)	600 psf
6. Elevator Machine Room	125 psf

3.4 Snow Loads

Where appropriate, drifting snow loads have been considered in accordance with Section 1608 of the Building Code. The primary design snow load information has been found in the structural drawings and listed in the table below (Table 3).

Table 3 – Snow Loads

Snow Load	Load Value
1. Ground Snow Load, P_g	25 psf
2. Flat Roof Snow Load, P_f	20 psf
3. Snow Exposure Factor, C_e	0.9
4. Snow Load Importance Factor, I_s	1.0
5. Thermal Factor, C_t	1.0

3.5 Wind Loads

The following design wind loads are determined on the basis of the reference standard ASCE 7-02. The primary design wind load information has been found in the structural drawings and listed in the table below (Table 4).

Table 4 – Wind Loads

Wind Load	Load value
1. Basic Wind Speed (3 sec gust), V	98 mph
2. Wind Importance Factor, I_w	1.0
3. Wind Exposure	B
4. Internal Pressure Coefficient	+/-0.18

3.6 Seismic Loads

The following design seismic loads are determined on the basis of the reference standard AISC 341-05. The primary design seismic load information has been found in the structural drawings and listed in the table below (Table 5).

Table 5 – Seismic Loads

Seismic Load	Load Value
1. Seismic Importance Factor, I_E	1.0
2. Spectral Response Acceleration, S_s	0.365
3. Spectral Response Acceleration, S_1	0.071
4. Site Class	D
5. Spectral Response Coefficient, S_{DS}	0.367
6. Spectral Response Coefficient, S_{D1}	0.114
7. Seismic Design Category	C
8. Design Base Shear, V	164,000 Ibs
9. Seismic Response Coefficient, C_s	0.16
10. Response Modification Factor, R	3
11. Seismic Force Resisting System	
a. Steel Moment Frames	
b. Ordinary Reinforced Concrete Shear wall	

[4] Joint Details and Connections

Joints and connections are very important components of the building construction because they provide a smooth or flexible place for the building to expand, contract, and move without overstressing the structure and causing cracking problems. This section outlines two different type of joint systems and briefly introduces steel connections.

4.1 Building Expansion Joints

The seismic joint between the new addition and the existing building serves as an expansion joint, which can not only absorb the heat-induced expansion and contraction of concrete slabs or walls, but also provides a space where the concrete slab can move due to the seismic or wind load without overstressing the concrete and causing cracking problems. As shown in Figure 11, the 4” seismic joint, formed with soft material, located between two concrete buildings will allow one of the two buildings to move independently from the other during a seismic or wind event without imposing force on the other building.

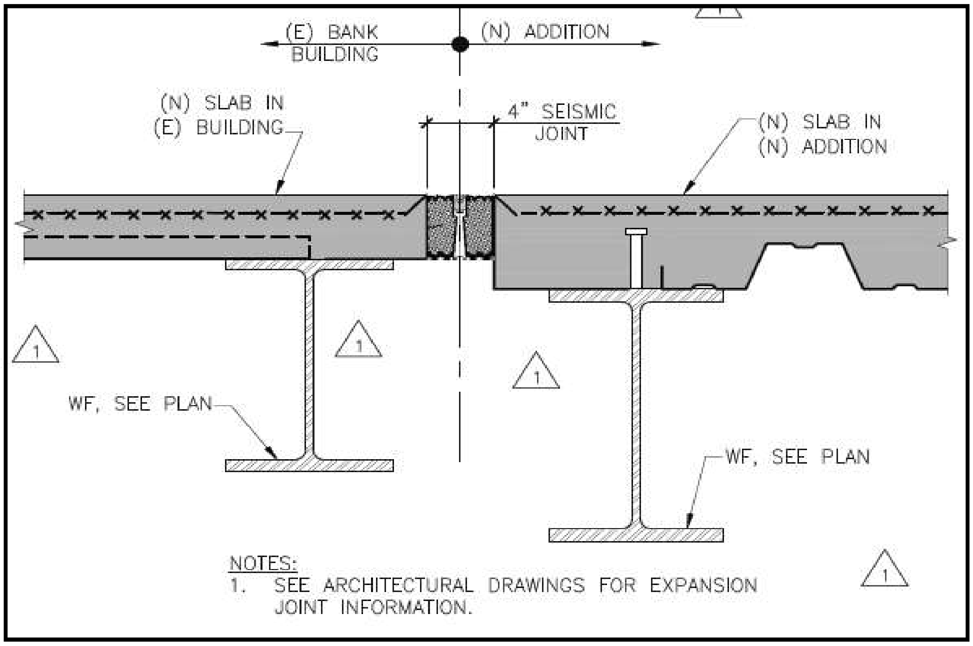


Figure11 – Seismic Joint Between Addition and Existing Building

4.2 Construction Joints

As shown in the figure below (Figure 12 & 13), two type of construction joints are utilized to design the connection of 706 Madison Avenue: the horizontal wall construction joint and the vertical construction joint. As shown in the figure below (Figure 11), the CONT. 1x2 or 12" LONG KEY @ 24" acts as a construction joint and is located at predetermined pour stops or where the first pour stops and the second pour will occur. The joint is to help provide continuity between pours to help maintain structural integrity in shear and reduce cracking.

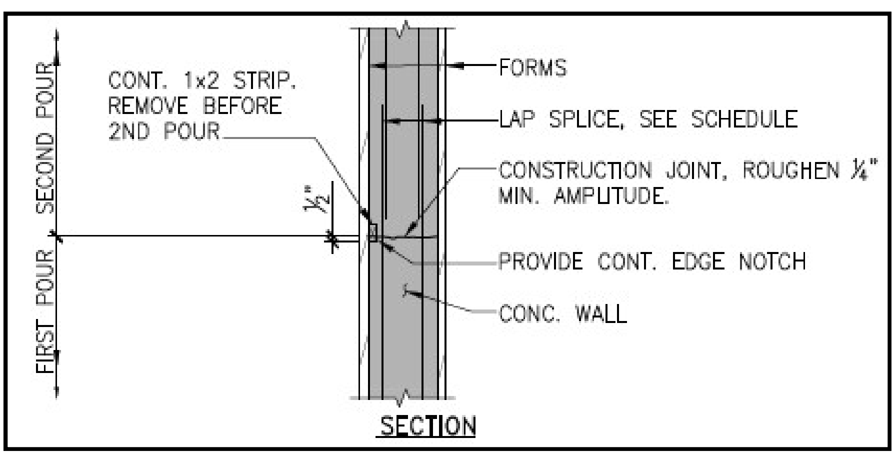


Figure 12 – Horizontal Wall Construction Joint

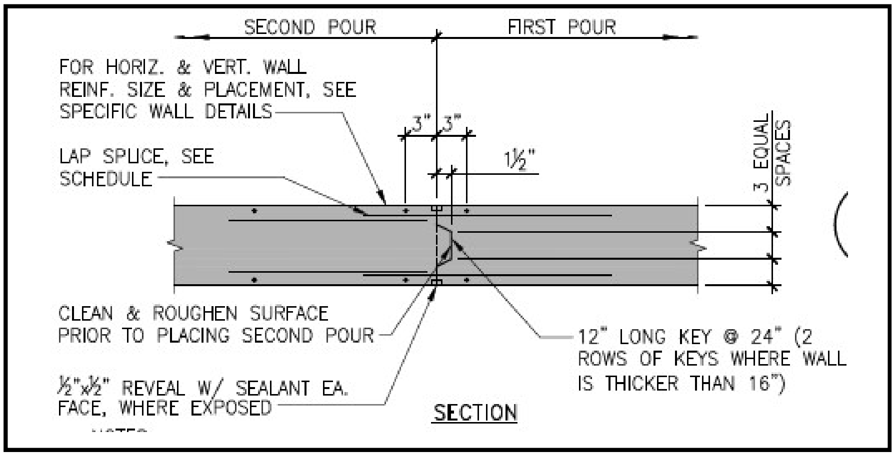


Figure 13 – Vertical Wall Construction Joint

4.3 Steel Connections

706 Madison Ave consists of a series of steel connections, which includes the beam shear connection, typical beam framing to spandrel Beam connection, beam-to-beam moment connection, wide flange column with web parallel to beam web, wide flange column with web perpendicular to beam web, typical moment frame connection and bolted wide flange brace connection, etc. Figure 14 indicates some types of steel connections and details of welding and bolting.

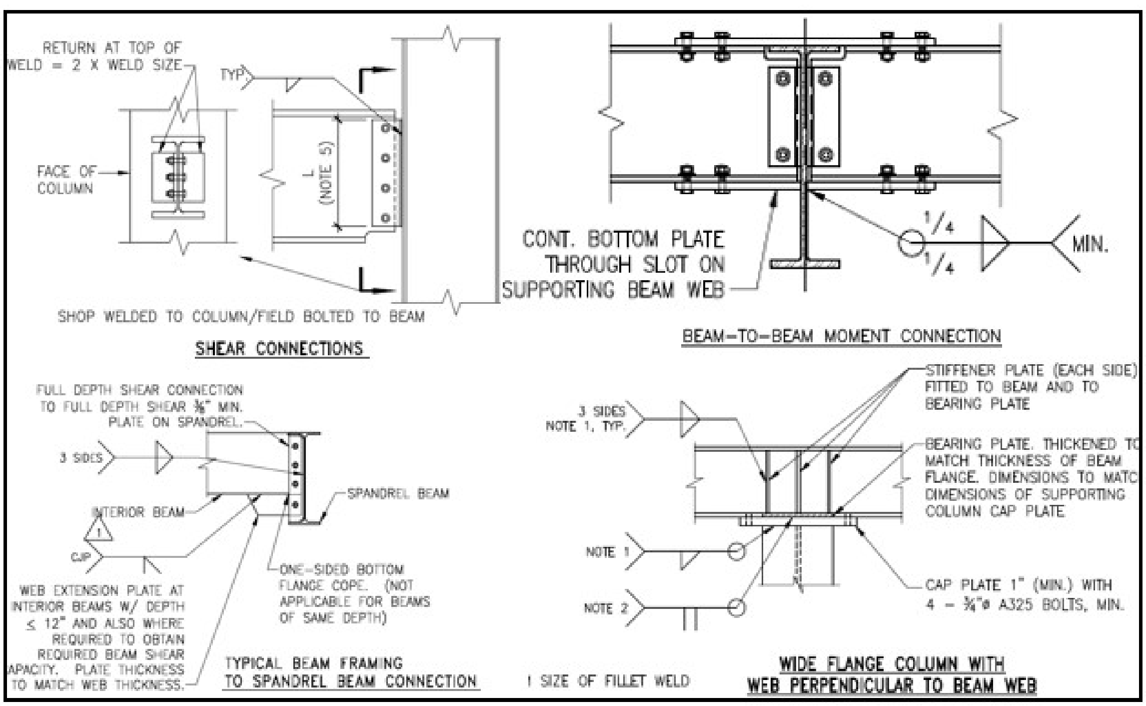


Figure 14 – Details of Steel Connections

[5] Proposal

5.1 Problem Statement

Upon completion of the initial analysis of 706 Madison Avenue, based on the information indicated in the previous reports, the current designs of the building have been proven to be sufficient to meet all necessary strength, code, and serviceability requirements. Additionally, the building meets the historical requirements.

The addition has been designed literally to meet owner's request that was to have a commodious room for retail use. The request has been fulfilled by eliminating interior columns and using transfer girders; however, the space in the addition is not capacious enough to be used efficiently for retail use. As shown in the typical floor plan in the previous reports, the area on the left is kind of narrow and wasted. Moreover, due to use of the transfer girders, the structural layout of the floors are irregular. It causes the structural design of the building impossibly having an alternative solution.

To continue to fulfill this request and pursue an alternate design solution, some design constraints must to be relaxed. My proposal is to have a reconstruction of the whole building as opposed to an addition of two separate buildings. The façade of existing building will be preserved in order to meet the historical requirements and Landmark Preservation Law. In addition, the spring 2017 will propose two-way slab concrete system rather than composite steel system for consideration of durability, finical saving, resource efficiency and energy efficiency.

5.2 Proposed Solution

The original walls between two buildings will be eliminated in the redesign of the building. Two column lines will be added in E-W direction and four column lines will be added in N-S direction. The columns are distributed uniformly. The 23' x 27' new typical bay size and concrete flat slab gravity system will be utilized for the redesigned structure. Preliminary beam and column sizes will be explored further to meet strength and serviceability requirement. For the lateral force resisting system, reinforced concrete moment frames will be utilized in the similar location where existing steel moment frame were. The façade of existing building will be temporarily supported during the construction and the pitched roof will be moved off and reuse at the end of the construction.

Methods of this approach will be discussed in Section 5.3. Research on the preservation of historic façade will be conducted as a breath topic in Section 5.4. The 2008 New York Building Code and minimum design loads from ASCE 7-02 will be referenced for the solution proposed above.

5.3 Solution Method

The design of the two-way slab system will be based on Chapter 13 of ACI 318-11 and Chapter 13 of Reinforced Concrete Mechanics and Design (Fourth Edition). Computer analysis of two-way flat slab system will utilize RAM Concept. Trial sizes, as outlined above will be input into the computer program. The design of reinforced concrete moment frames will utilize portal analysis and computer program ETABS2015. Throughout the gravity and lateral design process, notes from concrete design classes (AE 402 & AE 431) and other architectural engineering courses will be used as a reference. AE faculty members with relevant expertise will also be a resource for the redesign and historic preservation of the building.

5.4 Breadth Topics

5.4.1 Historic Façade Preservation (Facadism)

The construction of temporary structure to support the historic façade will be conducted in this breath. In order to preserve the façade of the building, the critical path of construction will be altered. The method and material of the temporary structure will also be discussed. Due to the new critical path of construction – in addition to the new cost of material and labor - might affect the overall project cost. Cost and schedule analysis will be used to determine the feasibility of the proposed project.

5.4.2 Indoor Air Quality

The existing three-story building is stretched upwards, creating a total of five-story high, 9612 SF new retail building at the corner of 706 Madison Avenue. Its location and added area give the building a higher ventilation requirement. To meet the minimum indoor air quality requirement, ASHRAE standard 62.1 will be used to calculate total outdoor cfm that must be provided by mechanical systems.

5.5 MAE Coursework

The redesign of the gravity force resisting and lateral force resisting systems of the proposed concrete structure will require the execution of three-dimensional modeling. The three-dimensional model will be constructed in Etabs, which have been learned from AE 530 – "Computer Modeling of Building Structures." SAP will also be utilized to verify a two-dimensional structures. Modeling the building in three dimensions will provide a greater understanding of building behavior and the outputs from it can be utilized to verify manual calculation. Additionally, coursework from AE 538: Earthquake Resistant Design of building will be used to provide seismic reinforcing detailing for the concrete moment frames.

5.6 Tasks and Tools

1. Research Phase

- Research modeling approach for design of concrete two-way slab
- Research modeling approach for design of concrete moment frame
- Research necessary governing code, references, standards, design guides, etc.
- Research the feasibility of reconstruction of historic buildings
- Research temporary structures to support historical building façade
- Research mechanical properties (thermal, moisture, etc.) of façade alternatives
- Research structural properties (earthquake, etc.) of façade alternatives
- Research architectural context of building site
- Research integrated design approaches

2. Structural Depth | Concrete Redesign

- Gravity Force Resisting System Design
  - i. Design
    1. Identify new gravity loading conditions based on ASCE 7 - 10
    2. Design two-way concrete slab system based on Chapter 13 of ACI 318-11
    3. Design primary beam members based on Chapter 5 of ACI 318-11
    4. Design primary girder structure members
    5. Design columns based on Chapter 11 of ACI 318-11
  - ii. Model
    1. Verify design RAM Concept
    2. Develop three-dimensional model in ETABS

- Lateral Force Resisting System Design
  - i. Design & Model
    1. Calculate new wind and seismic loads based on ASCE 7-10
    2. Define controlling lateral loading condition
    3. Design preliminary concrete moment frames using portal analysis and Chapter 21 of ACI 318-11 as a reference
    4. Analyze wind and seismic loads in ETABS
  - ii. Verification
    1. Validate ETABS model with manual calculations
    2. Verify reinforcing detail with seismic detailing from AE 538

3. Historical Façade Preservation Breath

- Construction Issues
    i. Determine the construction challenges
    ii. Select method and material of temporary structure
    iii. Determine schedule of preservation of façade coordinating with redesign alternative
- Cost Analysis
    i. Cost analysis of façade preservation
- Assess feasibility of redesign based on cost and difficulties

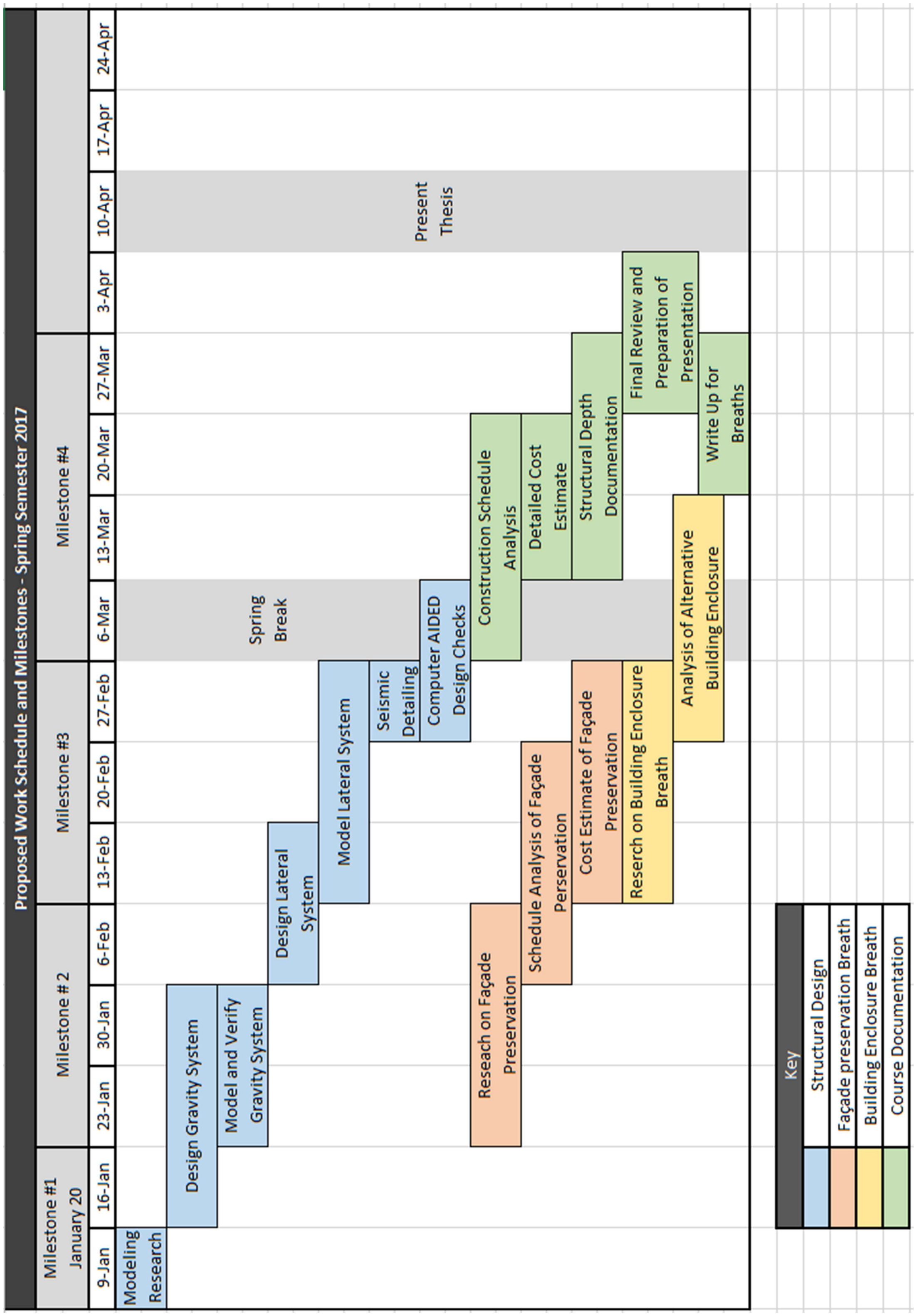
4. Building Enclosure Breath

- Ventilation requirement
    i. Determine ventilation requirements for the new building
- Ventilation Rate Calculation
    i. Calculate ventilation rates for the new building

5. Documentation

- Outline final report for BAM/MAE requirements
- Generate template for final presentation submission
- Complete final report document
- Complete final presentation file
- Update final documents on CPEP website
- Submit and present final documentation to jury

5.7 Schedule


Proposed Work Schedule and Milestones - Spring Semester 2017
Milestone #1 January 20
Milestone # 2
Milestone #3
Milestone #4
9-Jan
16-Jan
23-Jan
30-Jan
6-Feb
13-Feb
20-Feb
27-Feb
6-Mar
13-Mar
20-Mar
27-Mar
3-Apr
10-Apr
17-Apr
24-Apr
Modeling Research
Design Gravity System
Model and Verify Gravity System
Design Lateral System
Model Lateral System
Seismic Detailing
Computer AIDED Design Checks
Spring Break
Reseach on Façade Preservation
Schedule Analysis of Façade Perservation
Cost Estimate of Façade Preservation
Reserch on Building Enclosure Breath
Analysis of Alternative Building Enclosure
Construction Schedule Analysis
Detailed Cost Estimate
Structural Depth Documentation
Final Review and Preparation of Presentation
Write Up for Breaths
Present Thesis
Key
Structural Design
Façade preservation Breath
Building Enclosure Breath
Course Documentation

[6] Structural Depth | Concrete Redesign

6.1 Gravity Design

6.1.1 New Design Layout

The gravity system for the redesign of 706 Madison Avenue consists of two-way concrete slabs with edge beams and reinforced concrete columns. The new layout of the building has been designed as shown in Figure 15. The typical bay size is 23' x 27'. The elevation of the building does not change and can be reviewed in Figure 2. Snow load does not need to be recalculated since the building is on the same site. Snow load calculation can be found in the previous snow load section. Other loading conditions will be introduced in the next section. The gravity system has been designed in terms of applicable strength and serviceability criteria.

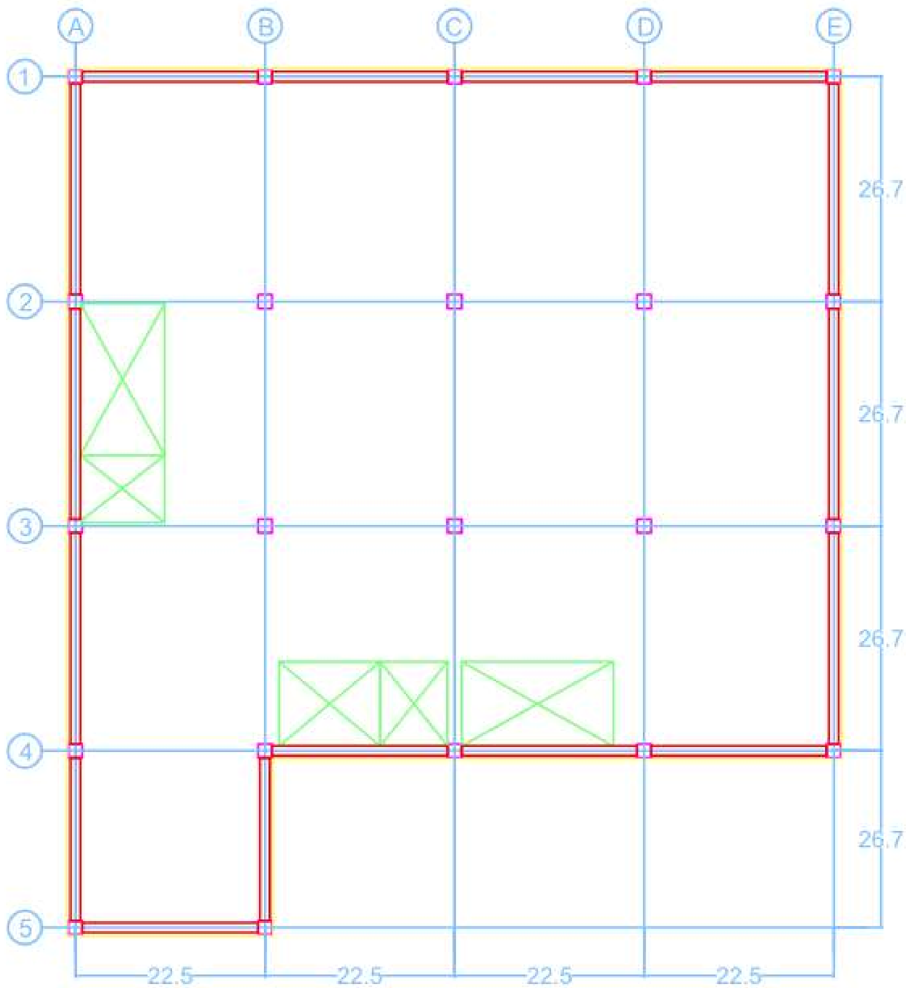


Figure 15 – New Design Layout

6.1.2 Gravity Loading

The gravity loading is shown in the Appendix 1.1, where roof loading, typical floor loading and wall loading are introduced. The load patterns consist of dead load, live load, superimposed dead load and roof live load. The dead load contains the self-weights of two-way slabs, beams and columns, finishes and the superimposed dead load. Other loads, including live load, roof live load are determined by ASEC 7-10. Based on the calculation from the previous report, the snow load does not control. Therefore, the minimum live loads (20psf) required by ASCE 7-20 is added on the roof for the design.

6.1.3 Gravity Load Path

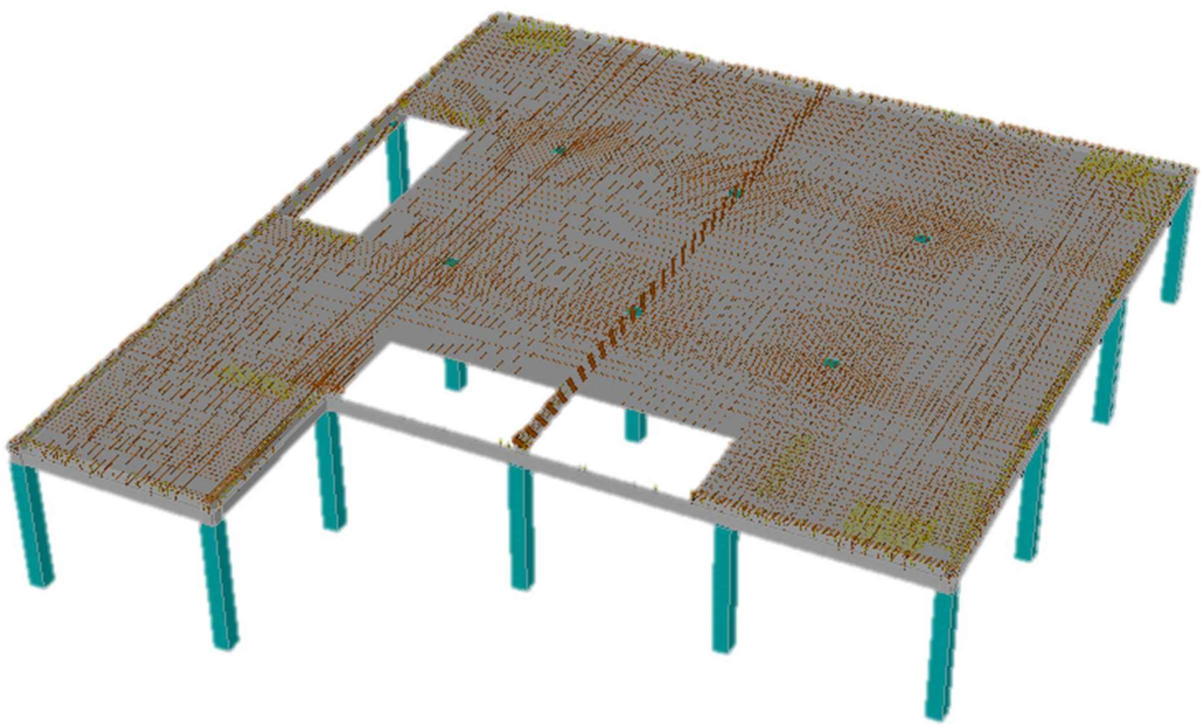
The typical gravity loads described above will be resisted by roof and floor two-way slab system and mostly transferred to the foundation directly through the reinforced concrete columns. Some loads closed to the edge beams are likely to be carried by the edge beams first and then transferred to foundation through exterior columns.

6.1.4 Member Size Estimation

This section explores ways of estimating trial sizes of structural members including the trial sizes of edge beams and columns, and the thickness of the two-way slab. Trial sizes of beams and columns are elected by the approximate structural analysis, which only considers the gravity loads without the lateral loads. The two-way slab thickness is determined by the deflection limit (L/33) in Table 8.3.1.1 ACI318-14. They are roughly selected in the first phase since a computer model then could be built with them, and used to analyze and design the structure.

Based on the calculations in Appendix 1.2, a 10" two-way slab, 14" by 28" beams and 20"by 20" columns are specified. However, they are not really the final design. The beams and columns will be designed in the lateral design section, where the lateral loads applied on the beams and columns are known. It means that the beams and columns will be designed to carry both gravity and lateral loads. The final design will be completed by checking the strength and serviceability of members in all applicable load combinations.

6.1.5 Two-way Slab Design (With Edge Beams)



RAM® Concept

6.1.5.1 Introduction of Two-way slab system

The two way-slab is chosen to be the framing system of 706 Madison Avenue due to several advantages of the system. First of all, it provides a flexibility in room layout since the partition wall can be placed anywhere and false ceiling can be omitted. Secondly, the reinforcement placement and the framework installation is easier. Thirdly, the building height can be reduced because less beams are used. Last, construction time is saved due to the easier reinforcement placement and framework installation.

The approaches of designing and analyzing the two-way slab system contain direct design method and RAM Concept modeling. Two approaches are used to verify and compare the results from each method and ensure the accuracy of the design. The two-way slab system specifies the normal weight concrete and Grade 40 rebar. The details of the design and the validation are showed in the following sections.

6.1.5.2 Direct Design Method

The direct design method is permitted to be used for design of the two-way slab while the building meets all the limitations listed in the chapter 8 of ACI 318-14. As shown is the Appendix 1.2, this building is verified to meet all the requirements. So the direct design method is allowed to use for the design of two-way slab of the building. Although the direct design method is a very approximate approach of design of two-way slab, it’s still an effective and fast way of checking and validating the RAM Concept model.

6.1.5.3 Punching Shear Check and Shear Reinforcement Design

The punching shear failure is one the most critical failure for the two-way slab system. It must be checked to determine if the thickness is adequate for shear or the shear reinforcement is needed. Due to the uniformity of the building, only two critical shear sections will need to be checked: the shear sections around interior columns (B2, C2, D2, B3, C3, and D3), and the shear sections around exterior columns (other columns). From Appendix 1.3, the calculations indicates that the interior columns don’t need any shear reinforcement and the exterior columns require #3 bars (2 branches) at 4”O.C. to prevent a shear failure. The shear reinforcement placement at exterior column E2 are showed in the figure 16.

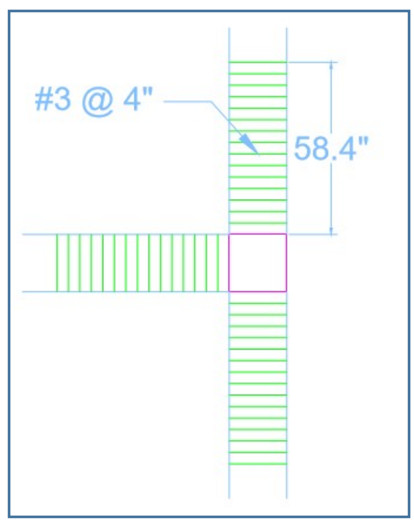


Figure 16 – Exterior Column Shear Reinforcement

6.1.5.4 Model Validation for Shear

The shear reinforcement designed by RAM Concept matches the results above. As shown in the Figure 17, the interior columns don't have any shear reinforcement around since by the calculation above 10" concrete slab is able to carry the punching shear force without shear reinforcement. It's also easier to see the shear reinforcement around the exterior columns. The shear reinforcement around the column E2 is #4 bars @ 6.85" O.C., which is 0.7 in^2/ft. The shear reinforcement calculated manually is #3 bars @ 4" O.C, which is 0.66 in^2/ft. The error of required shear enforcement is in 1%. So the RAM Concept model is validated.

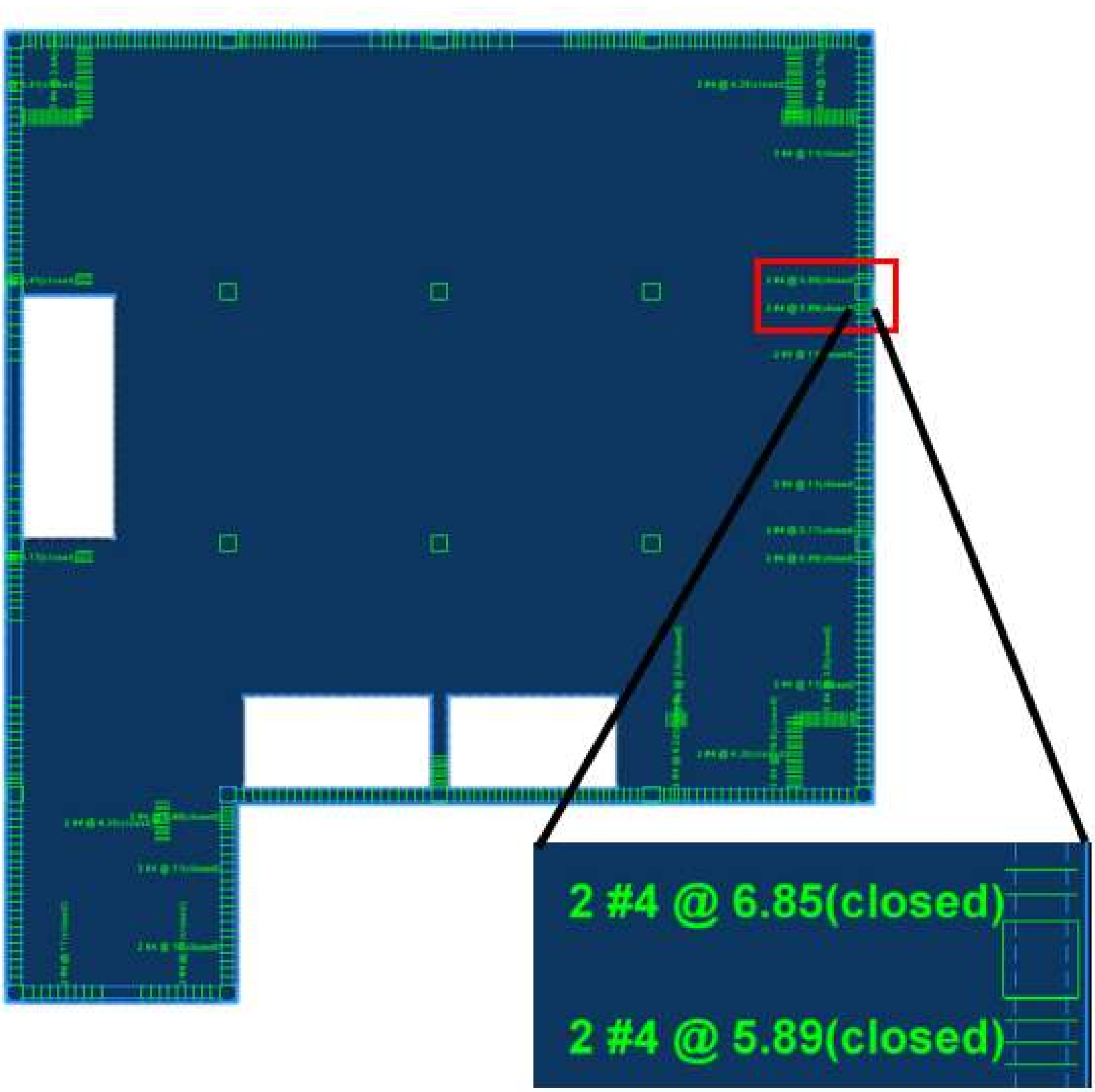


Figure 17 – RAM Concept Shear Reinforcement Plan (RAM Concept)

6.1.5.5 Flexural Reinforcement Design (DDM)

As shown in figure 18, the green area is chose for the flexural reinforcement design in east-west direction. The total applied moments are distributed in both column strips and middle strips with a certain ratio. The column strips will take more moments since they are stiffer than the middle strips. Especially the column strip at line E will take the significant portion of the total moment at the edge due to the stiffness of the edge beam. All the moments are calculated following the direct design method and values are showed in the figure 18. The flexural reinforcement will be determined by the applied moments in the slab and designed to meet both the strength and serviceability requirements.

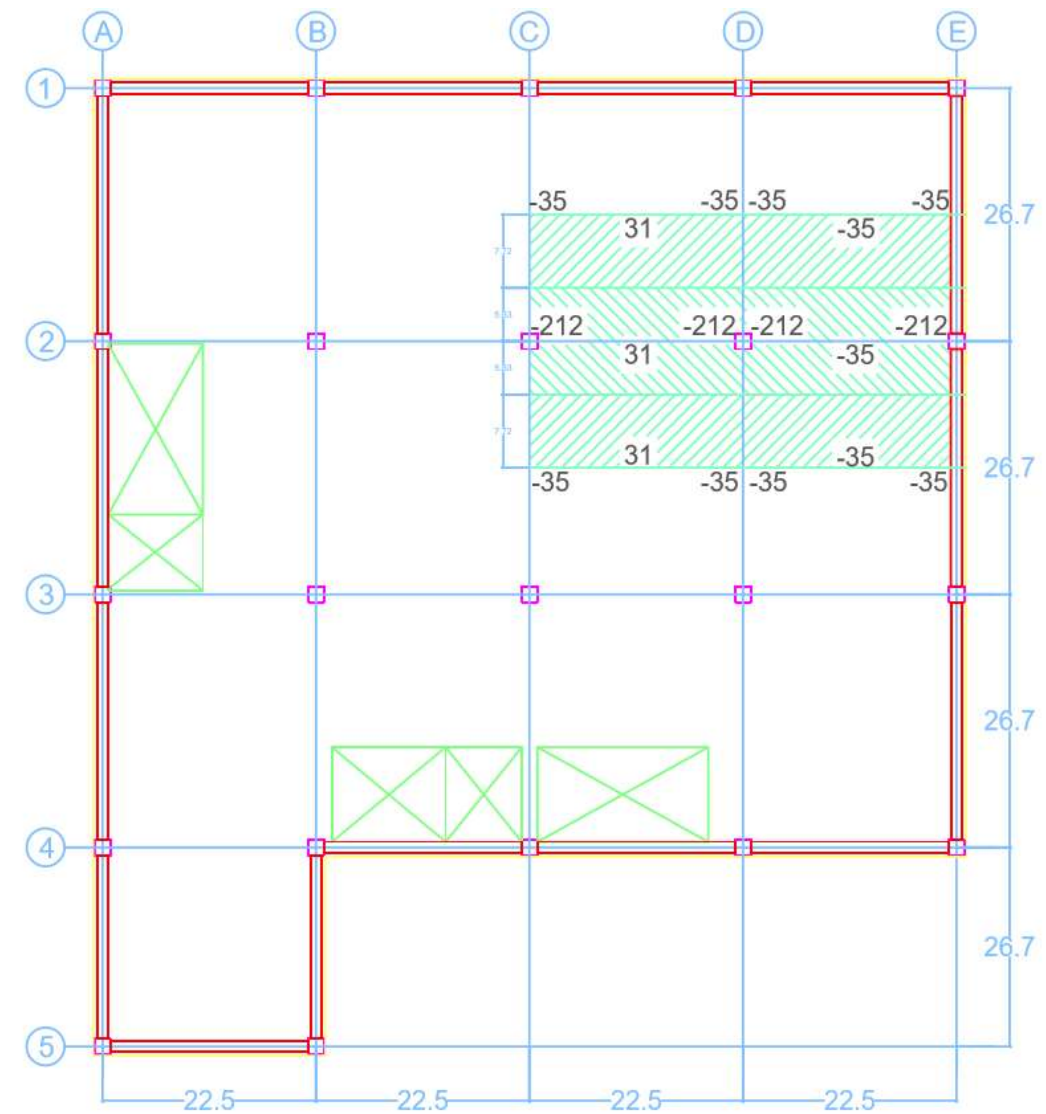


Figure 19 – Column & Middle Strip Moments in E-S Direction (DDM)

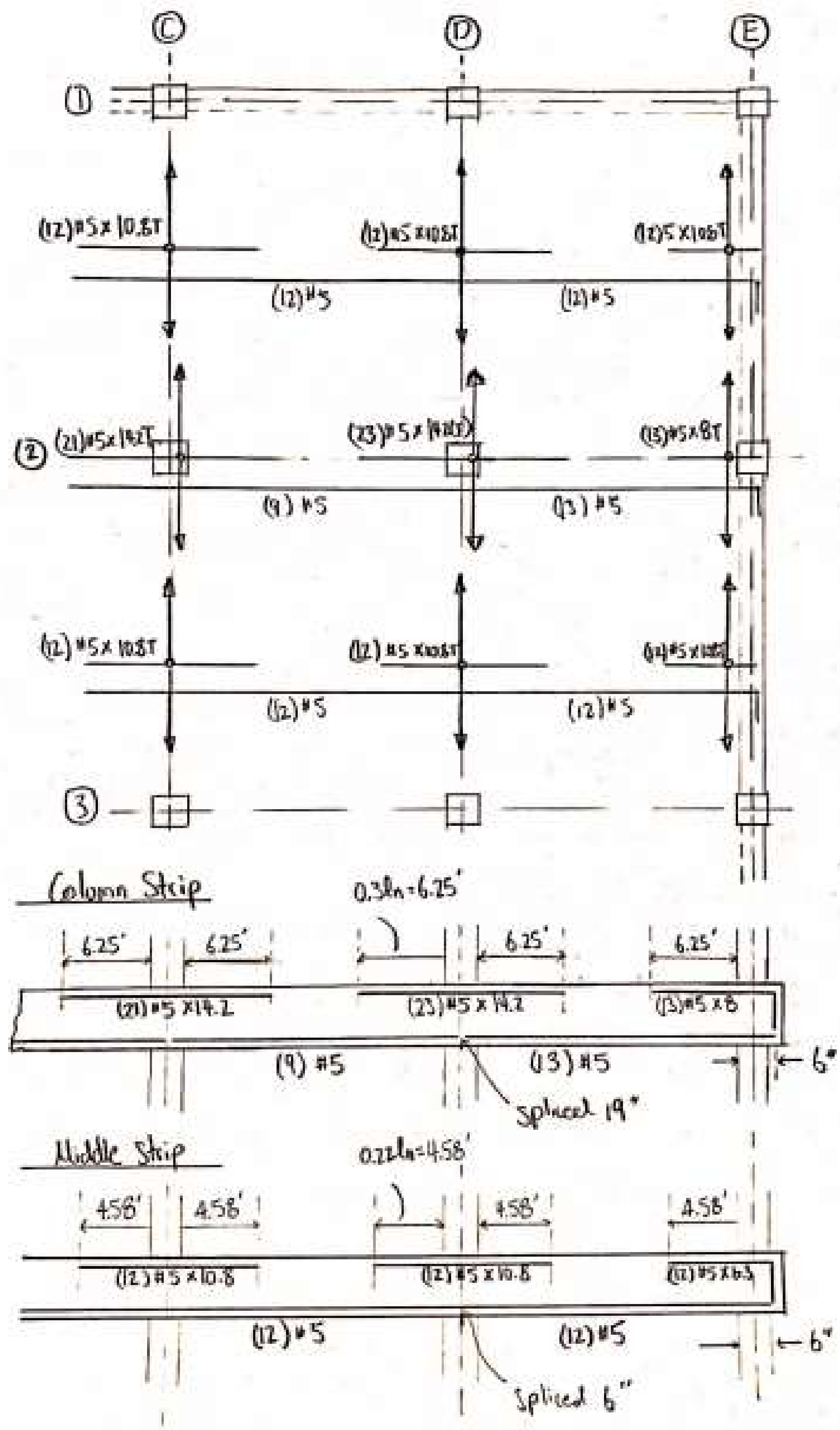


Figure 20 – Flexural Reinforcement Placement and detailing (DDM)

As shown in the figure 20, the #5 Grade 40 rebar is specified for the flexural reinforcement design over the whole slab. The picture above exhibits the placement of the flexural reinforcement and the picture below shows the reinforcement detailing in the column strip and middle strip. Based on the calculation in Appendix 1.3, more reinforcement is required at the top of slab to resist the negative moments at the columns ends, and less reinforcement is required at the bottom of slab to resist the positive moment at mid-spans. In addition, the middle strips have less reinforcement than the column strips, verifying that the column strips carry more loads than the middle strips. The development lengths of the reinforcement steel are determined by the figure 8.7.4.1.3a ACI318-14.

6.1.5.6 Model Validation for Flexural

- Column and Middle Strip moments

The moment demand has been calculated by RAM Concept as shown in the figure 21 and compared to the values from direct design method in Table 6. Two moment curves (green and red) are showing the maximum demand and minimum demand due to different load combinations. The critical moments will be elected for the following comparison.

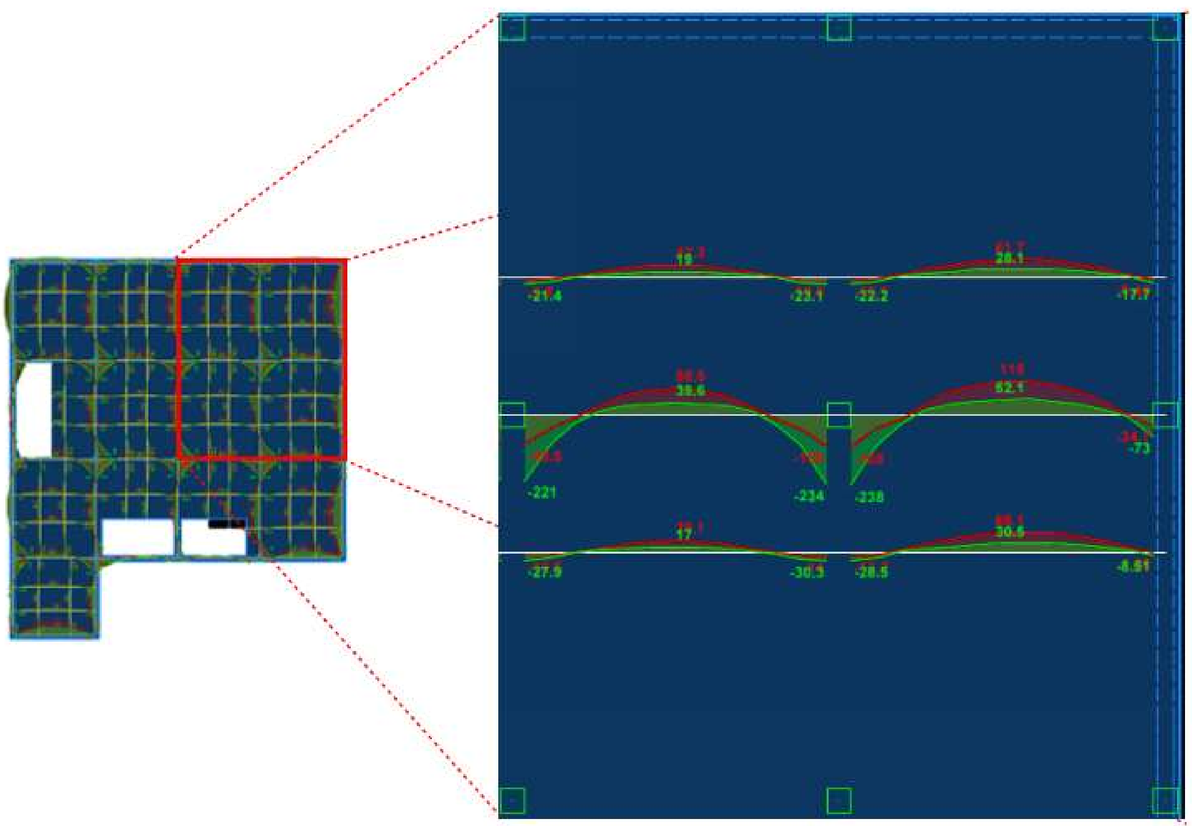


Figure 21 – Moment Demands of Column and Middle Strips (RAM)

Strip Moment Comparison										
	Column & Middle Strip Moments									
	C (-)		C-D (+)		D (-)		D-E (+)		E (-)	
	Col. Strip	Mid. Strip	Col. Strip	Mid. Strip	Col. Strip	Mid. Strip	Col. Strip	Mid. Strip	Col. Strip	Mid. Strip
DDM	212	70	91	62	228	70	130	88	125	5.2
RAM Concept	221	50	88.5	80	234	51	115	130	73	26.2
Error %	4.07	29	1.13	22.5	2.56	27.1	11.5	32.3	41.6	80
	Total Strip Moments									
DDM	282		153		298		218		130.2	
RAM Concept	271		168.5		285		245		99.2	
Error %	3.9		9.2		4.4		0.1		23.8	

Table 6 – Strip Moment Comparison

From the table above, it's found that the percentage errors of the column and middle strip moments between two approaches are relatively large. However, the percentage errors of the total strip moments are smaller. The percentage error of the total strip moment are less than 10%, which is acceptable. The phenomenon indicates that the amount of total strip moment distributed to column and middle strips between two approaches are different.

In the direct design method, the negative moments are distributed to the column and middle strips with approximate factors 0.75 and 0.25, and the positive moments are distributed to the column and middle strips with factors 0.6 and 0.4. However, the RAM program distributes the moments without the certain factors. The total moments would be distributed based on the real stiffness ratio of the structure. Since the upper part of the bay contains a parallel edge beam which is much stiffer than the slab. It could also affect the moment distribution in that area.

Because the total moment calculated by two approaches are very close. The RAM Concept model could still be validated.

- Flexural Reinforcement

Comparing the flexural reinforcement plan between the two approaches, the reinforcement placements are very similar. As shown in Figure 22, RAM model specifies 19 #5 bars at the top of the slab at the column C2, 21 #5 bars at the column D2 and 8 #5 bars at the column E2. The direct design method determines the reinforcement at the same places with the number of 21, 23, and 13. The direct design method specifies a bit more reinforcement, which is acceptable.

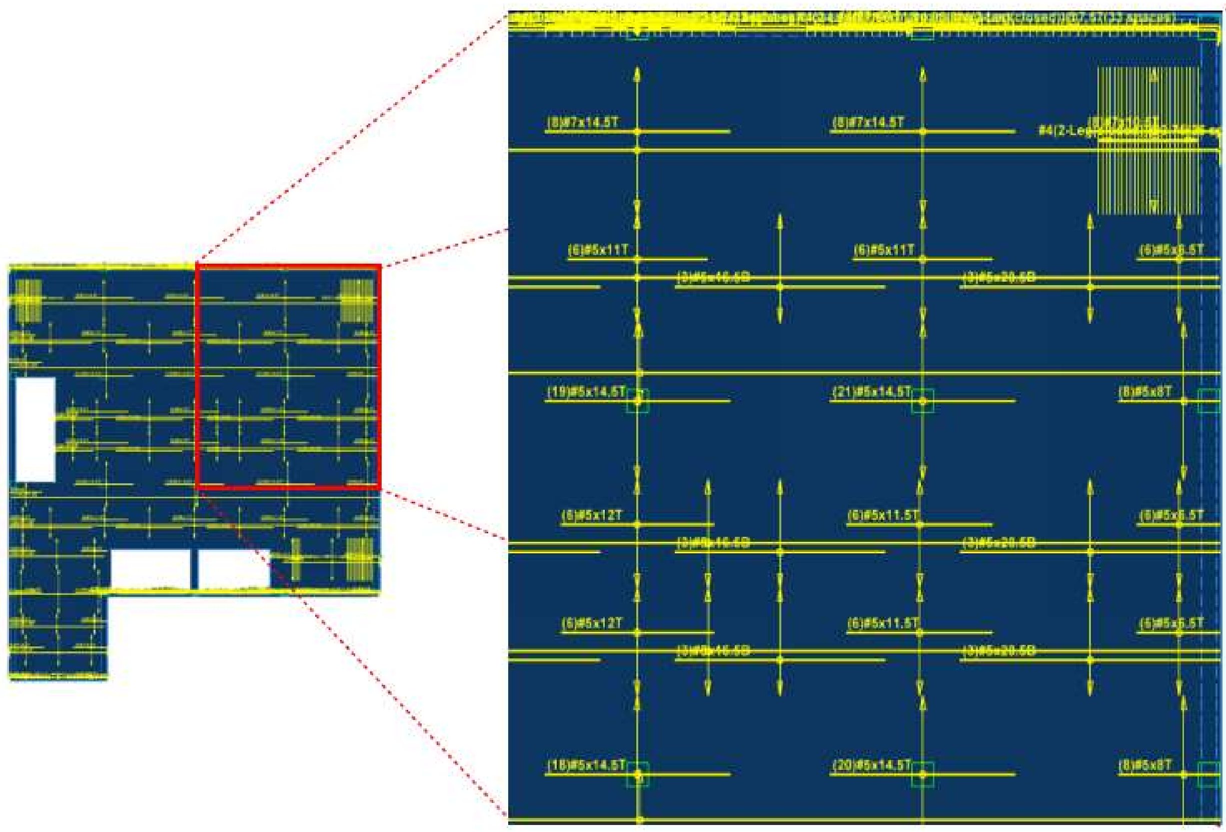


Figure 22 – Flexural Reinforcement Plan (RAM Concept)

6.1.5.7 Deflection of Two-way Slab

Because of the lack of the infill beams, the deflection of the two-way slab could be extremely large. It's necessary to design the two-way slab to meet the serviceability requirement. From Table 9.5 (b) ACI318-11, the deflection limitation of two-way slab not supporting or attached to nonstructural elements not likely to be damaged by large deflection is L/360. As shown in figure 23, the largest deflection along the column line 2 is 0.88". The following calculation presents that the deflection of the two-way slab meets the ACI code requirements.

L/360 = (22.5*12)/360 =1.125" > 0.88" So, the deflection is Ok.

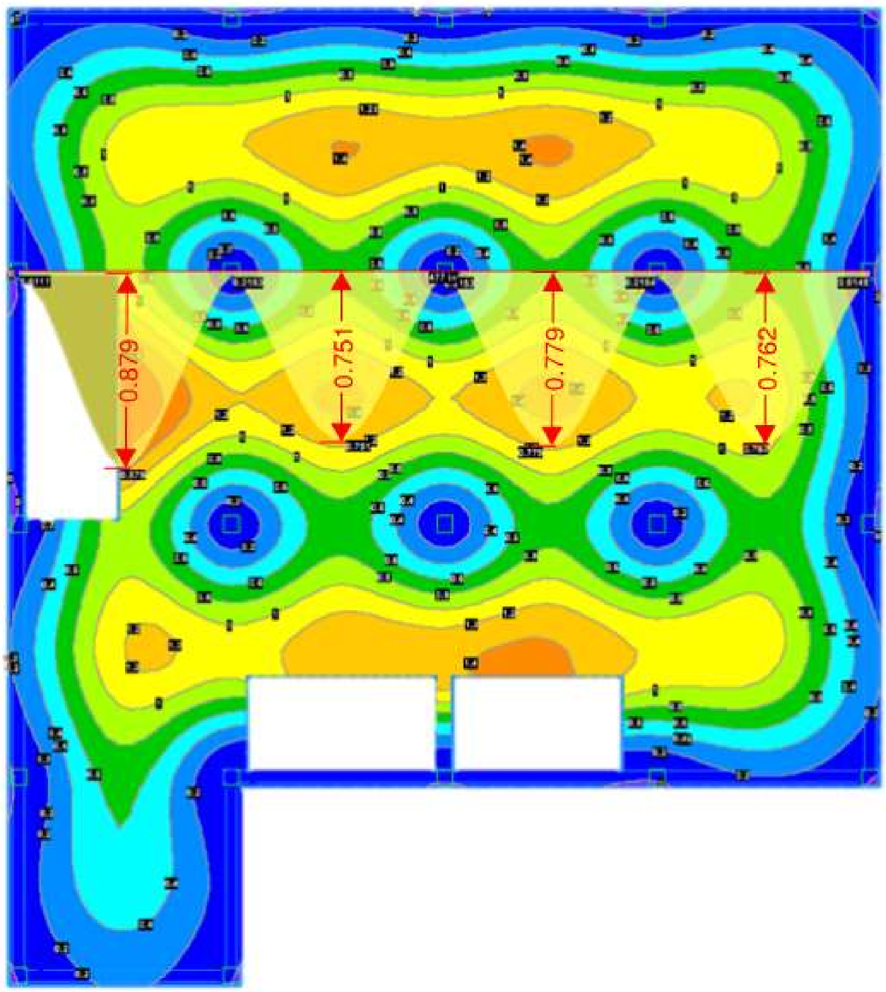


Figure 23 – Deflection of Two-way Slab (RAM Concept)

6.1.6 Gravity Column Design

All the interior columns are considered as gravity columns to resist only gravity loads. Although a portion of the unbalanced moment is distributed to the slabs, and the rest goes to the columns. The unbalanced moment calculated by the equation 13-7 ACI 318-1 is very small and doesn't really affect the column design. Therefore, the interior columns will be designed to carry the axial load. The axial reinforcement, ties, and splices will also be designed. The whole design process and calculations of are presented in Appendix 1.4.

As shown in figure 24, column C2 at ground floor has been designed to resist the gravity loads. The final design came up with a 20" by 20" concrete column with #4 ties at 18" O.C. The axial reinforcement contains 4 #9 bars, which provides a 4 square inch steel area to resist the axial loads. The splice length has determined to be 53.5".

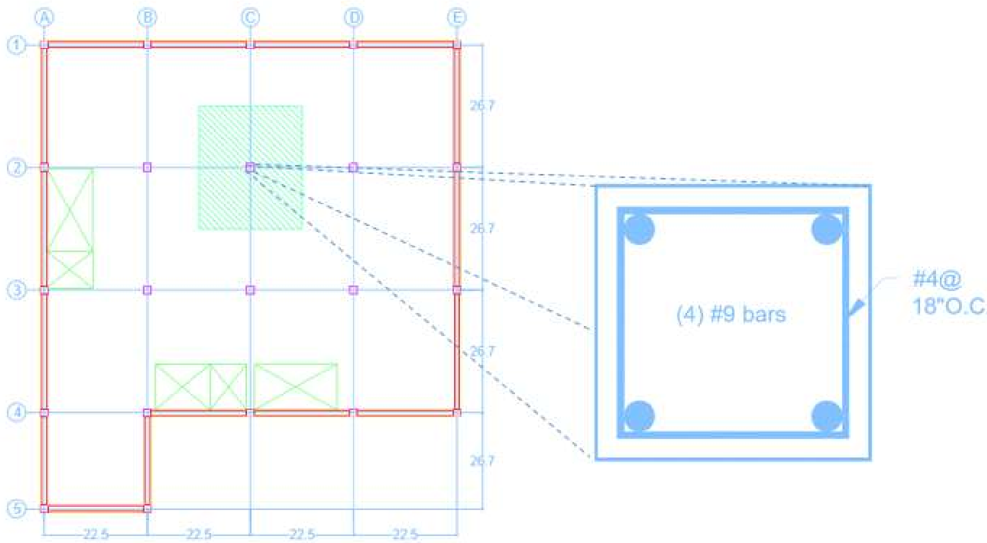


Figure 24 – Gravity Column Design (C2)

The reinforcement design of column C2 are verified by the ETABS model. ETABS specifies a same amount of steel (4in^2) for the column C2, which could be seen in Figure 25. In fact, interior columns are able to resist lateral loads since the two-way slab rigidly connected to the interior columns behaves as a moment frame. In order to decrease the capability of the interior columns to take lateral loads, it's easier to find in Figure 25 that all the interior column bases are pinned to foundation. Therefore, the 4 square inch steel reinforcement are eventually designed to resist gravity loads. The final design is verified.

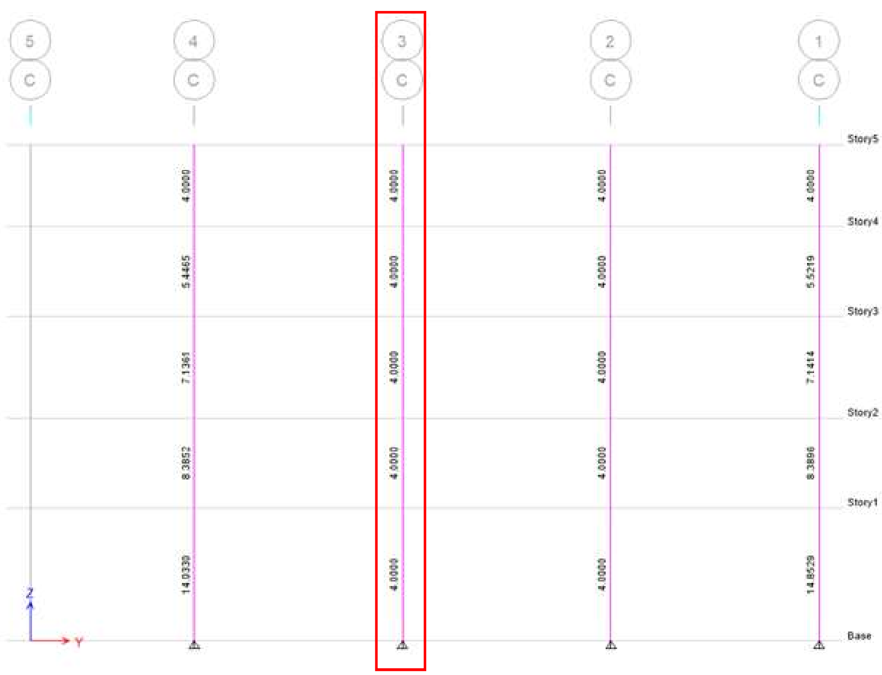
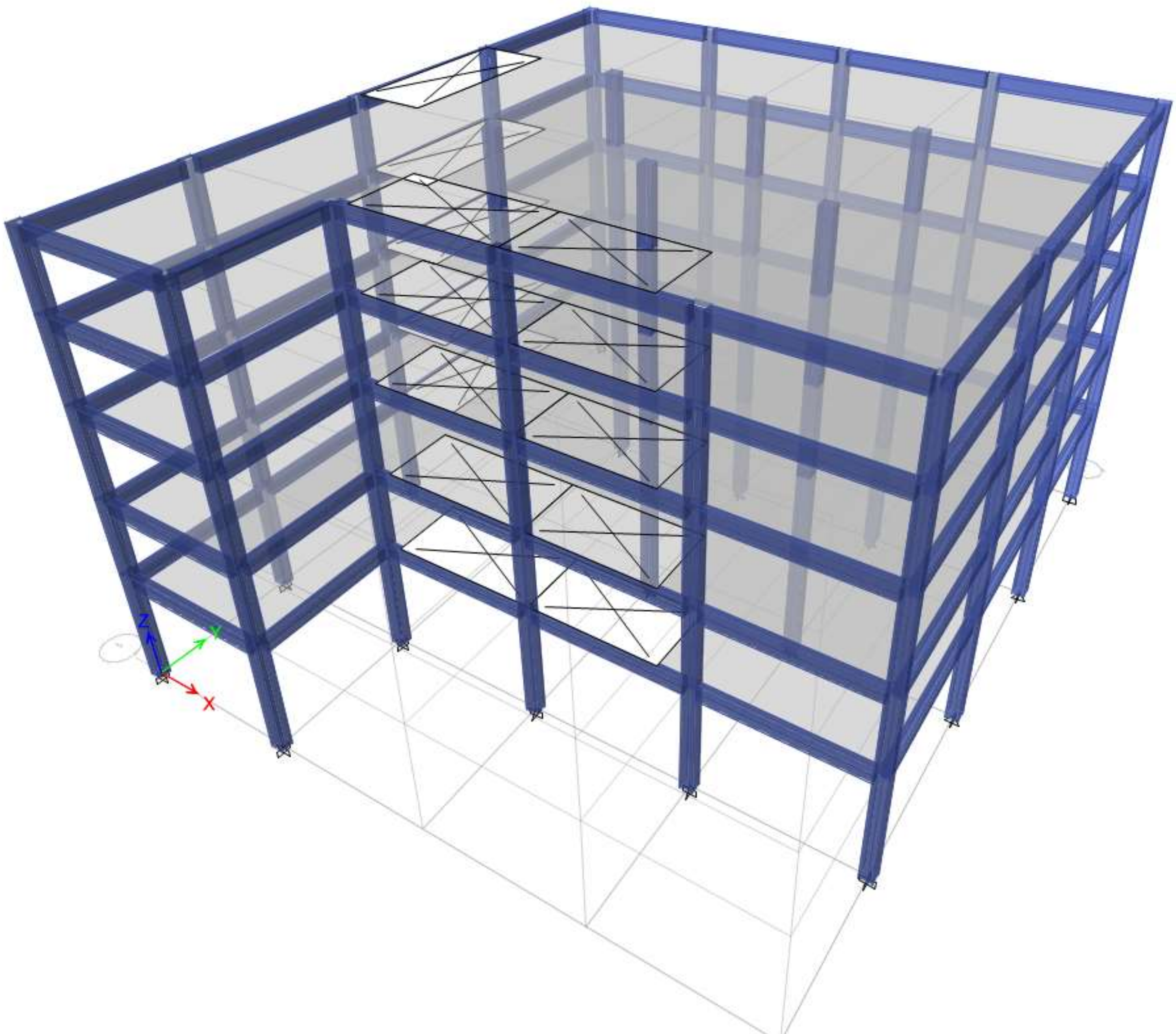


Figure 25 – Gravity Column Design (C2)

6.2 Lateral Depth



ETABS

6.2.1 New Lateral Load-Resisting System

The concrete moment-resisting frame is introduced as a new lateral load-resisting system for 706 Madison Avenue. The moment frame is an assemblage of edge beams and columns, with the beams rigidly connected to the columns, and is located at the perimeter of the building. As shown in figure 26, the moment frame 1, 2 and 3 are designed to resist the lateral loads in Y direction and the moment frame 4, 5 and 6 are designed to resist the lateral loads in X direction. In addition, the moment frames are also parts of the gravity system. So they must be finally designed to carry both gravity loads and lateral loads, with applicable load combinations.

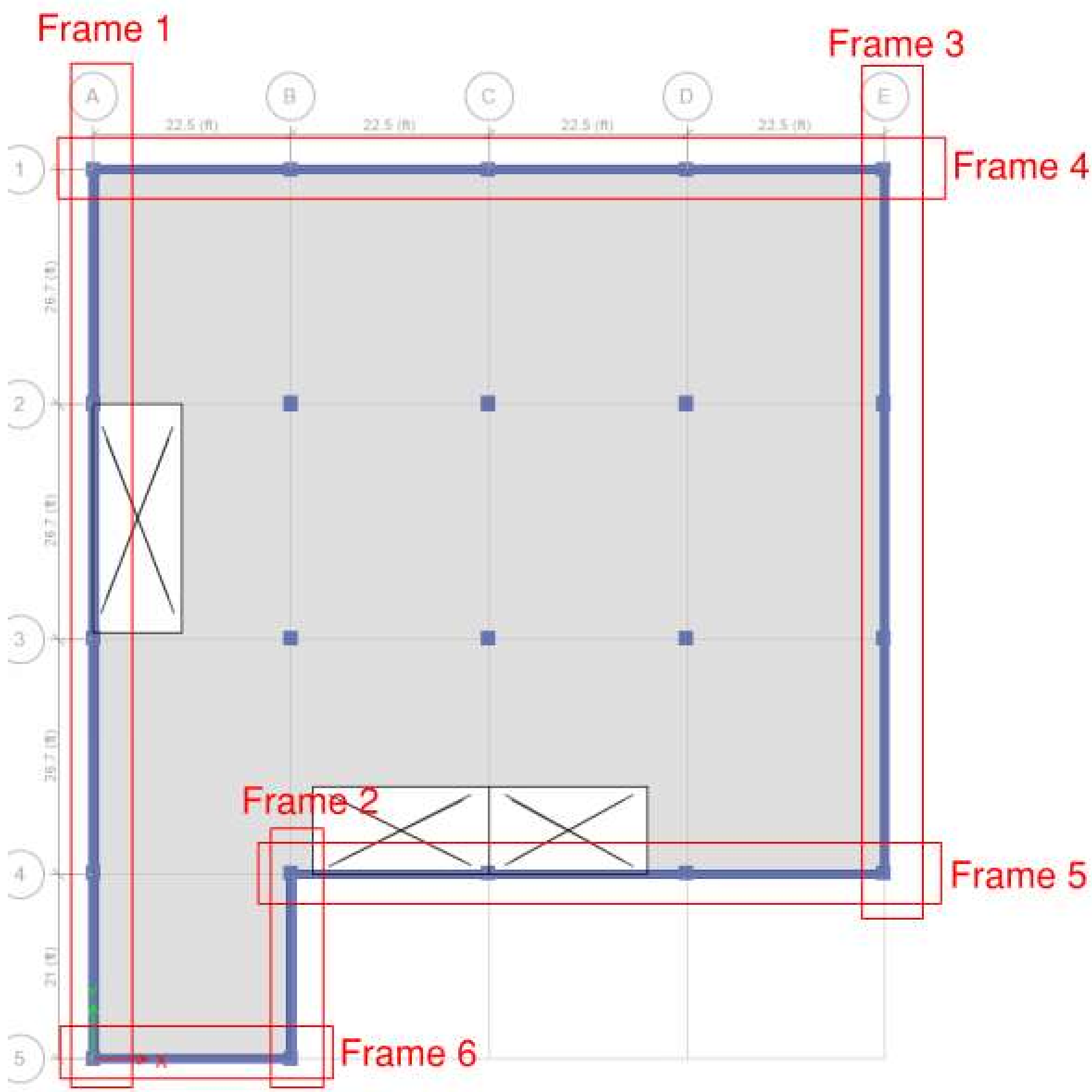


Figure 26 – Moment Frame Layout

6.2.2 Lateral Loading

Wind loads and seismic loads are considered as two most critical lateral loads when designing the lateral load-resisting members. They must be recalculated since the new design changed the building's material and layout. The applied wind and seismic loads are calculated manually following ASCE 7-10 and compared to the ETABS2015 model output. The manual calculations are attached in the Appendix 2. The ETABS output and the comparison are explored in the following section.

6.2.2.1 Wind Loading & Verification

According to ASCE 7-10, the wind load could be determined using Directional Procedure. Wind speed has been determined by ATC. Based on the frequency calculation (na>1Hz), the building is considered rigid, so a Gust-effect factor G = 0.85 could be used. The wind forces determined by ETABS has been shown in figure 27, and the wind load comparison between two approaches has been made in Table 7. From Table 7, it's easy to find that the errors of wind loading calculated between manual calculations and ETABS output are less than3 %. So the wind load calculations are verified.

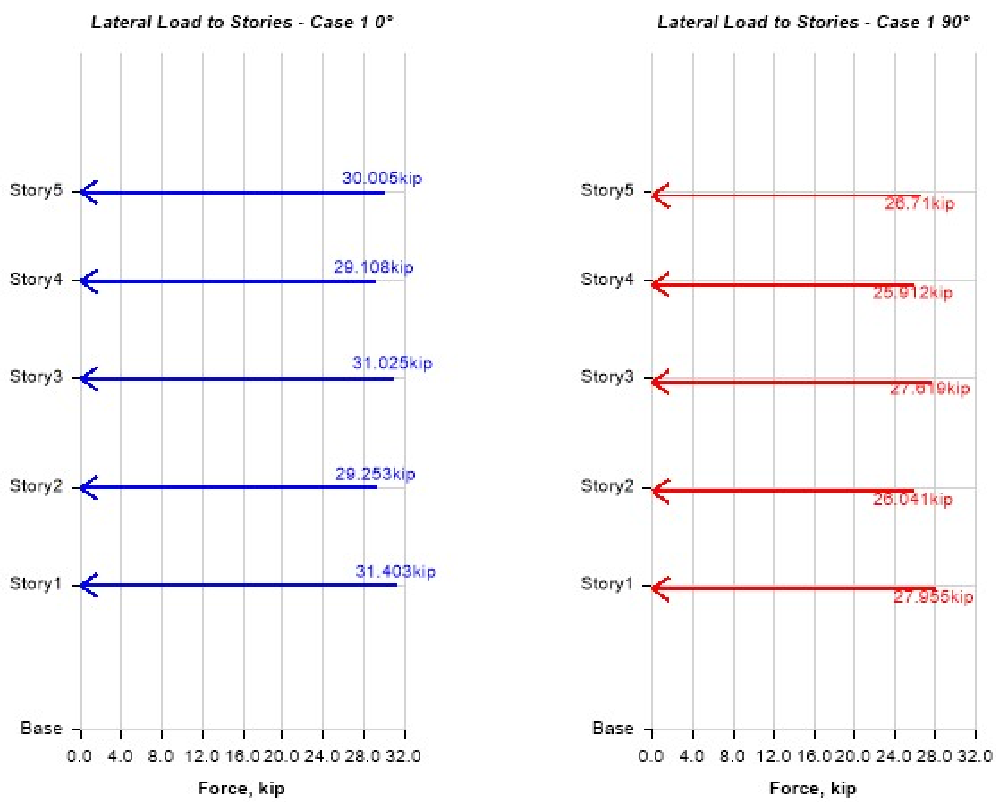


Figure 27 – Wind Load to Stories (ETABS Report)

Wind Load Comparison in X-direction						
	Manual Calculation (Notebook A)				ETAB2015 Model	
Story	Wind Load (psf)	Story Height (ft)	Dimension (ft)	Wind Load (k)	Wind Load (k)	% of Error
Parapet	63.15	2.5	100	15.79	30.01	-1.66
Roof	28.03	5.25	100	14.72		
4	27.17	11	100	29.89	29.11	-2.68
3	26.08	12.25	100	31.95	31.03	-2.98
2	24.55	12.25	100	30.07	29.25	-2.81
1	22.7	14.25	100	32.35	31.41	2.91

Wind Load Comparison in Y-direction						
	Manual Calculation (Notebook A)				ETAB2015 Model	
Story	Wind Load (psf)	Story Height (ft)	Dimension (ft)	Wind Load (k)	Wind Load (k)	% of Error
Parapet	63.15	2.5	90	14.21	26.71	-1.19
Roof	27.13	5.25	90	12.82		
4	26.27	11	90	26.01	25.91	0.37
3	25.18	12.25	90	27.76	27.52	0.87
2	23.65	12.25	90	26.07	26.04	0.13
1	21.8	14.25	90	27.96	27.96	-0.01

Table 7 – Wind Load Comparison

6.2.2.2 Seismic Loading & Verification

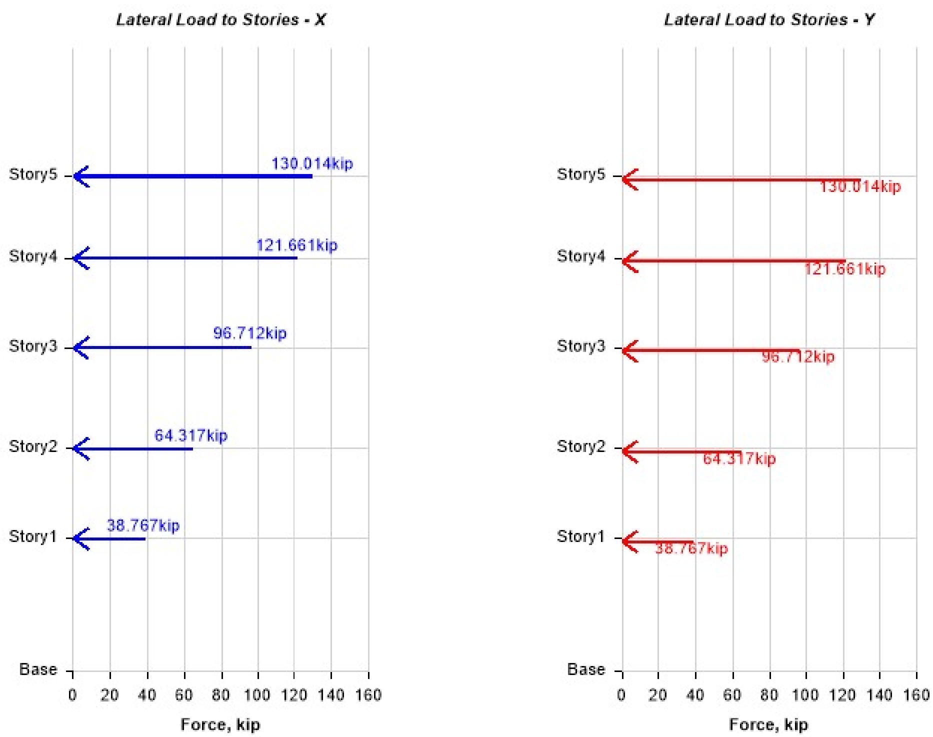


Figure 28 – Seismic Load to Stories (ETABS Report)

According to ASCE 7-10, a minimum weight of 10 psf partition load needs to be included in the calculations. Snow load is not considered when calculating seismic weight since the flat roof snow, Pf, doesn't exceeds 30 psf. The seismic forces determined by ETABS has been shown in figure 28. And the seismic load comparison between two approaches has been made in Table 8. By comparing the seismic loading between hand calculations and ETABS output (errors not exceed 5%), it's able to contend that the seismic calculation is fairly correct.

Seismic Load Comparison (X & Y direction)			
	Manual Calculation	ETAB Model	
Story	Seismic Load (k)	Seismic Load (k)	% of Error
Roof	133.1	130	-2.38
4	118.8	121.7	2.38
3	93.8	96.7	3.00
2	62.4	64.3	2.95
1	37.2	38.8	4.12
Base Shear	445.3	451.5	1.37

Table 8 – Seismic Load Comparison

6.2.2.3 Lateral Load Path

Lateral load path has been introduced in section 2.6. In the new design, the two-way flat slab system serves as a horizontal diaphragm that distributes the lateral wind and seismic forces to the lateral elements, which are concrete moment frames. Moment frames then carry the applied lateral loads to the building foundation. The foundation is designed to resist uplift resulting from the overturning moments caused by lateral loads.

6.2.3 Lateral Design:

Based on the wind and seismic load calculation above, it's able to conclude that the seismic loads control over wind loads for the lateral design. So the load combinations 1.2D+0.5L+1.0E+0.2S and 1.2D+1.6L+0.5Lr will be adopted for the design and analysis of the moment frames. Portal analysis is used to estimate the forces and moments in members. The ETABS model provides a detailed design of the structure, which will be validated by the portal analysis. The moment frames will be designed to meet axial, flexural, shear, and serviceability requirements with the most critical load combination. All calculations of portal analysis are listed in the Appendix 2.3.

6.2.3.1 Introduction of Portal Analysis

A particular benefit of the portal methods is the ability to quickly estimate member forces for subsequent selection of preliminary member sizes of the structure. Once the preliminary member sizes are selected, an exact structural analysis can proceed. Several assumptions are made when using the portal analysis:

- All frame member joints are rigid
- All lateral loads are applied at joints.
- Each column is deformed under load so that PI occurs at midheight
- Each girder is deformed under load so that a PI occurs at midspan
- At each level, the interior columns may be considered to resist twice as much shear compared to the exterior columns.

6.2.3.2 Seismic Load Distribution

In order to obtain final designs of the moment frames, the forces and moments due to lateral loads must be determined. Before using portal analysis to determine the forces and moments in the frames, it's required to know how much lateral forces are resisted by each frame.

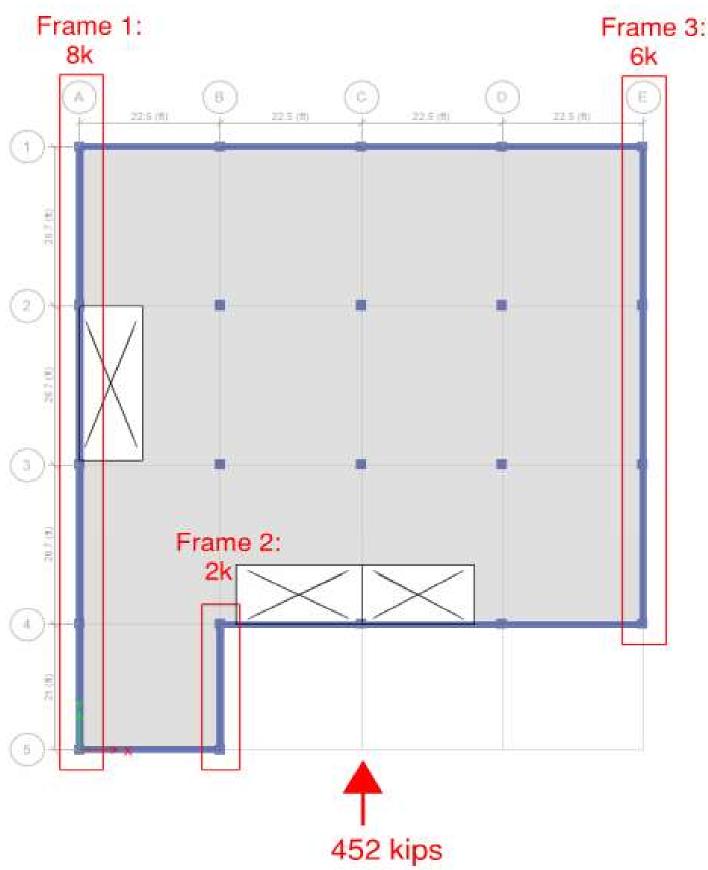


Figure 29 – Seismic Load Distribution Plan

As shown in figure 29, total seismic shear (452k) in N-S direction is resisted by three moment frames. Because of the uniformity of the structure, frame 1 could be considered to have 8 stiffness units. Frame 2 has 2 stiffness units and frame 2 has 6 stiffness units. The total seismic load will be distributed to the frames based on their relative stiffness. The distributed forces are presented in Table 9 and Figure 30.

Seismic Load Distribution				
Story	Seismic Load (kips)	Frame 1 (8/16)	Frame 2 (2/16)	Frame 3 (6/16)
5	130	65.0	16.3	48.8
4	121.7	60.9	15.2	45.6
3	96.7	48.4	12.1	36.3
2	64.3	32.2	8.0	24.1
1	38.8	19.4	4.9	14.6
	451.5	225.8	56.4	169.3

Table 9 – Seismic Load Distribution

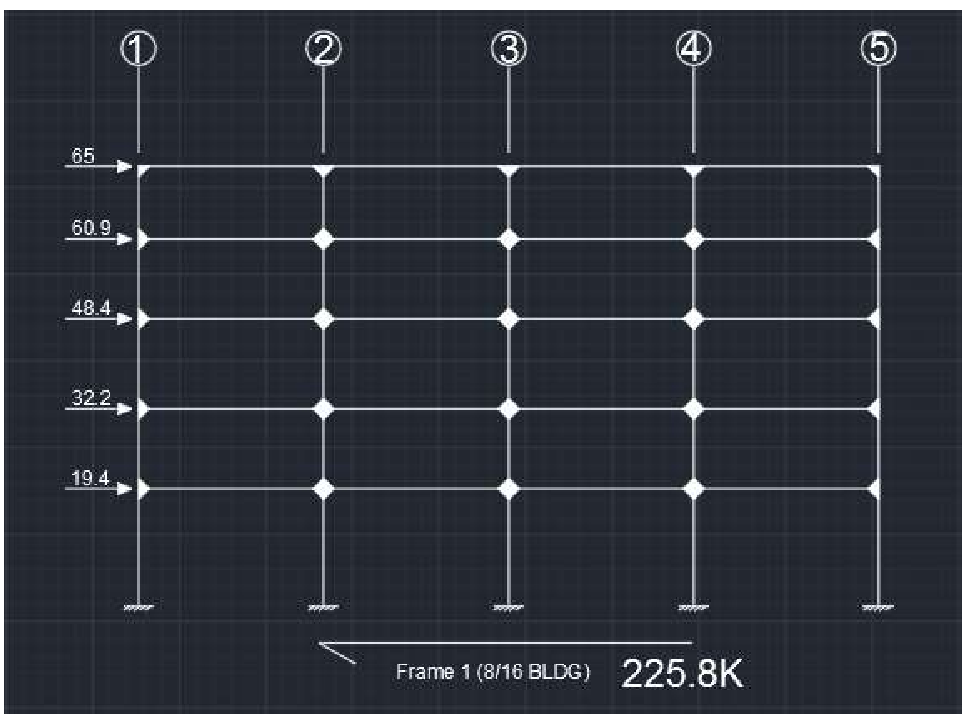


Figure 30 – Seismic Loads to Frame 1

6.2.3.3 Portal Analysis for Frame 1

Base on the analysis above, it's able to find that 225.8 kips seismic shear are distributed in frame 1. It means that the frame 1 will be designed to carry at least 225.8 kips lateral loads. Following the assumptions listed in Section 5.3.1.1, the shear forces carried by columns could be determined and showed in Table 10 and Figure 31. The moments could also be calculated and presented in Table 11 & Figure 32. All calculations including beam shears and moments are presented in Appendix 2.3.

Level	Seismic Force	Shears in Columns				
		1	2	3	4	5
Roof	65	8.13	16.25	16.25	16.25	8.13
5	60.9	15.74	31.48	31.48	31.48	15.74
4	48.4	21.79	43.58	43.58	43.58	21.79
3	32.2	25.81	51.63	51.63	51.63	25.81
2	19.4	28.24	56.48	56.48	56.48	28.24

Table 10 – Column Shear Forces

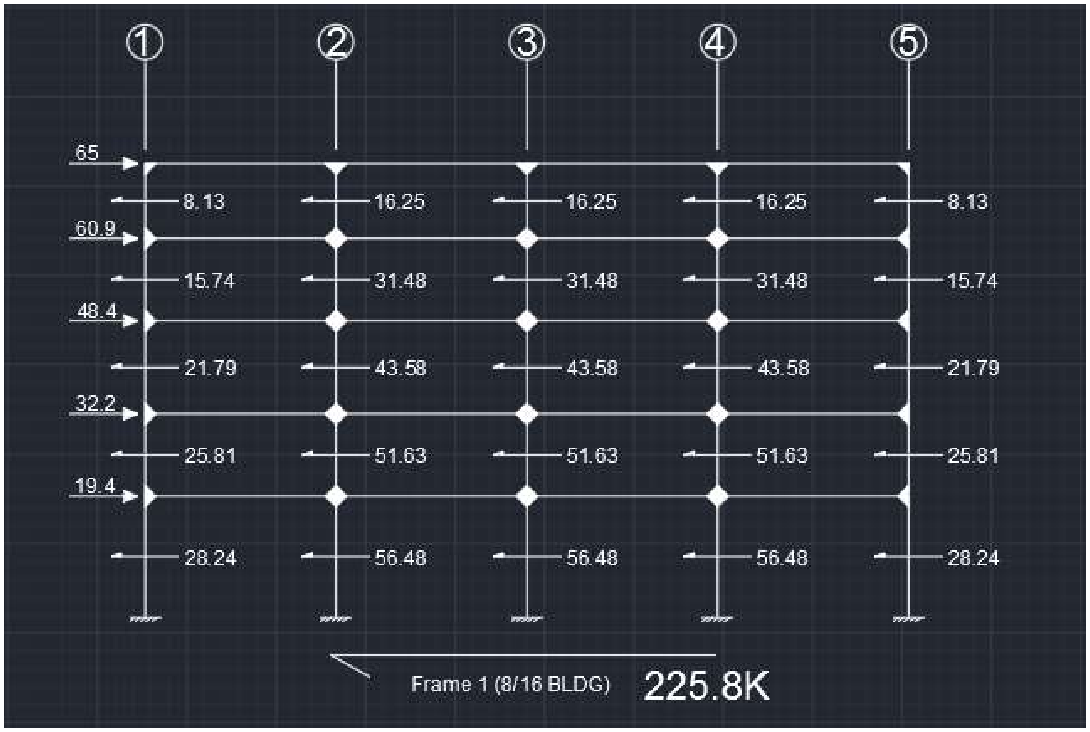


Figure 31 – Column Shear Forces

Level	Story Height	Moments in Columns				
		1	2	3	4	5
Roof	10.5	42.66	85.31	85.31	85.31	42.66
5	11.5	90.49	180.98	180.98	180.98	90.49
4	13	141.62	283.24	283.24	283.24	141.62
3	11.5	148.42	296.84	296.84	296.84	148.42
2	17	240.02	480.04	480.04	480.04	240.02

Table 11 – Column Moments

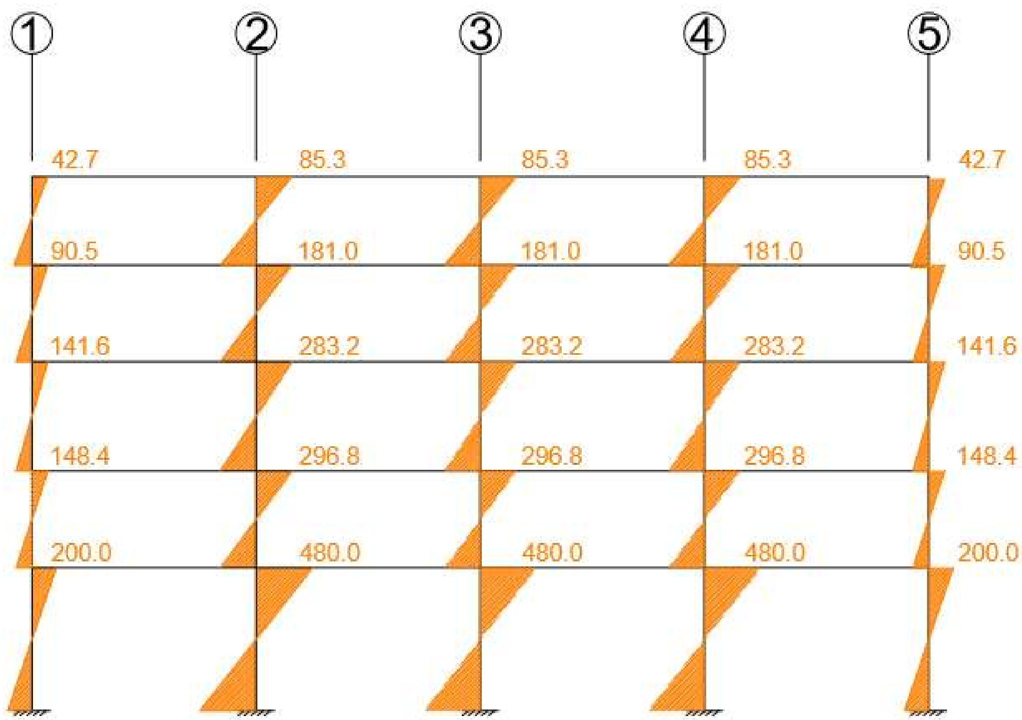


Figure 32 – Column Moments

Note: Only column shears and moments are presented here in order to validate the ETABS model. Shears and moments in beams has also been calculated and showed in Appendix 2.3.

6.2.3.4 ETABS Model Validation

The portal analysis considers that only moment frames are resisting the lateral loads. In fact, the 10" two-way slab rigidly connected columns act as the moment frames to resist lateral loads. In order to validate the ETABS model, the bases of the interior columns are assigned to be pinned-pinned. Therefore, the interior columns are not able to resist any lateral loads and more loads are distributed to the moment frames. The shear forces determined by ETABS are showed in Figure 33 and the comparison of shear force by two approaches are presented in Table 12.

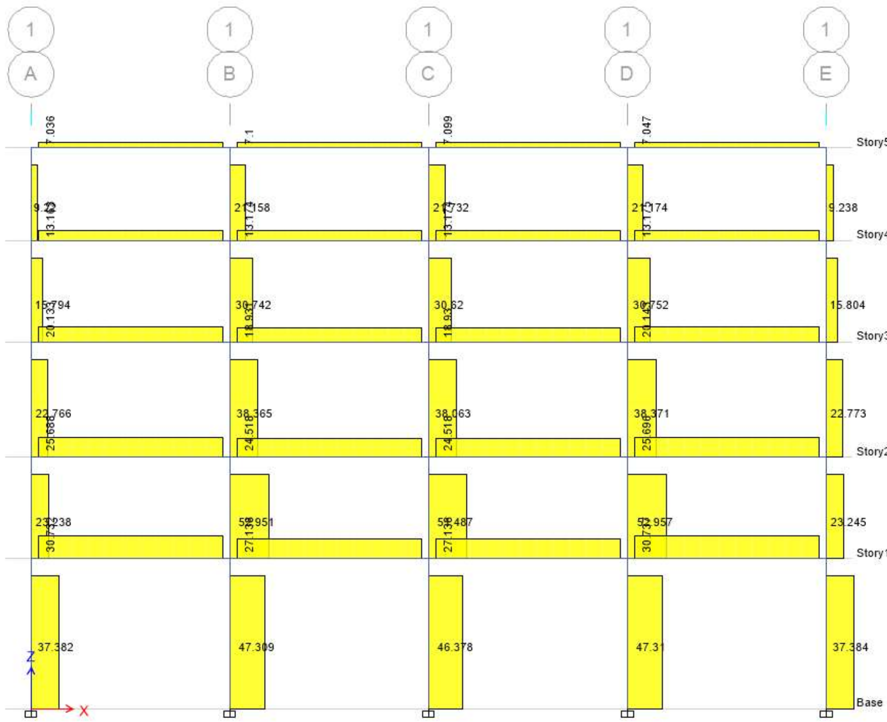


Figure 33 – Shear Force Diagram (ETABS)

Level	Error % of Shear Forces in Clomns				
	1	2	3	4	5
Roof	11.88	23.20	23.20	23.20	11.88
5	0.40	2.34	2.34	2.34	0.40
4	3.98	11.94	11.94	11.94	3.98
3	9.97	2.50	2.50	2.50	9.97
2	24.46	16.23	16.23	16.23	24.46

Table 12 – Colum Shear Comparison

The moments determined by ETABS are showed in Figure 34 and the comparison of moments by two approaches are presented in Table 13.

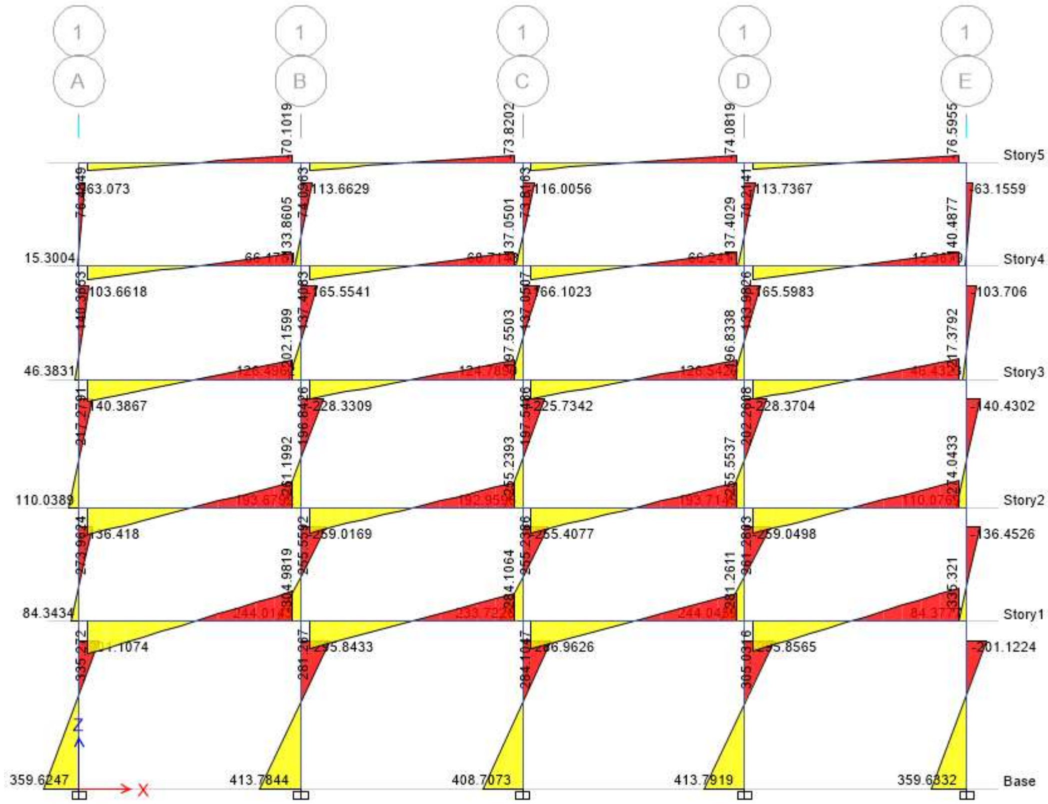


Figure 34 – Moment Diagram (ETABS)

Level	Error % of Moments in Columns				
	1	2	3	4	5
Roof	8.10	5.21	5.21	5.21	8.10
5	12.74	8.50	8.50	8.50	12.74
4	0.86	7.85	7.85	7.85	0.86
3	8.10	12.75	12.75	12.75	8.10
2	33.25	13.80	13.80	13.80	33.25

Table 13 – Colum moment Comparison

From Table 12 and 13, it's easy to notice that the errors are relatively large at the ground floor. The result is predicted and could be explained by two reasons. Firstly, exterior columns are not assigned to be pinned-pinned. So they could assemble with slabs acting as the moment frames to resist lateral loads in another direction. Secondly, the columns at the ground floor are much stiffer at the base due to the fixed condition. It means that moments at base are much bigger than the moments at top of the column. However, the portal analysis assume that each column is deformed under load so that PI occurs at mid-height. This is why relatively large errors exist at the ground level.

Based on the data and the explanation above, it's able to contend that errors are acceptable and the ETABS model are validated.

6.2.3.5 Estimation Sizes for the Members in Frame 1

According to the calculation in Appendix 2.3, 12D+0.5L+1.0E is the most critical load combination for designing members. The governed shear force and moment determined by two approaches will be used for the member design. In figure 35, beam 2-3 and column 2 at the ground floor are designed to meet both strength and serviceability requirements with the critical load combination.

The final design of the beam is 14" x 32" with 8 #8 bars at top and 3 #7 bars at bottom. The detailed shear reinforcement and development length can be reviewed in the figure 35. The calculation of the beam design is attached in Appendix 2.4.

The final design of the column is 20"x20" with 12 #10 bars for axial and flexural reinforcement and #3 stirrups for shear reinforcement at 18" O.C. The splice length is determined to be 60 ft. The configuration is exhibited in Figure 35 and the calculation of the column design is attached into Appendix 2.4.

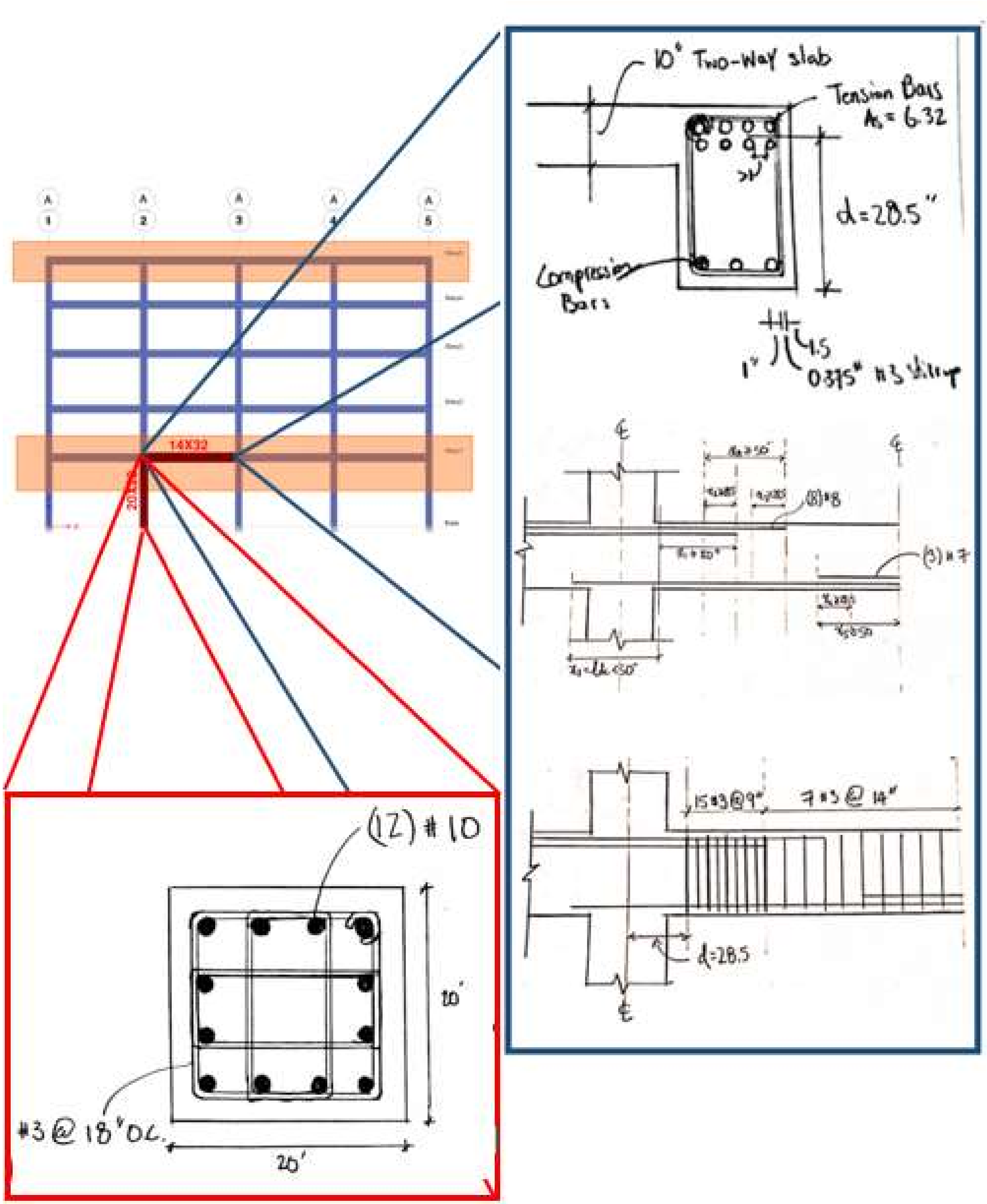


Figure 35 – Beam and Colum Detailing

6.2.3.6 ETABS Final Design

The concrete frames are remodeled with the estimated members. ETABS starts to design and check all the members and verifies that all concrete frames passed the design check. Figure 36 show the final reinforcement design from ETABS.

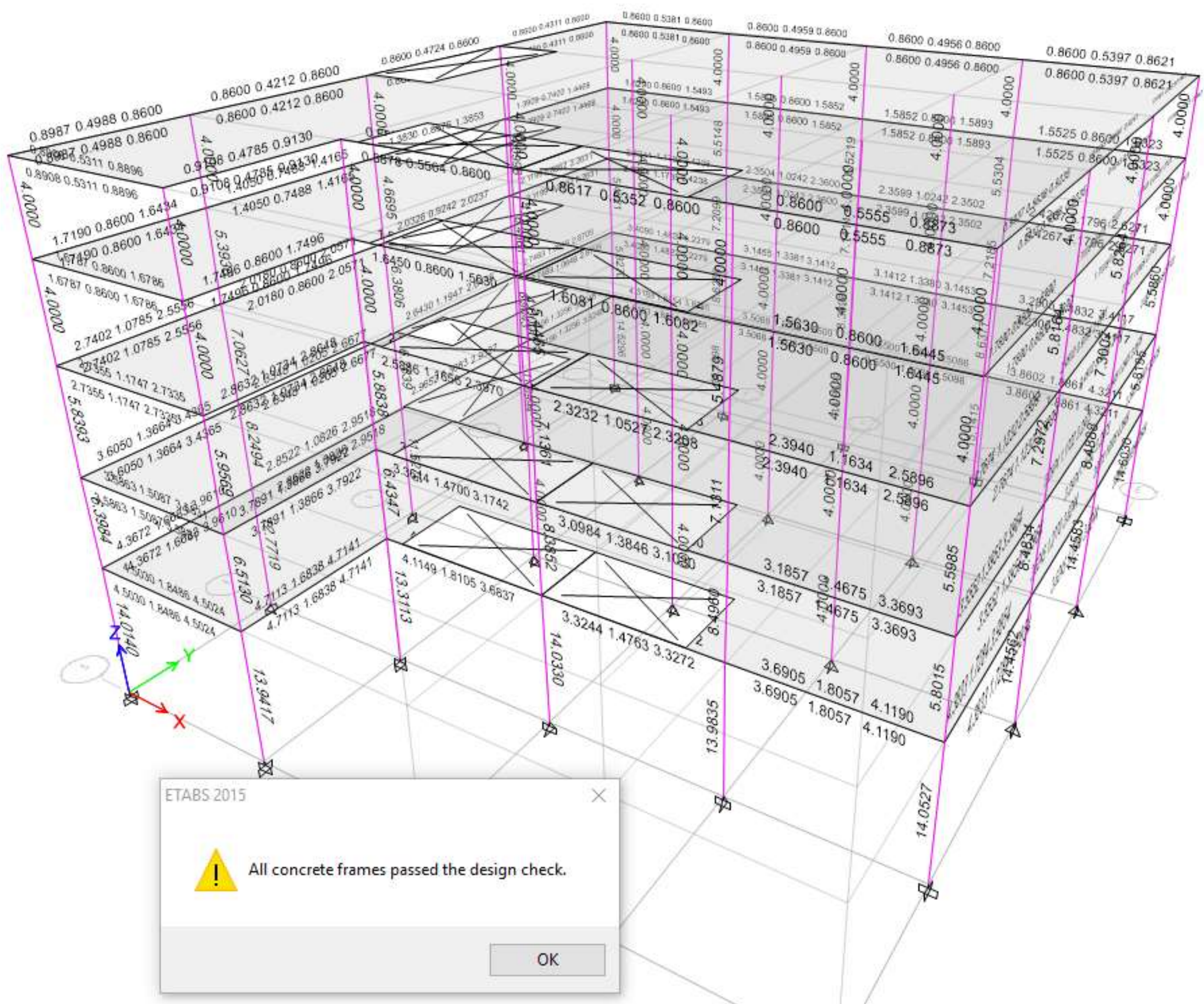


Figure 36 – Final Design by ETABS

6.2.4 Lateral Drift Check

Wind   Seismic

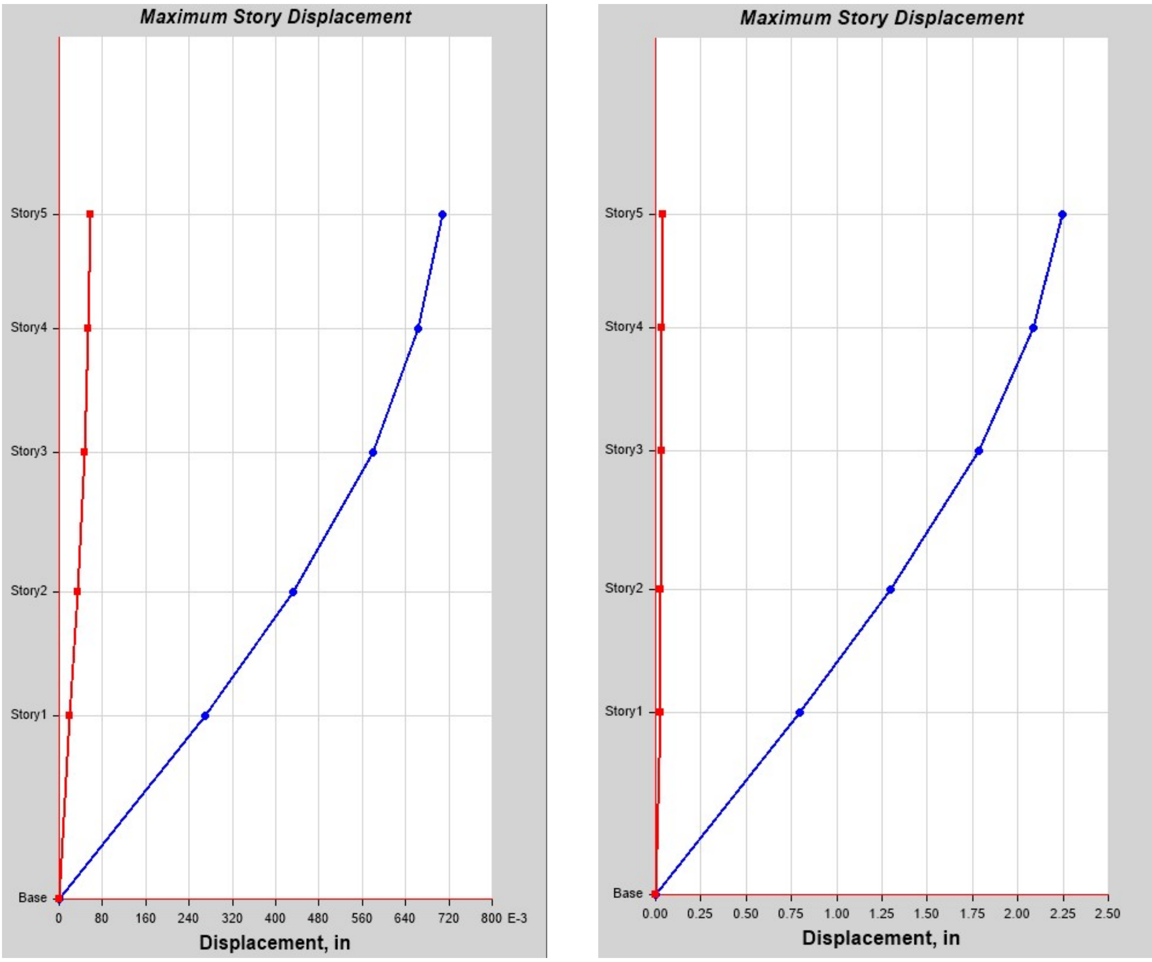


Figure 37. Wind & Seismic Drift

From graphs above, drifts due to the seismic load are critical. They are compared to code and industry acceptable values. Table 14 verifies that all drifts pass the deflection requirements. (Wind: H/400 (Industry); Seismic: 0.020hsx (ASCE 7-10))

Drift Comparsion				
	Model	Code	Industry	Good ?
Wind	0.72	/	1.905	Good
Seismic	0.75	4.08		Good
Note: Story drift for seismic is calculated on story 1, which is 0.75. (hsx = 17')				

Table 14. Drift Comparison with Codes/Industry Acceptable Values

[7] Cost Estimation & Schedule

7.1 Cost Estimation

The total amount of materials of one floor has been determined by RAM Concept. The material and labor rates are found in RS Means 2017 and inputted in the estimation sheet. The total structural cost of one floor has been estimated to be $150000, which is $20.19 per square feet.

Estimate

Concrete Costs								
Materials:	131	per	yd³	x	259.8	yd³	=	34030
Labor:	0	per	yd³	x	259.8	yd³	=	0
Total:	131	per	yd³	x	259.8	yd³	=	34030
Post-Tensioning Costs								
Materials:	0	per	pounds	x	0	pounds	=	0
Labor:	0	per	pounds	x	0	pounds	=	0
Total:	0	per	pounds	x	0	pounds	=	0
Formwork Costs								
Materials:	4.22	per	ft²	x	7428	ft²	=	31340
Labor:	6.9	per	ft²	x	7428	ft²	=	51250
Total:	11.12	per	ft²	x	7428	ft²	=	82600
Mild Steel Reinforcing Costs								
Materials:	940	per	tons	x	18.9	tons	=	17770
Labor:	755	per	tons	x	18.9	tons	=	14270
Total:	1695	per	tons	x	18.9	tons	=	32040
SSR Costs								
Materials:	2	per	stud	x	444	studs	=	888
Labor:	1	per	stud	x	444	studs	=	444
Total:	3	per	stud	x	444	studs	=	1332
Total Costs								
Materials:	11.31	per	ft²	x	7428	ft²	=	84030
Labor:	8.881	per	ft²	x	7428	ft²	=	65970
Total:	20.19	per	ft²	x	7428	ft²	=	150000

Figure 38. Cost Estimation Sheet

[8] Breadth Topics

8.1 Facades Preservation

8.1.1. Background

The existing building's façade contains a typical masonry wall system with brick veneers, double –hung aluminum-clad wood windows, flat brick arched lintels, marble cornice and string course, slate mansard roof, etc. as shown in Figure 39.

Figure 39 Façade East Elevation

The existing building of 706 Madison Avenue was constructed in 1920's with three stories. Today, it is a very important part of the City's Heritage and protected under the Landmark Law. Therefore, any minor alternation of the façade must be approved in advance by Landmark Preservation Commission (LPC). To meet the historical requirements, the existing façade needs to be preserved when the design of the new building was found. The façade preservation breadth will be explored in the following sections with regarding to scheme, temporary support system, cost and construction coordination.

8.1.2. Scheme

The re-development involved total demolition of the existing building's interior and the insertion of a new framed structure behind the preserved façade. The new structure, which comprised of a reinforced concrete moment frame and two-way slab floors, incorporated two additional floor levels, occurring between the third and fifth floor levels. The existing slate mansard roof will be taken off, stored and added to the top of the new building.

8.1.3. Temporary Support system

A single bay structural steel framework, comprising four stanchions and four horizontal wind girders, was erected immediately behind the façade before any demolition of the existing interior tool place. The façade was tied back to this temporary framework at five levels using temporary resin-anchor ties between the front member of each wind girder and the façade masonry. Figure 40 shows the layout of the temporary support system.

One of the problems in forming the temporary resin-anchor ties between the façade and its support system was the variation in the gap width between the wind girder members and the façade masonry caused by the latter's unevenness. This was overcome by using steel packing plates to fill the gap and achieve the solid connection at each tie position.

In addition, it came up with an idea that the temporary support system could be used as parts of the new structure. To be specific, the temporary support ultimately formed part of the new structure, its four stanchions being encased in concrete and incorporate as elements of the reinforced concrete moment frame. The temporary support system's four horizontal wind girders were also retained as permanent structural elements by casting them into the concrete slab. The only elements of the temporary support system not incorporated into the new structure were the vertical bracing members and the temporary resin-anchor ties.

8.1.4. Cost

Historic preservation represents a tradeoff between saving old buildings and making way for new ones. This tradeoff carries economic consequences. Although façade preservation results in a saving of façade materials for the new building, the budget could be drove up due to the difficulty of the construction and the labor fee.

Other economic consequences rather than the direct cost of the construction also needs to be considered. Preservation advocates often argue that saving historic building increases property tax revenue and thus fills city coffers. However, the entrepreneurs argue that preservation leads to increased property values overlooks the long-term economic appreciation of redevelopment. Therefore, it's really difficult to estimate a specific cost of the façade preservation.

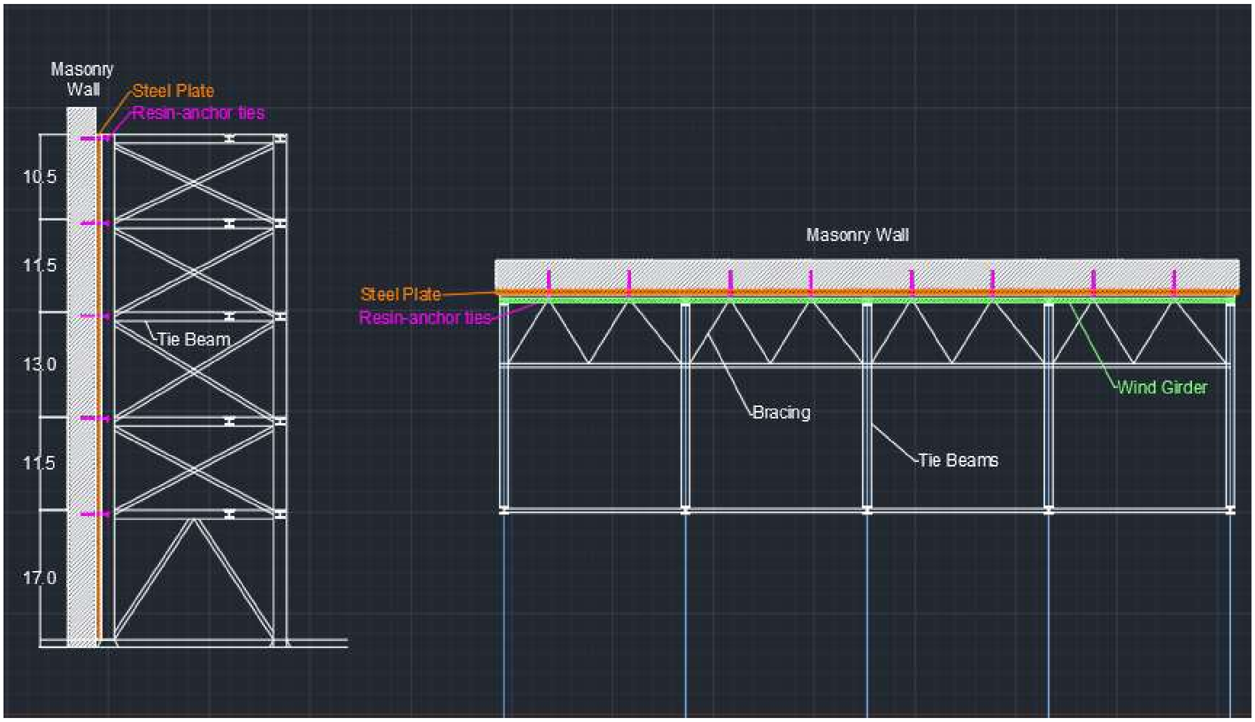


Figure 40 Section and Plan of Temporary Support System

8.1.5. Temporary Work Coordination

Coordination is a key appointment for façade retention work. It ensures that all components of the design and detailing will fit together and result in a safe and stable structure all times until the permanent work entirely replaces the temporary work. For this particular project, the coordinator must ensure that the demolition of the existing structure could commence only when all of the temporary resin-anchor ties had been completed, and the façade secured to its temporary system. The temporary resin-anchor ties were cut off using oxyacetylene burning equipment. It should be make clear, however, that none of the temporary ties were disconnected until all of the permanent façade-ties had been installed.

References:

1. Highfield David. (1991). The construction of New Buildings Behind Historic Facades. New York: E.& F.N. Spon

2. Washington Emily. (May 2, 2012). “Historic Preservation and Its Costs.” Retrieved from https://www.city-journal.org/html/historic-preservation-and-its-costs-11014.html

3. Bussell, M. (2003). Retention of masonry facades – best practice site handbook. London: CIRIA C589

8.2 Indoor Air Quality

8.2.1 Background

Because the redesign adds two more floor levels in the existing building, the original air handling units might not provide enough ventilation to the new building, resulting discomfort, reduced efficiency, and sickness to occupants. Therefore, a new ventilation rate needs to be calculated to meet the minimum indoor air quality set by ASHRAE Standard 62.1.

8.2.2 Ventilation Rate Calculation

The following steps describe the computation process based on section 6 of Standard 62.1.

6.2.2.1 Breathing Zone Outdoor Airflow

V_{bz} is the breathing zone outdoor airflow for occupiable spaces. R_p, P_z and R_a can be found in Table 6-1 from Standard 62.1.

$$V_{bz} = R_p \cdot P_z + R_a \cdot A_z$$

where

A_z = net occupied floor area (ft^2)

P_z = zone population

R_p = outdoor airflow rate per person (cfm/person)

R_a = outdoor airflow rate per area (cfm/ft^2)

6.2.2.2 Zone Air Distribution Effectiveness

The zone air distribution effectiveness E_z is determined by Table 6-2 in Standard 62.1. Because the system is ceiling supply of cool air, E_z is 1.0.

6.2.2.3 Zone Outdoor Airflow

V_{oz} is the amount of outdoor that must be supplied by mechanical systems. It is the ratio between breathing zone outdoor flow and distribution effectiveness.

$$V_{oz} = V_{bz}/E_z$$

6.2.5 Multiple-Zone Recirculating System

Because the building is conditioned by VAV with reheat system, multiple-zone recirculating calculation should be used.

6.2.5.1 Primary Outdoor Air Fraction

The first step is to find out Z_p which is the zone primary outdoor air fraction. For VAV, V_{pz} is the minimum primary airflow.

$$Z_p = V_{oz}/V_{pz}$$

6.2.5.2 System Ventilation Efficiency

System ventilation efficiency E_v, is listed in Table 6-3 in Standard 62.1.

6.2.5.3 Uncorrected Outdoor Air Intake

The equation below can be used to determine uncorrected outdoor air intake V_{ou}.

$$V_{ou} = D\Sigma_{\text{all zones}}(R_p \cdot P_z) + \Sigma_{\text{all zones}}(R_a \cdot A_z)$$

In the equation above, D is the occupant diversity to adjust occupancy variations and can be determined as below:

$$D = P_s / \Sigma_{\text{all zones}} P_z$$

where P_s is the total population in the zone.

6.2.5.4 Outdoor Air Intake

The last step is to use V_{ou} and E_v to find outdoor air intake flow, V_{ot}.

$$V_{ot} = V_{ou}/E_v$$

The detailed calculation and final result are shown in Table 15 below. Because the sub-cellar and cellar levels are mechanical and electric rooms, ASHRAE does not require to supply outside air to those spaces. The occupied zones are from first floor to fifth floor. The total ventilation rate is 14,978 cfm. New air handling units should be selected according to that.

706 Madison Ave Ventilation Rate													
Room Name	Az(SF)	# of peopleper 1000 SF	Ra(CFM/SF)	Pz(# of people)	Rp(CFM/Person)	Vbz	Ez	Voz	Vpz	Zp	Vou	Ev	Vot
Bathroom-Women x4	248	N/A					1	N/A	720	N/A			720
Bathroom-Men x4	248						1		720				720
First Floor Retail	6017	40	0.06	240.68	7.5	2166.12	1	2166.12	6700	0.3233	2166.12	0.8	2708
Second Floor Retail	6017	40	0.06	240.68	7.5	2166.12	1	2166.12	6700	0.3233	2166.12	0.8	2708
Third Floor Retail	6017	40	0.06	240.68	7.5	2166.12	1	2166.12	6700	0.3233	2166.12	0.8	2708
Forth Floor Retail	6017	40	0.06	240.68	7.5	2166.12	1	2166.12	6700	0.3233	2166.12	0.8	2708
Fifth Floor Retail	6017	40	0.06	240.68	7.5	2166.12	1	2166.12	6700	0.3233	2166.12	0.8	2708
												Sum	**14978**

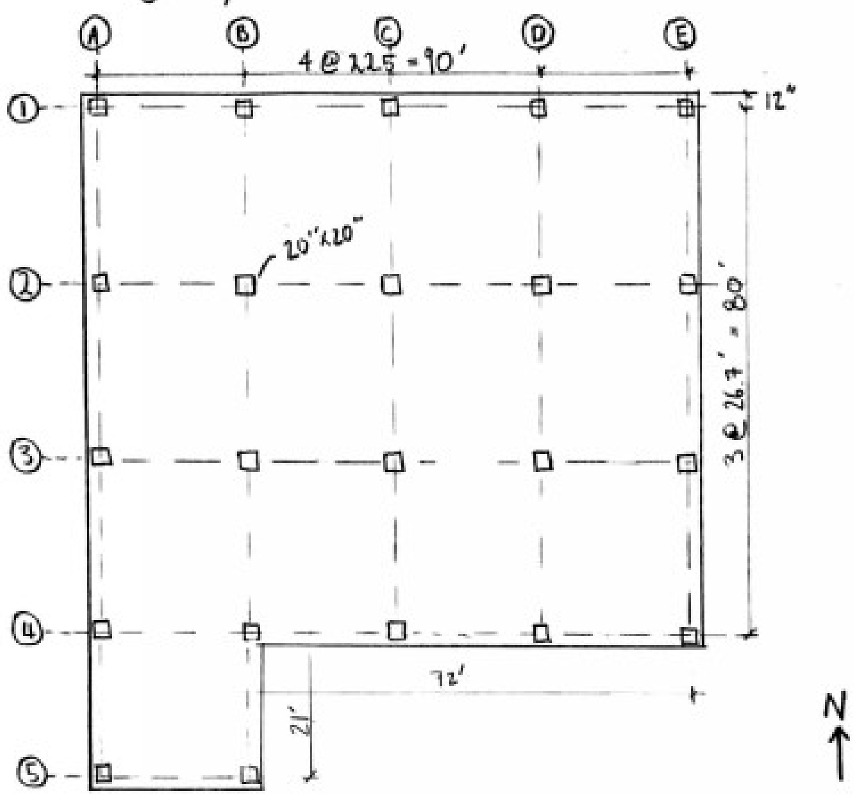
Table 15: Ventilation Rate

[9] Conclusion

The existing structural condition of 706 Madison Avenue have been introduced and analyzed in the first part of the report. Upon investigation of the existing building, a proposal of the redesign for 706 Madison Avenue has been achieved. The new design of the building changed the original architectural layout, materials and structural systems. Two-way concrete slab floor systems and reinforced concrete moment-resisting fames have been utilized in redesign of the building. The structural depths focus on designing such systems using applicable approaches. Based on the calculations and modeling, it's able to conclude that the new structural systems passed all the design check. In addition, a cost analysis and two breadth topics have been discussed in this report to assess feasibility of the redesign for 706 Madison Avenue.

Appendix 1: Calculations for Gravity Design and Analysis

New Design Layout



Proposed Solution

Gravity : Two-Way Slabs

Lateral : Concrete Shear Walls

Computer Software

→ RAM Concept

→ Etabs

Breadth Topics

- Facade Preservation (const.)
- Building Enclosure (mech.)

1.1 Gravity Loading

Gravity

- Roof loading:

Dead load:

Roof Membrane:	1 psf
2" Rigid Insulation:	3 psf
10" two-way slab:	125 psf
SDL:	25 psf
	154 psf

15 psf Beams
10 psf Colums

Live Load: 20 psi (Snow load doesn't control)

- Floor Loading:

Dead Load:

Finishes	2 psf
10" two-way slab	125 psf
SDL:	25 psf
	152 psf

Live load: 75 psf (above 1^{st} floor)

- Wall load (from previous)

@ Roof

$W_{wall} = 700$ plf

@ floors

$W_{wall(5)} = 1056$ plf

$W_{wall(4\&3)} = 1176$ plf

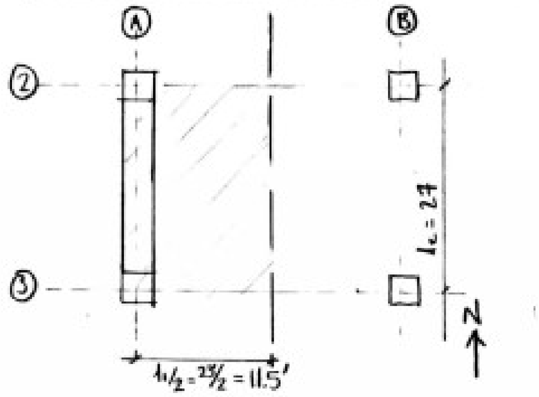
$W_{wall(2)} = 1368$ plf

- Load Path

The typical gravity loads, including dead, live, snow, and rain will be resisted by the roof or floor two-way slabs transmitted to the concrete columns, and transfer to the foundation through the concrete columns.

1.2 Member Size Estimation

- Select Beam Trial Size (A2-3 @ 1st floor)



DD = 142 psf
LL = 75 psf
WL = 1368 plf

$l_2 = 27'$
$l_1 = 23'$

$f'_c = 4000$ psi
$f_y = 60$ ksi

$$LL_R = 75 * \max \begin{cases} 0.5 \\ 0.25 + \dfrac{15}{\sqrt{2(11.5)(27)}} \end{cases} = 64 \text{ psf}$$

$$W = 1.4(142 * 11.5 + 1368) = 4.2 \text{ klf}$$

$$W = 1.2(142 * 11.5 + 1368) + 1.6(64)(11.5) = 4.8 \text{ klf} \quad \Leftarrow \text{controls}$$

$$M_{(A2\text{-}3)} = \frac{w l_2^2}{8} = \frac{4.8(27)^2}{8} = 437^{1k}$$

$$\rho = \frac{0.25 f'_c \beta_1}{f_y} = \frac{0.25 * 4 * 0.85}{60} = 0.0142$$

$$M_n = M_u/\phi = 437^{1k}/0.9 = 486^{1k} \quad (\phi = 0.9 \text{ is assumed})$$

$$\omega = \frac{\rho f_y}{f'_c} = 0.0142 * \frac{60}{4} = 0.213$$

$$R = \omega f'_c (1 - 0.59\omega) = 0.213(4)(1 - 0.59 * 0.213) = 0.745 \text{ ksi}$$

$$\left.\begin{matrix} M_n = R b d^2 \\ b \propto d/2 \end{matrix}\right\} \Rightarrow d^3 = \frac{486 \times 12}{0.5 \times 0.745} = 15656.4 \text{ in}^3$$

$$\Rightarrow d \geq 25"$$

choose 14" X 28"

TWO-WAY Slab Design (without edge beams)

- Direct Design Method

– Check applicability of DDM.

a) 3 or more continuous spans

b) ratio of longer/shorter span < 2

$27/23 = 1.2 < 2$ OK

c) Successive span lengths diff by no more than 1/3 of the longer span

$23/27 = .85 \geq 0.67$ OK

d) column offset < 10% no column offset

e) loads are uniformly distributed
(exception is made for wall load)

f) Service LL ≤ 2 * unfactored DL

$100\,psf \leq 2 * [150\,pcf\,(\frac{10''}{12}) + 15] = 280\,psf$ OK

e) no beams (ACI Sec 8.10.2.7 does not apply)

– Select Column Trial Size (B2 @ ground floor)

Interior Col: $f'_c = 4\,ksi$, & $F_y = 60\,ksi$ $\rho_g \simeq 0.015$

$LL_R = 100 * \max \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{(4)(27)(23)}} \end{array} \right. = 55\,psf$

$DL = 142\,psf$

$1.4(142) = 198.8\,psf$
$1.2(142) + 1.6(55) = 258.4\,psf$ ← controls

$\Rightarrow P_u = 258.4\,psf\,(27')(23')(4) = 642^k$

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

$\geq \frac{642}{0.4(4 + 60 * 0.015)} = 328\,in^2$

choose a square column 20" X 20" (@ ground floor)

1.3 Two-way Slab Design

1.31 Punching Shear Check

- Select slab Thickness (based on deflection)

From Table 8.3.1.1,

$f_y = 60\text{ksi}$, w/ edge beams

$l_n = 27 \times 12 - 20 = 304$ in

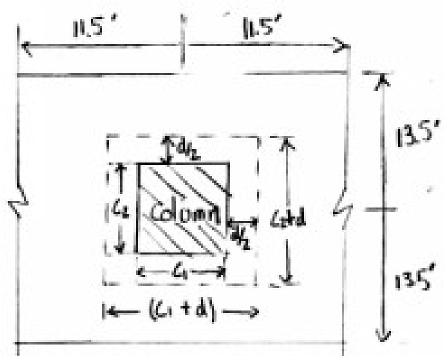
$t_{s\,min} = \dfrac{l_n}{33} = \dfrac{304}{33} = 9.2'' \Rightarrow 10''$ slab

- Check thickness adequate for shear (column B2)

$q_u = 258.4$ psf

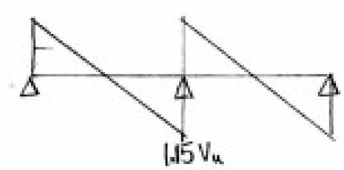
$d_{ave} \simeq 10'' - 1.5'' \cong 8.5''$

$b_o = 4(20 + 8.5) = 114$ in



$b_o = 2(c_1+d) + 2(c_2+d)$

* First span



$$V_u = 258.4\left[(11.5 \times 1.15 + 11.5)(13.5 \times 1.15 + 13.5) - \frac{(20+8.5)^2}{144}\right]$$

$= 184$ Kips

$$v_u = \frac{V_u}{b_o d} = \frac{184000}{(114)(8.5)} = 190 \text{ psi}$$

$\beta = 20/20 = 1$; $\alpha_s = 40$

$$V_c = \begin{cases} 4\sqrt{f'_c}\, b_o d & \Leftarrow \text{governs} \\ \left(2 + \frac{4}{1}\right)\sqrt{f'_c}\, b_o d = 6\sqrt{f'_c}\, b_o d \\ \left[\frac{40(8.5)}{114} + 2\right]\sqrt{f'_c}\, b_o d = 5\sqrt{f'_c}\, b_o d \end{cases}$$

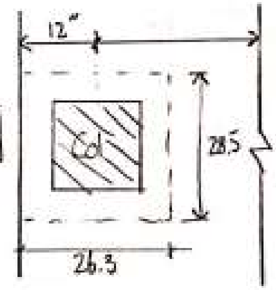
$\phi V_c = 0.75(4)\sqrt{4000}\,(114)(8.5) = 184^k \geq V_u$ OK

- Check punching shear (cont.) (Column A2)

$b_o = 28.5 + 2(26.3) = 81.1''$

$V_u = 258.4\left[(13.5 \times 1.15 + 13.5)(11.5+1) - \left(\frac{28.5 \times 26.3}{144}\right)\right] + \left[1.2 \times 1368 \times (13.5 \times 1.15 + 13.5)\right]$

$= 140$ kips



$d_s = 30$

$\frac{30(8.5)}{81.1} + 2 = 5.1 > 4$

$\Rightarrow \phi V_c = 4(0.75)\sqrt{4000}\,(81.1)(8.5) = 131$ kips $< V_u$

- Verify adequacy of dimensions

Max allowable shear $= \phi\, 6\sqrt{f'_c}\, b_o\, d$

$= 0.75\,(6)\sqrt{4000}\,(81.1)(8.5)$

$= 196\text{k} > V_u$

$\therefore$ section dimensions are adequate for reinforcement.

- Determine the extent to which the reinforcement is needed.

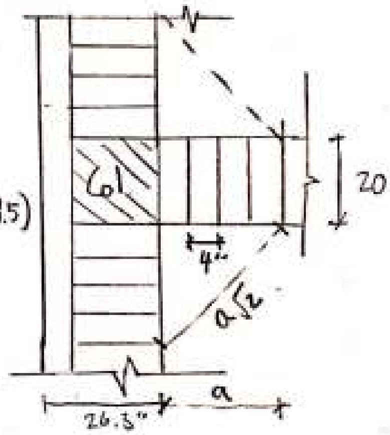
$\phi V_c = \phi\, 2\sqrt{f'_c}\, b_o\, d$

$b_o = 2(26.3) + 20 + 2a\sqrt{2}$

$= 72.6 + 2\sqrt{2}a$

$140{,}000 = 0.75(2)\sqrt{4000}\left[72.6 + 2\sqrt{2}a\right](8.5)$

$a = 58.4$ in



- Shear Reinforcement

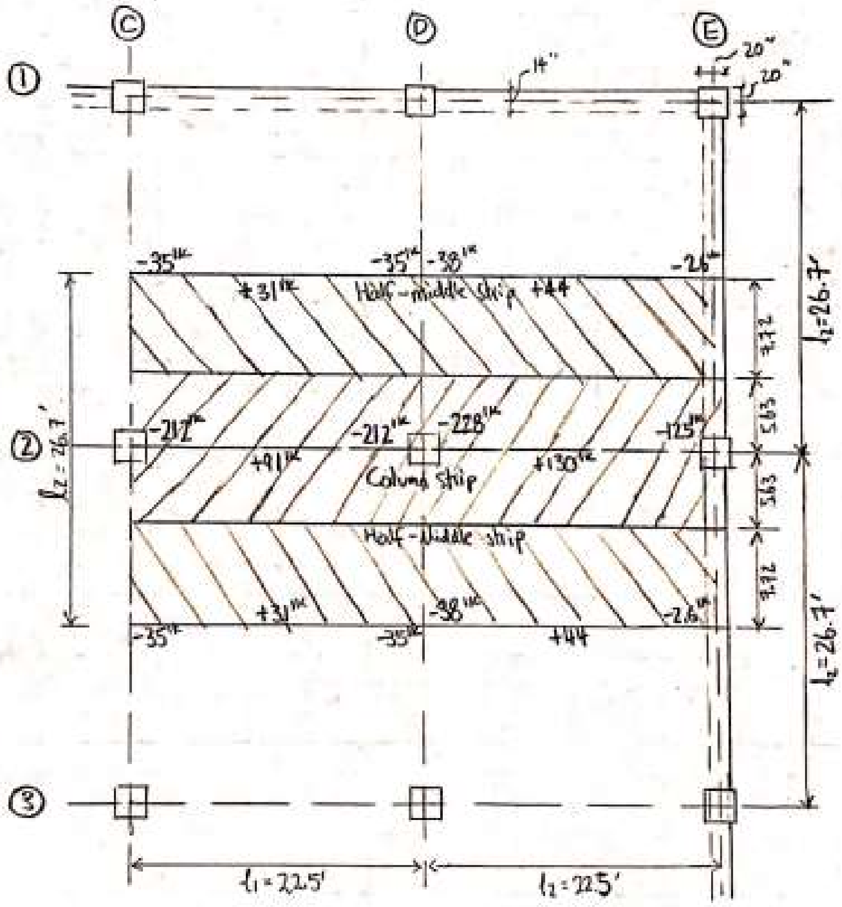
$V_s = \frac{1}{3}\left(\frac{140 - 131}{0.75}\right) = 4$ kips

use #3 stirrups $\quad V_s = \frac{A_v f_y d}{s} \Rightarrow s = \frac{(2 \times 0.11)(60)(8.5)}{4} = 28''$

Max allowable spacing $d/2 = 4.25 \Rightarrow$ use 4" spacing

TWO-WAY SLAB DESIGN

- Direct Design Method (DDM) - E-W direction
- Info: h = 10"
  LL = 75 psf
  SDL = 25 psf
  Story ht. = 17' (1st story)



Col. Strip Width = $l_{min}/2$ = 22.5/4 = 11.25'

Middle Strip Width = 26.7 - 11.25 = 15.45'

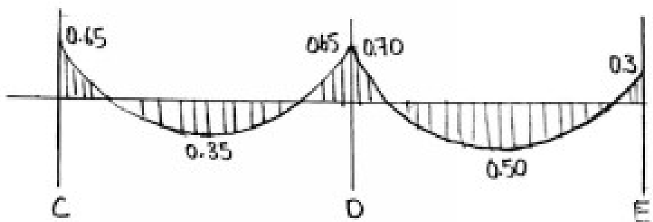
l_n = 22.5 - 20/12 = 20.83'

$q_u = 1.2\left(\frac{10}{12}\times 150 + 25\right) + 1.6(75) = 300$ psf

Moments in span 2C 2E

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{.3(26.7)(20.83)^2}{8} = 434^{'k}$$

Divide Mo into (+ve) & (-ve) moments



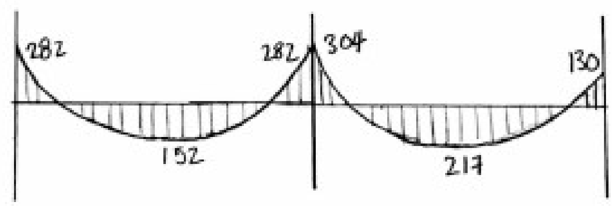
$M_{+ve} = 0.35(434) = 152^{'k}$

$M_{-ve} = -0.65(434) = -282^{'k}$

$M_{-ve} = -0.7(434) = -304^{'k}$

$M_{+ve} = 0.5(434) = 217^{'k}$

$M_{-ve} = -0.3(434) = -130^{'k}$



Divide the moments between column & middle strips

*Interior Span (CD)

Negative moments

$$\frac{\alpha_{f1} l_2}{l_1} = 0 \quad \text{(no beams)}$$

col. strip (-ve) moment $= 0.75(-282) = -212^{'k}$

Mid. strip (-ve) moment $= 0.25(-282) = -70.5^{'k}$

Col. strip (+ve) moment $= 0.60(152) = 91^{'k}$

Mid. strip (+ve) moment $= 0.40(152) = 61^{'k}$

* Exterior Span (DE)

Interior (-ve) moment : (-304^{1k})

$$\frac{\alpha f_1 l_2}{l_1} = 0 \quad \text{(no beams)}$$

Col. strip (-ve) moment = 0.75 (-304) = -228^{1k}

Mid. strip (-ve) moment = 0.25 (-304) = -76^{1k}

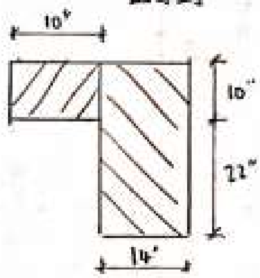
Positive moment : (217^{1k})

Col. strip (+ve) moment = 0.6 (217) = 130^{1k}

Mid. strip (+ve) moment = 0.4 (217) = 87^{1k}

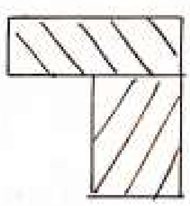
Exterior (-ve) moment : (-130^{1k})

$$\beta_t = \frac{E_{cb}\, C}{2E_{cs}\, I_s}$$



$$C = \left(1 - 0.63 \times \frac{14}{32}\right)\left(\frac{14^3 \times 32}{3}\right) + \left(1 - 0.63 \times \frac{10}{10}\right)\left(\frac{10^3 \times 10}{3}\right)$$

$= 22435 \text{ in}^4 \leftarrow$ used



$$C = \left(1 - 0.63 \times \frac{10}{24}\right)\left(\frac{10^3 \times 24}{3}\right) + \left(1 - 0.63 \times \frac{14}{22}\right)\left(\frac{14^3 \times 22}{3}\right)$$

$= 17955 \text{ in}^4$

$$I_s = \frac{(26.7 \times 12)(10)^3}{12} = 26700 \qquad E_{cb} = E_{cs}$$

$$\beta_t = \frac{22435}{2(26700)} = 0.42$$

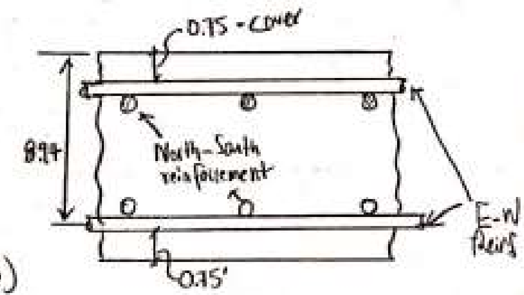
$\beta_t = 0$ 100% moment to col. strip
$\beta_t = 2.5$ 75% moment to col. strip
$\Big\}$ $\beta_t = 0.96$
96% moment to col. strip.

Exterior col. strip (-ve) moment : $0.96(-130) = -125^{1k}$

Ext. Mid. strip (-ve) moment = $0.04(-130) = -5.2^{1k}$

Calculation of the Required Area of steel

$$A_s = \frac{M_u}{\phi f_y j d}$$



$d = 10 - 0.75 - \frac{1}{2}$ bar diameter

$= 8.94$ (assuming #5 bars)

Assume $j = 0.925$

largest $M_u = 180^{1k}$
(@ 1st interior negative col. strip)

$$A_{s\,(req'd)} = \frac{228 \times 12000}{0.9(40{,}000)(0.925)(8.94)} = 9.2\ in^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{9.2 \times 40000}{0.85(4000)(11.25 \times 12)} = 0.8''$$

Table A-4 (textbook)

$a/d = 0.8/8.94 = 0.09 \Rightarrow$ tension-controlled limit $\Rightarrow \phi = 0.9$

$C = 0.8/0.85 = 0.94$

$$\varepsilon_t = \frac{(d-c)}{c} \times 0.003 = \frac{(8.94 - 0.94)}{0.94} \times 0.003 = 0.026 > 0.005$$

$jd = d - a/2 = 8.94 - 0.8/2 = 8.54''$

$$A_s = \frac{M(k\text{-}ft) \times 12000}{0.9(40000)(8.54)}$$

$= \underline{0.039\, M_u\,(k\text{-}ft)}$

ACI 318-11 §13.3.1

$A_{s\,min} = 0.002bh = 0.002(11.25 \times 12)(10) = 2.7\ in^2 < 9.2\ in^2$ OK

col. strip

$A_{s\,min} = 0.002(15.45 \times 12)(10) = 3.71\ in^2$ (Middle strip)

Spacing:

$S = \min \begin{cases} 2h = 20'' \\ 18'' \end{cases} = 18''$ ctrl (ACI318-11 §13.3.2)

Min. number of bars:

Col. strip $= \frac{11.25(12)}{18} = 7.5 \Rightarrow 8$

Mid strip $= \frac{15.45(12)}{18} = 10.3 \Rightarrow 11$

Col. strip

- Negative Moment (A_s @ top) *

$A_s(C) = 0.039(-212) = 8.27\ in^2 = (23)\ \#5\ (T)$

$A_s(D) = 0.039(228) = 8.90\ in^2 = (21)\ \#5\ (T)$

$A_s(E) = 0.039(-125) = 4.88\ in^2 = (13)\ \#5\ (T)$

- Positive Moment (A_s @ bot)

$A_s(CD) = 0.039(91) = 3.55\ in^2 = (12)\ \#5\ (B)$

$A_s(DE) = 0.039(130) = 5.07\ in^2 = (12)\ \#5\ (B)$

Mid. strip

- Negative Moment (A_s @ Top)

$A_s(C) = 0.039(71) = 2.77\ in^2 < A_{min} = 3.71\ in^2 \Rightarrow (12)\ \#5\ (T)$

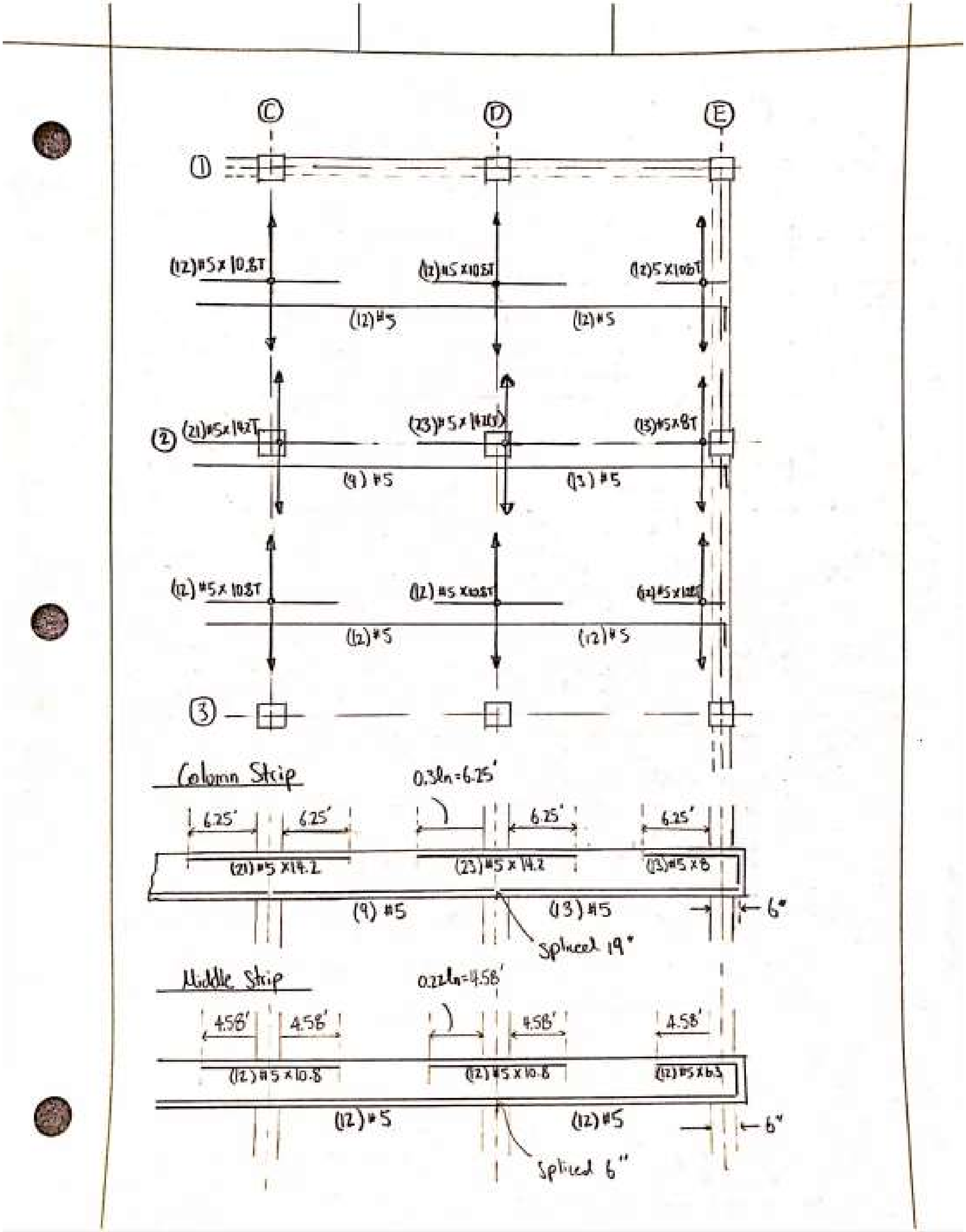
$A_s(D) = 0.039(76) = 3.0\ in^2 \Rightarrow 3.71\ in^2 \Rightarrow (12)\ \#5\ (T)$

$A_s(E) = 0.039(5.2) = 0.20\ in^2 \Rightarrow 3.71\ in^2 \Rightarrow (12)\ \#5\ (T)$

- Positive Moment (A_s @ bot)

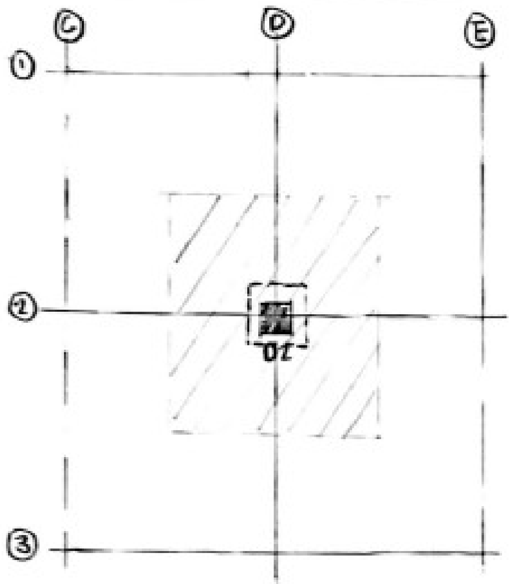
$A_s(CD) = 0.039(61) = 2.38\ in^2 \Rightarrow 3.71\ in^2 \Rightarrow (12)\ \#5\ (B)$

$A_s(DE) = 0.039(87) = 3.40\ in^2 \Rightarrow 3.71\ in^2 \Rightarrow (12)\ \#5\ (B)$


C
D
E
1
2
3
(12)#5x10.8T
(12)#5
(21)#5x14.2T
(23)#5x14.2
(13)#5x8T
(9)#5
(13)#5
Column Strip
0.3ln=6.25'
6.25'
(21)#5x14.2
(23)#5x14.2
(13)#5x8
6"
Spliced 19"
Middle Strip
0.22ln=4.58'
4.58'
(12)#5x10.8
Spliced 6"

1.4 Gravity Column Design

GRAVITY COLUMN DESIGN (D2)



$A = 26.7(22.5) = 601\ ft^2$

$K_{LL} = 4$

$P_o = (152)(600)(5)$
$= 456^k$

$L_r = 20\,psf$
$L_f = 75\,psf$

$$LL_f = 75 \times \left| \begin{matrix} 0.4 \\ \max\ 0.25 + \dfrac{15}{\sqrt{4(2404)}} \approx 0.4 \end{matrix} \right.$$

$= 30\,psf$

$P_r = (20\,psf)(601) = 12^k$

$P_L = 4(30)(601) = 72^k$

LC: $1.4D = 1.4(456) = 638^k$

$1.2D + 1.6L + 0.5L_r = 1.2(456) + 1.6(72) + 0.5(12) = 668^k$

$\Rightarrow P_u = 668^k$

Calculate M_u from Two-way slab Analysis:

$$M_u = 0.07\left[(q_{Du} + 0.5\,q_{Lu})\ell_2 \ell_n^2 - q'_{Du}\ell_2'(\ell_n')^2\right]$$

$$= 0.07\left[\left(1.2\times150 + 1.6(0.5)(75)\right)\times 26.7\times 20.83^2 - (1.2)(150)\times 26.7\times 20.83^2\right]$$

$= 49'^{k}$

Design Column with

$P_u = 668^k\ ;\ M_u = 49'^{k}$

$f'_c = 4000\,psi \quad f_y = 60\,ksi$

Select trial Size

$$A_{g(trial)} \geqslant \frac{P_u}{0.4(f'_c + f_y \rho_g)} \qquad \rho_g \simeq 0.015$$

$$\geqslant \frac{668^k}{0.4(4 + 60 \times 0.015)}$$

$$\geqslant 341 \text{ in}^2$$

choose a square column 20" x 20"

check Slenderness

$$\frac{K l_u}{r} \leqslant 34 + 12 \frac{M_1}{M_2} \leqslant 40$$

$K = 0.67 \qquad M_1/M_2 = -0.5$

$r = 0.3h$

$$\frac{K l_u}{r} = \frac{0.67(17 \times 12)}{0.3(20)} = 23 \leqslant 34 + 12(-0.5) = 28 \leqslant 40$$

∴ Slenderness can be neglected

Find Real ρ_g (from Interaction graph)

$$e = \frac{M_u}{P_u} = \frac{69(12)}{668} = 1.24''$$

$$\frac{e}{h} = \frac{1.24}{20} = 0.062$$

$$\gamma = \frac{20-5}{20} = 0.75$$

$$\left.\begin{aligned} \frac{\phi P_n}{A_g} &= \frac{P_u}{A_g} = \frac{668}{20(20)} = 1.67 \text{ ksi} \\ \frac{\phi M_n}{A_g h} &= \frac{M_u}{A_g h} = \frac{69 \times 12}{20^3} = 0.10 \text{ ksi} \end{aligned}\right\} \rho_g = 0.01 \text{ from (E-4-60-0.75)}$$

$$A_{st} = \rho_g A_g$$
$$= 0.01(20)(20) = 4 \text{ in}^2$$

use (4) #9 bars 2 on each side

Check max load Capacity

$P_o = 0.85(4)(20^2 - 4) + (4)(60)$
$= 1571^k$

$\phi P_{n,max} = 0.65(0.8)(1571) = 817^k > 668^k$ ok

Design Splice :

$l_d = \left(\frac{f_y \psi_t \psi_e}{20\lambda\sqrt{f'_c}}\right) d_b$

$= \left(\frac{60000(1)(1)}{20(1)\sqrt{4000}}\right) * 1.128$

$= 53.5''$

Select ties : (#4)

spacing : $S \leq 16(1.128) = 18''$

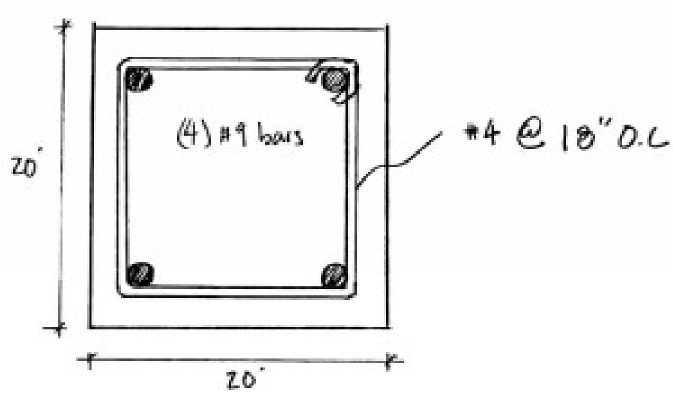
$S \leq 48(d_t) = 24''$

$S \leq 20''$

$\therefore$ use $S = 18''$

Final interior Column Design :

20" x 20" w/ #4 ties @ 18" O.C.



Appendix 2: Calculations for Lateral Design and Analysis

2.1 Wind Load Calculation (ASCE 7-10)

3/5/2017 Search Results for Map

ASCE 7 Windspeed | ASCE 7 Ground Snow Load | Related Resources | Sponsors | About ATC | Contact

Search Results

Query Date: Sun Mar 05 2017
Latitude: 40.7660
Longitude: -73.9699

ASCE 7-10 Windspeeds
(3-sec peak gust in mph*):

Risk Category I: 105
Risk Category II: 115
Risk Category III-IV: 122
MRI 10-Year:** 76
MRI 25-Year:** 85
MRI 50-Year:** 90
MRI 100-Year:** 96

ASCE 7-05 Windspeed:
104 (3-sec peak gust in mph)
ASCE 7-93 Windspeed:
80 (fastest mile in mph)

*Miles per hour
**Mean Recurrence Interval

Users should consult with local building officials to determine if there are community-specific wind speed requirements that govern.

Print your results

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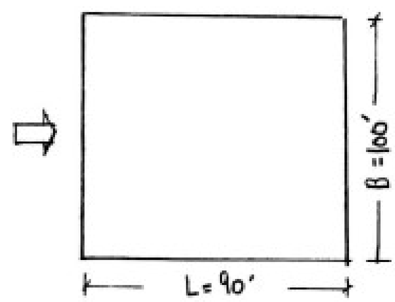
http://windspeed.atcouncil.org/index.php?option=com_content&view=article&id=10&dec=1&latitude=40.766025&longitude=-73.969948&risk_category_i=105&ris... 1/1

WIND LOAD

E-W WIND (ASCE7-10)

- Risk Category : II
- Wind Speed : 115 mph (ATC)
- Wind Directionality factor : $K_d = 0.85$



- Wind Important Factor : $I_w = 1.0$
- Exposure Category : B
- Topographic Factor : $K_{zt} = 1.0$ (except for isolated escarpments, ridges, bluffs)
- Velocity Pressure Coefficients, K_z (Table 27.3-1)

Story	Ht z (ft)	Story Ht (ft)	K_z	q_z
1	17	17	0.60	17.14
2	28.5	11.5	0.69	19.87
3	41.5	13	0.77	22.12
4	53	11.5	0.82	23.72
5	63.5	10.5	0.87	24.98
Parapet	66		0.88	25.26

Eqn:

$K_z = 0.00256\, K_z K_{zt} K_d v^2 I$

(27.3-1)

- Gust Effect Factor:

  For concrete moment-resisting frame buildings:

  $n_a = 43.5/h^{0.9} = 43.5/(63.5)^{0.9} = 1.04 > 1.0$

  ∴ Rigid

  $\Rightarrow G = 0.85$ (26.9.1)

- Enclosed Building ⇒ Internal Pressure Coefficient, $GP_i = \pm 0.18$ (T26.11-1)
- Because $h > 60'$. "Chapter 27 - Directional Procedure" is the correct method to use to determine MWFRS.

- External Pressure Coefficient, C_p (Fig. 27.4-1)

$L/B = 90/100 = 0.9$; $h/L = 63.5/90 = 0.706$
$A = 90 \times 100 = 9000 \Rightarrow$ Reduction Factor $= 0.8$

– Walls : $C_{p,\text{windward}} = 0.8$
$C_{p,\text{leeward}} = 0.52$

– Roofs : $C_{p,\text{roof}\,(0-31.75)} = -.96$
$C_{p,\text{roof}\,(31.75-63.5)} = -.82$
$C_{p,\text{roof}\,(63.5-90)} = -.58$

Location	z (ft)	q_z (psf)	C_p	q_zGC_p (psf)	GC_{pi}	q_iGC_{pi} (psf)	Net Pressure (psf)
Windward	17	17.14	0.8	11.66	0.18	4.5	$q_zGC_p - q_iGC_{pi}$
	28.5	19.87		13.51			
	41.5	22.12		15.04			
	53	23.72		16.13			
	63.5	24.98		16.99			
Leeward	All	24.98	0.52	11.04			
Parapet (WW)	66	25.26			1.5		37.89
Parapet (LW)	66	25.26			1.0		25.26
Roof (0-31.75)	63.5	24.98	0.96	20.37	0.18	4.5	
(31.75-63.5)	63.5	24.98	0.82	19.41			
(63.5-90)	63.5	24.98	0.58	12.32			

$^{*}P = qGC_p - q_i(GC_{pi})$ (lb/ft²) (27.4-1)

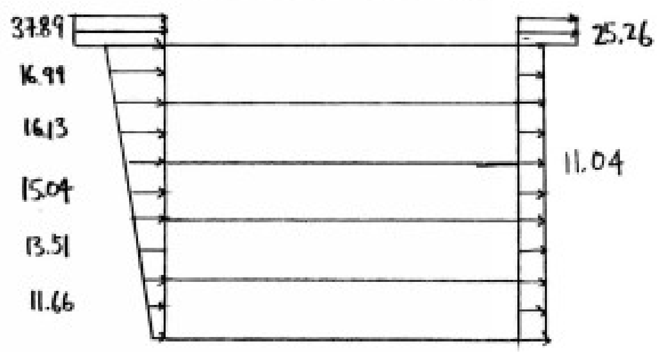
- Pressure & Story Forces

z	q_z	$P_{windward}$	$P_{leeward}$	trib Width	trib Ht.	Story force	
17	17.14	11.66	11.04	100	14.25	32.34	
28.5	19.87	13.51	11.04	100	12.25	30.07	
41.5	22.12	15.04	11.04	100	12.25	31.95	
53	23.72	16.13	11.04	100	11.0	29.89	
63.5	24.98	16.99	11.04	100	5.25	14.71	30.50
66	25.26	37.89	25.26	100	2.5	15.78	

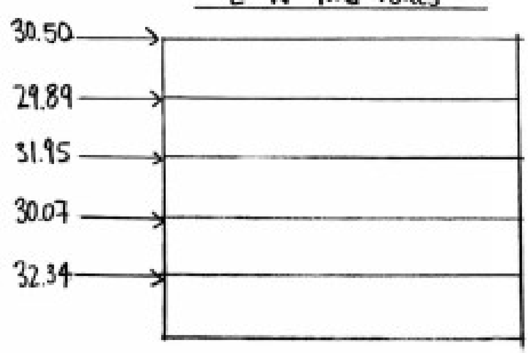
Base Shear = 154.75

- Graphs:

E-W Wind Pressures



E-W Wind Forces



Base shear = 154.75

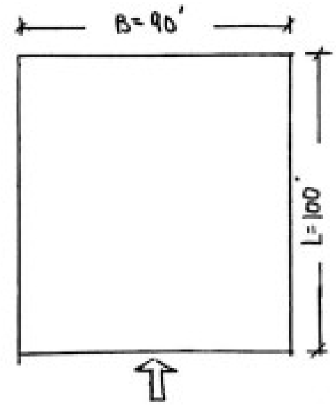
N-S WIND

- External Pressure Coefficient, C_p

$L/B = 100/90 = 1.11$
$h/L = 63.5/100 = 0.635$
$A = 9000 \Rightarrow RF = 0.8$

- Walls: $C_{p\,windward} = 0.8$
$C_{p\,leeward} = 0.48$

- Roof: $C_{p\,(0-31.75)} = -0.94$
$C_{p\,(31.75-63.5)} = -0.85$
$C_{p\,(63.5-100)} = -0.55$



Location	z (ft)	q_z (psf)	C_p	$q_z G C_p$ (psf)	GC_{pi}	$q_h GC_{pi}$ (psf)	Net Pressure (psf)
Windward	17	17.14	0.8	11.66	0.18	4.5	
	28.5	19.87		13.51			
	41.5	22.12		15.04			
	53	23.72		16.13			
	63.5	24.98		16.99			
Leeward	All	24.98	0.48	10.14			
Parapet (ww)	66	25.26			1.5		37.89
(LW)	66	25.26			1.0		25.26
Roof (0-31.75)	63.5	24.98	0.94	19.96	0.18	4.5	
(31.75-63.5)	63.5	24.98	0.85	18.05			
(63.5-100)	63.5	24.98	0.55	11.68			

$P = qGC_p - q_i(GC_{pi})$ (lb/ft²) (27.4-1)

- Pressure & Story forces

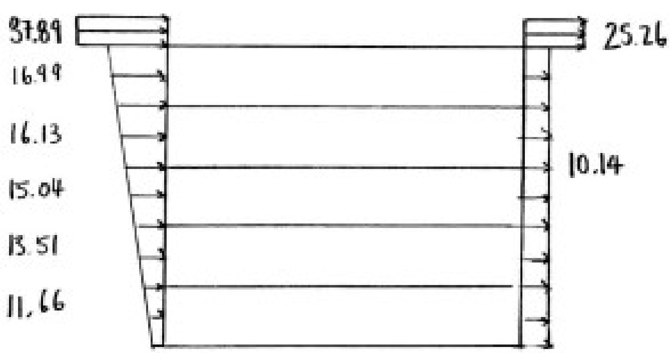
z	q_z	$P_{windward}$	$P_{leeward}$	trib Width	trib Ht	Story forces
17	17.14	11.66	10.14	90	14.25	27.96
28.5	19.87	13.51	10.14	90	12.25	26.08
41.5	22.12	15.04	10.14	90	12.25	27.77
53	23.72	16.13	10.14	90	11.0	26.01
63.5	24.98	16.99	10.14	90	5.25	12.82
66	25.26	37.89	25.26	90	2.5	14.21

27.02

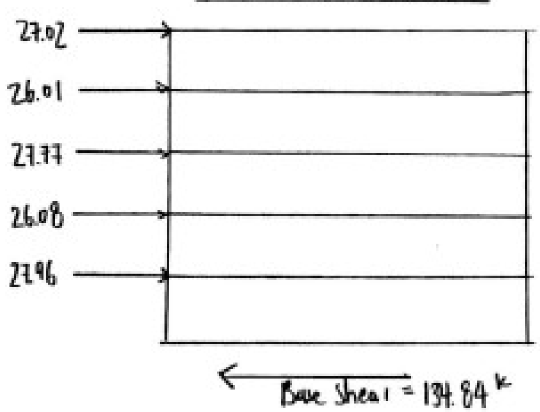
Base Shear = 134.84

- Graphs:

N-S Wind Pressure



N-S Wind Forces



2.2 Seismic Load Calculation (ASCE 7-10)

SEISMIC LOAD (ASCE7-10)

· Site Class : D "Stiff soil" (§11.4.2)

· $S_s = 0.280g$ $S_{MS} = 0.441g$ $S_{DS} = 0.294g$
$S_1 = 0.072g$ $S_{M1} = 0.172g$ $S_{D1} = 0.115g$ (From USGS)

· Seismic Design Catagory

$0.167 \leq S_{DS} < 0.33 \Rightarrow B$
$0.067 \leq S_{D1} < 0.133 \Rightarrow B$ $\therefore$ SDC B (Table 11.6-1 & 11.6-2)

Table 12.6-1 . ELF is permitted, use ELF Procedure

· Ordinary reinforced concrete moment frames (Table 12.2-1)

$R = 3$, $\Omega_0 = 3$, $C_d = 2.5$

$I_e = 1.0$ (Table 1.5-2)

$C_t = 0.016$, $x = 0.9$ (Table 12.8-2 : Concrete moment-resisting frames)

$h_n = 63.5'$

$T_a = 0.016(63.5)^{0.9} = 0.671\,s$ (Eqn. 12.8-7)

$T_L = 6s$ (Figure 22-12)

· Seismic Responce Coefficient, C_s

$T_a = 0.671s < T_L = 6s$, use Eq 12.8-2 & 12.8-3

$$\left.\begin{array}{l} C_s = \dfrac{S_{DS}}{(R/I_e)} = \dfrac{0.294}{(3/1)} = 0.098 \\ C_{s\,max} = \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{0.115}{0.671(3)} = 0.057 \\ C_{s\,min} = \max \left| \begin{array}{l} 0.044 S_{DS} I_e = 0.044(0.294)(1) = 0.013 \\ 0.01 \Leftarrow \text{controls} \end{array} \right. \end{array}\right\} \therefore C_s = 0.057$$

· Total Seismic Weights :

Floor Area : $90 \times 80 + 21 \times 23 - \underset{\text{opening}}{640} = 7043\ ft^2$

Wall perimeter : $2(90 + 101) = 382\ ft$

* 10psf partition load has been considered.

$W_{roof} = 164(7043) + 700(382) = 1422.5$ kips

$W_{floor(5)} = 162(7043) + 1056(382) = 1544.4$ kips

$W_{floor(4\&3)} = 162(7043) + 1176(382) = 1590.2$ kips

$W_{floor(2)} = 162(7043) + 1368(382) = 1663.5$ kips

$W_{total} = 1422.5 + 1544.4 + 2(1590.2) + 1663.5 = 7811$ kips

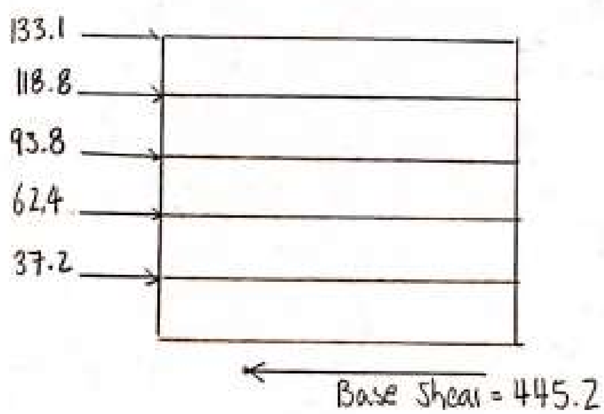
- Seismic Base Shear

$V = C_s W = 0.057(7811) = 445.2$ kips $K = 1.0855$

h_x (ft)	W_x (kips)	$W_x h_x^k$	C_{vx}	F_x (kips)
17	1663.5	36030.9	0.084	37.2
28.5	1590.2	60351.2	0.140	62.4
41.5	1590.2	90749.2	0.211	93.8
53	1544.4	114937.4	0.267	118.8
63.5	1422.5	128814	0.299	133.1
Σ	7811	430882.7	1	445.2

$F_x = C_{vx} V$ (12.8-11)

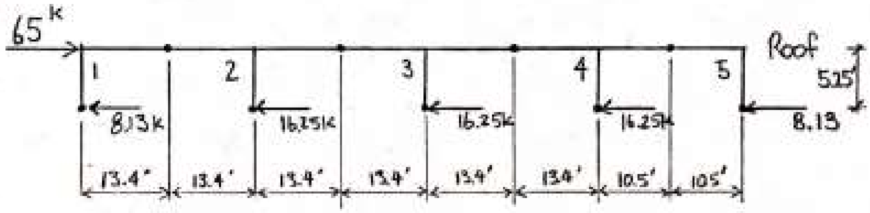
$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (12.8-12)



Seismic Loads

2.3 Portal Analysis

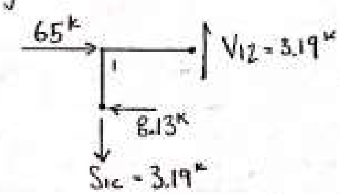
Portal Analysis for Elements @ Roof & 1st Floor

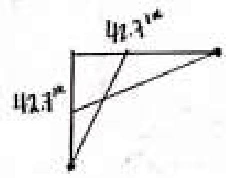


- Add forces on the roof and distribute 2:1 to interior and exterior columns as assumed by PAM:

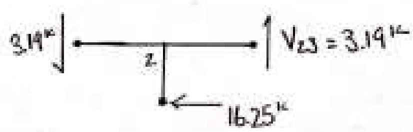
  $P_{ext} = 65/8 = 8.13^{k}$; $P_{INT} = 65/4 = 16.25\,k$

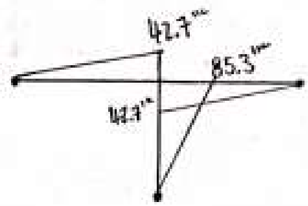
- Girder moments and shears plus column axial force found from joint FBDs:



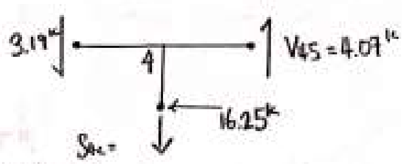


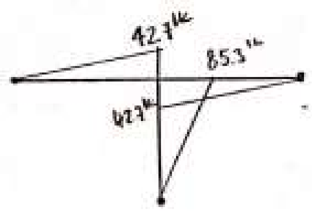
$M_g = M_c = (8.13)(5.25) = 42.7'^{k}$
$V_{12} = 42.7 / 13.4 = 3.19^{k}$
$S_{1c} = V_{12} = 3.19^{k}$



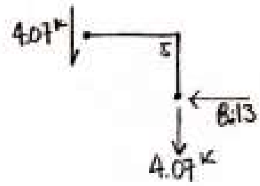


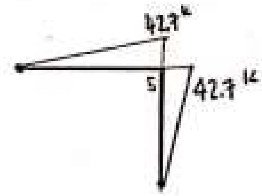
$V_{23} = 3.19^{k}$
$M_c = 16.25\,(5.25) = 85.3'^{k}$
$M_g = 3.19\,(13.4) = 42.7'^{k}$



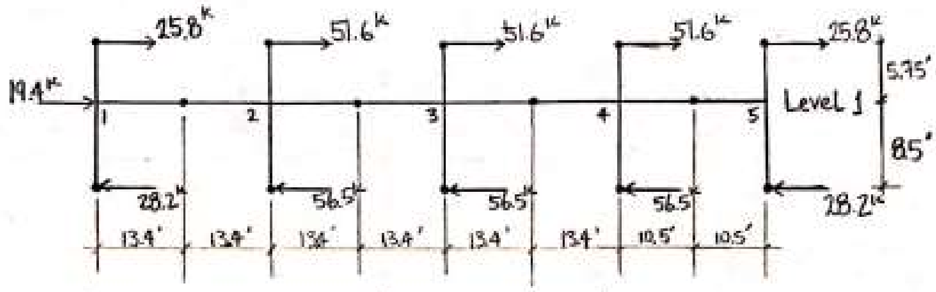


$\Sigma M_4 = 0$
$V_{45} = 42.7/10.5 = 4.07^{k}$
$S_{4c} = 4.07 - 3.19 = 0.88^{k}$





@ 1st Floor



- Add forces from above 1st floor and distribute 2:1 to interior and exterior columns as assumed by the PAM:

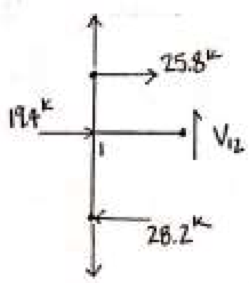
  ΣForces from above 1st floor = $65 + 60.9 + 48.4 + 32.2 = 206.5$ K

  $P_{ext} = 206.5/8 = 25.8^K$ & $P_{int} = 206.5/4 = 51.6^K$

- Add story force to lateral loads from above and distribute to columns below level 1:

  $P_{ext} = (206.5 + 19.4)/8 = 28.2^K$; $P_{int} = (206.5 + 19.4)/4 = 56.5^K$

- Beams and column moments can be determined from equilibrium:



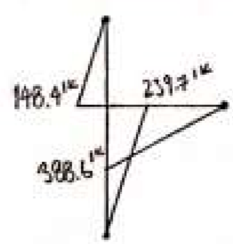
$\Sigma M_1 = 0$

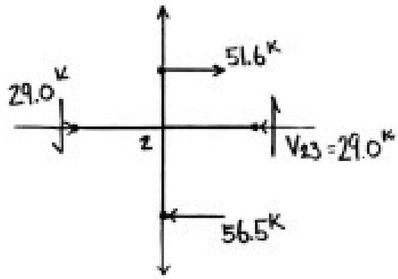
$V_{12} = (25.8 \times 5.75 + 28.2 \times 8.5)/13.4 = 29.0^K$

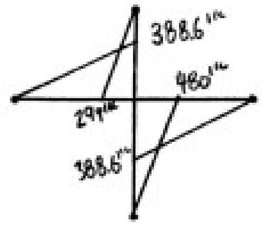
$M_g = (29.0)(13.4) = 388.6^{\prime K}$

$M_{cu} = (25.8)(5.75) = 148.4^{\prime K}$

$M_{cd} = (28.2)(8.5) = 239.7^{\prime K}$







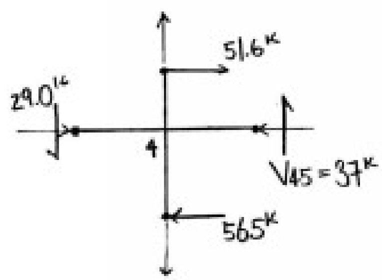
$\sum M_{girders} = \sum M_{columns}$

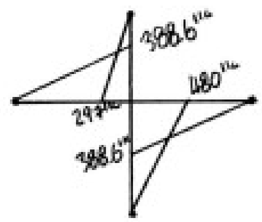
$(29.0 + V_{23})(13.4) = (51.6)(5.75) + (56.5)(8.5) \Rightarrow V_{23} = 29.0^{k}$

$M_g = 388.6^{'k}$

$M_{cu} = (51.6)(5.75) = 297^{'k}$

$M_{cd} = (56.5)(8.5) = 480^{'k}$





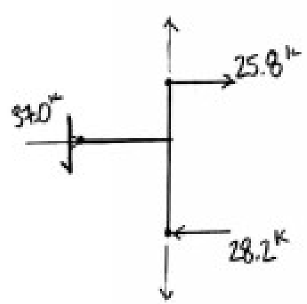
$\sum M_{girder} = \sum M_{columns}$

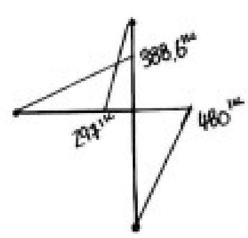
$V_{45} = (29.0)(13.4)/10.5 = 37^{k}$

$M_g = 388.6^{'k}$

$M_{cu} = 297^{'k}$

$M_{cd} = 480^{'k}$

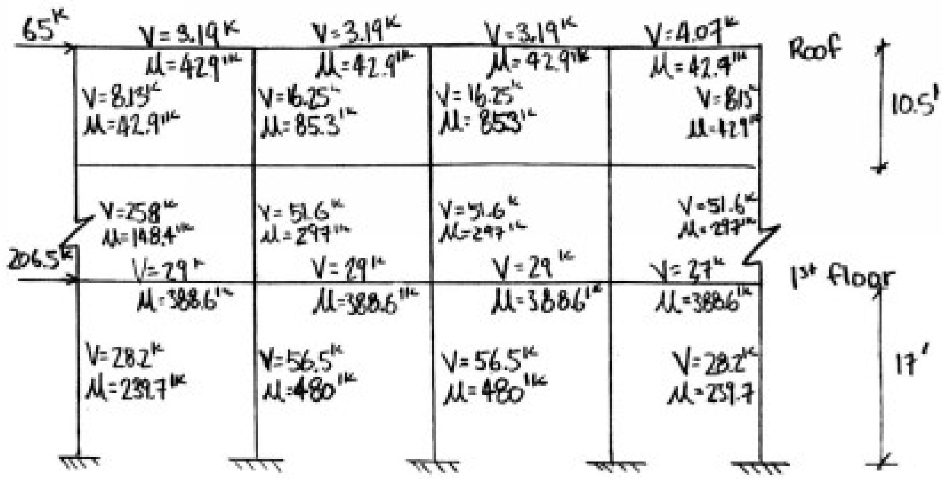




$\sum g = 388.6^{'k}$

$\sum cu = 297^{'k}$

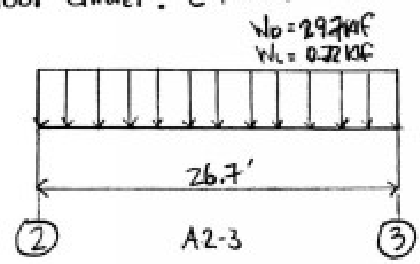
$\sum cd = 480^{'k}$


65k
V=3.19k
M=42.9'k
V=3.19k
M=42.9'k
V=3.19k
M=42.9'k
V=4.07k
M=42.9'k
Roof
10.5'
V=8.13k
M=42.9'k
V=16.25k
M=85.3'k
V=16.25k
M=85.3'k
V=8.15k
M=42.9'k
V=25.8k
M=148.4'k
V=51.6k
M=297'k
V=51.6k
M=297'k
V=51.6k
M=297'k
206.5k
V=29k
M=388.6'k
V=29k
M=388.6'k
V=29k
M=388.6'k
V=27k
M=388.6'k
1st floor
17'
V=28.2k
M=239.7'k
V=56.5k
M=480'k
V=56.5k
M=480'k
V=28.2k
M=239.7

ESTIMATION SIZES FOR THE MEMBERS IN FRAME A

*Approximate Analysis of Floor and Roof Girders

• Floor Girder: @ 1st floor



extract 10 psf for clgs.

$W_D = (142\ psf)(22.5/2) = 1.60$ klf

$W_{wall} = 1.37$ klf (from Gravity Section)

$W_L = (64\ psf)(22.5/2) = 0.72$ klf

$LL_R = 64$ psf (see previous)

1.2D + 1.6L

$W_u = 1.2(2.97) + 1.6(0.72) = 4.72$ klf

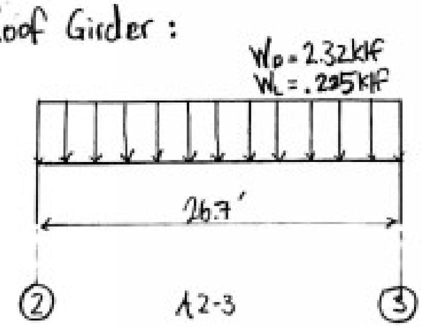
$M_u = \frac{W_u l^2}{11} = \frac{(4.72)(26.7)^2}{11} = 306^{'k}$

1.2D + 0.5L

$W_u = 1.2(2.97) + 0.5(0.72) = 3.9$ klf

$M_u = \frac{W_u l^2}{11} = \frac{3.9(26.7)^2}{11} = 253^{'k}$

• Roof Girder:



$W_D = (144\ psf)(22.5/2) = 1.62$ klf

$W_{wall} = .7$ klf (from Gravity Section)

$W_L = (20\ psf)(22.5/2) = .225$ klf

1.2D + 1.6Lr

$W_u = 1.2(2.32) + 1.6(.225) = 3.1$ klf

$M_u = \frac{W_u l^2}{11} = \frac{(3.1)(26.7)^2}{11} = 201^{'k}$

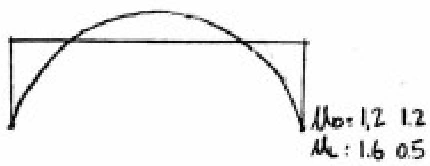
1.2D + 0.5Lr

$W_u = 1.2(231) + 0.5(.225) = 2.9$ Klf

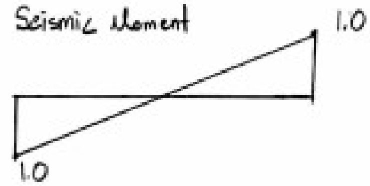
$M_u = \frac{W_u l^2}{11} = \frac{2.9(26.7)^2}{11} = 188^{1K}$

Estimate Size for Typical Floor Girder

Gravity Moment



Seismic Moment



1.2D + 1.6L

$M_{nt} = 306^{1K}$

$M_{lt} = 0^{1K}$

$M_u = 306^{1K}$

1.2D + 0.5L + 1.0E

$M_{nt} = 253^{1K}$

$M_{lt} = 389^{1K}$

$M_u = 253 + 389 = 642^{1K}$ ✓

So. LC: 1.2D+0.5L+1.0W Contros, $M_u = 642^{1K}$

Estimate Size for Typical Roof Girder

1.2D + 1.6L

$M_{nt} = 201^{1K}$

$M_{lt} = 0^{1K}$

$M_u = 201^{1K}$

1.2D + 0.5L + 1.0E

$M_{nt} = 188^{1K}$

$M_{lt} = 43^{1K}$

$M_u = 188 + 43 = 231^{1K}$

∴ 1.2D + 0.5L + 1.0W contros

$M_u = 231^{1K}$

2.3 Beam Design

- Beam Design (A2-3 @ 1st floor)

$L = 26.7'$
$f'_c = 4$ ksi
$f_y = 60$ ksi
$M_u = 642^{1k}$

- Calculate a tentative ρ

$$\rho = \frac{0.25 f'_c \beta_1}{f_y} = \frac{0.25(4)(0.85)}{60} = 0.0142$$

$$M_n = \frac{M_u}{\phi} = \frac{642^{1k}}{0.9} = 713^{1k} \quad (\text{Assume } \phi = 0.9)$$

$$\omega = \frac{\rho f_y}{f'_c} = 0.0142 * \frac{60}{4} = .213$$

$$R = \omega f'_c (1 - 0.59\omega)$$
$$= .213(4)(1-0.59 * .213) = 0.745 \text{ ksi}$$

$$M_n = R bd^2 \Rightarrow bd^2 = \frac{713 \times 12}{0.745} = 11485 \text{ in}^3$$

Potiential Configurations ($b \simeq d/2$)

$$11485 = (d/2)(d)^2 \Rightarrow d = 28.5'' \Rightarrow 14'' \times 29.5''$$

Choose 14 x 32 ($d = 29.5''$, $c = 2.5''$)

Req. As:

$$R = \frac{M_u}{\phi bd^2} = \frac{713 \times 12}{0.9(14)(29.5)^2} = 0.78 \text{ Ksi}$$

$$\rho = \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(.78)}{0.85(4)}}\right] = 0.015$$

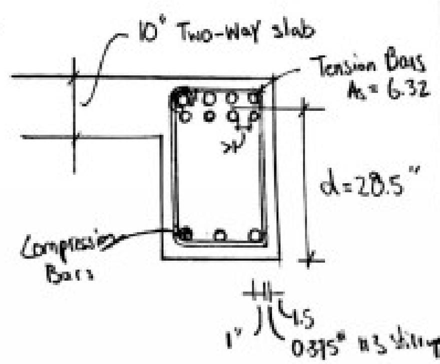
$$A_s = 0.015(14)(29.5) = 6.2 \text{ in}^2$$

use (8) #8, $A_s = 6.32$ in^2
↳ 2 rows

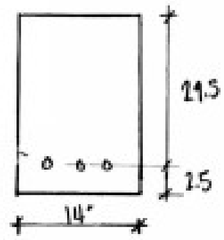
Min Spaing of Bars:

$$7 + 2(0.375 + 1.5) = 10.75'' < 14''$$

$\therefore$ OK.



Midspan Reinf. Requirement



* Assume Moment due to Seismic @ midspan is 0.

$W_u = 4.72$ Klf

$M_u = \frac{Wl^2}{16} = \frac{4.72(26.7)^2}{16} = 210^{\text{'k}}$

$R = \frac{M_u}{\phi b d^2} = \frac{210 \times 12}{0.9(14)(29.5)^2} = 0.230$

$\rho = \frac{0.85(4)}{60}\left(1 - \sqrt{1 - \frac{2(0.23)}{0.85(4)}}\right) = 0.0035$

$A_s = \rho b d = 0.0035(14)(29.5) = 1.45\ in^2$

use 3 #7 $\Rightarrow A_s = 3(0.6) = 1.8\ in^2$

check $\rho_{(0.003)} = \frac{0.32 f'_c \beta_1}{f_y} = \frac{0.32(4)(0.85)}{(60)} = 0.0181$

$0.0181 > 0.0035 \Rightarrow \phi = 0.9$

*Check $A_{s\,min}$

$A_{s\,min} = \frac{3\sqrt{4000}}{60000} \times 14 \times 29.5 = 1.3\ in^2$

but not less than $\frac{200 \times 14 \times 29.5}{60000} = 1.38\ in^2 < 1.45\ in^2$ ok

$\therefore$ Use (3) #7 @ middle of the beam.

· Check Reinf. Requirement:

$A_{smin} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60000} * 14 \times 29.5 = 1.31 \text{ in}^2$

but not less than $\frac{200 b_w d}{f_y} = \frac{200(14)(29.5)}{60000} = 1.38 \text{ in}^2 < 6.32 \text{ in}^2$

$\therefore A_{smin}$ is satisfied.

· Flexural Strength (ϕM_n)

$T = C$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60)(6.32)}{0.85(4)(14)} = 8.0''$

$c = a/\beta_1 = 9.4''$

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

$= 0.9(6.32)(60)(28.5 - 8/2)$

$= 697^{1k} > 642^{1k}$

Verify strain in steel

$\varepsilon_s = \frac{(d-c)}{c} \varepsilon_{cu} = \frac{(28.5-9.4)}{9.4} * 0.003 = 0.0061 > 0.005$

$\Rightarrow \phi = 0.9$

· Check Shear Strength (ϕV_n)

1.2D + 1.6L

$V_u = \frac{4.72(26.7)}{2} = 63^k$

1.2D + 0.5L + 1.0E

from Seismic (see previous)

$V_u = \frac{3.9(26.7)}{2} + 29 = 81^k$

$V_u @ d = 81 - \frac{81 * 28.5}{13.4(12)} = 66.6^k$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{4000}(14)(28.5) = 50.5^k$

$\phi V_c = 0.75(50.5) = 37.9^k$

$V_u > \phi V_c \Rightarrow$ Shear Reif. are required.

Shear Reinforcement

$V_s = \frac{V_u}{\phi} - V_c = \frac{66.6}{0.75} - 50.5 = 38.3^k$

check $8\sqrt{f'_c}\,b_w d = 202^k > V_s$ section is adequate

Stirrup Spacing

Using #3 stirrup 2 branch

$S \leq \frac{A_v f_{yt} d}{V_s} = \frac{0.22(60)(28.5)}{38.3^k} \leq 9.8$ use $S = 9''$

check Min. Rft requirement

$$A_{v,min} = \begin{cases} 0.75\sqrt{f'_c}\,\frac{b_w S}{f_{yt}} = \frac{0.75\sqrt{4000}(14)(9)}{60000} = 0.1 \text{ in}^2 \\ 50\,\frac{b_w S}{f_{yt}} = \frac{50(14)(9)}{60000} = .105 \text{ in}^2 < 0.22\text{in}^2 \quad \text{OK} \end{cases}$$

check Max. spacing requirements

$V_s < 4\sqrt{f'_c}\,b_w d$

$\Rightarrow S_{max} = \frac{d}{2} = 14.25''$ or $24''$

governs for this check $> 9''$ OK

∴ Final design: #3 stirrup 2 branch @ 9" O.C.

Development Length (ld)

$l_d = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\psi_t \psi_e \psi_s}{C_b + K_{tr}}$

$C_b = \min \begin{cases} \text{side cover} = 2.5'' \\ \frac{1}{2}S = \frac{1}{2}(14 - 2\times 2.5)/3 = 1.5'' \Leftarrow \text{Governs} \end{cases}$

$K_{tr} = \frac{A_{tr} f_{yt}}{1500 \times S \times n}$

$= \frac{40(0.22)}{6 \times 4}$

$= .367$

$S = 6''$

$n = 4$

$A_{tr} = 2\times 0.11 = 0.22 \text{ in}^2$

$f_{yt} = 60000$ psi

$\frac{C_b + K_{tr}}{d_b} = \frac{1.5 + 0.367}{1} = 1.87 \leq 2.5$ OK

$\psi_t = 1.3$ (>12" of concrete underneath bar)

$\psi_e = 1.0$ (Uncoated bars)

$\psi_s = 1.0$ ($\geq$ No.7 bars)

$\lambda = 1.0$ (Normal Weight Concrete)

$l_d = \frac{3}{40(1)} * \frac{60000}{\sqrt{4000}} * \frac{1.3(1)(1)}{1.85} = \underline{50.0 \text{ in}}$

$x_1 \geq l_d = 50"$

$x_2 \geq \left| \begin{array}{l} d = 28.5" \Leftarrow \text{controls.} \\ 12d_b = 12" \end{array} \right.$

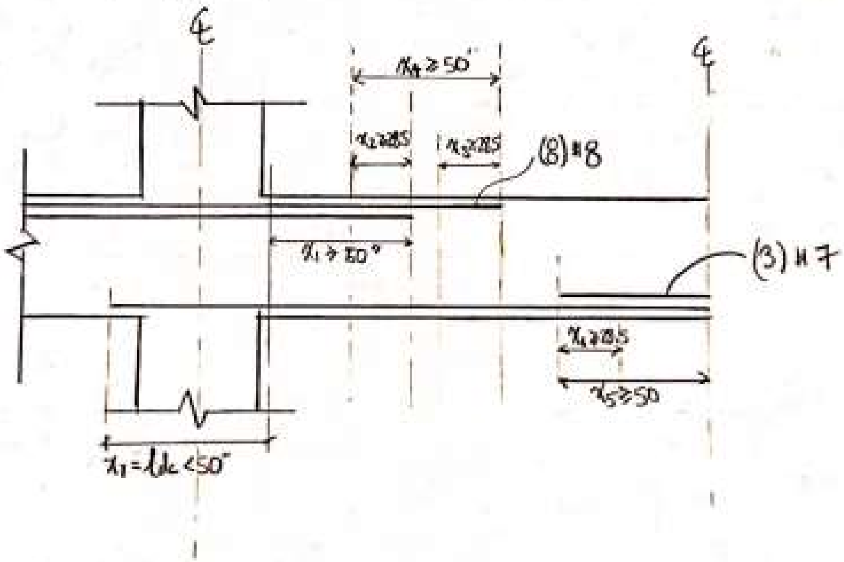
$x_3 \geq \left| \begin{array}{l} d = 28.5" \Leftarrow \text{controls} \\ 12d_b = 12" \\ 1/16(26.7 \times 12 - 20) = 18.8" \end{array} \right.$

$x_4 = x_1 \geq 50"$

$x_5 \geq l_d = 50"$

$x_6 = \underset{max}{} \left| \begin{array}{l} d = 28.5" \Leftarrow \text{controls} \\ 12d_b = 12" \end{array} \right.$

§9.7.3.4 ACI 318-14



Shear Design

$V_u = 81^k \quad V_n = V_u/\phi = 81^k/0.75 = 108^k$

$V_u @ d = 66.6^k \quad V_n = 66.6^k/0.75 = 88.8^k$

$V_c = 50.5^k \quad \phi V_c = 37.9^k$

$V_n \leq \frac{1}{2} V_c = 25.3^k$ No need for shear Reinf.

V_n @ midspan $= 29^k/0.75 = 38.7^k > 25.3^k \Rightarrow$ Shear Ref. needed for the whole beam.

↑ from Seismic

$V_n \leq V_c = 50.5^k$ Provide Min. Shear reinf.

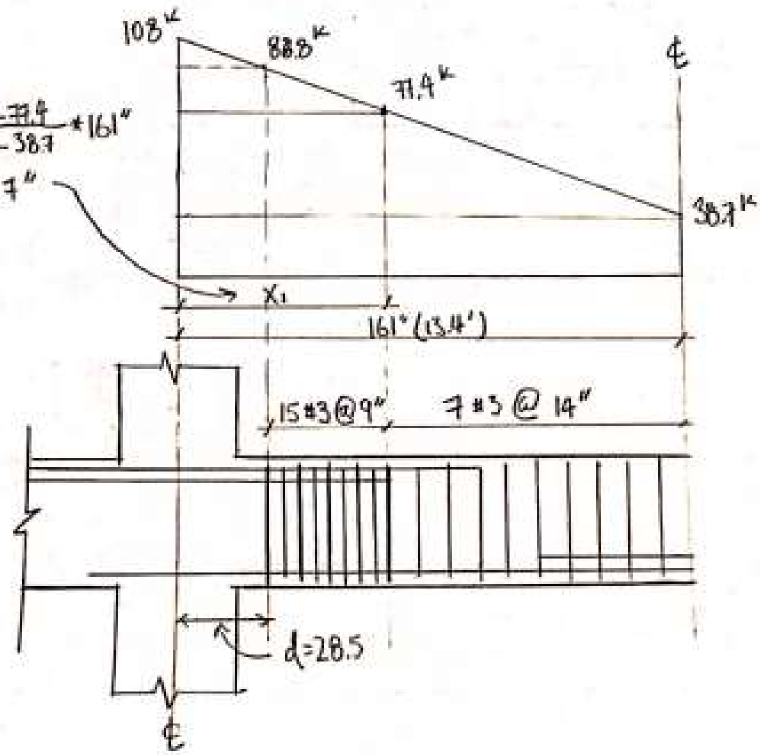
$$S_{max} = \frac{A_v f_{yt}}{0.75\sqrt{f'_c}\, b_w} = \frac{0.22\,(60000)}{0.75\sqrt{4000}\,(14)} = 19.9''$$

$$S_{max} = \frac{A_v f_{yt}}{50\, b_w} = \frac{0.22 \times 60000}{50\,(14)} = \underline{14''} \Leftarrow \text{controls} \leq \frac{d}{2} = 14.5'' \text{ or } 24''$$

$$V_s = \frac{A_v f_{yt}\, d}{S_{max}} = \frac{0.22 \times 60 \times 28.5}{14} = 26.9^k$$

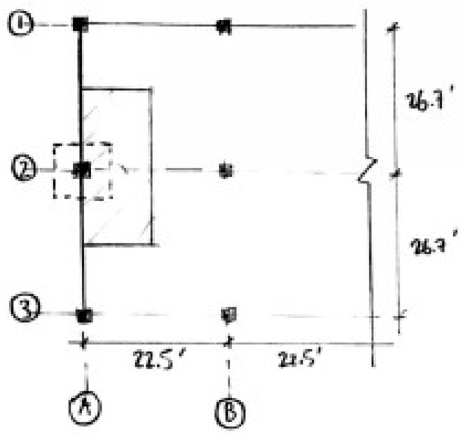
$$V_n = V_c + V_s = 50.5 + 26.9 = 77.4^k$$

$$X_1 = \frac{108 - 77.4}{108 - 38.7} \times 161'' = 71.7''$$



2.4 Column Design

*Approximate Analysis for Interior Column (A2)



$A = 26.7 \times 22.5/2 = 300\ ft^2$

$K_{LL} = 4$

$P_D = (152psf)(300)(5) + \underbrace{(700 + 1056 + 2 \times 1176 + 1368)}_{\text{wall load}}(26.7)$

$= 374^k$

$L_r = 20\ psf$

$L_F = 75\ psf$

$K_{LL}A = 1200\ ft^2$

$LL_r = 75 \times \max \left| \begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(1200)}} \end{array} \right. = 35\ psf$

$P_r = (20psf)(300) = 6^k$

$P_L = 4(35)(300) = 42^k$

Gravity Case:

$1.4D = 1.4(374) = 524^k$ ← controls

$1.2D + 1.6L = 516^k$

1.4 D

$P_u = 524^k$

$M_u = 0^{1k}$

1.2D + 0.5L + 1.0E

$P_u = 1.2D + 0.5L$

$= 1.2(374) + 0.5(42)$

$= 470^k$

$M_u = 480^{1k}$

Column Design

1.2D + 0.5L + 1.0E

$P_u = 470^k$; $M_u = 480^{'k}$; $V_u = 56.5^k$

Select materials & trial size

$f'_c = 4000\,psi$ & $f_y = 60\,ksi$ $\rho_g \approx 0.015$

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

$$\geq \frac{470}{0.4(4+60\times0.015)}$$

$$\geq 240\,in^2$$

choose a square column 16" x 16"

$$e = \frac{M_u}{P_u} = \frac{480(12)}{470} = 12.26''$$

$$\frac{e}{h} = \frac{12.26}{16} = 0.766$$

$$\gamma = \frac{16-5}{16} = 0.69$$

$$\frac{\phi P_n}{A_g} = \frac{P_u}{A_g} = \frac{470}{16\times16} = 1.84\,ksi$$

$$\frac{\phi M_n}{A_g h} = \frac{M_u}{A_g h} = \frac{480\times12}{16^3} = 1.41\,ksi$$ (can't find ρ_g in chart)

choose 20" x 20"

$$\frac{e}{h} = \frac{12.26}{20} = 0.613 > 0.2$$ ⇒ Two sided layout is more efficient.

$$\gamma = \frac{20-5}{20} = 0.75$$

$$\frac{\phi P_n}{A_g} = \frac{470}{20\times20} = 1.18\,ksi$$

$$\frac{\phi M_n}{A_g h} = \frac{480\times12}{20^3} = 0.72\,ksi$$

From chart ($\gamma = 0.75$),

$\Rightarrow \rho_g = 0.035$

$A_{st} = \rho_g A_g = 0.035(20)(20) = 14\ in^2$

use 12 #10 (4 on each side) $\Rightarrow A_{st} = 12(1.27) = 15.24\ in^2$

check max. load capacity

$\phi P_n = 0.8\phi[0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$

$= 0.8(0.65)[0.85(4)(400 - 15.24) + 60(15.24)]$

$= 1156^k > 524^k$ ok

Design of Splice

$l_d = \left(\frac{f_y \psi_t \psi_e}{20 \lambda \sqrt{f'_c}}\right) d_b$

$= \left(\frac{60000 \times 1 \times 1}{20 \times 1 \times \sqrt{4000}}\right)(1.27)$

$= 60.2''$

Splice length $= 1.3 l_d = 78.3''$

Select ties: (#3)

spacing: $S \le 16 d_b = 16(1.27) = 20.3''$

$S \le 48 d_t = 48(0.375) = 18''$ ⇐ controls

$S \le 20''$

Check Shear

$V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right) \lambda \sqrt{f'_c}\, b_w d$

$= 2\left(1 + \frac{450000}{2000(400)}\right)\sqrt{4000}\,(20)(17.5)$

$= 69.2^k > V_u = 56.5^k$

$0.5\phi V_c = 0.5(0.75)(69.2) = 26^k$

$\frac{1}{2}\phi V_c \le V_u \le \phi V_c$

$V_s = \frac{V_u}{\phi} - V_c = \frac{56.5}{0.75} - 69.2 = 6.13^k$

$S = \frac{A_v f_{yt} d}{V_s} = \frac{0.22(60)(17.5)}{6.13} = 37.7" > 18"$

$\therefore$ use #3 ties @ 18" O.C.

<u>check slenderness</u>

$\frac{K l_u}{r} \leq 34 + 12 \frac{M_1}{M_2} \leq 40$

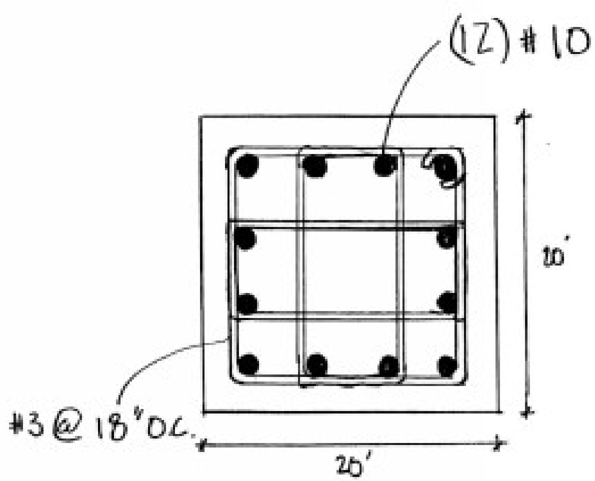
$K = 0.67$ $\quad M_1/M_2 = -0.5$

$r = 0.3h$

$\frac{K l_u}{r} = \frac{0.67(17 \times 12)}{0.3(20)} = 23$ &

$34 - 12(0.5) = 28$

$23 \leq 28 \leq 40$ $\quad \therefore$ Good. Neglect Slenderness.



Appendix 3: Calculations for Lateral Design and Analysis

TABLE 6-1 MINIMUM VENTILATION RATES IN BREATHING ZONE *(continued)*
(This table is not valid in isolation; it must be used in conjunction with the accompanying notes.)

Occupancy Category	People Outdoor Air Rate R_p		Area Outdoor Air Rate R_a		Notes	Default Values: Occupant Density (see Note 4)	Default Values: Combined Outdoor Air Rate (see Note 5)		Air Class
	cfm/person	L/s·person	cfm/ft²	L/s·m²		#/1000 ft² or #/100 m²	cfm/person	L/s·person	
Office Buildings									
Office space	5	2.5	0.06	0.3		5	17	8.5	1
Reception areas	5	2.5	0.06	0.3		30	7	3.5	1
Telephone/data entry	5	2.5	0.06	0.3		60	6	3.0	1
Main entry lobbies	5	2.5	0.06	0.3		10	11	5.5	1
Miscellaneous Spaces									
Bank vaults/safe deposit	5	2.5	0.06	0.3		5	17	8.5	2
Computer (not printing)	5	2.5	0.06	0.3		4	20	10.0	1
Electrical equipment rooms	–	–	0.06	0.3	B	–			1
Elevator machine rooms	–	–	0.12	0.6	B	–			1
Pharmacy (prep. area)	5	2.5	0.18	0.9		10	23	11.5	2
Photo studios	5	2.5	0.12	0.6		10	17	8.5	1
Shipping/receiving	–	–	0.12	0.6	B	–			1
Telephone closets	–	–	0.00	0.0		–			1
Transportation waiting	7.5	3.8	0.06	0.3		100	8	4.1	1
Warehouses	–	–	0.06	0.3	B	–			2
Public Assembly Spaces									
Auditorium seating area	5	2.5	0.06	0.3		150	5	2.7	1
Places of religious worship	5	2.5	0.06	0.3		120	6	2.8	1
Courtrooms	5	2.5	0.06	0.3		70	6	2.9	1
Legislative chambers	5	2.5	0.06	0.3		50	6	3.1	1
Libraries	5	2.5	0.12	0.6		10	17	8.5	1
Lobbies	5	2.5	0.06	0.3		150	5	2.7	1
Museums (children's)	7.5	3.8	0.12	0.6		40	11	5.3	1
Museums/galleries	7.5	3.8	0.06	0.3		40	9	4.6	1
Residential									
Dwelling unit	5	2.5	0.06	0.3	F,G	F			1
Common corridors	–	–	0.06	0.3					1
Retail									
Sales (except as below)	7.5	3.8	0.12	0.6		15	16	7.8	2
Mall common areas	7.5	3.8	0.06	0.3		40	9	4.6	1
Barbershop	7.5	3.8	0.06	0.3		25	10	5.0	2
Beauty and nail salons	20	10	0.12	0.6		25	25	12.4	2
Pet shops (animal areas)	7.5	3.8	0.18	0.9		10	26	12.8	2
Supermarket	7.5	3.8	0.06	0.3		8	15	7.6	1
Coin-operated laundries	7.5	3.8	0.06	0.3		20	11	5.3	2

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TABLE 6-2 Zone Air Distribution Effectiveness

Air Distribution Configuration	E_z
Ceiling supply of cool air.	1.0
Ceiling supply of warm air and floor return.	1.0
Ceiling supply of warm air 15°F (8°C) or more above space temperature and ceiling return.	0.8
Ceiling supply of warm air less than 15°F (8°C) above space temperature and ceiling return provided that the 150 fpm (0.8 m/s) supply air jet reaches to within 4.5 ft (1.4 m) of floor level. ***Note:*** For lower velocity supply air, $E_z = 0.8$.	1.0
Floor supply of cool air and ceiling return provided that the 150 fpm (0.8 m/s) supply jet reaches 4.5 ft (1.4 m) or more above the floor. ***Note:*** Most underfloor air distribution systems comply with this proviso.	1.0
Floor supply of cool air and ceiling return, provided low-velocity displacement ventilation achieves unidirectional flow and thermal stratification.	1.2
Floor supply of warm air and floor return.	1.0
Floor supply of warm air and ceiling return.	0.7
Makeup supply drawn in on the opposite side of the room from the exhaust and/or return.	0.8
Makeup supply drawn in near to the exhaust and/or return location.	0.5

1. "Cool air" is air cooler than space temperature.
2. "Warm air" is air warmer than space temperature.
3. "Ceiling" includes any point above the *breathing zone*.
4. "Floor" includes any point below the *breathing zone*.
5. As an alternative to using the above values, E_z may be regarded as equal to air change effectiveness determined in accordance with ANSI/ASHRAE Standard 129[16] for all air distribution configurations except unidirectional flow.

TABLE 6-3 System Ventilation Efficiency

Max (Z_p)	E_v
≤0.15	1.0
≤0.25	0.9
≤0.35	0.8
≤0.45	0.7
≤0.55	0.6
>0.55	Use Appendix A

1. "Max Z_p" refers to the largest value of Z_p, calculated using Equation 6-5, among all the zones served by the system.
2. For values of Z_p between 0.15 and 0.55, one may determine the corresponding value of E_v by interpolating the values in the table.
3. The values of E_v in this table are based on a 0.15 average outdoor air fraction for the system (i.e., the ratio of the *uncorrected outdoor air intake* V_{ou} to the total *zone primary airflow* for all the zones served by the air handler). For systems with higher values of the average outdoor air fraction, this table may result in unrealistically low values of E_v and the use of Appendix A may yield more practical results.