Project Report on

"Vertical Extension of Civil Engineering Building"



By

MD. MASUD AZAD	**1901012**
NAZMUL HASAN	**1901014**
MD ABU SAYED	**1901049**
IQBAL MAHMUD	**1901119**

A project report submitted in partial fulfilment of the requirements for the degree of BACHELOR of SCIENCE in Civil Engineering

Department of Civil Engineering

CHITTAGONG UNIVERSITY OF ENGINEERING AND TECHNOLOY

JULY 2025

Declaration

We hereby declare that the work contained in this thesis has not been previously submitted to meet requirements for an award at this or any other higher education institution. To the best of our knowledge and belief, the project contains no material previously published or written by another person except where due reference is cited. Furthermore, the report complies with PLAGIARISM and ACADEMIC INTEGRITY regulation of CUET.

Name: MD. MASUD AZAD

ID: 1901012

Name: NAZMUL HASAN

ID: 1901014

Name: MD ABU SAYED

ID: 1901049

Name: IQBAL MAHMUD

ID: 1901119

Department of Civil Engineering

Chittagong University of Engineering & Technology (CUET)

Copyright ©MD MASUD AZAD, NAZMUL HASAN, MD ABU SAYED, IQBAL MAHMUD; 2025

This work may not be copied without permission of the author(s) or Chittagong University of Engineering & Technology.

Approval by the Supervisor

This is to certify that **MD. MASUD AZAD**, **NAZMUL HASAN, MD ABU SAYED**, **IQBAL MAHMUD** has carried out this research work under my supervision, and that he has fulfilled the relevant Academic Ordinance of the Chittagong University of Engineering and Technology, so that he is qualified to submit the following Thesis in the application for the degree of BACHELOR of SCIENCE in Civil Engineering. Furthermore, the Thesis complies with the PLAGIARISM and ACADEMIC INTEGRITY regulation of CUET.

Dr. Muhammad Abdur Rahman Bhuiyan
Professor
Department of Civil Engineering
Chittagong University of Engineering & Technology

Table of Contents

Table of Contents i

Chapter 1: INTRODUCTION 9

1.1 General: 9

1.2 Background 9

1.3 Aims and Objectives 11

1.4 Significance, Scope and Definitions 11

1.5 Justification for Vertical Expansion 12

1.6 Selection of Composite Structure 12

Chapter 2: LITERATURE REVIEW 14

2.1 Introduction 14

2.2 Vertical Expansion of Existing Structures 14

2.3 Composite Structural Systems 14

2.4 Structural Analysis Software 15

2.5 Seismic Considerations in Vertical Expansion 15

2.6 Availability of Steel Materials in BangladesH 15

2.7 Analysis and design of steel building system 15

Chapter 3: METHODOLOGY 18

3.1 Problem Identification 20

3.2 Possible solutionS 21

3.3 Strategies 22

3.4 Engineering/Design constraints 23

Chapter 4: SEISMIC EVALUATION OF EXITING BUILDING 25

4.1 Description of the building 25
4.1.1 Characteristics of the Building 25
4.1.2 Outline of Building 25

4.2 Calculation of gravity load 26
4.2.1 Calculation of Unit Weight of Structure 27
4.2.2 Summary of Floor Weight of Structure 27

4.3 Example of Axial Force on Column(C2) 28

4.4 Structural Weight of each Floor and Cumulative Weight 29
4.4.1 Vertical Distribution of lateral Forces according to BNBC 2020 30

4.5 Seismic demand index of structure Iso, Ctu, Sd 30
4.5.1 Ultimate Strength of Column 32
4.5.2 Material Properties 32
4.5.3 Reinforcing Bar 32
4.5.4 Ultimate Flexural Strength 32

4.5.5 Ultimate Shear Strength 33
4.6 Calculation of Column Ductility Index F 36
4.6.1 Ductility Index 37
4.7 Summary of ductility indices in X-direction 40
4.8 Effective Strength Factor 43
4.9 Strength Index 44
4.10 Seismic index of structure Is and Ctu, Sd 46
Chapter 5: ANALYSIS AND DESIGN OF PROPOSED STEEL BUILDING SYSTEM AS PER BNBC 2020 49
5.1 Existing Plan 49
5.1.1 Typical Floor Plan 49
5.1.2 Ground Floor Plan 50
5.1.3 Column Schedule 50
5.2 Proposed Layout Plan 51
5.3 Analysis 51
5.4 Reaction analysis at base plan 52
5.5 Deformed shape for Load 52
5.6 Shell forced for Load 53
5.7 Maximum story displacement for earthquake 54
5.8 Maximum story displacement for wind 55
5.9 Maximum story Drift for earthquake 56
5.10 Story Shear for earthquake 57
5.11 Overturning moment for earthquake 58
5.12 Story stiffness for earthquake 59
5.13 Torsion for Loads 60
5.14 Moment, Shear and Axial force for Dead Load 60
Chapter 6: DETAILING 63
6.1 Detailing of Foundation 64
6.2 Base Plate Layout 66
6.3 Column Layout 68
6.4 Reinforcement Detail 72
6.5 Connection Detail 74
6.6 Beam Layout 77
6.7 Deck slab layout 82
6.8 Elevation Section 85
Chapter 7: RESULT AND DISCUSSION 88
7.1 General 88
7.2 Shear wall design results 88

7.3 Shear wall design preferences 89
7.4 Spandrel design chart from etabs 90
7.5 Pier longitudinal reinforcing result 91
7.6 Pier longitudinal reinforcing result 91
7.7 P-M colors/shear ratio values 93
7.8 P-M ratio colours 93
7.9 Cont. plate areas/doubler plate thickness 94
7.10 Steel beam/column capacity ratios 95
7.11 P-M colors/beam end shear force 96
7.12 Steel frame design check result details 97
7.13 Strength summary 99
7.14 Composite column design check result details 101
7.15 Maximum Story Displacement Check 103
7.16 Drift Ratio Check 104
7.17 Torsional Irregularity Check 105
7.18 Soft Story and Extreme Soft Story Check 107
7.19 P- Delta Check 109
7.20 Structural Checks and Stability 110
7.21 Discussion 110
7.21.1 Story Displacement and Drift Behaviour 110
7.21.2 Structural Stability and Soft Story Control 111
7.21.3 Code Compliance and Safety Verification 111
7.21.4 Seismic Safety Overview 111
7.21.5 Summary of Structural Performance 111

Chapter 8: CONCLUSIONS 113

8.1 General 113
8.2 Limitation of the Work: 114
8.2.1 Foundation Construction Without Interference 114
8.2.2 Time-of-Day Restrictions 114
8.2.3 Logistical and Access Limitations 114
8.2.4 Cost Implications 114
8.3 Future scope of the work 115
8.3.1 Structural Framing System 115
8.3.2 Foundation 115
8.3.3 Floor Systems 115
8.3.4 Selection of structural materials 115

Bibliography 117

Appendices 118

Finite Element Modelling (FEM) 118
1.1 Initialization of country codes and standard codes 118
1.2 Grid points creation & Structural Generation 119

1.3 Defining the properties 120
1.4 Supports assigning and load defining 124

List of Figures

Fig. No. **Figure Caption** **Page No.**

Figure 1.1: Sattelite view (Source: Google maps) 10
Figure 4.1 Existing Building Plan 26
Figure 4.2 Flowchart to calculate Ductility 36
Figure 5.1 Typical Floor Plan 49
Figure 5.2 Ground Floor Plan 50
Figure 5.3 Column Schedule (Black) 50
Figure 5.4 Proposed Layout Plan 51
Figure 5.5 Run for Analysis 51
Figure 5.6 Reaction on Base level for Dead Load 52
Figure 5.7 Deformed shape for Dead Load (Contours) 53
Figure 5.8 Shell forces for Dead Load (Contours) 53
Figure 5.9 Maximum Story Displacement for Ex 54
Figure 5.10 Maximum Story Displacement for Ey 54
Figure 5.11 Maximum Story Displacement for Wx 55
Figure 5.12 Maximum Story Displacement for Wy 55
Figure 5.13 Maximum Story Drift for Ex 56
Figure 5.14 Maximum Story Displacement for Ey 56
Figure 5.15 Story Shears Ex 57
Figure 5.16 Story Shear Ey 57
Figure 5.17 Overturning moment for Ex 58
Figure 5.18 Overturning moment for Ey 58
Figure 5.19 Story stiffness for Ex 59
Figure 5.20 Story stiffness for Ey 59
Figure 5.21 Torsion for Dead Load 60
Figure 5.22 Moment For Dead Load 61
Figure 5.23 Shear for dead load 61
Figure 5.24 Axial Force for dead load 62
Figure 6.1 Reinforcement Details of MAT Foundation. 64
Figure 6.2 Typical Section of MAT Foundation. 65
Figure 6.3 Base Plate Layout Plan 66
Figure 6.4 Detail on Base Plate. 67
Figure 6.5 Column Layout Plan (upto 1st Floor Level) 68
Figure 6.6 Column Layout Plan (1st Floor Level to 2nd Floor Level) 69

Figure 6.7 Column Layout Plan (2nd Floor Level to 4th Floor Level) 70
Figure 6.8 Column Layout Plan (4th Floor Level to Above Level) 71
Figure 6.9 Details of all Column Layout (C1, C2, C3) 72
Figure 6.10 Reinforcement Detail of Shear Wall 73
Figure 6.11 Connection Detail of Column (C1 to C2, C2 to C2) 74
Figure 6.12 Connection Detail of Column (C3 to C2, C3 to C3) 75
Figure 6.13 Connection Detail of Column to Beam and Beam to Beam 76
Figure 6.14 1st Floor Steel Beam Layout Plan 77
Figure 6.15 2nd Floor Steel Beam Layout Plan 78
Figure 6.16 3rd & 4th Floor Steel Beam Layout Plan 79
Figure 6.17 5th to Roof Floor Steel Beam Layout Plan 80
Figure 6.18 Schedule of Steel Beam 81
Figure 6.19 1st Floor Deck Slab Layout Plan 83
Figure 6.20 2nd Floor Deck Slab Layout Plan 83
Figure 6.21 Deck Slab Detail 84
Figure 6.22 Section on Grid 01 & 18 85
Figure 6.23 Section on Grid 02-05 & 14-17 86
Figure 6.24 Section on Grid 09 & 10 87
Figure 7.1 Shear Wall Design Preferences 89
Figure 7.2 Shear Wall Design Result Criteria 90
Figure 7.3 ETABS Shear Wall Design 91
Figure 7.4 P-M Ratio value in elevation 92
Figure 7.5 P-M Colors/Shear Ratio values 93
Figure 7.6 P-M Ratio Colors 94
Figure 7.7 Cont. Plate Areas/DOUBLER Plate thickness in elevation 94
Figure 7.8 Steel Beam/Column Capacity Ratios 95
Figure 7.9 P-M Colors/Beam end Shear force 96
Figure 7.10 Values in preference 97
Figure 7.11 Deflection Details 98
Figure 7.12 Strength Summary 100
Figure 7.13 Composite Column Design Preferences 101
Figure 7.14 ETABS Composite Column Design 103
Figure 0.1 Required Code setup 118
Figure 0.2 Grid System Chart 119
Figure 0.3 Elevation data of structure 119

Figure 0.4 Defining material properties....120
Figure 0.5 Selecting of Beam Section 121
Figure 0.6 Selection of Column Section....121
Figure 0.7 Selection of Deck Sectio 122
Figure 0.8 Assign Deck Section 122
Figure 0.9 Assign Beam and Column Section....123
Figure 0.10 Defining Load Pattern 124
Figure 0.11 Assigning Loads....125
Figure 0.12 Defining Seismic Loads 126
Figure 0.13 Defining Wind Loads....126
Figure 0.14 Modeling 128

List of Tables

Table No. **Table Caption** **Page No.**

Table 1.1 Basic information....11
Table 1.2 Justification For Vertical Expansion 12
Table 4.1 Basic Details 25
Table 4.2 Gravity Loads(Typical Floor)....27
Table 4.3 Gravity Loads(Roof)....27
Table 4.4 Summary of Floor weight of the structure....27
Table 4.5 Column Index 28
Table 4.6 Axial Force on Column(C2) 28
Table 4.7 Axial Force on Column 29
Table 4.8 Structural weight of each floor and cumulative weight....29
Table 4.9 Vertical Distribution of lateral Forces according to BNBC 2020 30
Table 4.10 Summary of strength of Column 35
Table 4.11 Summary of ductility indices in X-direction: 40
Table 4.12 Summary of effective strength factor in X-direction....43
Table 4.13 Strength Index....45
Table 4.14 Basic Seismic Index in X and Y direction....46
Table 4.15 Result of 2nd Level Screening....48
Table 7.1 Maximum Story Displacement Check....104
Table 7.2 Drift ration check for Ex in X direction....105
Table 7.3 Drift ratio checks for Ex in Y direction 105
Table 7.4 Torsional Irregularity Check....106
Table 0.1 Load Combinations....127

Chapter 1: INTRODUCTION

1.1 GENERAL:

The Chittagong University of Engineering and Technology (CUET) is facing significant pressure due to rapid growth in student intake and academic functions. The **Civil Engineering building**, built in 1968 as a three-story RCC structure, now struggles to support the expanding requirements of the department. With limited land for horizontal development and no feasible option for relocation, a sustainable and technically feasible alternative is required.

This project proposes a **seven-story vertical extension**, constructed using a **composite structural system**, built independently over and around the existing RCC building, resulting in a **total of ten stories**. Importantly, this structure will **not impose any additional load** on the existing building. All vertical loads will be transferred directly to a **new mat foundation**, ensuring full structural independence.

The design, analysis, and validation of the proposed structure were carried out using modern software tools such as **ETABS, AutoCAD, and IDEA StatiCa**, following the guidelines of **BNBC 2020** and LRFD standards. Seismic performance, soft story behavior, and lateral stiffness were also carefully addressed.

1.2 BACKGROUND

The Civil Engineering Department at CUET currently has an intake of 132 students per batch, across four academic years, totalling over 528 undergraduate students. The department also supports several postgraduate students and hosts numerous laboratories, offices for faculty, conference rooms, and lounges.

Additionally, the building accommodates the Water Resources Engineering (WRE) department, which has 30 students per batch across four years—bringing in 120 more

students. Combined with the faculty offices, research spaces, and common facilities, the current building is under constant spatial stress.

Due to increasing enrolment, new labs, faculty recruitments, and research activities, space demand is rising rapidly. However:

There is no available land for horizontal expansion.

The building is centrally located, and relocation is impractical.

Continuous academic operation prohibits demolition or major interruption.

Thus, vertical expansion was determined as the only feasible solution.

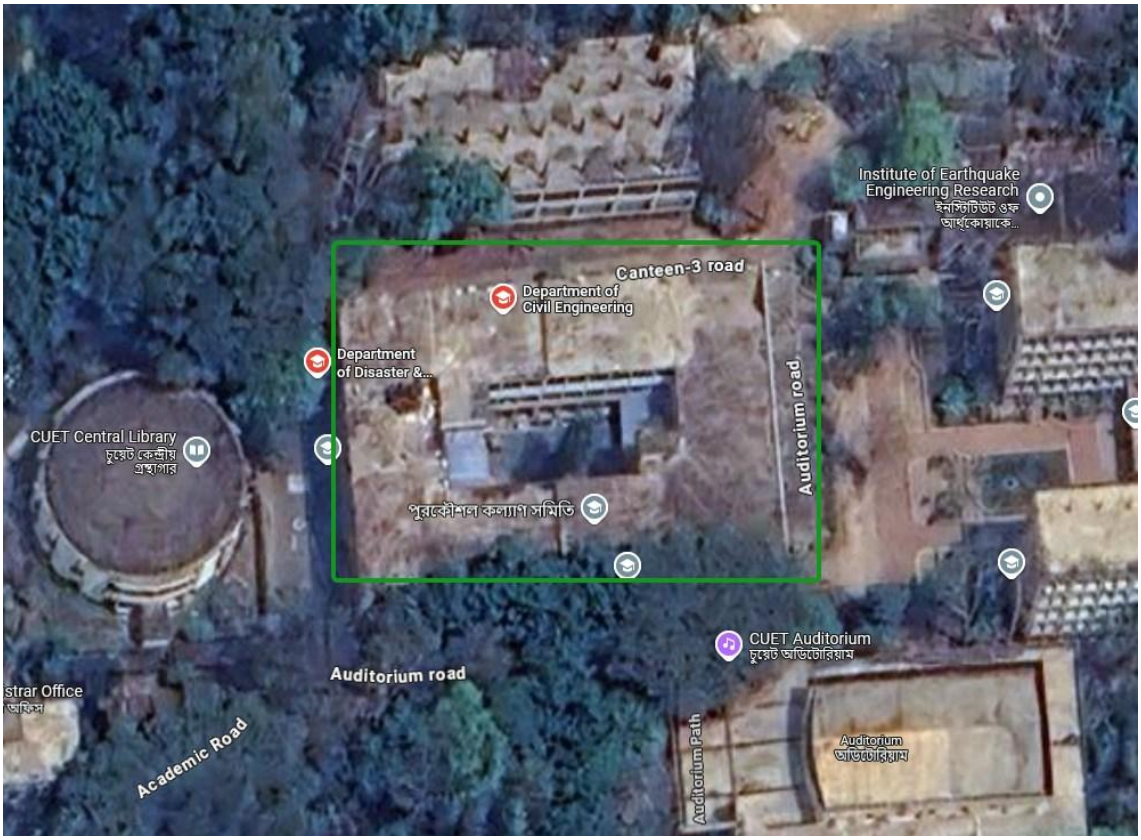


Figure 1.1: Sattelite view (Source: Google maps)

Table 1.1 Basic information

	Description
Structural system	Composite column and steel beam
Floor Area	28110 sq-ft
Number of stories	7 (1st floor is 39 ft)
Foundation type	Mat foundation (Assumed)
Construction materials	Concrete with stone chips for foundation, composite concrete filled steel HSS for column, Steel beams, Steel and concrete for deck slab

1.3 AIMS AND OBJECTIVES

The main objectives of this research were:

- To expand the capacity of an existing RCC structure without imposing any structural load or alteration.
- To ensure continuous operational use of the existing building during construction.

1.4 SIGNIFICANCE, SCOPE AND DEFINITIONS

This thesis presents a novel solution to a growing urban challenge—how to expand upward without interrupting existing building use or compromising safety. The strategy of introducing a structurally independent vertical extension over an operational structure can be a model for future vertical densification in urban Bangladesh and beyond. It contributes practical design methodology, construction feasibility analysis, and software-based validation for engineers, architects, and policy makers seeking sustainable and non-disruptive expansion techniques.

1.5 JUSTIFICATION FOR VERTICAL EXPANSION

Table 1.2 Justification For Vertical Expansion

CRITERIA	DESCRIPTION
STUDENT LOAD	CIVIL: 132×4 = 528 STUDENTS; WRE: 30×4 = 120 STUDENTS; PLUS, FACULTY AND RESEARCH STAFF.
SPACE NEEDS	MULTIPLE LARGE LABORATORIES, CLASSROOMS, FACULTY OFFICES, LOUNGES, AND MEETING SPACES.
NO HORIZONTAL EXPANSION	SURROUNDED BY OTHER BUILDINGS; ROADS.
OPERATIONAL BUILDING	CANNOT DEMOLISH OR DISTURB THE FUNCTIONING OF THE CURRENT STRUCTURE.
VERTICAL SOLUTION	ADDS 7 STORIES WITHOUT ALTERING OR LOADING THE EXISTING FRAME, SUPPORTED ON NEW MAT FOUNDATION.

1.6 SELECTION OF COMPOSITE STRUCTURE

After comparative analysis, the composite structural system was chosen due to its:

- Structural efficiency in high-rise, vertical expansions.
- Seismic resilience, meeting all BNBC 2020 safety checks.
- Construction speed and modularity, ideal for congested sites.
- Lighter weight, which reduces foundation pressure compared to rcc.
- Independence from existing building, ensuring no load or interaction with the old rcc frame.

This makes composite construction the optimal solution for this vertical expansion project at cuet.

Chapter 2: LITERATURE REVIEW

2.1 INTRODUCTION

The vertical expansion of existing structures has become an increasingly viable solution to address space constraints in educational institutions. This chapter reviews pertinent literature on vertical extensions, composite structural systems, the utilization of structural analysis software, seismic considerations, and the availability of steel materials in Bangladesh, all of which are integral to the proposed expansion of the Civil Engineering Department building at CUET.

2.2 VERTICAL EXPANSION OF EXISTING STRUCTURES

Vertical extensions involve adding new stories atop existing buildings, offering a sustainable approach to increasing usable space without the need for new land acquisition. Gillott [1] emphasizes that vertical extensions can effectively reduce resource consumption and carbon emissions associated with new constructions. However, such projects require careful assessment of the existing structure's capacity to support additional loads.

2.3 COMPOSITE STRUCTURAL SYSTEMS

Composite structures, combining steel and concrete, offer enhanced strength, durability, and efficiency. These systems are particularly advantageous for vertical expansions due to their high strength-to-weight ratio and ease of construction. A study by Alam et al. [2] highlights the growing importance of steel–concrete composite structures in Bangladesh, noting their benefits over traditional RCC structures, including better seismic performance and cost-effectiveness.

2.4 STRUCTURAL ANALYSIS SOFTWARE

Advanced structural analysis software plays a crucial role in designing and evaluating complex structures. ETABS, developed by Computers and Structures, Inc., is widely recognized for its capabilities in modelling, analysis, and design of multi-story buildings. According to a comparison guide [3], ETABS stands out for its user-friendly interface and comprehensive analysis features, making it a preferred choice among structural engineers.

2.5 SEISMIC CONSIDERATIONS IN VERTICAL EXPANSION

Bangladesh's location in a moderate seismic zone necessitates careful seismic evaluation of building expansions. Composite structures have demonstrated favourable seismic performance due to their ductility and energy dissipation properties. Katsimpini et al. [4] conducted an in-depth analysis of the seismic performance characteristics of steel–concrete composite structures, underscoring their resilience in earthquake-prone areas.

2.6 AVAILABILITY OF STEEL MATERIALS IN BANGLADESH

The availability and utilization of steel materials are critical factors in the feasibility of composite structures. Alam et al. [2] note that while the use of steel–concrete composites is gaining traction in Bangladesh, challenges remain in terms of production capacity and public perception. Addressing these challenges is essential for the broader adoption of composite systems in the country's construction industry.

2.7 ANALYSIS AND DESIGN OF STEEL BUILDING SYSTEM

The analysis and design of steel building systems—particularly for vertical extensions—have become increasingly relevant in modern structural engineering. In projects where the goal is to add new stories above existing buildings without imposing additional loads or requiring structural intervention in the original construction,

independent composite steel systems have proven to be a reliable and efficient solution [1].

Composite structures, which combine steel beams and columns with concrete slabs, offer a favorable balance of strength, ductility, and weight efficiency. These systems are particularly suited for multi-story vertical additions, as they reduce dead load compared to traditional RCC structures and allow for quicker and less intrusive construction [2]. Their adaptability to irregular footprints also makes them ideal for retrofitting over existing buildings that remain operational during construction.

To design and analyze such complex structures, modern software tools have become essential. ETABS, developed by Computers and Structures, Inc., is one of the most widely used programs for high-rise steel and composite structures. Its ability to perform static and dynamic analyses, including P-Delta effects, story drift checks, torsional irregularity detection, and modal response evaluations, makes it highly suitable for vertical expansion projects [3].

Shear walls and other lateral load-resisting systems also play a critical role in ensuring structural safety and performance under seismic conditions. According to Katsimpini et al. [4], composite steel systems integrated with shear walls provide superior energy dissipation and lateral stiffness, reducing the risk of soft-story mechanisms, which are commonly observed in tall, slender buildings lacking adequate lateral support.

In Bangladesh, the implementation of such systems must adhere to national codes such as the Bangladesh National Building Code (BNBC 2020) [6], which provides guidelines for design load combinations, structural detailing, and performance requirements. Additionally, the Seismic Evaluation Manual (2nd Edition) published under the BSPP project offers critical tools for assessing seismic resilience, especially for retrofitting and expansion scenarios [5].

Recent studies in the local context have also emphasized the growing role of steel–concrete composite structures. Alam et al. [2] highlighted the material's increasing adoption in Bangladesh, noting that it meets both economic and structural performance demands, particularly in educational and institutional infrastructure.

In summary, the literature supports the viability of independently constructed composite steel systems for vertical expansion projects. These systems, when designed using advanced software like ETABS and verified against national codes and seismic guidelines, provide a technically sound solution for increasing usable space without compromising safety or operational continuity.

Chapter 3: METHODOLOGY

Based on the provided project report, the methodology for the vertical extension of the Civil Engineering building at Chittagong University of Engineering & Technology (CUET) followed a systematic process of structural design, analysis, and verification. The core objective was to design a structurally independent seven-story composite steel building over an existing, operational three-story RCC structure. This approach ensured that no additional loads were imposed on the original 1968 building, allowing it to remain in continuous use throughout the construction process.

The methodology can be broken down into the following key stages:

1. Conceptualization and Layout

The project began by establishing the need for vertical expansion due to increasing student enrolment at CUET. Since demolition of the existing, functional building was not feasible, the concept of a standalone vertical extension was adopted. A modified layout plan was developed for a new seven-story structure that would be built around and over the existing three-story building, creating a combined ten-story facility. The new structure was designed to be completely independent, with steel columns positioned to bypass the old frame and transfer all loads to a new foundation.

2. Structural Modelling and Software Utilization

The entire seven-story extension was modelled as a composite steel building using the software ETABS (Version 18.0.1). The modelling process included:

Initialization: The model was set up using built-in settings for U.S. Customary units and design codes such as AISC 360-10 for steel and ACI 318-11 for concrete.

Grid and Story Definition: A detailed grid system and story elevation data were created to define the building's geometry. The ground floor was set at a height of 39 ft to clear the existing structure, with subsequent floors having a 12 ft height.

Component Assignment: Structural components were assigned within the model, including steel beams, composite columns, and a filled deck slab system.

In addition to ETABS for primary analysis, AutoCAD was used for creating detailed structural drawings, and IDEA StatiCa was used for connection design.

3. Material and Section Property Definition

Specific material properties were defined in the ETABS model to ensure the analysis accurately reflected real-world behaviour. The key materials specified were:

Concrete: Compressive strength (f'c) of 4000 psi was used for the mat foundation, composite deck slabs, and the concrete fill in the columns.

Structural Steel: A36 grade steel, with a yield strength of 36,000 psi, was specified for all steel beams and columns.

Reinforcing Steel: Rebar with a yield strength of 60,000 psi was used for the foundation and deck slabs.

The structural system consisted of composite columns (concrete-filled steel HSS), steel beams, and a composite deck slab.

4. Load Definition and Analysis

The structure was analysed according to the Bangladesh National Building Code (BNBC 2020) and the Load and Resistance Factor Design (LRFD) methodology.

Load Patterns: Various load types were defined, including dead load, live load, floor finish, partition walls, and lateral loads for earthquake and wind.

Seismic and Wind Loads: Seismic loads were defined based on ASCE 7-10, with specific parameters set for the site class and spectral accelerations. Wind loads were also defined according to ASCE 7-10, with a specified wind speed of 178.955 mph.

Load Combinations: A comprehensive set of 31 load combinations was applied to the model to simulate worst-case scenarios as per code requirements.

Structural Analysis: The model was run for analysis to determine reactions at the base, deformed shapes, shell stresses, and member forces.

5. Design Verification and Stability Checks

Following the analysis, a series of checks were performed to verify the structural stability and safety of the design. These included:

Displacement and Drift: Maximum story displacement and drift were checked for both earthquake and wind load to ensure they were within the allowable limits of BNBC 2020.

Soft Story Check: The model was checked for soft story and extreme soft story conditions. Initial analysis indicated a potential soft story issue, which was mitigated by incorporating shear walls in the building's longer direction.

Torsional Irregularity: The building was checked for torsional irregularity and found to be regular.

P-Delta Effects: The design accounted for P-Delta effects, which were checked and passed.

Seismic Safety: A seismic evaluation confirmed that the structure's capacity exceeded the seismic demand (Iso = 0.234).

6. Detailing and Documentation

The final stage of the methodology involved creating detailed drawings for construction. This included layouts and reinforcement details for the mat foundation, base plates, columns, beams, connections, and deck slabs, all generated using AutoCAD based on the ETABS analysis.

3.1 PROBLEM IDENTIFICATION

The primary problem with the existing Civil Engineering building at CUET is its inability to meet the growing demand for academic space due to increasing student enrolment.

Key issues identified in the report include:

• Insufficient Space The university's expansion has created a need for more infrastructural and academic space, which the current three-story building cannot accommodate.

• Infeasibility of Traditional Expansion The building, constructed in 1968, is structurally sound and remains in continuous academic use. This makes demolition, reconstruction, or disruptive structural alterations unfeasible options for expansion.

• Operational Continuity A major constraint is the need for the existing building to remain fully operational during any expansion project.

3.2 POSSIBLE SOLUTIONS

Structural System Comparison for CUET Project

Criteria	RCC Structure	Steel Structure	Composite Structure (Selected)
Seismic Performance	Poor (brittle under stress)	Good	Excellent (high ductility and energy dissipation)
Construction Over Existing Building	Difficult (wet construction, vibrations)	Easier than RCC	Easiest (minimal disruption, dry assembly)
Foundation Load	High	Moderate	Lower (due to lighter superstructure)
Structural Independence	Hard to achieve	Feasible	Easily achievable (ideal for vertical extension)
Suitability for CUET Project	Not Suitable	Partially Suitable	Most Suitable

The proposed solution to the space shortage at the CUET Civil Engineering building is a vertical extension that is structurally independent of the existing building.

The key features of this solution are:

• *Independent Seven-Story Structure:* The design involves constructing a new seven-story steel-composite building that stands on its own, positioned around and above the existing three-story RCC structure. This creates a combined ten-story facility without altering the original building.

• *No Load on Existing Building:* The new extension is designed to be entirely separate, transferring all its loads to a new foundation. This ensures no structural stress is added to the old RCC frame.

• *Continuous Operation:* This method allows the existing building to remain fully operational and in continuous use during the construction of the extension.

• *Modern Materials and Design:* The new structure utilizes a composite system with concrete-filled steel HSS for columns, steel beams, and a composite deck slab. The

entire project is designed and analysed using advanced software like ETABS and complies with the Bangladesh National Building Code (BNBC 2020).

• *New Foundation:* The extension is supported by its own independent mat foundation, which is designed to carry all loads from the new seven-story structure.

3.3 STRATEGIES

Based on the provided reports, the following strategies were employed to implement the vertical extension of the Civil Engineering building at CUET:

• **Structural Independence:** The core strategy was to design a seven-story steel-composite structure that is completely independent of the existing building. The new structure is built around and above the old one, ensuring no structural load is transferred to the original 1968 frame. This approach allows the existing building to remain fully operational during construction.

• **Advanced Modelling and Analysis:** The project utilized a suite of modern software tools for design and verification.

- ETABS (Version 18.0.1) was used for the primary structural modeling and analysis.
- AutoCAD was used for the detailed structural drawings.
- IDEA StatiCa was employed for the design of the connections.

• **Code-Compliant Design:** The entire design and analysis process strictly adhered to the Bangladesh National Building Code (BNBC 2020) and adopted the Load and Resistance Factor Design (LRFD) methodology to ensure safety and compliance. A comprehensive set of 31 load combinations was evaluated.

• **Specific Material Selection**: A composite structural system was strategically chosen for the new building. This included:

- Composite columns made of concrete-filled steel HSS (Hollow Structural Sections).
- Steel beams.
- A composite deck slab system made of steel and concrete.
- A new, independent mat foundation to support the entire load of the extension.

• **Lateral Stability Management:** To ensure the stability of the new ten-story structure, specific measures were taken. When initial analysis revealed a potential "soft story" issue, the design was modified to include shear walls along the building's longer direction15. This strategy effectively increased lateral stiffness and resolved the structural irregularity.

• **Comprehensive Safety Verification:** The design was validated through a series of rigorous checks to ensure structural integrity. These included evaluations of story displacement and drift, P-Delta effects, and torsional irregularity, all of which passed within acceptable safety margins. A seismic evaluation was also performed for the entire site to confirm safety.

3.4 ENGINEERING/DESIGN CONSTRAINTS

Based on the provided reports, the engineering and design process for the vertical extension of the CUET Civil Engineering building was governed by several key constraints:

• **Preservation of the Existing Structure:** The most significant constraint was that the existing three-story RCC building could not be demolished or structurally altered. It had to remain fully operational and in continuous use throughout the project.

• **Structural Independence:** A direct consequence of the primary constraint, the new seven-story extension had to be designed as a completely independent structure. It was required to transfer all of its own loads to a new foundation without imposing any stress on the old RCC frame.

• **Regulatory Compliance:** The entire design had to fully comply with the standards of the Bangladesh National Building Code (BNBC 2020). This included meeting all requirements for lateral stability, such as story displacement and drift limits.

• **Spatial and Geometric Limitations**: The new building had to be designed to fit precisely around and above the existing structure. This meant placing new columns in open areas around the original building's perimeter and within its central courtyard. The first floor of the new extension had to be elevated to 39 feet to clear the existing three-story building.

• **Lateral Stability Requirements:** The new, taller structure presented a stability challenge. Initial analysis identified a potential "soft story" behaviour, which created a

constraint to redesign the system to ensure lateral stiffness and eliminate this vulnerability. This was resolved by incorporating shear walls.

• **Functional and Economic Constraints**: The project report acknowledges several limitations that acted as constraints on its real-world viability:

- The proposed trial structure is unserviceable as it lacks stairs and a lift.
- The cost of this specific solution was noted as "extremely high which is unrealistic".
- The report anticipates that workers and engineers would face significant complications during the construction phase.

Chapter 4: SEISMIC EVALUATION OF EXITING BUILDING

4.1 DESCRIPTION OF THE BUILDING

4.1.1 Characteristics of the Building

Some of the important characteristics of the building are:

1) Typical classroom with frame structure constructed in 1968.

2) No earthquake resisting design was performed.

3) No grade beams exist.

4) Low strength concrete was used.

4.1.2 Outline of Building

Table 4.1 Basic Details

Usage	Classroom Building
Number of Storey	Three
Structural type	R.C. Framed Structure
Foundation type	Shallow Foundation
Year of design	1967
Year of Construction	1968

4.2 CALCULATION OF GRAVITY LOAD

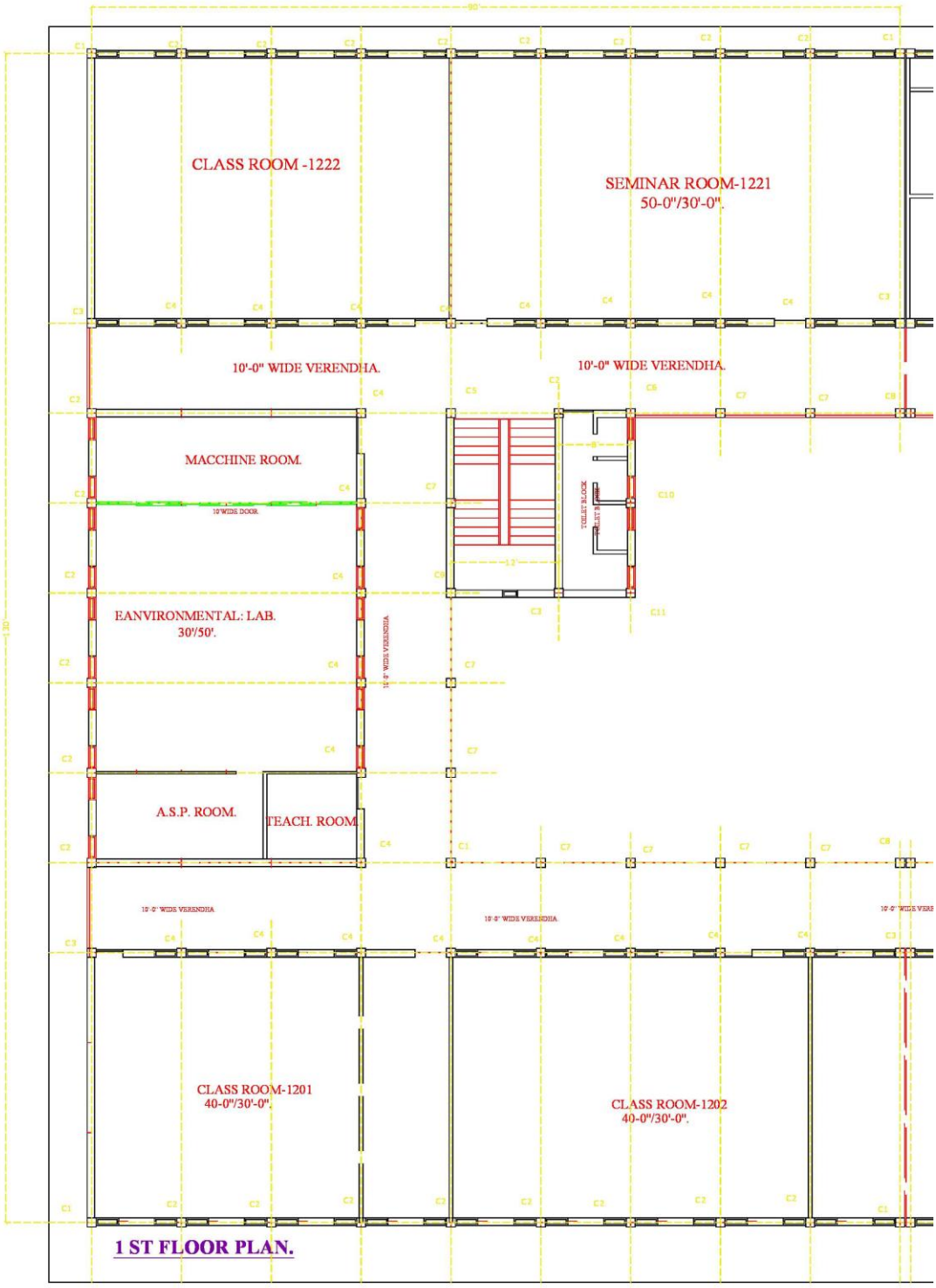


Figure 4.1 Existing Building Plan

4.2.1 Calculation of Unit Weight of Structure

The whole building is divided into two parts joined with an expansion joint. West part and east part. We did the calculation for seismic analysis for west part.

Table 4.2 Gravity Loads(Typical Floor)

Unit Area Weight (Typical Floor)		
Load type	Load	Unit
Live Load	2	kN/m^2
Dead Load	3.54	kN/m^2
Seismic Weight	2*25%=0.5	kN/m^2
Total Weight	7	kN/m^2

Reference: Table 6.2.2(Floor-150mm) and Table 6.2.3(Classroom) BNBC 2020

Table 4.3 Gravity Loads(Roof)

Unit Area Weight (Roof)		
Load type	Load	Unit
Live Load	1	kN/m^2
Dead Load	3.54	kN/m^2
Seismic Weight	1*25%=0.25	kN/m^2
Total Weight	5	kN/m^2

Reference: Table 6.2.2(Floor-150mm) and Table 6.2.3(Roof) BNBC 2020

4.2.2 Summary of Floor Weight of Structure

Table 4.4 Summary of Floor weight of the structure

Structural Weight				
Storey	Floor Area(ft^2)	Floor Area(m^2)	Floor Weight W_i(kN)	$\sum W_i$(kN)
3	9600	891.9	4459.3	4459.3
2	9600	891.9	6243.1	10702.4
1	9600	891.9	6243.1	16945.5

Table 4.5 Column Index

Column Index	No of Column	Area per column
C1	5	75
C2	23	150
C3	5	100
C4	22	200
C5	1	140
C6	1	65
C7	9	50
C8	2	25
C9	1	110
C10	1	40
C11	1	20

4.3 EXAMPLE OF AXIAL FORCE ON COLUMN(C2)

Table 4.6 Axial Force on Column(C2)

Col Index	Floor	Area(m^2) ×Unit Area Weight(kN/m^2)	Floor Weight(kN)	Axial Force(kN)
C2	3	320.5×5	1602.577	1602.577
	2	320.5×7	2243.608	3846.186
	1	320.5×7	2243.608	6089.794

Table 4.7 Axial Force on Column

Floor	3	2	1
	Load (Roof) 5kN/m^2	Load (Slab)=7 kN/m^2	Load (Slab)=7 kN/m^2
C1	174.193	418.064	661.934
C2	1602.577	3846.186	6089.794
C3	232.258	557.418	882.579
C4	2043.867	4905.281	7766.694
C5	65.032	156.077	247.122
C6	30.193	72.464	114.735
C7	209.032	501.676	794.321
C8	23.226	55.742	88.258
C9	51.097	122.632	194.167
C10	18.581	44.593	70.606
C11	9.290	22.297	35.303

4.4 STRUCTURAL WEIGHT OF EACH FLOOR AND CUMULATIVE WEIGHT

Table 4.8 Structural weight of each floor and cumulative weight

Floor	Floor weight (kN) W_i	Cumulative weight (kN) Summation of Wi	Vertical distribution factor $\frac{n+1}{n+i}$
3	4459.346	4459.346	$\frac{3+1}{3+3}$ =0.67
2	6243.084	10702.430	$\frac{3+1}{3+2}$=0.8
1	6243.084	16945.514	$\frac{3+1}{3+1}$=1

4.4.1 Vertical Distribution of lateral Forces according to BNBC 2020

Table 4.9 Vertical Distribution of lateral Forces according to BNBC 2020

k=1 (Structural Period, T=0.34)			Vertical Distribution of lateral forces according to BNBC 2020						
Story	h(m)	hi(m)	Wi	∑Wi	wi*hi^k	Fx (Horizontal Earthquake Load)	Vi(Seismic Shear)	Ci(Seismic Shear Coefficient)	Vertical distribution factor
3	3.048	9.144	4459.34	4459.346	40776.2	1029.879	1029.8	0.23	1.49
2	3.048	6.096	6243.08	10702.43	65242.0	1647.807	2677.6	0.25	1.25
1	3.048	3.048	6243.08	16945.51	51649.9	1304.514	3982.2	0.23	1
				∑Wi*hi^k =157668.202		V=3982.2			

4.5 SEISMIC DEMAND INDEX OF STRUCTURE ISO, CTU, SD

$I_{S0} = \frac{2}{3} ZIC_{SE}$

Where,

Z: Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC 2020

I: Structure importance factor, as defined in Section 2.5.5. I of BNBC 2020

C_{SE}: Normalized acceleration response spectrum for seismic evaluation, which is a function of structure (building) period and soil type (site class).

The building is located in Chittagong, the earthquake zone is Zone III, 0.28. The building is an educational institution building.

Z= 0.28 (Zone III)

I= 1.25 (Occupancy Category III)

S= SC

Calculation of C_S (BNBC 6.2.35)

$C_S = 0.8 \times S \times \frac{1+T}{T_B(2.5\eta-1)}$ for $0 \leq T \leq T_B$

$C_S = 0.8 \times 2.5 \times S \times \eta$ for $T_B \leq T \leq T_C$

$C_S = 0.8 \times 2.5 \times S \times \eta \times \frac{T_C}{T}$ $for T_C \leq T \leq T_D$

$C_S = 0.8 \times 2.5 \times S \times \eta \times \frac{T_C \times T_D}{T^2}$ $for T_D \leq T \leq 4\ sec$

h_n: Height of building (m)

S: Soil factor which depends on site class

T: $0.0466.(ho)^{0.9}$

T_B: Lower limit of the period of the constant spectral acceleration branch.

T_C: Upper limit of the period of the constant spectral acceleration branch

T_D: Lower limit of the period of the constant spectral displacement branch

ξ= Viscous damping ratio= 4%

η: Damping correction factor as a function of damping with a reference value of

η =1 for 5% viscous damping.

$h_n = 9.144\text{m}(30\text{ ft})$

S= 1.15(SC)

$T = 0.0466.(h_n)^{0.9} = 0.0466.(9.144)^{0.9} = 0.3425$ s

T_D: 2 s

T_C: 0.6 s

T_B: 0.2 s

η: $\sqrt{\frac{10}{5+4}} = 1.05$ BNBC: (6.2.36)

As, $T_B < T < T_C$

C_S: $C_S = 2.5 \times 1.15 \times 1.05 = 3.01$

$S_a = \frac{2\ ZI}{3\ R} C_s$ (BNBC 6.2.34)

$S_a = \frac{2 \times 0.28 \times 1.25}{3 \times 3} \times 3.01 = 0.234$ [This value also denoted by C_{SE}]

According to the second level seismic capacity evaluation, the seismic capacity of a structure is evaluated based on the performance of vertical elements on the assumption that beams are strong enough and will not fail.

4.5.1 Ultimate Strength of Column

GF C2 is shown as an example:

Calculation procedure of CI column at ground floor is shown as below: Assessment Direction: X

Location: Grid

Floor: 2^{nd} floor

Column: C2

Storey height: h= 10'-0" =3.048m

Clear height: h_0=10'-6" =9.5'= 2.9m

Standard clear height of column, H_0=2.9m

4.5.2 Material Properties

Compressive strength of existing concrete (F_c) = 13.8 N/mm^2(2000 psi)

Tensile strength of existing rebar (σ_y)= 275 N/mm^2 (40 Grade)

Axial force N = 265 kN

Column size, bD = 300mm × 300mm

4.5.3 Reinforcing Bar

Total reinforcing bars, 4-Φ13, a_g= 4*129=516 mm^2

Tensile reinforcing bars, 2-Φ13, a_t= 2*129=258 mm^2

Tie Φ10, a_w= 71 mm^2

4.5.4 Ultimate Flexural Strength

$0.4bDF_C$= 0.4×300×300×13.8

= 496 kN

$N_{MAX}= a_g\sigma_y+bDF_C$

= 516 ×275+300 ×300 ×13.8

=1357 kN

$N_{MIN}= a_g\sigma_y$

= 516 ×275 N

=142 kN

According to Eq-3.8 of BSPP [5] manual v2,

For $0.4bDFC \geq N > 0$

$$M_{u2} = \left(0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_C}\right)\right) \text{N·mm}$$

$$M_{u2} = \left(0.8 \cdot 258 \cdot 275 \cdot 300 + 0.5 \cdot 265 \cdot 300 \cdot \left(1 - \frac{265}{300 \cdot 300 \cdot 13.8}\right)\right) \text{N·mm}$$

= 48296720.0 N·mm

= 48.3 kN·m

$Q_{mu}=\frac{2Mu}{ho}$

$= \frac{2\times48.3}{2.09}$ kN

=33.31 kN

4.5.5 Ultimate Shear Strength

Shear strength of column according to Eq-3.7 [Modified supplementary provision 1.1 ultimate strength Al. 1-2]

$$Qsu = k_r\{\frac{0.053Pt^{0.23}(18 + Fc)}{\frac{M}{Q.d} + 0.12} + 0.85\sqrt{p_w.\sigma_{wy}} + 0.1\sigma_0\}.b.j$$

[Supplementary Provisions 1 Column with Modified to consider low Strength Concrete]

k_r: reduction factor of low strength concrete

Kr=0.244+0.056σb

Kr=0.244+0.056*13.8=1

p_t= Tensile reinforcement ratio (%)

$$pt = \frac{at}{bD} \times 100\%$$

$$pt = \frac{258}{300 \times 300} \times 100\%$$

$$= 0.3\%$$

Aw= Cross sectional area of shear reinforcement= 71 mm^2

S= Spacing of shear reinforcement (In case tie is 90-degree hook: s= 2xs (Twice spacing length)).

s= 2*8 inch= 400 mm

Pw= Shear reinforcement ratio,

Pw= 0.012

$s\sigma_{wy}$ = Yield strength of shear reinforcement bar= 275 N/mm^2

σ_0 =Axial stress in column=$\frac{N}{bD} = \frac{265}{300 \times 300} =$ 2.94 N/mm^2< 8 N/mm^2

D= Effective depth of column

M/Q= Shear span length. Default value of $h_0/2$= 2.9/2=1.45 m= 1450 mm

H_0=2.9 m

$$\frac{M}{Q.d} = \frac{1450}{300 - 50} = 5.57 > 3$$

$$\frac{M}{Q.d} = 3$$

$$Qsu = 1.\{\frac{0.053 \times 0.3^{0.23}(18 + 13.5)}{3 + 0.12} + 0.85\sqrt{0.012 \times 2.94} + 0.1 \times 2.94\}.300 \times 240$$

= 161551.34 N

= 161.55 kN

3.3.2 Strength of Column

Qmu=26.6 kN, Qsu=161.6 kN

Qmu<Qsu

So, Flexural failure type. Qu= 26.6 kN [C2 Column]

Table 4.10 Summary of strength of Column

Strength of Members								
Col	Story	N(kN)	h0/D	Mu(kN-m)	Qmu(kN)	Qsu(kN)	Q(kN)	Failure Type
C1	3	34.839	9.667	22.104	12.195	142.765	12.195	Flexural
	2	83.613	9.667	28.707	15.838	146.667	15.838	Flexural
	1	132.387	9.667	34.722	19.156	150.569	19.156	Flexural
C2	3	69.677	9.667	26.880	14.830	145.552	14.830	Flexural
	2	167.225	9.667	38.659	21.328	153.356	21.328	Flexural
	1	264.774	9.667	48.089	26.531	161.160	26.531	Flexural
C3	3	46.452	9.667	23.729	13.091	143.694	13.091	Flexural
	2	111.484	9.667	32.216	17.774	148.897	17.774	Flexural
	1	176.516	9.667	39.659	21.880	154.100	21.880	Flexural
C4	3	92.903	9.667	29.898	16.495	147.411	16.495	Flexural
	2	222.967	9.667	44.336	24.460	157.816	24.460	Flexural
	1	353.032	9.667	54.596	30.121	168.221	30.121	Flexural
C5	3	65.032	9.667	26.261	14.488	145.181	14.488	Flexural
	2	156.077	9.667	37.432	20.651	152.464	20.651	Flexural
	1	247.122	9.667	46.557	25.685	159.748	25.685	Flexural
C6	3	30.193	9.667	21.444	11.831	142.394	11.831	Flexural
	2	72.464	9.667	27.249	15.033	145.775	15.033	Flexural
	1	114.735	9.667	32.613	17.993	149.157	17.993	Flexural
C7	3	23.226	9.667	20.445	11.280	141.836	11.280	Flexural
	2	55.742	9.667	25.006	13.796	144.438	13.796	Flexural
	1	88.258	9.667	29.305	16.168	147.039	16.168	Flexural
C8	3	11.613	9.667	18.753	10.346	140.907	10.346	Flexural
	2	27.871	9.667	21.113	11.648	142.208	11.648	Flexural
	1	44.129	9.667	23.407	12.914	143.509	12.914	Flexural
C9	3	51.097	9.667	24.370	13.445	144.066	13.445	Flexural
	2	122.632	9.667	33.566	18.518	149.789	18.518	Flexural
	1	194.167	9.667	41.499	22.895	155.512	22.895	Flexural
C10	3	18.581	9.667	19.772	10.908	141.465	10.908	Flexural
	2	44.593	9.667	23.472	12.949	143.546	12.949	Flexural
	1	70.606	9.667	27.003	14.898	145.627	14.898	Flexural
C11	3	9.290	9.667	18.411	10.157	140.722	10.157	Flexural
	2	22.297	9.667	20.311	11.206	141.762	11.206	Flexural
	1	35.303	9.667	22.170	12.231	142.803	12.231	Flexural

4.6 CALCULATION OF COLUMN DUCTILITY INDEX F

The calculation example of F-index is as shown below. To calculate ductility index of each vertical member following flowchart is as follows

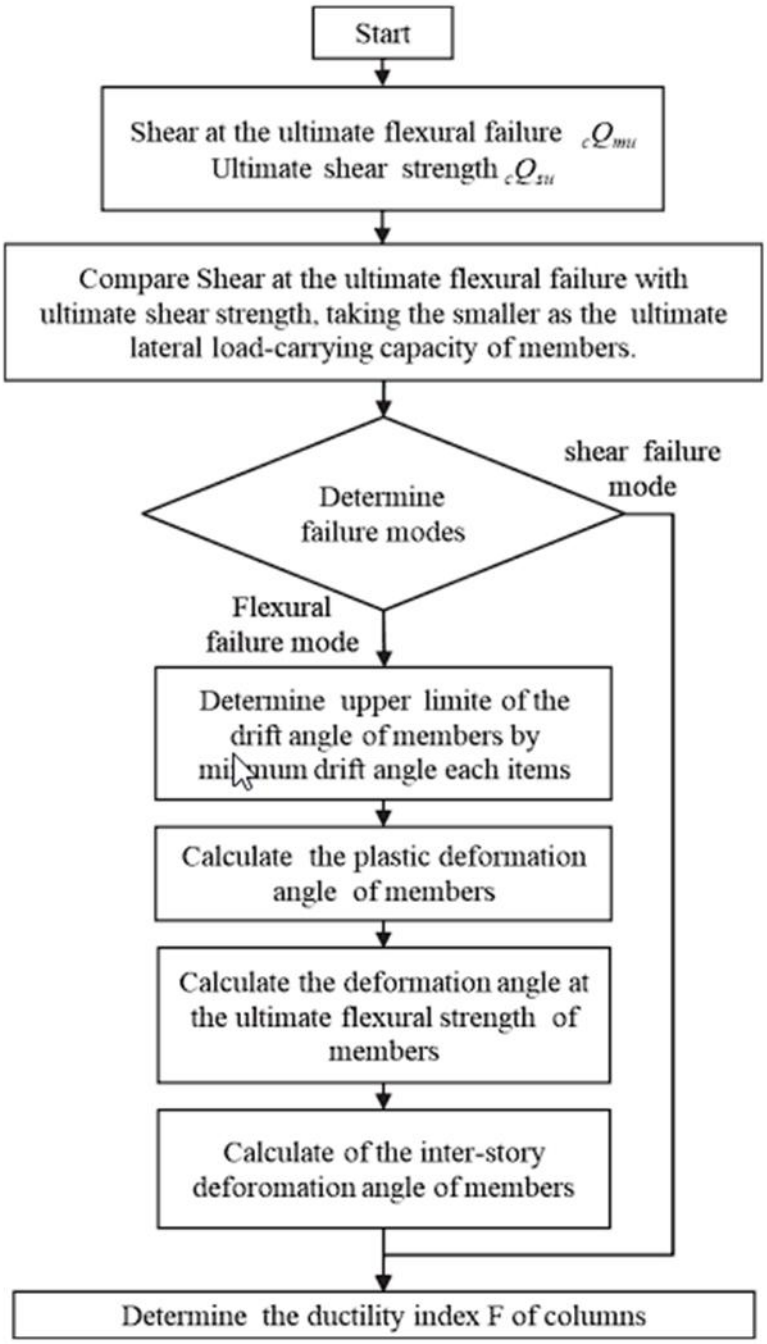


Figure 4.2 Flowchart to calculate Ductility

4.6.1 Ductility Index

Calculation procedure of ductility index of Cl column at ground floor in X-direction is shown as below:

4.6.1.1 Upper limit of deflection angle of flexural column cRmax

cRmax= minimum of { cRmax(n), cRmax(s), cRmax(t) , cRmax(b) , cRmax(h) , cRmax(p) , cRmax(a) , cRmax(b)}

[cRmax(n)] Upper limit of the deformation angle of the flexural column determined by the axial force

$$\eta = \frac{165 \times 10^3}{300 \times 300 \times 13.8}$$

= 0.2134

pw=0.0006 ≥0.2 % with 90 degree hook

cRmax(n)=cR30 $(\frac{cR_{250}}{cR_{30}})^{\eta}$ ≤cR30 for other case $\eta = 0.2134$

$= \frac{1}{30} * (\frac{\frac{1}{250}}{\frac{1}{30}})^{0.2134} \leq \frac{1}{30}$

$=0.0212 \leq \frac{1}{30}$

[cRmax(s)] Upper limit of deformation angle of flexural column determined by shear force

cRmax(s)= $\frac{min\ of\ (Qmu,Qsu)}{bj}$

$= \frac{min\ of\ (14.83\ ,145.55)}{bj}$

=0. 00021

$$\frac{c\tau u}{Fc} = \frac{0.00021}{13.8} = 0.000001 \neq \geq 0.2$$

cRmax(s)=cR30 =$\frac{1}{30}$

[cRmax(t)] Upper limit of deformation angle of flexural column determined by tensile pt= 0.3% < 1.3%

cRmax(t)=$\frac{1}{30}$

[cRmax(b)] Upper limit of deförmation angle of flexural column determined by spacing of hoop

$$\frac{s}{d} = \frac{400}{10} > 8$$

cRmax(b)=$\frac{1}{50}$

[cRmax(h)] Upper limit of deformation angle of flexural column determined by clear height

$$\frac{ho}{D} = \frac{2900}{300} = 9.67 > 2$$

cRmax(h)=$\frac{1}{30}$

[cRmax(p)] Upper limit of deformation angle of flexural column determined by poor beam-column joint for low strength concrete

cRmax(p)=$\frac{1}{100}$

[cRmax(a)] Upper limit of deformation angle of flexural column determined by anchor length for low strength concrete
Fc>=13.5
cRmax(a)=$\frac{1}{100}$

[cRmax(B)] Upper limit of deformation angle of flexural column determined by clear height. In this building there is no standing wall which makes $\frac{h0}{D} \geq$

cRmax(3)=$\frac{1}{30}$

Therefore, cRmax=$\frac{1}{100}$

4.6.1.2 Yield deflection angle of the Column cRmy

$$\frac{\boldsymbol{h0}}{\boldsymbol{D}} = \frac{\mathbf{2900}}{\mathbf{300}} = \mathbf{9.67} \geq \mathbf{3}$$

cRmy=$\frac{1}{150} \leq \frac{1}{100} = cRmax$

cRmy= $\frac{1}{150}$

4.6.1.3 Inter storey drift angle at the flexural yielding of column Rmy

$$\frac{h0}{H} = \frac{2900}{2900} = 1$$

Rmy=$\frac{ho}{H}.cRmy$

= $\mathbf{1} \times \frac{\mathbf{1}}{\mathbf{150}} = \frac{\mathbf{1}}{\mathbf{150}}$

4.6.1.4 Plastic deformation angle Of column cRmp

S = s= spacing of ties = 200 mm > 100 mm

Q= 1.1

$$cRmp = 10\frac{cQsu}{cQmu - q} \times \frac{1}{cRmy} \geq 0$$

=$10\frac{161.16}{26.531-1.1} \times \frac{1}{150}$

= 0.42

4.6.1.5 Deformation angle at ultimate flexural strength of column cRmu

$$\boldsymbol{cRmu = \frac{1}{150} + 0.42}$$

= 0.4267 >Rmax $\frac{1}{150}$

cRmu=$\frac{1}{150}$

Inter storey drift angle at the ultimate flexural strength of column Rmu

$$\frac{ho}{H} = \frac{2900}{2900} = 1$$

Rmu= $1 * \frac{1}{150} \geq \frac{1}{250}$

Rmu=$\frac{1}{150}$

Ductility Index

Since, Rmu>Ry= $\frac{1}{150}$

$$F = \frac{\sqrt{2 * \frac{Rmy}{Ry} - 1}}{0.75(1+0.05\frac{Rmu}{Ry})} \leq 3.2$$

$$F = \frac{1}{0.75(1 + 0.05\frac{\frac{1}{150}}{\frac{1}{150}})}$$

$$= 1.27 \leq 3.2$$

4.7 SUMMARY OF DUCTILITY INDICES IN X-DIRECTION

Table 4.11 Summary of ductility indices in X-direction:

Column	Story	Qmu(kN)	Qsu(kN)	Q(kN)	Failure	cRmax(n)	cRmax(s)	cRmax(t)	cRmax(b)	cRmax(h)	cRmax(P)	cRmax(a)	cRmax(B)	cRmax	cRmy	Rmy	cRmp	cRmu	F	Column	Story
C1	3	12.195	142.765	12.195	Flexural	0.031	0.033	0.033	0.020	0.033	0.010	0.010	0.033	0.010	0.007	0.007	0.515	0.010	1.754	C1	3
	2	15.838	146.667	15.838	Flexural	0.029	0.004	0.033	0.020	0.033	0.010	0.010	0.033	0.004	0.004	0.004	0.398	0.004	1.270		2
	1	19.156	150.569	19.156	Flexural	0.027	0.004	0.033	0.020	0.033	0.010	0.010	0.033	0.004	0.004	0.004	0.334	0.004	1.270		1
C2	3	14.830	145.552	14.830	Flexural	0.030	0.004	0.033	0.020	0.033	0.010	0.010	0.033	0.004	0.004	0.004	0.424	0.004	1.270	C2	3

C6	C5			C4			C3				
3	1	2	3	1	2	3	1	2	3	1	2
11.831	25.685	20.651	14.488	30.121	24.460	16.495	21.880	17.774	13.091	26.531	21.328
142.394	159.748	152.464	145.181	168.221	157.816	147.411	154.100	148.897	143.694	161.160	153.356
11.831	25.685	20.651	14.488	30.121	24.460	16.495	21.880	17.774	13.091	26.531	21.328
Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural
0.032	0.022	0.026	0.030	0.018	0.023	0.028	0.025	0.028	0.031	0.021	0.025
0.033	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.033	0.004	0.004
0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033
0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020
0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033
0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033
0.010	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.010	0.004	0.004
0.007	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.007	0.004	0.004
0.007	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.007	0.004	0.004
0.531	0.260	0.312	0.434	0.232	0.270	0.383	0.297	0.357	0.479	0.253	0.303
0.010	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.010	0.004	0.004
1.754	1.270	1.270	1.270	1.270	1.270	1.270	1.270	1.270	1.754	1.270	1.270
C6			C5			C4			C3		
3	1	2	3	1	2	3	1	2	3	1	2

C10	C9			C8			C7				
3	1	2	3	1	2	3	1	2	3	1	2
10.908	22.895	18.518	13.445	12.914	11.648	10.346	16.168	13.796	11.280	17.993	15.033
141.465	155.512	149.789	144.066	143.509	142.208	140.907	147.039	144.438	141.836	149.157	145.775
10.908	22.895	18.518	13.445	12.914	11.648	10.346	16.168	13.796	11.280	17.993	15.033
Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural	Flexural
0.032	0.024	0.027	0.031	0.031	0.032	0.033	0.029	0.030	0.032	0.027	0.029
0.033	0.004	0.004	0.033	0.033	0.033	0.033	0.004	0.033	0.033	0.004	0.004
0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033
0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020
0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033
0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033	0.033
0.010	0.004	0.004	0.010	0.010	0.010	0.010	0.004	0.010	0.010	0.004	0.004
0.007	0.004	0.004	0.007	0.007	0.007	0.007	0.004	0.007	0.007	0.004	0.004
0.007	0.004	0.004	0.007	0.007	0.007	0.007	0.004	0.007	0.007	0.004	0.004
0.577	0.285	0.344	0.467	0.486	0.539	0.610	0.390	0.455	0.557	0.353	0.418
0.010	0.004	0.004	0.010	0.010	0.010	0.010	0.004	0.010	0.010	0.004	0.004
1.754	1.270	1.270	1.754	1.754	1.754	1.754	1.270	1.754	1.754	1.270	1.270
C10			C9			C8			C7		
3	1	2	3	1	2	3	1	2	3	1	2

	2	12.949	143.546	12.949	Flexural	0.031	0.033	0.033	0.020	0.033	0.010	0.010	0.033	0.010	0.007	0.007	0.485	0.010	1.754		2
	1	14.898	145.627	14.898	Flexural	0.030	0.004	0.033	0.020	0.033	0.010	0.010	0.033	0.004	0.004	0.004	0.422	0.004	1.270		1
C11	3	10.157	140.722	10.157	Flexural	0.033	0.033	0.033	0.020	0.033	0.010	0.010	0.033	0.010	0.007	0.007	0.621	0.010	1.754	C11	3
C11	2	11.206	141.762	11.206	Flexural	0.032	0.033	0.033	0.020	0.033	0.010	0.010	0.033	0.010	0.007	0.007	0.561	0.010	1.754		2
C11	1	12.231	142.803	12.231	Flexural	0.031	0.033	0.033	1.020	0.033	0.010	0.010	1.033	0.010	0.007	0.007	0.513	0.010	1.754		1

4.8 EFFECTIVE STRENGTH FACTOR

Effective strength factor indicates the ratio of restoring force at the ultimate deflection angle of the first group (R1) to ultimate strength. The practical calculation method is as follows. Effective strength factor is calculated using the ratio of Rito is the deflection angle at yielding. Effective strength factors for the column on the ground floor are calculated as follows. In this floor, the failure mode of all columns is flexural and minimum ductility index I.

The effective strength factor for flexural column, $\alpha m = 0.3 + 0.7 \frac{R1}{Rmy}$

Table 4.12 Summary of effective strength factor in X-direction

Storey		Col. ID	Qmu(kN)	Qsu(kN)	Failure Mode	Rmy	F	1.27<=F
3		C1	12.195	142.765	Flexural	0.007	1.754	1.000
		C2	14.830	145.552	Flexural	0.004	1.270	1.000
		C3	13.091	143.694	Flexural	0.007	1.754	1.000
		C4	16.495	147.411	Flexural	0.004	1.270	1.000
		C5	14.488	145.181	Flexural	0.004	1.270	1.000
		C6	11.831	142.394	Flexural	0.007	1.754	1.000
		C7	11.280	141.836	Flexural	0.007	1.754	1.000
		C8	10.346	140.907	Flexural	0.007	1.754	1.000

		C9	13.445	144.066	Flexural	0.007	1.754	1.000
		C10	10.908	141.465	Flexural	0.007	1.754	1.000
		C11	10.157	140.722	Flexural	0.007	1.754	1.000
2		C1	15.838	146.667	Flexural	0.004	1.270	1.000
		C2	21.328	153.356	Flexural	0.004	1.270	1.000
		C3	17.774	148.897	Flexural	0.004	1.270	1.000
		C4	24.460	157.816	Flexural	0.004	1.270	1.000
		C5	20.651	152.464	Flexural	0.004	1.270	1.000
		C6	15.033	145.775	Flexural	0.004	1.270	1.000
		C7	13.796	144.438	Flexural	0.007	1.754	1.000
		C8	11.648	142.208	Flexural	0.007	1.754	1.000
		C9	18.518	149.789	Flexural	0.004	1.270	1.000
		C10	12.949	143.546	Flexural	0.007	1.754	1.000
		C11	11.206	141.762	Flexural	0.007	1.754	1.000
1		C1	19.156	150.569	Flexural	0.004	1.270	1.000
		C2	26.531	161.160	Flexural	0.004	1.270	1.000
		C3	21.880	154.100	Flexural	0.004	1.270	1.000
		C4	30.121	168.221	Flexural	0.004	1.270	1.000
		C5	25.685	159.748	Flexural	0.004	1.270	1.000
		C6	17.993	149.157	Flexural	0.004	1.270	1.000
		C7	16.168	147.039	Flexural	0.004	1.270	1.000
		C8	12.914	143.509	Flexural	0.007	1.754	1.000
		C9	22.895	155.512	Flexural	0.004	1.270	1.000
		C10	14.898	145.627	Flexural	0.004	1.270	1.000
		C11	12.231	142.803	Flexural	0.007	1.754	1.000

4.9 STRENGTH INDEX

$$C = \frac{Qu}{\sum W}$$

Qu= The strength of column

$\sum W$= The weight of building supported by the storey concerned.

Calculation procedure of strength index for C2 column at ground floor are shown here:

Qu= 14.83 kN

$\sum W$= 4459.346 kN

No of C2 column= 23

$$C = \frac{14.83 * 23}{4459.346} = 0.076$$

Table 4.13 Strength Index

Story	Col. ID	No of Column	∑Wi(kN)	Qu(kN)	C	F	Effective strength factor for various 1st group
							1.27<=F
3	C1	5	4459.346	12.195	0.014	1.754	1
	C2	23		14.830	0.076	1.270	1
	C3	5		13.091	0.015	1.754	1
	C4	22		16.495	0.081	1.270	1
	C5	1		14.488	0.003	1.270	1
	C6	1		11.831	0.003	1.754	1
	C7	9		11.280	0.023	1.754	1
	C8	2		10.346	0.005	1.754	1
	C9	1		13.445	0.003	1.754	1
	C10	1		10.908	0.002	1.754	1
	C11	1		10.157	0.002	1.754	1
2	C1	5	10702.430	15.838	0.018	1.270	1
	C2	23		21.328	0.110	1.270	1
	C3	5		17.774	0.020	1.270	1
	C4	22		24.460	0.121	1.270	1
	C5	1		20.651	0.005	1.270	1
	C6	1		15.033	0.003	1.270	1
	C7	9		13.796	0.028	1.754	1
	C8	2		11.648	0.005	1.754	1
	C9	1		18.518	0.004	1.270	1
	C10	1		12.949	0.003	1.754	1
	C11	1		11.206	0.003	1.754	1
1	C1	5	16945.514	19.156	0.021	1.270	1
	C2	23		26.531	0.137	1.270	1
	C3	5		21.880	0.025	1.270	1
	C4	22		30.121	0.149	1.270	1
	C5	1		25.685	0.006	1.270	1
	C6	1		17.993	0.004	1.270	1
	C7	9		16.168	0.033	1.270	1
	C8	2		12.914	0.006	1.754	1
	C9	1		22.895	0.005	1.270	1
	C10	1		14.898	0.003	1.270	1
	C11	1		12.231	0.003	1.754	1

Table 4.14 Basic Seismic Index in X and Y direction

Story	Sum of wt	(n+1) /(n+1)	Group	Failure	Ductility, F	Column	∑Qu(kN)	C	E1	E2	E
3	4552.249	0.667	1	Flexural	1.270	C2,4,5	718.457	0.158	0.200	0.952	0.952
		0.667	2	Flexural	1.754	C1,3,6,7,	294.982	0.065	0.114	1.213	1.213
2	10925.398	0.8	1	Flexural	1.270	C1,2,3,4,5,6	1250.930	0.114	0.145	1.107	1.107
		0.8	2	Flexural	1.754	C7,8,10,	171.611	0.016	0.028	1.416	1.416
1	17298.546	1	1	Flexural	1.270	C1,2,3,4,5,6,	1705.022	0.099	0.125	1.368	1.368
		1.000	2.000	Flexural	1.754	C8,11	429.820	0.025	0.044	1.779	1.779

4.10 SEISMIC INDEX OF STRUCTURE IS AND CTU, SD

The calculation procedure for Iso is shown as an example. The structure is assumed not to have the second-class prime element. The irregularity index and time index are also assumed as 1.0. C_{TU}. S_D is the index for the cumulative strength of a structure at the ultimate limit state.

Since the structure is assumed not to have the second-class prime element, if Eq. (4) is applied, the ultimate limit state is the deformation corresponding to the maximum F index of the groups. Thus, C_{TU} is the accumulated strength at the maximum F index of the groups.

On the other hand, if Eq. (5) is applied, the ultimate state is the deformation at Fl, and C_{TU} is the strength at F1

The seismic demand index of structure Isa should be calculated by Equation (4.3.1)

$$Iso = \frac{2}{3} \times Z \times I \times Cse$$

Where, Z= Seismic zone coefficient, as defined in Section 2.5.4.2 of BNBC 2020= 0.28 (Chittagong)

I= Structure importance factor, as defined in Section 2.5.5.1 of BNBC 2020= 1.25

Cse= Normalized acceleration response spectrum for seismic evaluation, which is a function of structure (building) period and soil type (Site Class), TO determine CSE, the peak value of cs derived from equation 6.2.35a to 6.2.35d of BNBC 2020 has been reduced to 80% considering inelastic behaviour at post-yield stage. After the peak value range, CSE will remain the same (without reduction) as cs of BNBC 2020, considering additional load effect and uncertainty from multimodal behaviour for mid and high-rise buildings. (Refer to 4.3.5) = 3.02

$$Iso = \frac{2}{3} \times 0.28 \times 1.25 \times 3.02 = \ 0.705$$

1. Safety check

$Ctu.Sd \geq 0.4 * \frac{2}{3} * Z * I * Cse$ where $0.4 \ \times \frac{2}{3} ZICse \leq 0.3$

$0.4 \ \times \frac{2}{3} * 0.28 * 1.25 * (3.02 * 0.8) = \ 0.226 \leq 0.3$ [ok]

Table 4.15 Result of 2nd Level Screening

C	Ductility, F	Failure Type	E	T	Sd	(n+1)/(n+1)	Ctu*Sd	Seismic Demand, Iso=0.234 / Seismic Capacity, Is	Result
0.158	1.270	Flexural	0.200	1	0.9	0.667	0.095	0.857	OK
0.065	1.754	Flexural	0.114	1	0.9	0.667	0.039	1.091	OK
0.114	1.270	Flexural	0.145	1	0.9	0.8	0.082	0.997	OK
0.016	1.754	Flexural	0.028	1	0.9	0.8	0.011	1.274	OK
0.099	1.270	Flexural	0.125	1	0.9	1	0.089	1.232	OK
0.025	1.754	Flexural	0.044	1	0.9	1	0.022	1.601	OK

Chapter 5: ANALYSIS AND DESIGN OF PROPOSED STEEL BUILDING SYSTEM AS PER BNBC 2020

5.1 EXISTING PLAN

This chapter discusses the transformation from the existing RCC building layout to a newly proposed steel-composite structure. A revised plan was developed to create a vertically extended system that stands independently around and above the old structure. The new design ensures no load is transferred to the existing frame, with all analysis and design performed according to BNBC 2020 using ETABS.

5.1.1 Typical Floor Plan

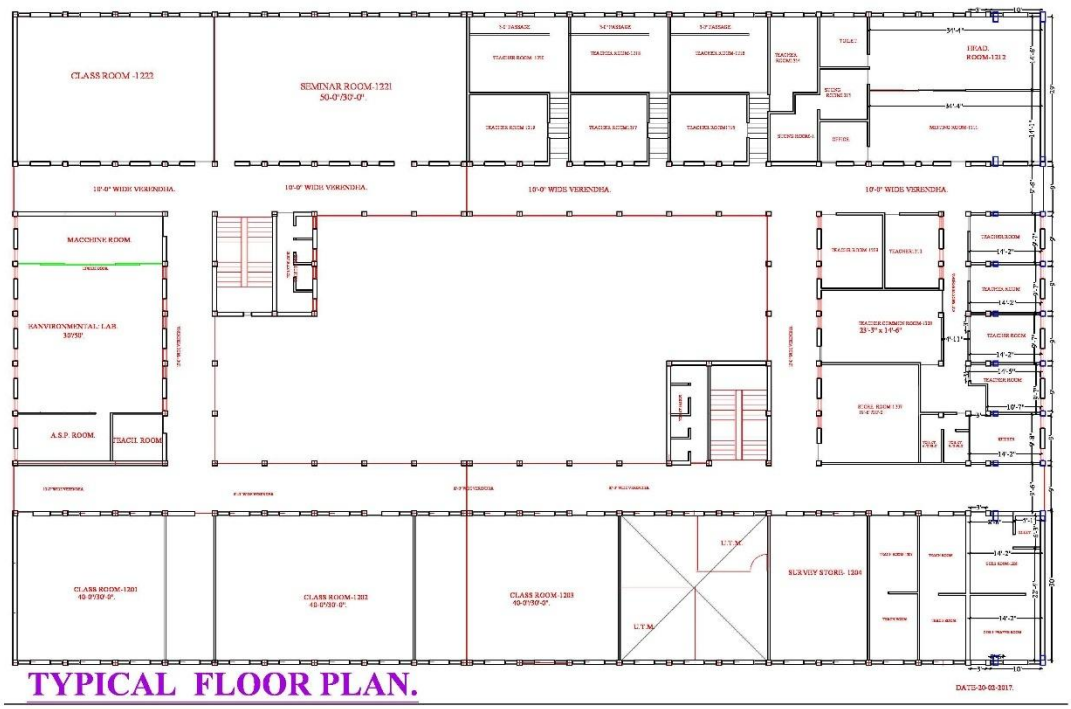


Figure 5.1 Typical Floor Plan

5.1.2 Ground Floor Plan



Figure 5.2 Ground Floor Plan

5.1.3 Column Schedule

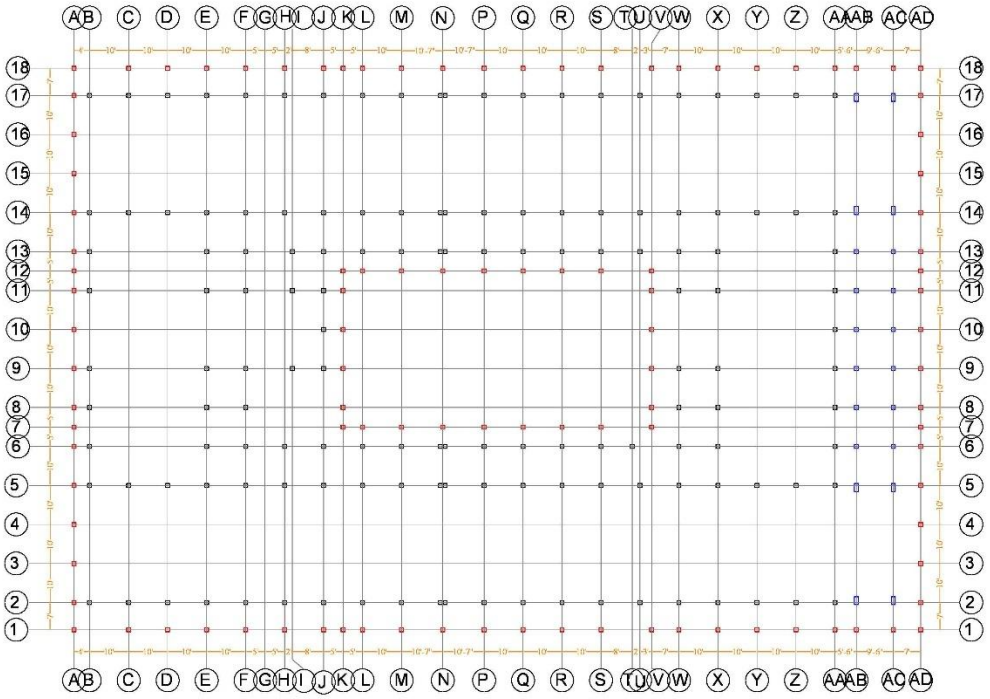


Figure 5.3 Column Schedule (Black)

5.2 PROPOSED LAYOUT PLAN

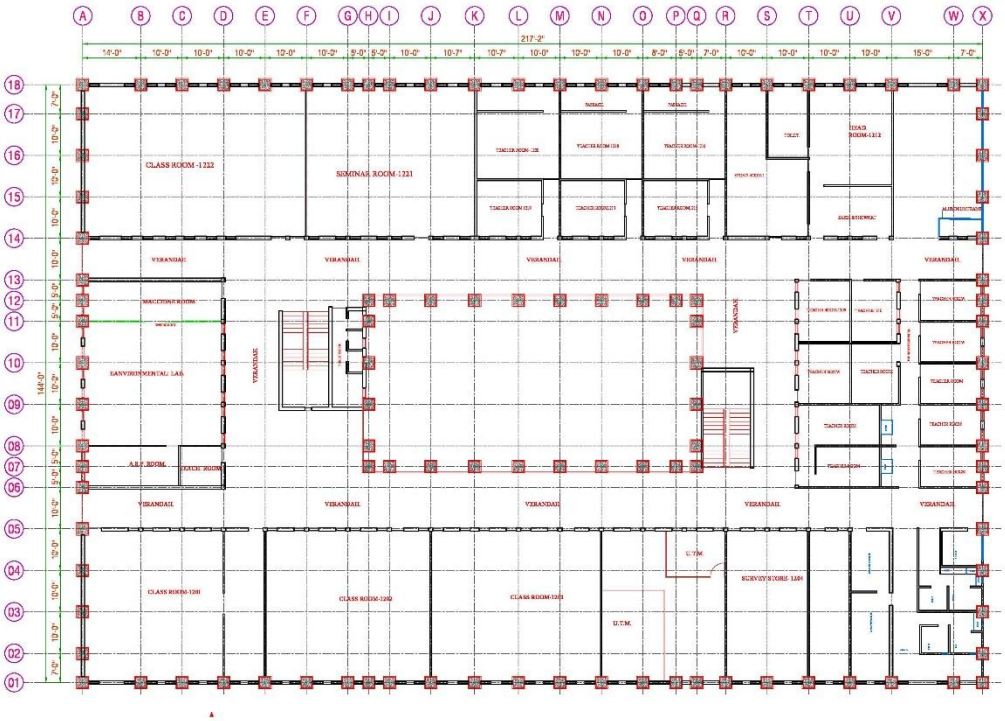


Figure 5.4 Proposed Layout Plan

5.3 ANALYSIS

After Completion of the model, the model will be run for analysis and design.

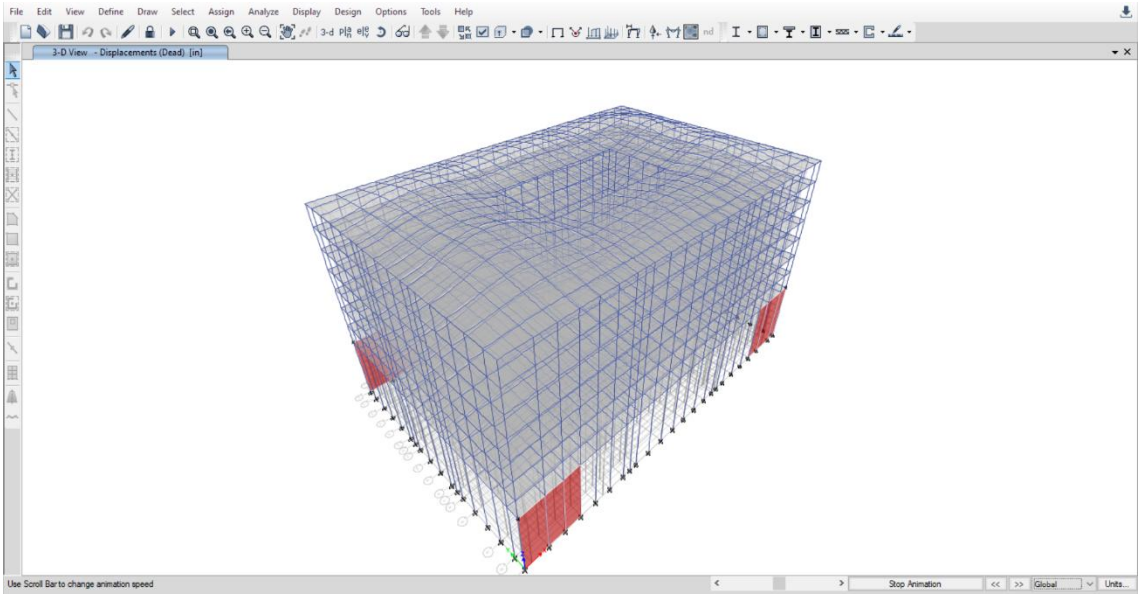


Figure 5.5 Run for Analysis

5.4 REACTION ANALYSIS AT BASE PLAN

Base Reaction (Dead Load) refers to the force exerted by the permanent weight of a structure that must be resisted by the foundation. It is essential for designing foundations, validating load transfers, and cross-checking with manual calculations. This value confirms whether the structure's dead loads are properly accounted for and safely transmitted to the ground. Accurate base reaction analysis ensures the building remains stable and secure under its own weight, preventing structural failures. It's a critical step in structural analysis and design, especially for ensuring the integrity of footings, piles, or other foundation elements.

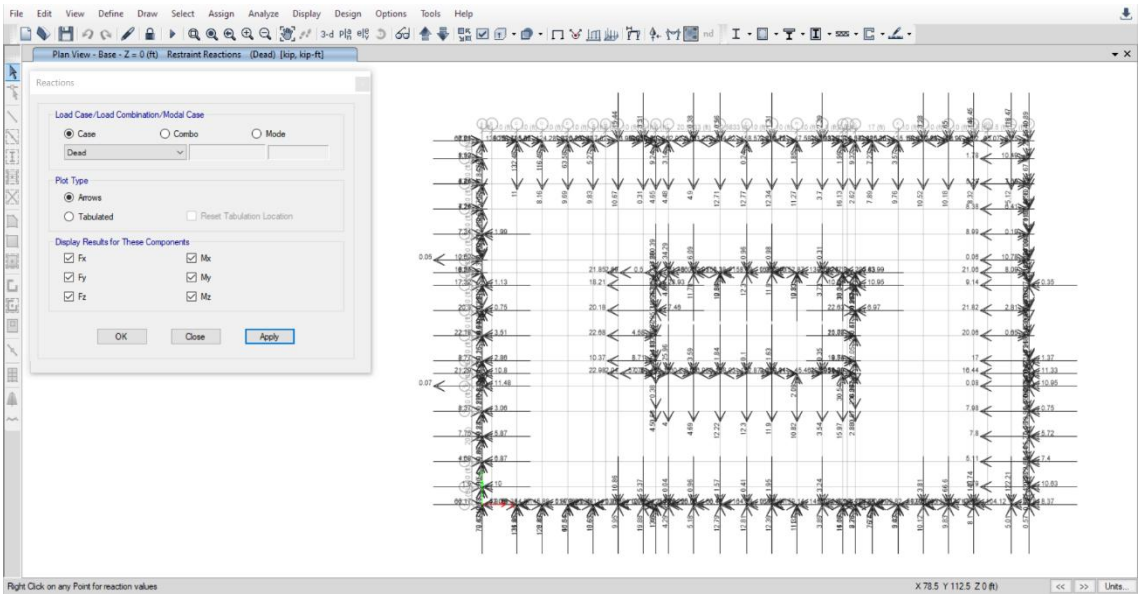


Figure 5.6 Reaction on Base level for Dead Load

5.5 DEFORMED SHAPE FOR LOAD

The deformed shape visually represents how a structure displaces under its own weight. It typically includes a colour contour map indicating the magnitude of displacement. This view is crucial for identifying serviceability issues like excessive sagging or deflection, verifying model accuracy, and guiding design improvements. It helps spot problems such as missing supports, weak or under-designed members, and incorrect input data. Reviewing the deformed shape ensures the structure behaves as expected under permanent loads and supports informed engineering decisions during analysis and design.

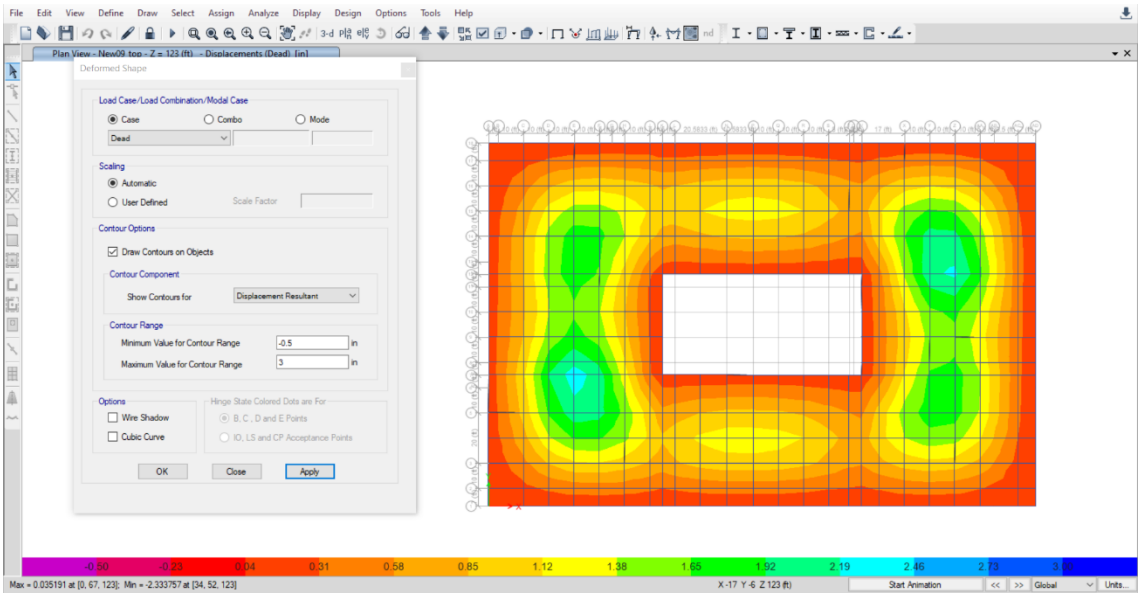


Figure 5.7 Deformed shape for Dead Load (Contours)

5.6 SHELL FORCED FOR LOAD

Shell load refers to the internal forces developed within slabs and walls due to the structure's self-weight. These forces—bending moments (Mx, My), membrane forces (Fx, Fy), shear (Vz), and torsion/shear (M_{xy}, F_{xy})—are essential for assessing structural behavior. Engineers use them to design appropriate reinforcement, control cracking, and ensure serviceability. Results are typically visualized through color contour plots and detailed numerical tables, helping identify stress concentrations or weak zones. Accurate interpretation of shell forces supports safe and efficient design of slab and wall elements in structural systems.

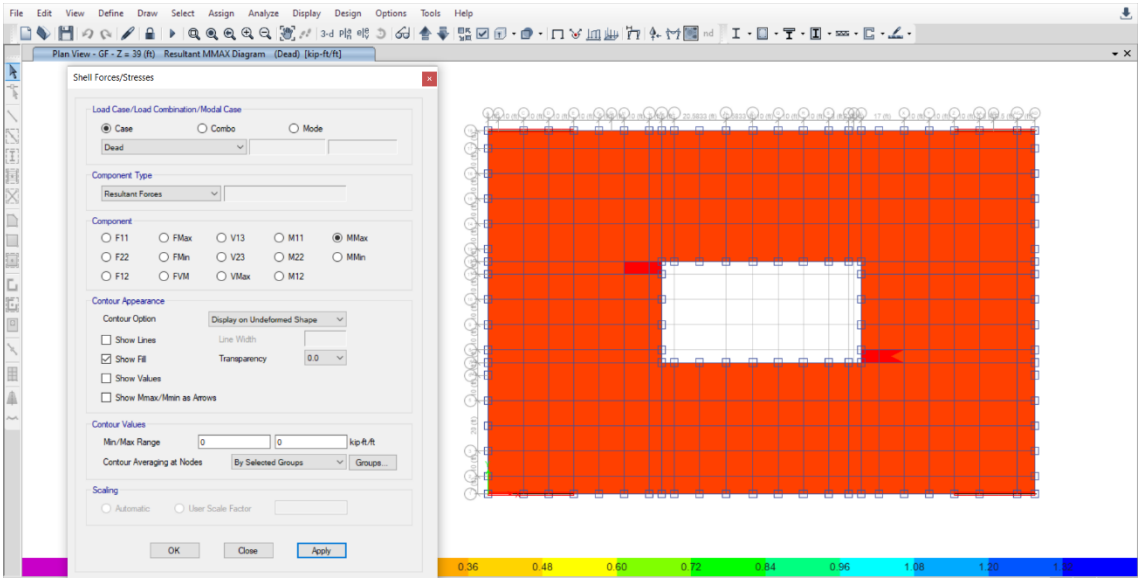


Figure 5.8 Shell forces for Dead Load (Contours)

5.7 MAXIMUM STORY DISPLACEMENT FOR EARTHQUAKE

Maximum story displacement refers to the greatest horizontal movement of any floor level in a building during an earthquake. It is used to assess how much the structure sways under seismic forces. The main purpose is to ensure that the displacement remains within acceptable code limits to maintain safety and functionality. It is significant because excessive displacement can lead to structural or non-structural damage, such as cracking, failure of joints, or glass breakage. Evaluating this helps in designing effective lateral load-resisting systems and controlling inter-story drift, ensuring the building remains stable, serviceable, and earthquake-resistant.

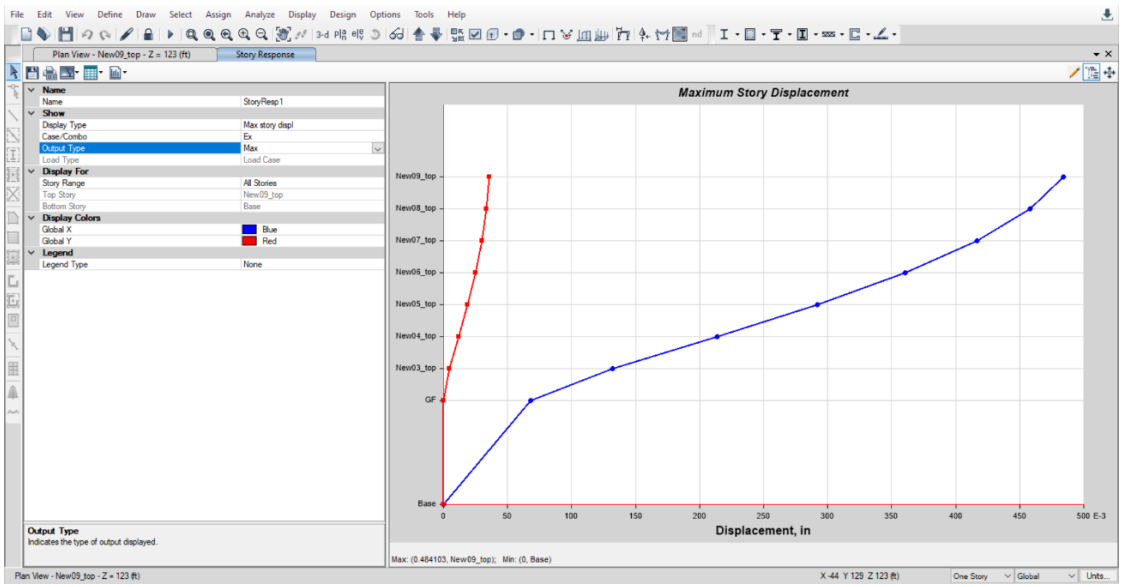


Figure 5.9 Maximum Story Displacement for Ex

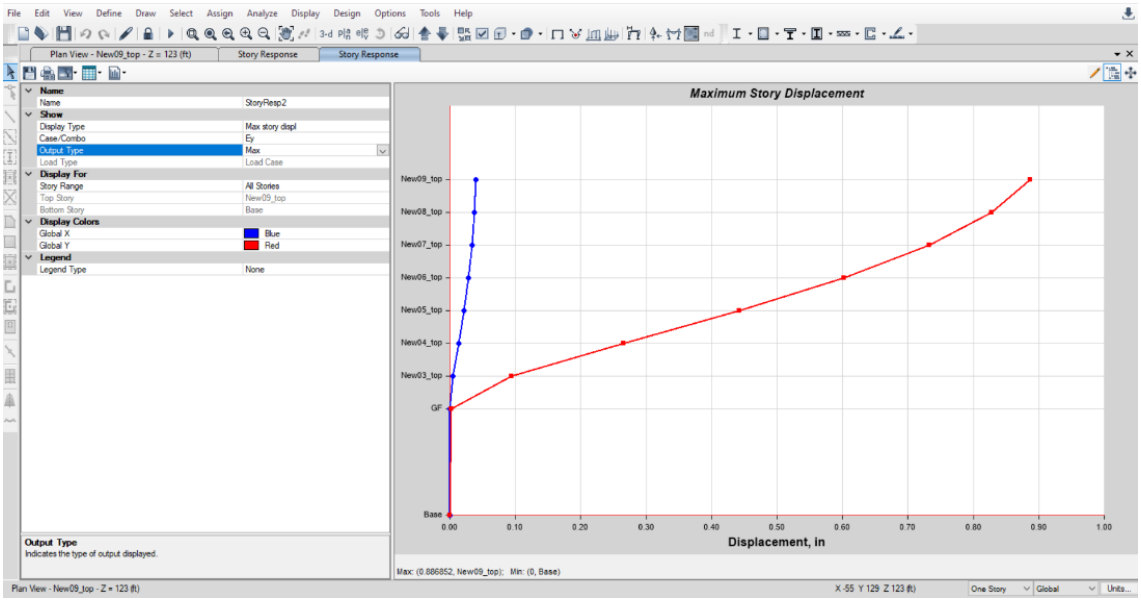


Figure 5.10 Maximum Story Displacement for Ey

5.8 MAXIMUM STORY DISPLACEMENT FOR WIND

Maximum story displacement for wind load refers to the largest lateral movement of any floor level caused by wind forces. It is crucial for evaluating how much a building sways under wind pressure. The purpose is to ensure occupant comfort, structural safety, and compliance with design codes. Excessive displacement can cause discomfort to occupants, affect structural integrity, and damage non-structural elements like partitions or glass. Assessing this helps in designing effective lateral load-resisting systems such as bracings or shear walls. By keeping wind-induced movement within acceptable limits, engineers ensure both stability and usability of the structure under wind loading conditions.

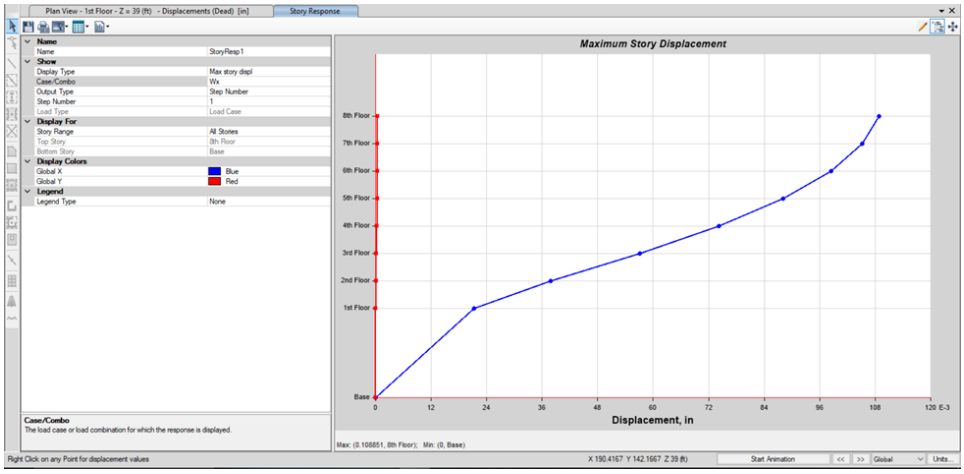


Figure 5.11 Maximum Story Displacement for Wx

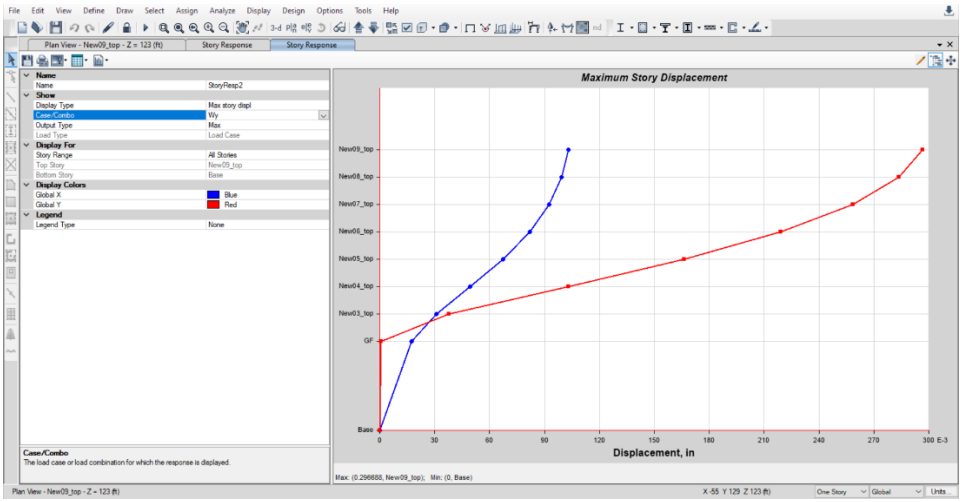


Figure 5.12 Maximum Story Displacement for Wy

5.9 MAXIMUM STORY DRIFT FOR EARTHQUAKE

Maximum story drift for earthquake load is the largest relative lateral displacement between two adjacent floors caused by seismic forces. It indicates how much one floor moves compared to the floor below during an earthquake. The purpose is to control structural deformation, prevent damage to both structural and non-structural elements, and ensure occupant safety. Excessive drift can lead to failures such as cracking, joint damage, or collapse. Assessing story drift is vital for designing lateral load-resisting systems like shear walls and braces. Keeping drift within code limits ensures the building maintains stability, integrity, and serviceability during and after seismic events.

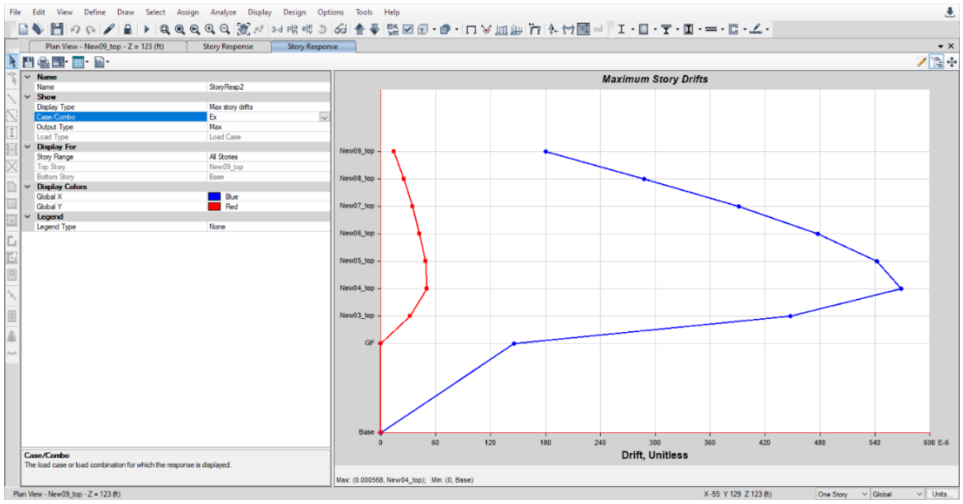


Figure 5.13 Maximum Story Drift for Ex

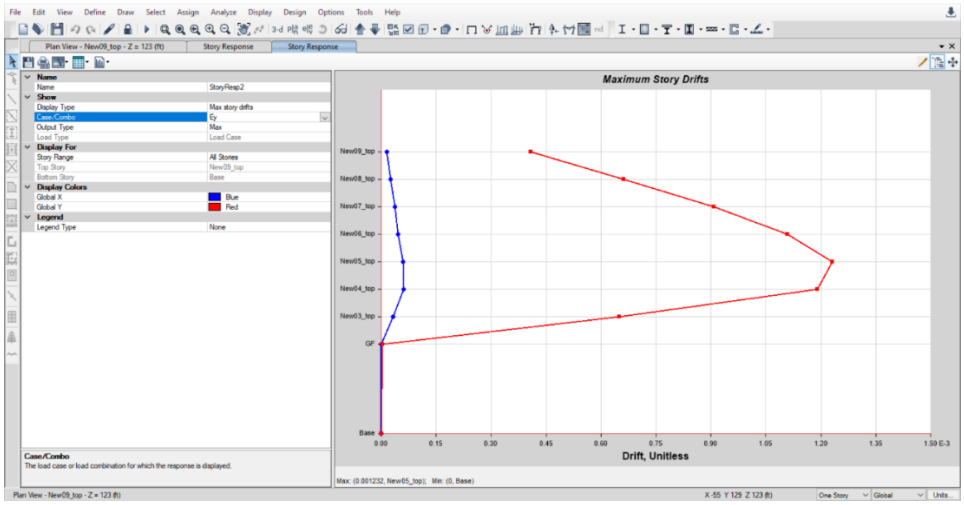


Figure 5.14 Maximum Story Displacement for Ey

5.10 STORY SHEAR FOR EARTHQUAKE

Story shear refers to the total horizontal force acting on each floor due to seismic loads. It plays a critical role in designing lateral load-resisting elements like shear walls and braces, and in checking overall structural stability. Story shear values are used to identify weak or soft stories and verify that the building can resist earthquake-induced forces. These forces are typically shown in tables or graphs plotting shear force against story height. The pattern generally shows increasing shear from top to bottom, with the ground floor experiencing the maximum force.

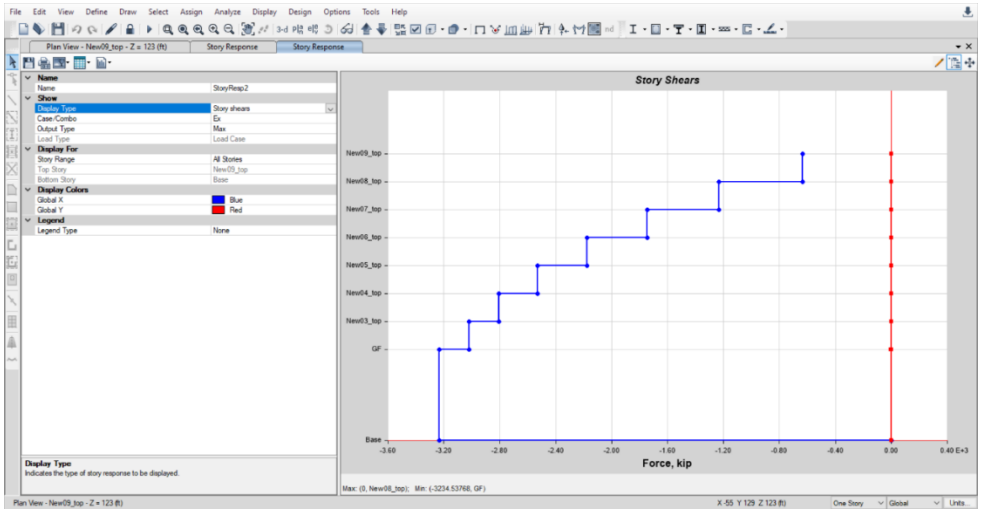


Figure 5.15 Story Shears Ex

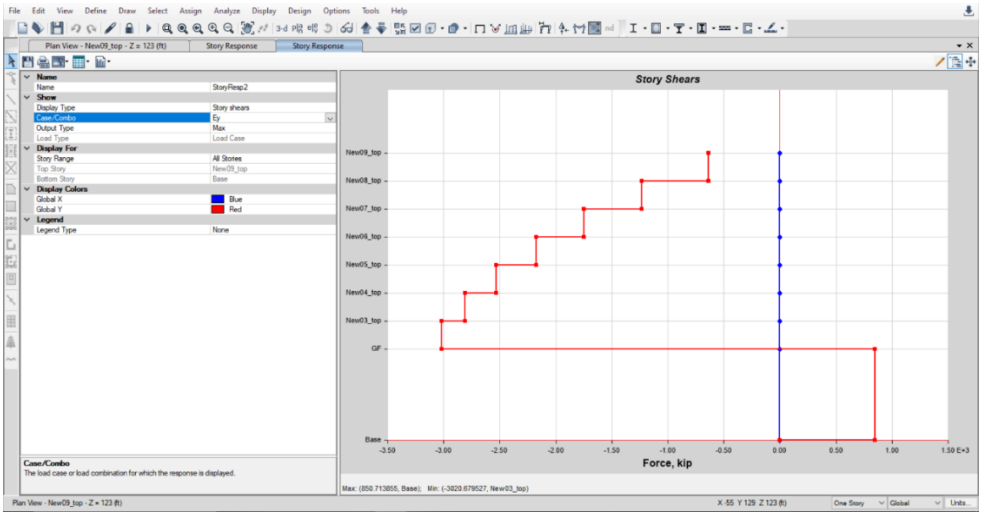


Figure 5.16 Story Shear Ey

5.11 OVERTURNING MOMENT FOR EARTHQUAKE

Overturning moment is the rotational force at a structure's base caused by lateral seismic loads acting at a height. It is critical for evaluating whether a building might tip or rotate during an earthquake. This moment must be resisted by factors like the structure's dead load, foundation weight, anchorage, or soil pressure. It's primarily used in foundation design, uplift checks, and ensuring seismic stability. In ETABS, it is shown as Mx and My at the base level, resulting from lateral load cases (E_X, E_Y). Proper assessment ensures the structure remains stable during seismic events.

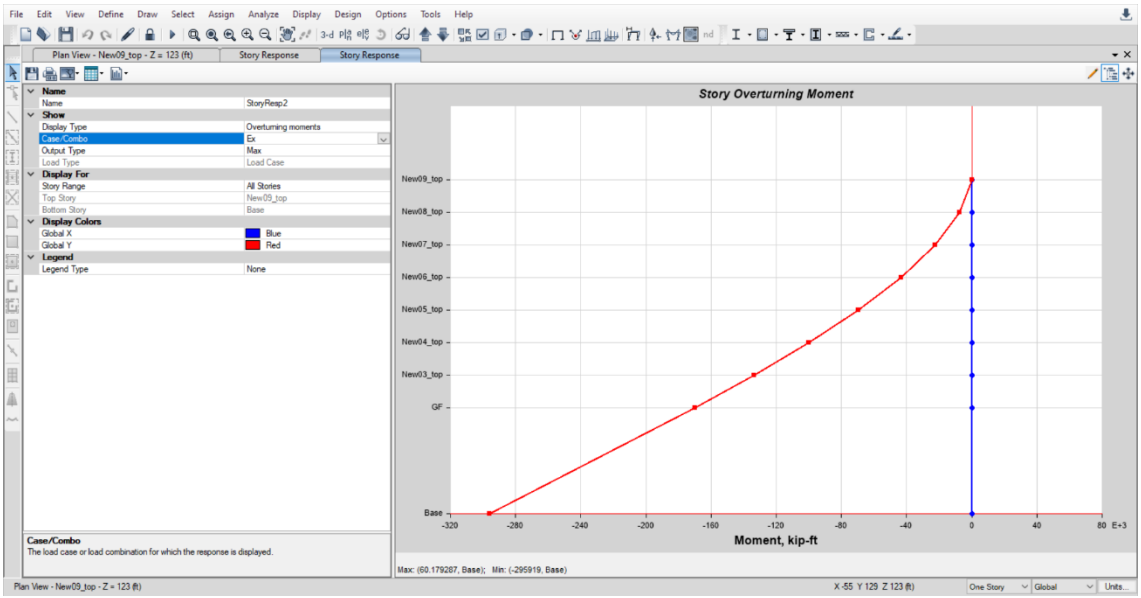


Figure 5.17 Overturning moment for Ex

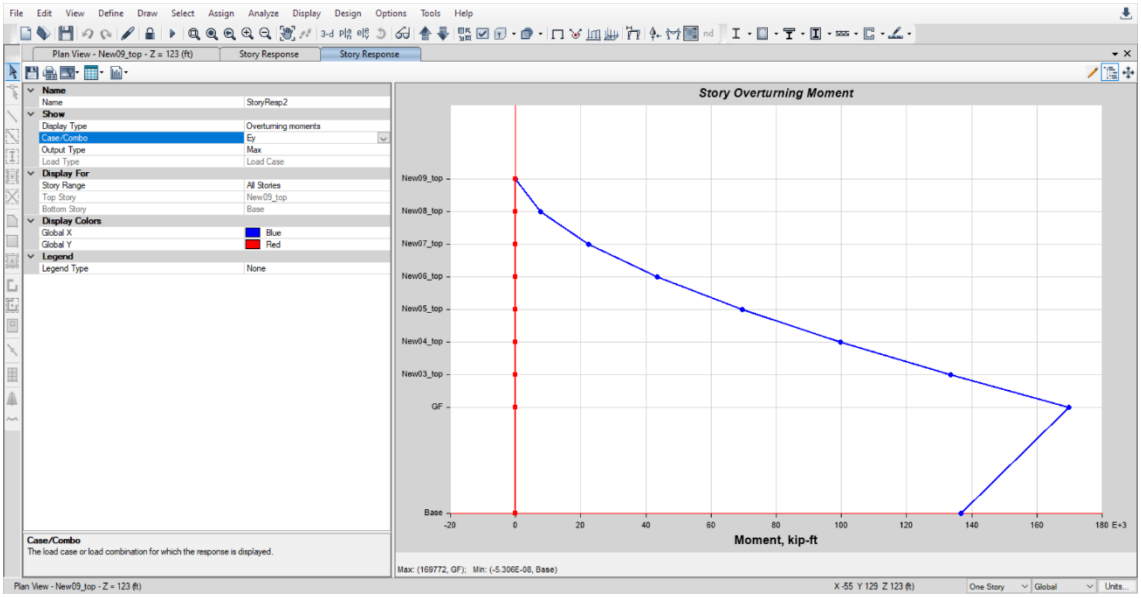


Figure 5.18 Overturning moment for Ey

5.12 STORY STIFFNESS FOR EARTHQUAKE

Story stiffness measures a floor's resistance to lateral movement during wind or seismic loads, calculated as force divided by displacement. It helps assess how rigid or flexible each story is. The main purpose is to ensure proper load distribution and detect soft stories, which are vulnerable during earthquakes. It plays a key role in designing lateral-resisting systems like shear walls and braces. Evaluating story stiffness is essential for seismic safety, code compliance, and preventing structural irregularities that may lead to collapse or damage under lateral forces.

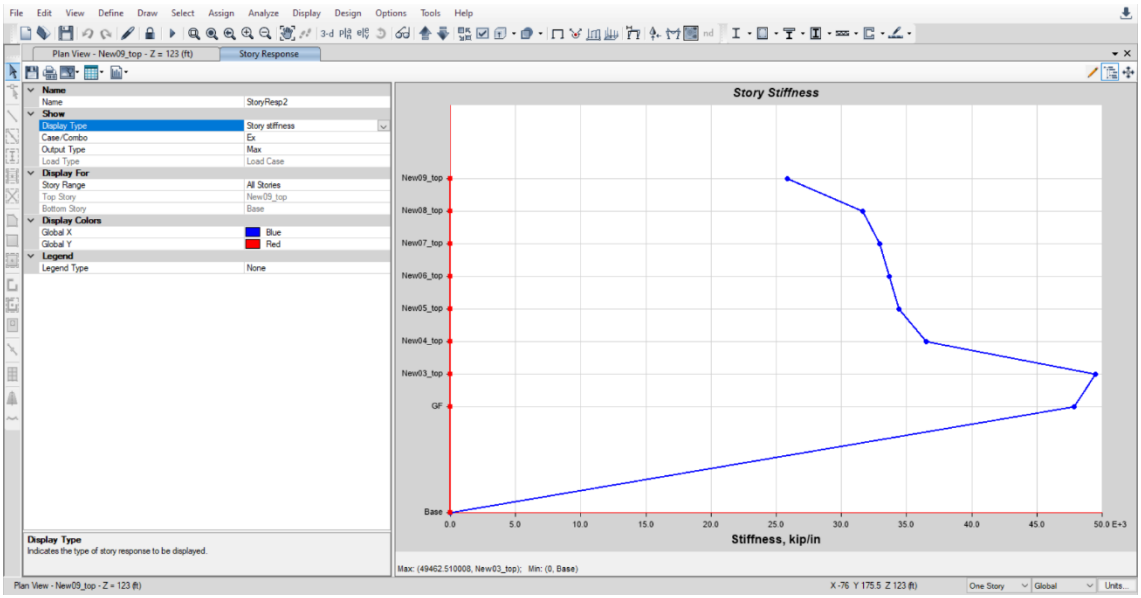


Figure 5.19 Story stiffness for Ex

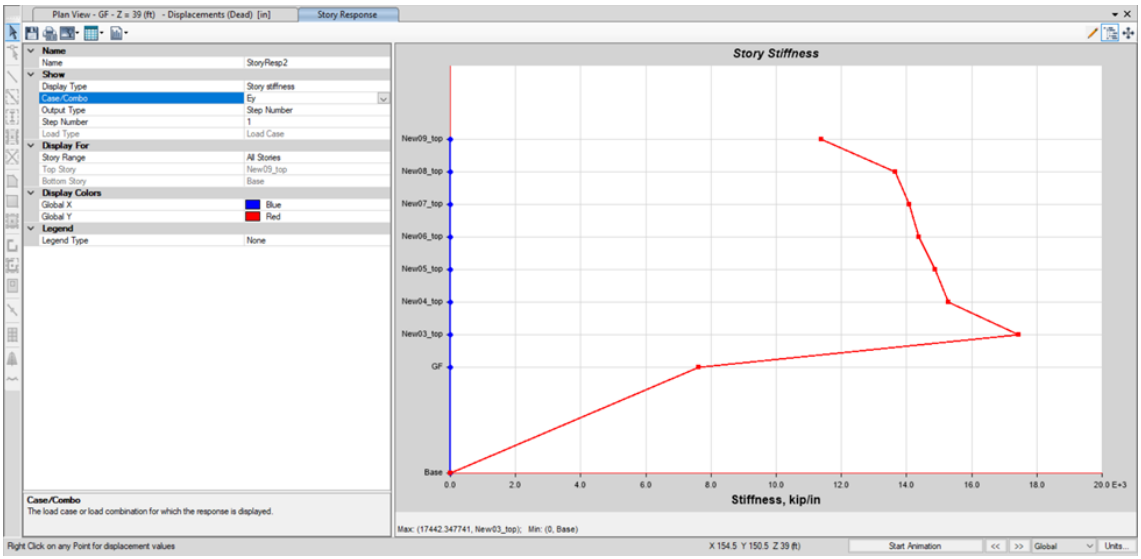


Figure 5.20 Story stiffness for Ey

5.13 TORSION FOR LOADS

Torsion for dead load refers to the twisting effect or torque that occurs in a structural member (like a beam or slab) due to the permanent weight of the structure itself, typically when loads are applied eccentrically (off-center) or when the geometry causes asymmetry. Although dead load usually causes bending and shear, in irregular or cantilevered structures, it can also produce torsion. Understanding torsion from dead load is important for designing beams, slabs, or edge members to prevent cracking, excessive rotation, or structural failure. Proper reinforcement and detailing are required to resist such torsion and ensure long-term structural integrity.

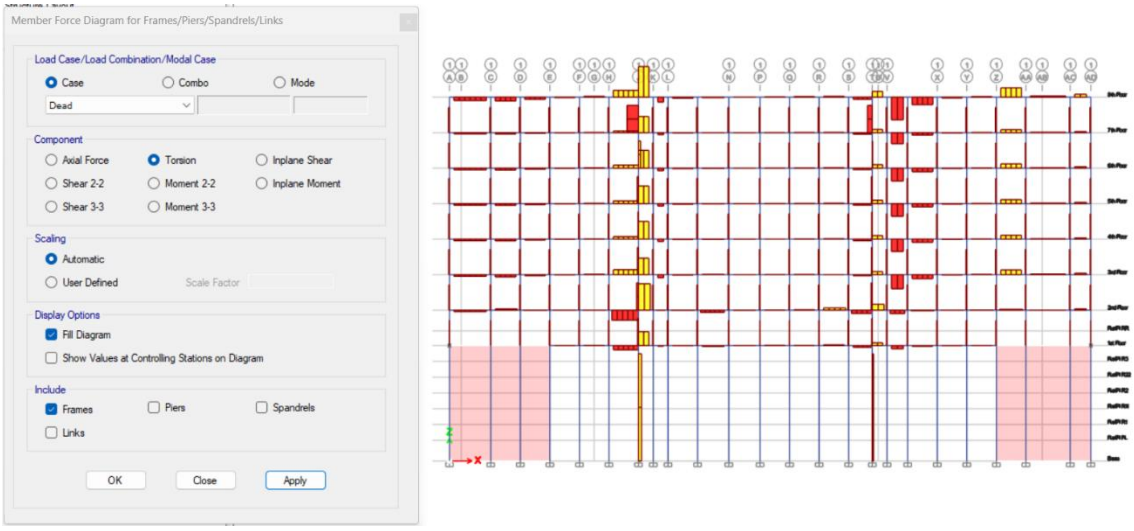


Figure 5.21 Torsion for Dead Load

5.14 MOMENT, SHEAR AND AXIAL FORCE FOR DEAD LOAD

Maximum moment for dead load refers to the highest bending moment in a structural member (like a beam or slab) caused solely by the dead load—which includes the permanent, stationary weight of the structure itself (e.g., concrete, steel, finishes). It is a critical design factor used to ensure that members have adequate strength and reinforcement to resist bending under constant weight. Engineers calculate it to prevent failure or excessive deflection over time. This moment is essential in structural analysis, as it influences material selection, cross-sectional dimensions, and safety margins for long-term structural stability.

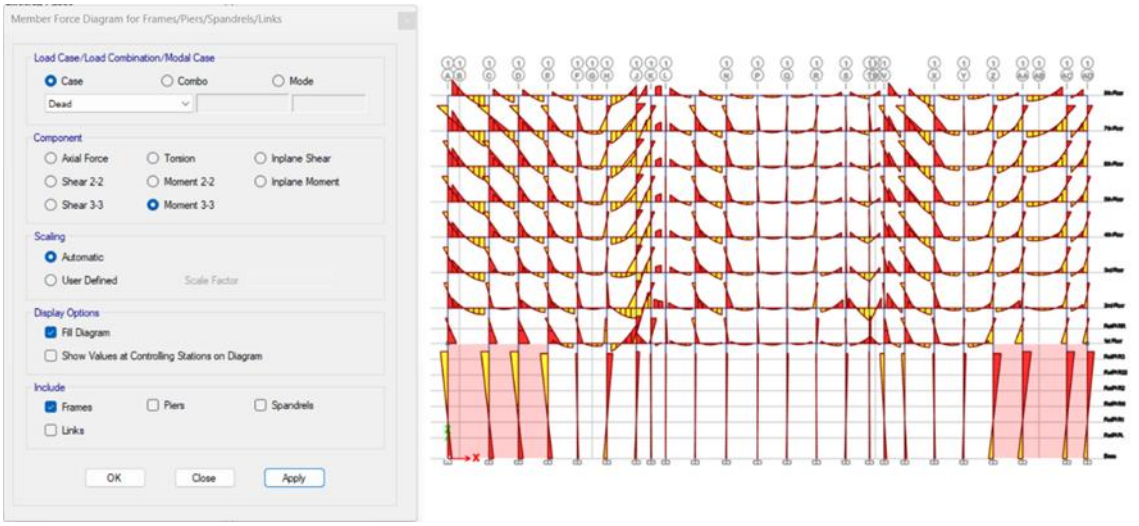


Figure 5.22 Moment For Dead Load

Shear for dead load refers to the internal shear force developed in structural elements (like beams or slabs) due to the permanent, static weight of the structure itself—such as concrete, walls, finishes, etc. This shear force is crucial in designing cross-sections to prevent shear failure, especially near supports where shear is typically highest. It ensures that materials and reinforcements can safely resist sliding or diagonal cracking. Although shear from dead load is usually lower than from live or dynamic loads, its constant nature makes it vital in long-term safety and stability. It also contributes to overall load combinations in structural design.

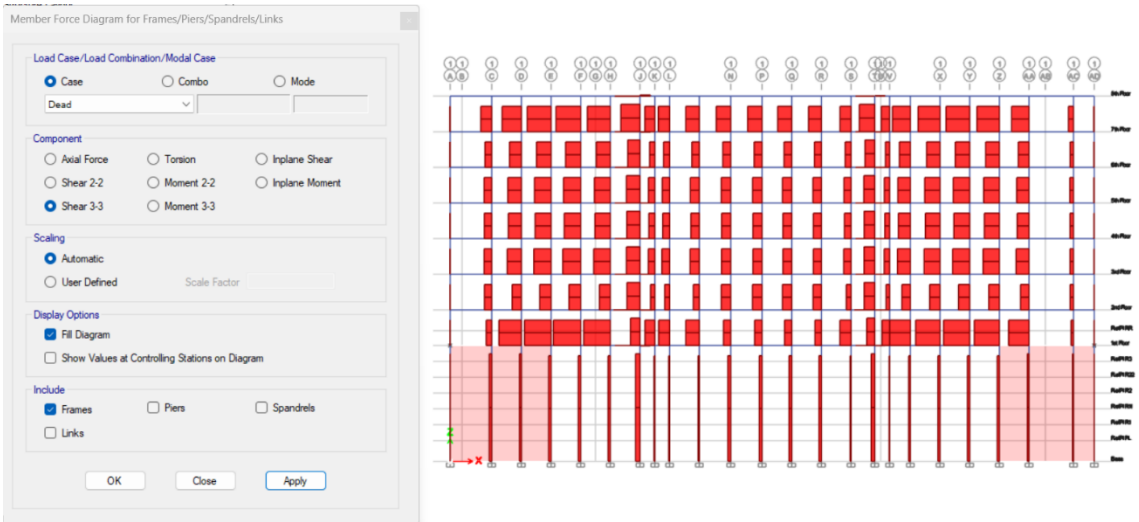


Figure 5.23 Shear for dead load

Axial force for dead load refers to the compressive or tensile force acting along the length of a structural member (like a column, wall, or truss member) due to the permanent weight of the structure itself—such as slabs, beams, roofing, and finishes. This force is essential in structural design, especially for columns and load-bearing walls, as it directly affects their size, material strength, and stability. Accurate calculation of axial force from dead load helps prevent buckling or crushing failures. Since dead load is constant, it forms the baseline axial force considered in all load combinations for safe and durable design.

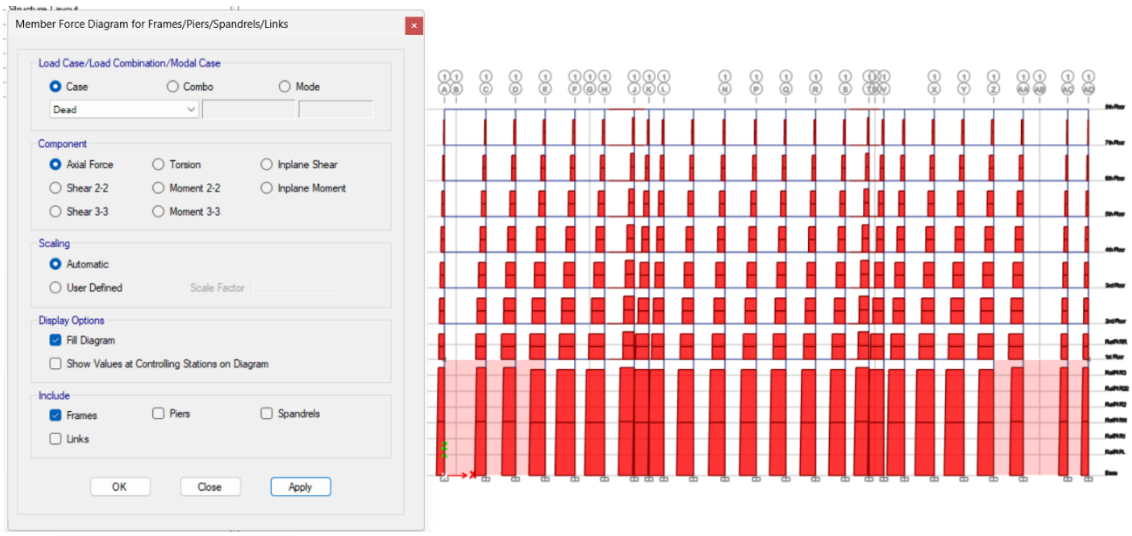


Figure 5.24 Axial Force for dead load

Chapter 6: DETAILING

This chapter presents the detailed structural drawings for the proposed seven-story composite steel structure. The detailing includes key elements such as beams, columns, floor systems, and connections, following the standards used in the design phase. This chapter ensures that the design is practically implementable and accurately documented for execution.

6.1 DETAILING OF FOUNDATION

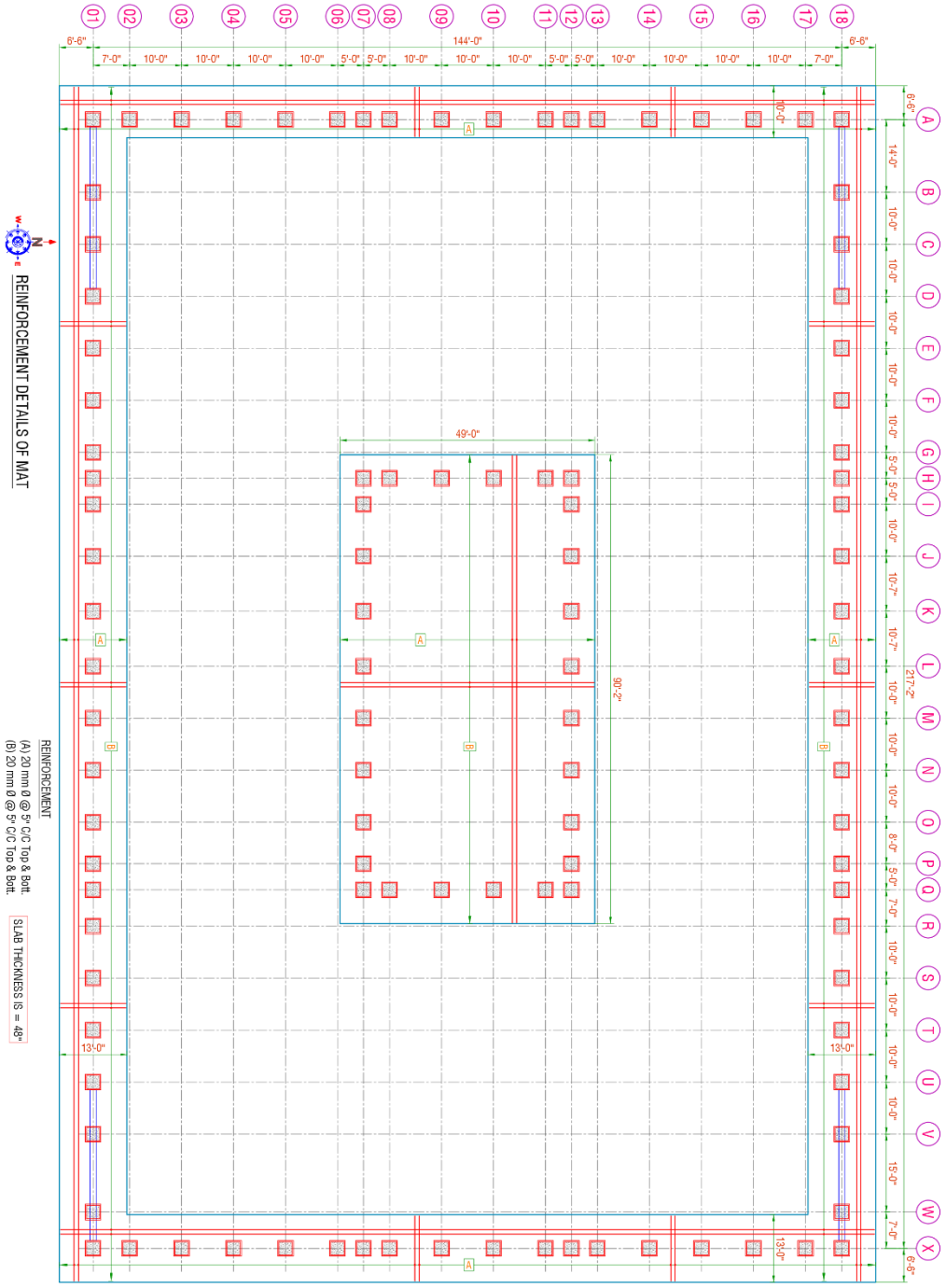


Figure 6.1 Reinforcement Details of MAT Foundation.

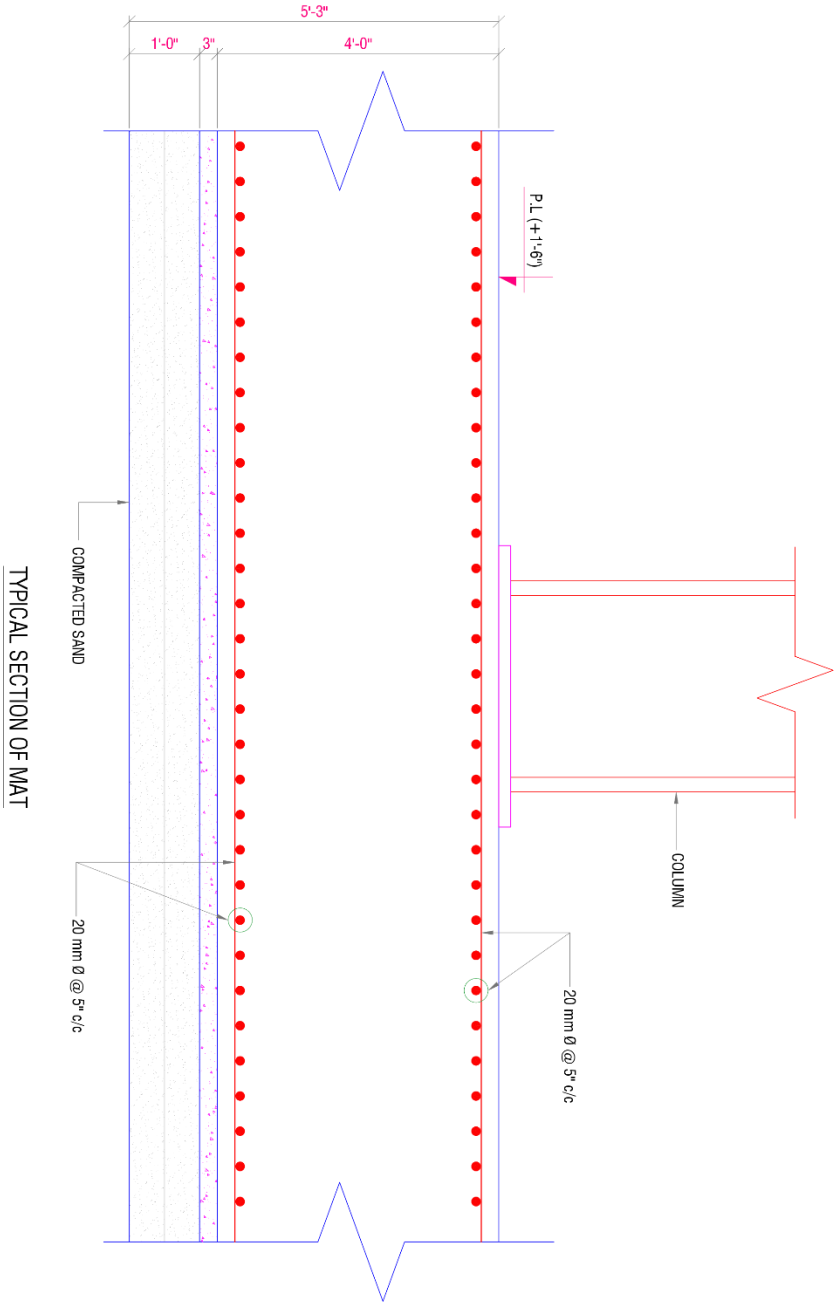


Figure 6.2 Typical Section of MAT Foundation.

6.2 BASE PLATE LAYOUT

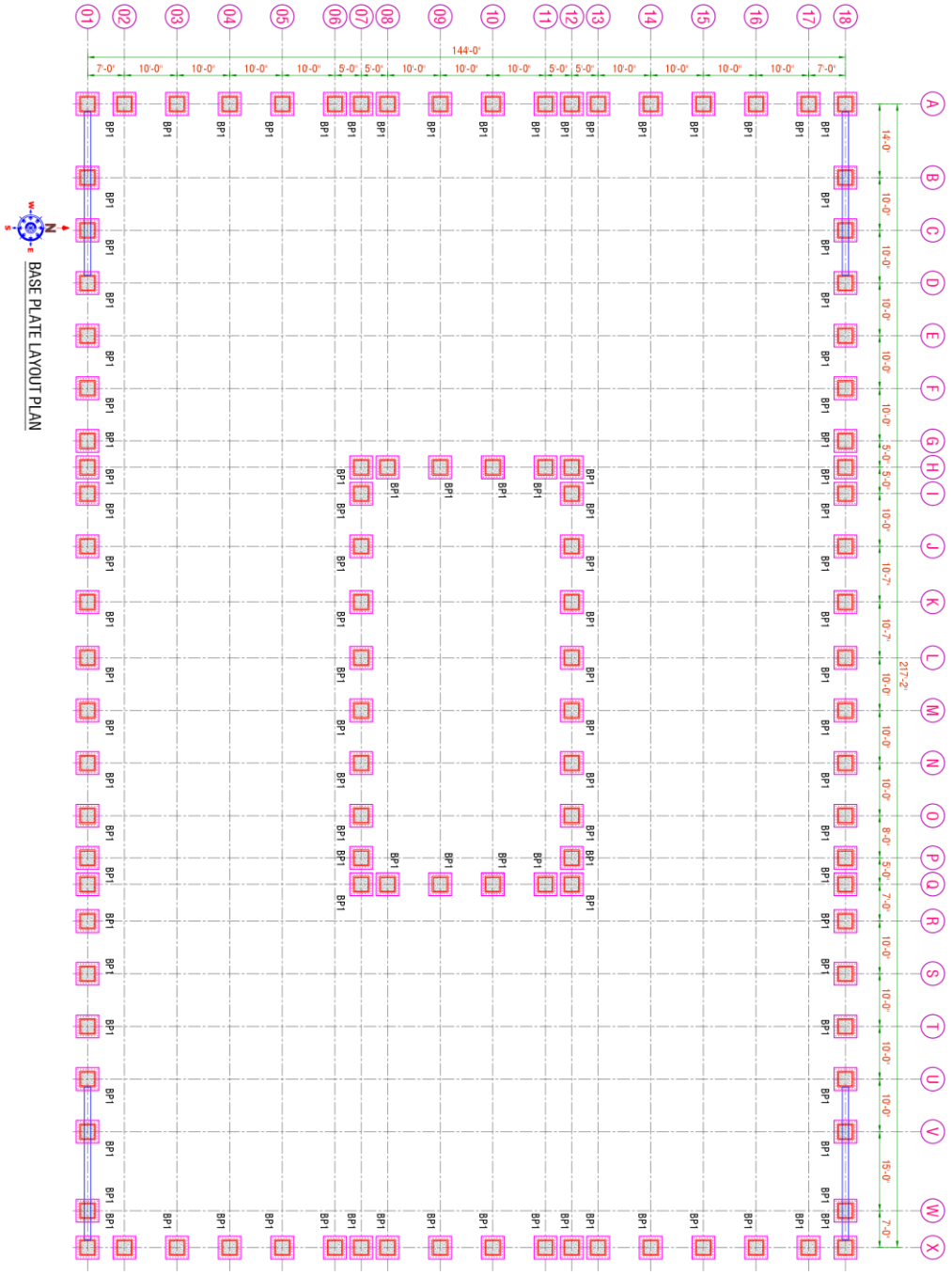


Figure 6.3 Base Plate Layout Plan

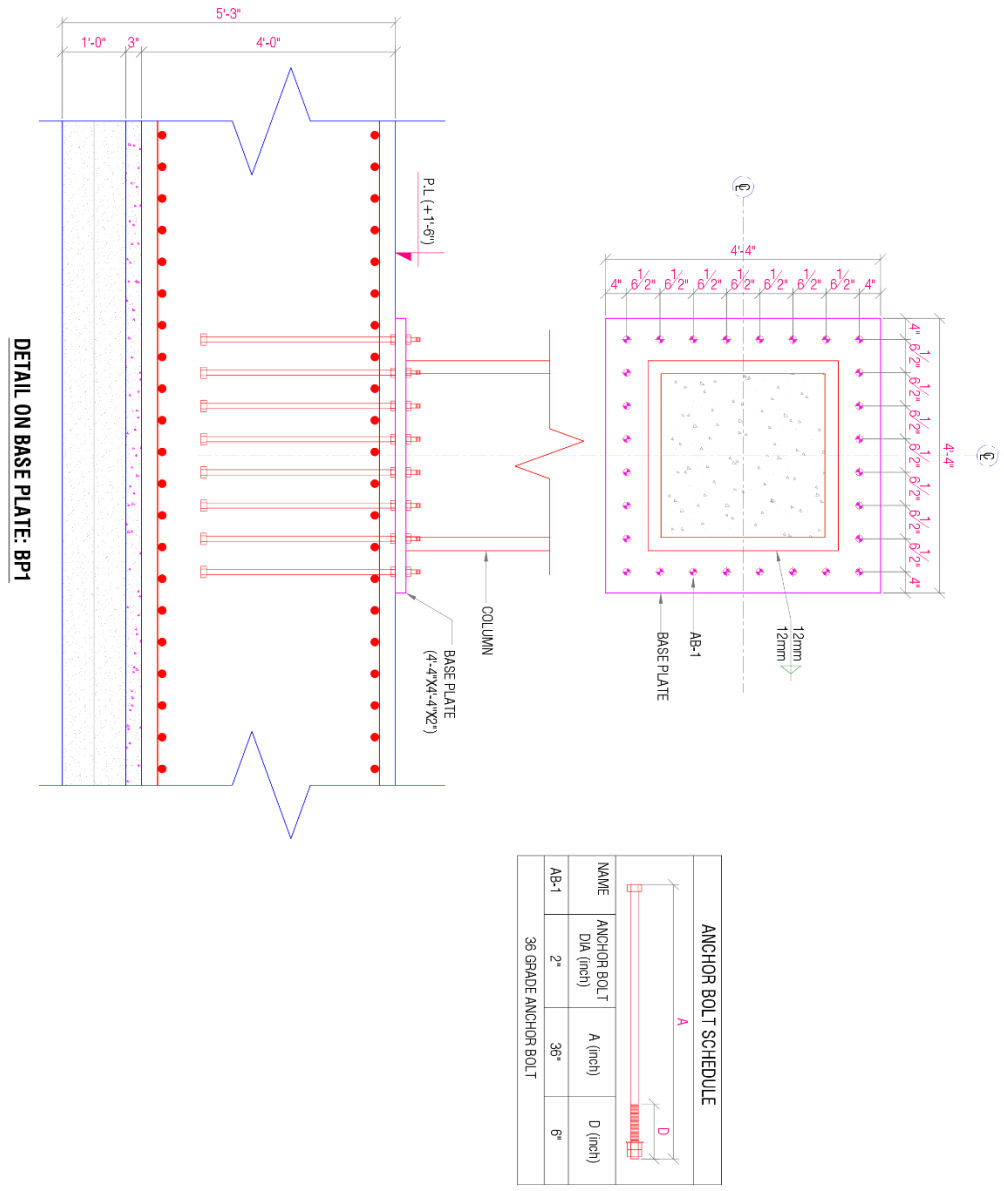


Figure 6.4 Detail on Base Plate.

6.3 COLUMN LAYOUT

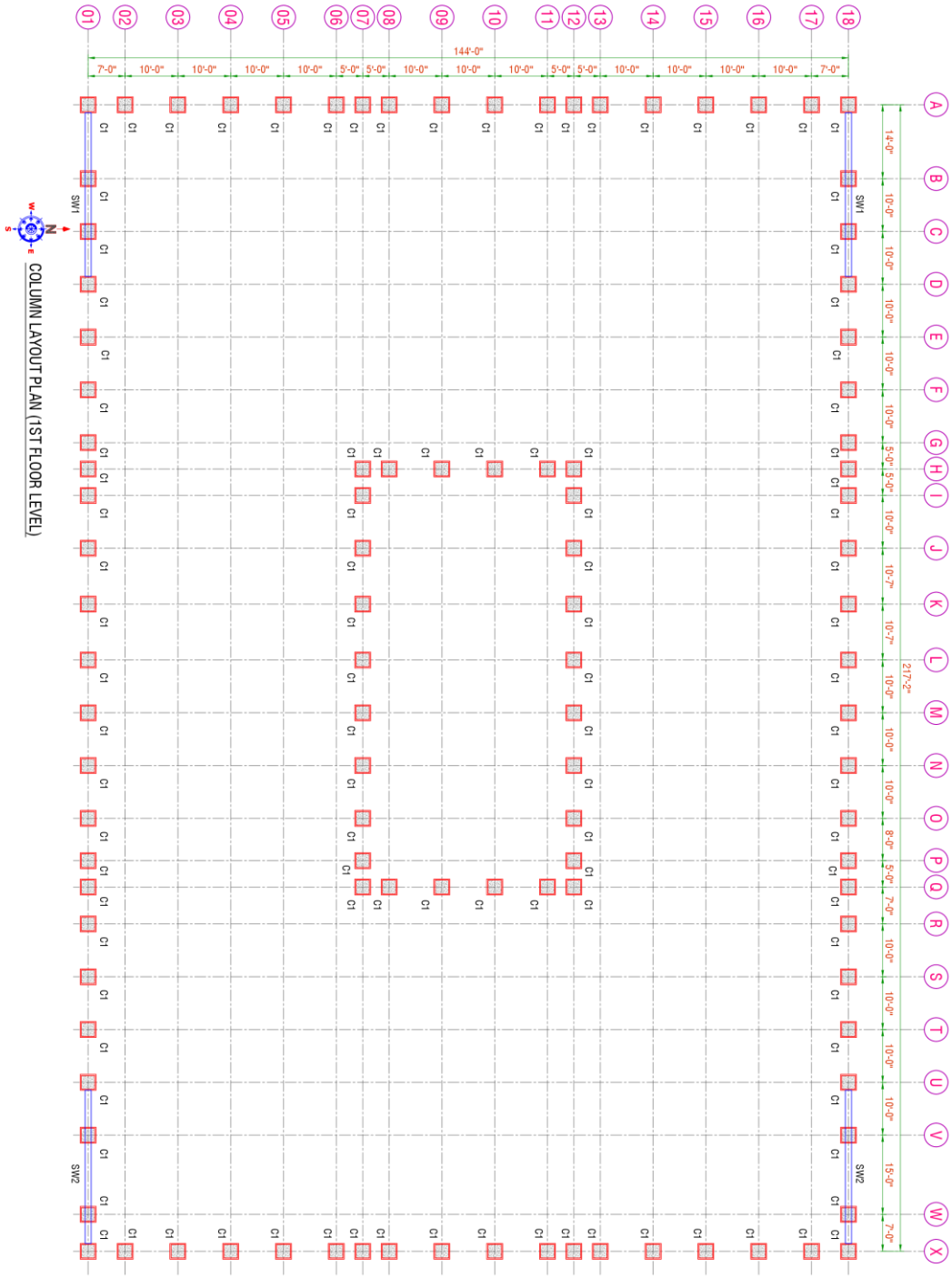


Figure 6.5 Column Layout Plan (upto 1st Floor Level)

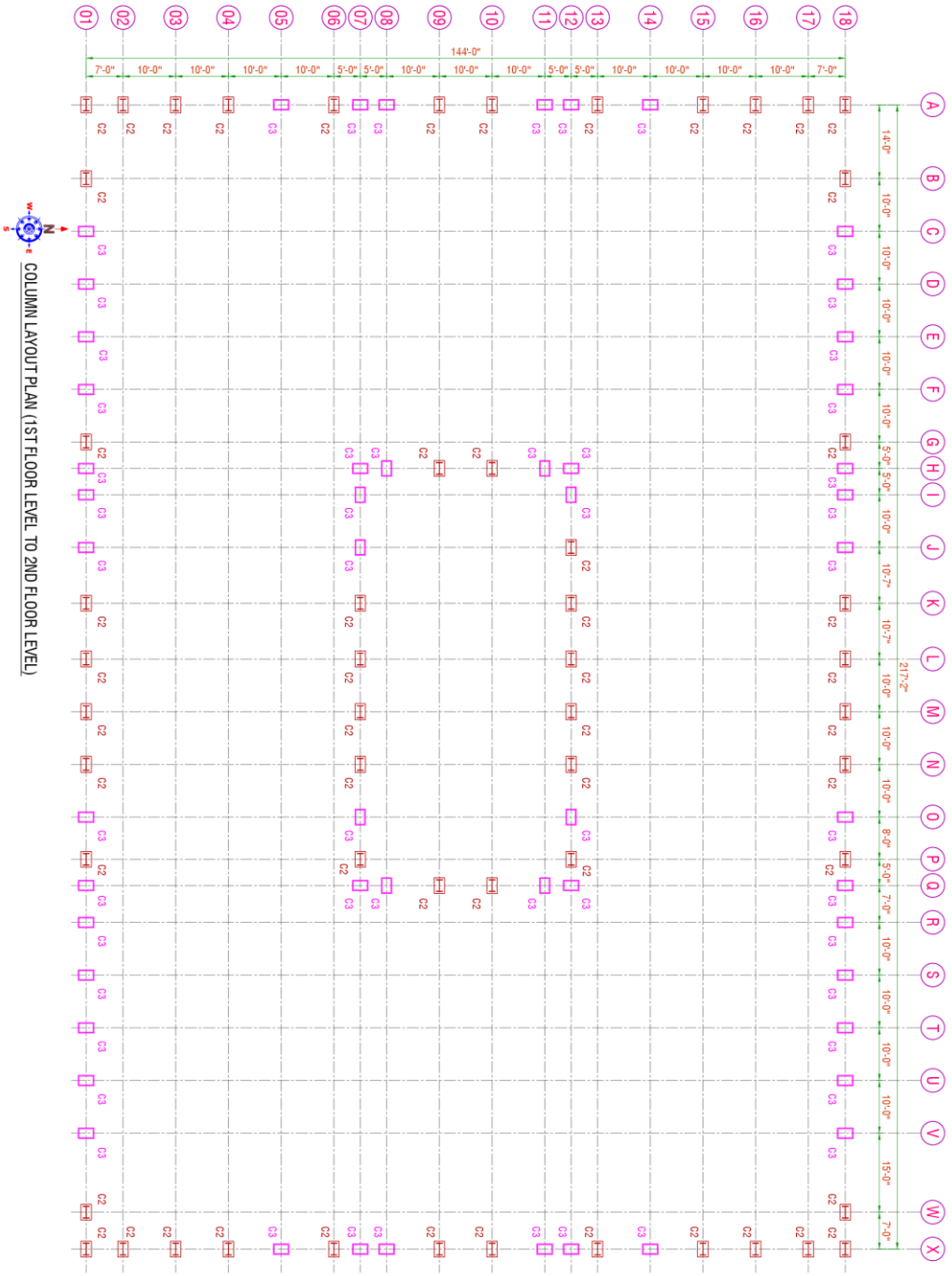


Figure 6.6 Column Layout Plan (1st Floor Level to 2nd Floor Level)

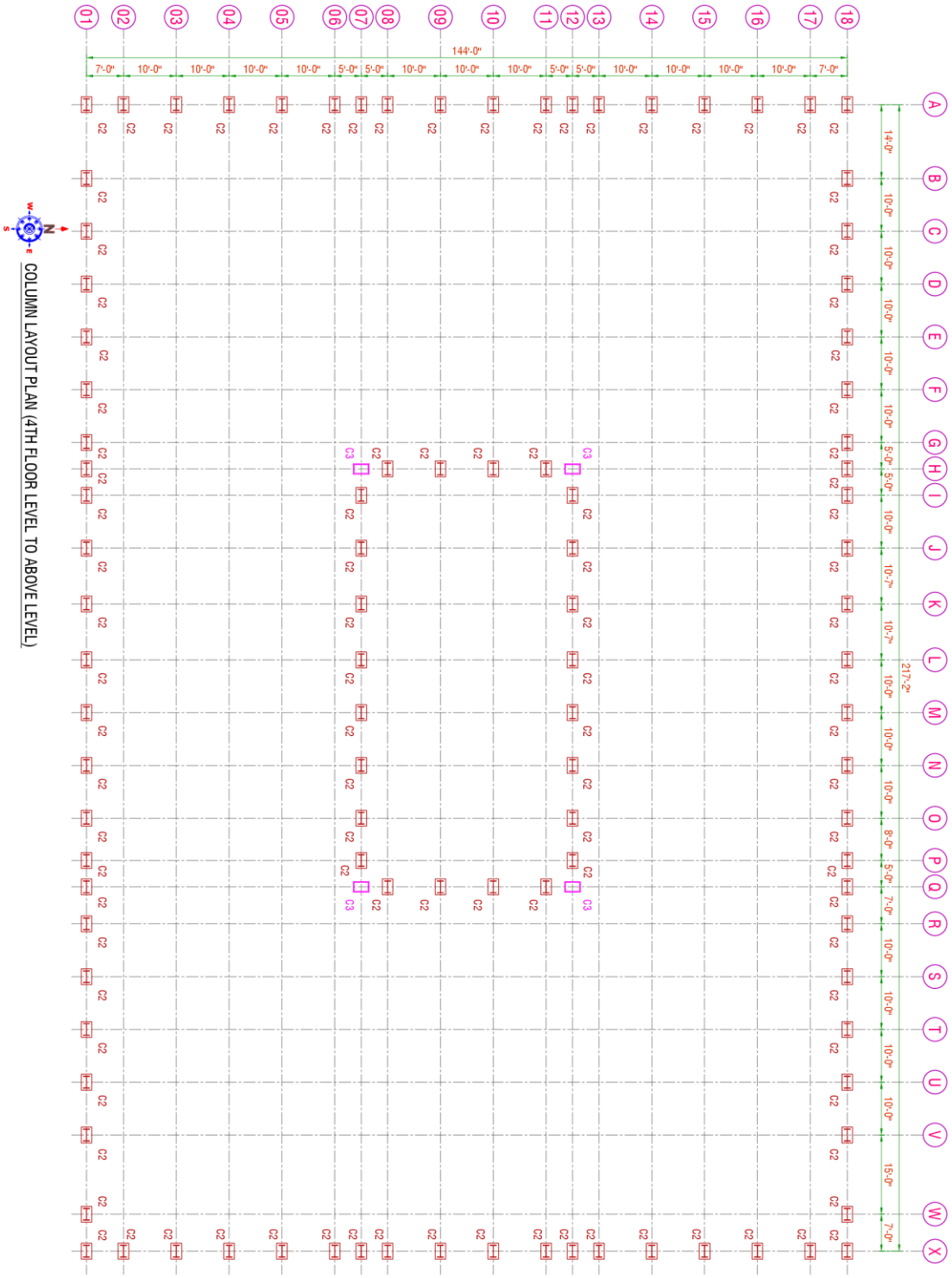


Figure 6.7 Column Layout Plan (2nd Floor Level to 4th Floor Level)

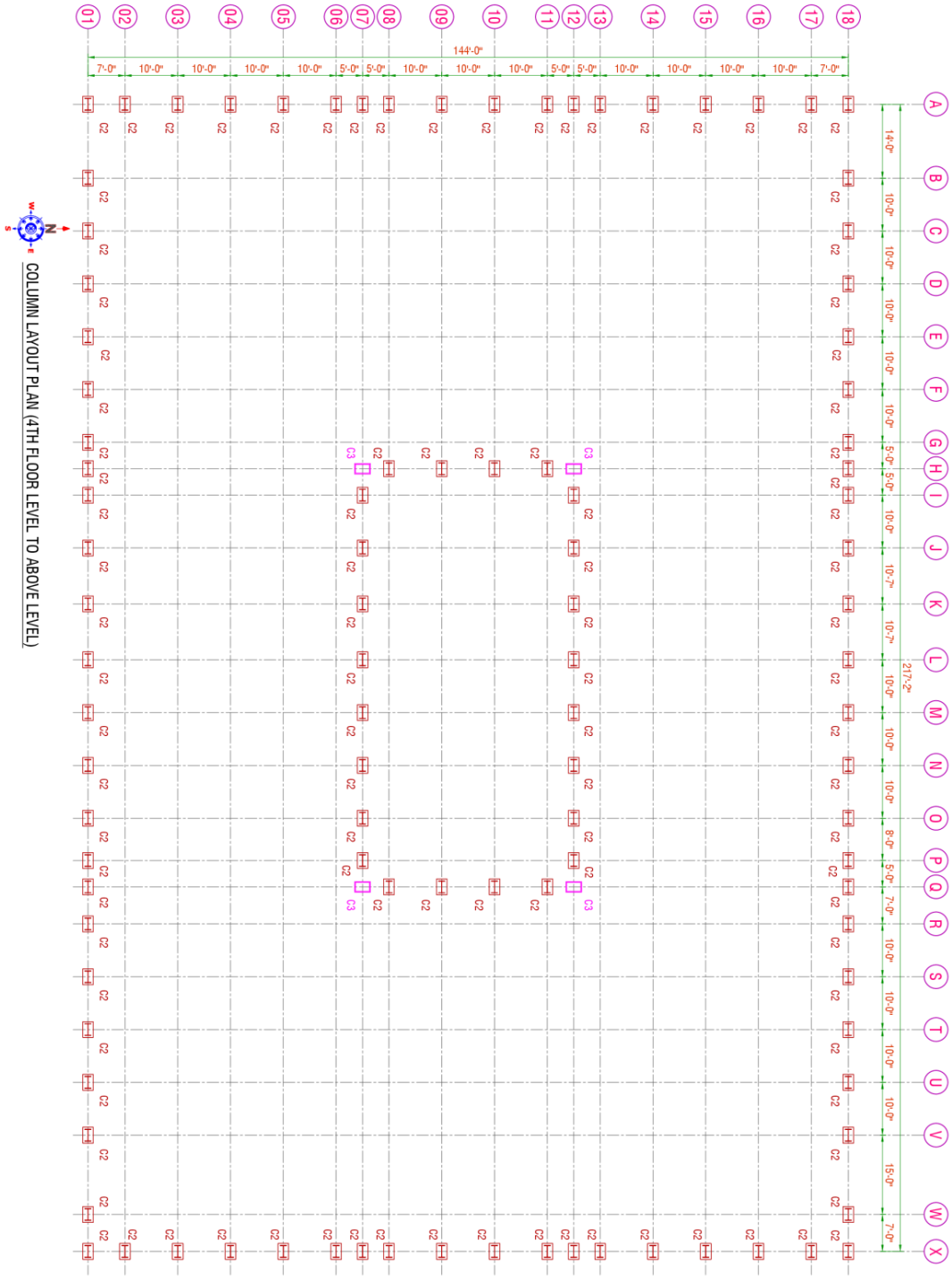


Figure 6.8 Column Layout Plan (4th Floor Level to Above Level)

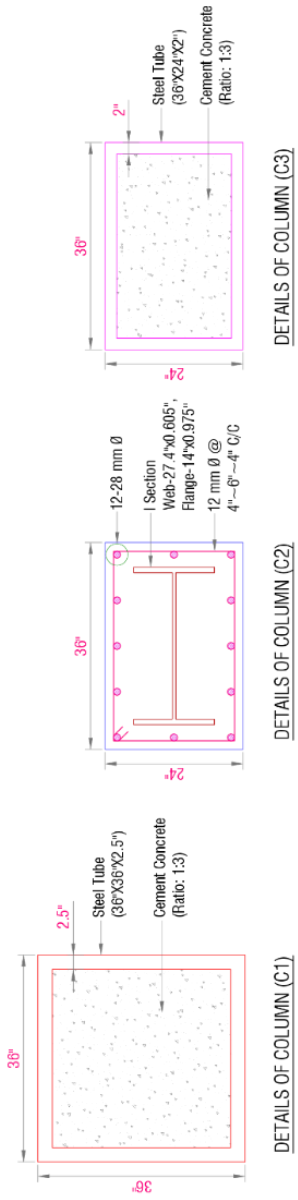


Figure 6.9 Details of all Column Layout (C1, C2, C3)

6.4 REINFORCEMENT DETAIL

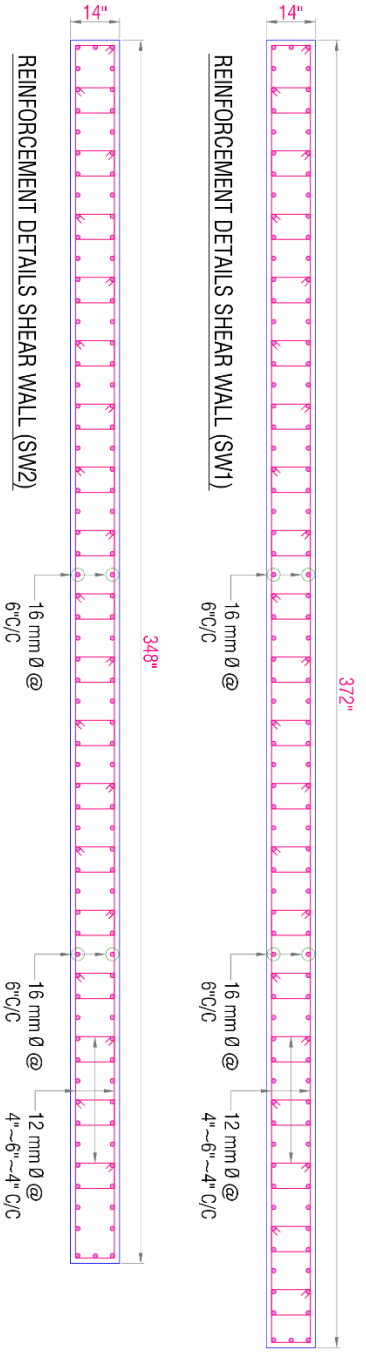


Figure 6.10 Reinforcement Detail of Shear Wall

6.5 CONNECTION DETAIL

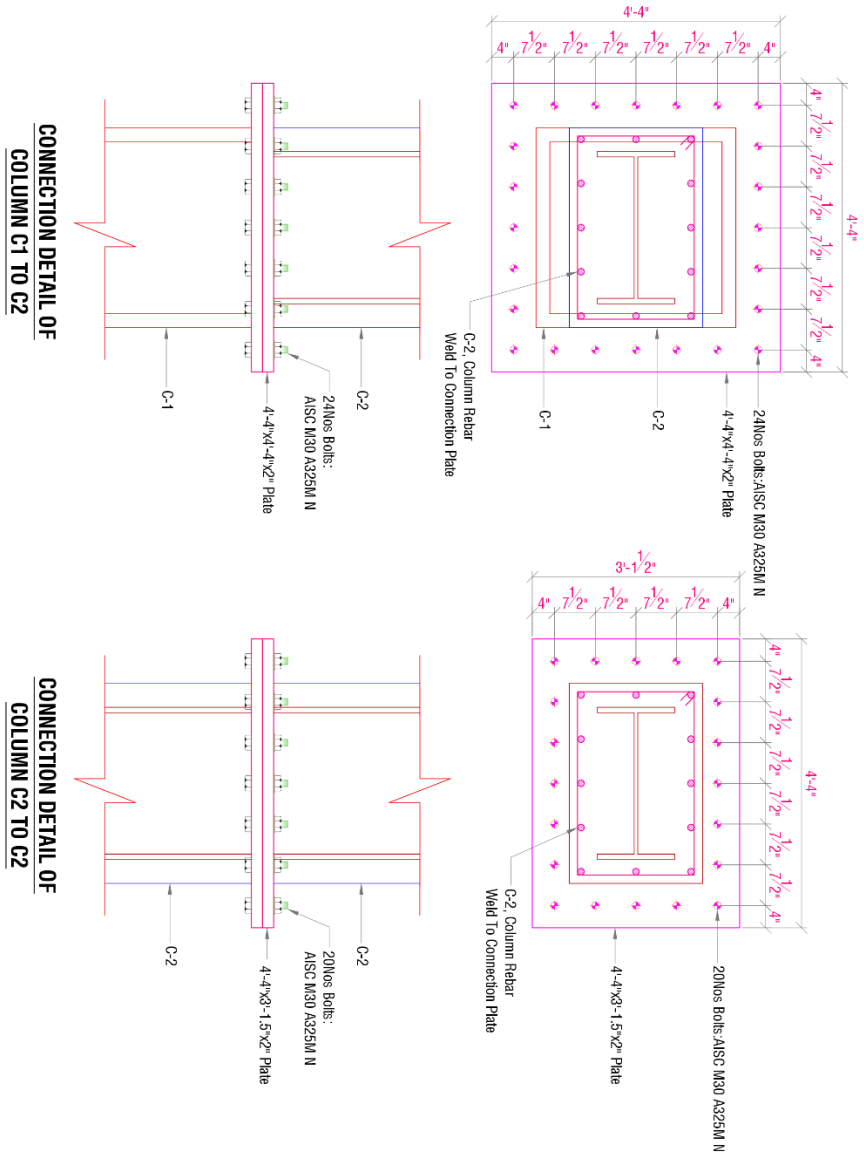


Figure 6.11 Connection Detail of Column (C1 to C2, C2 to C2)

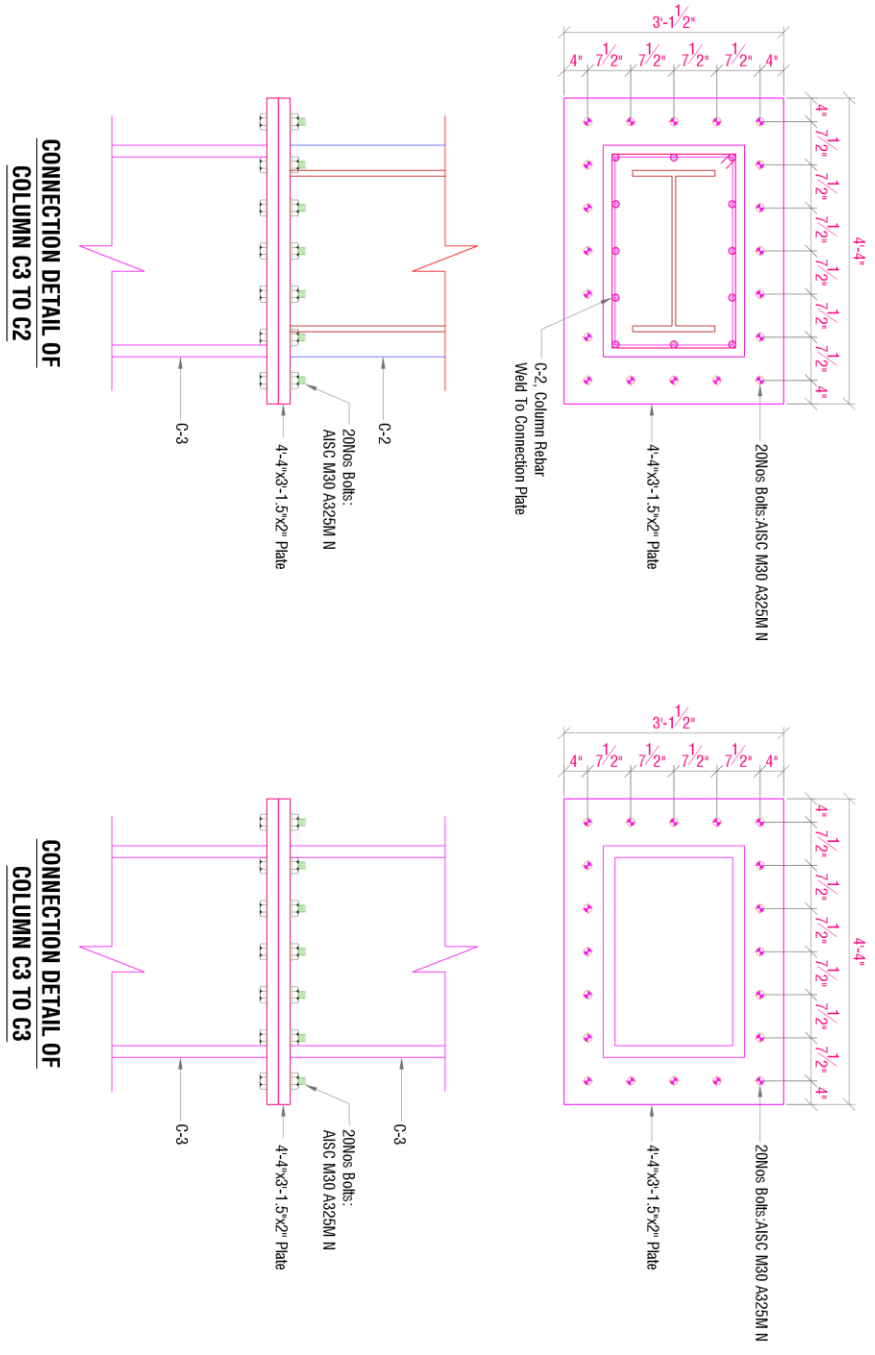


Figure 6.12 Connection Detail of Column (C3 to C2, C3 to C3)

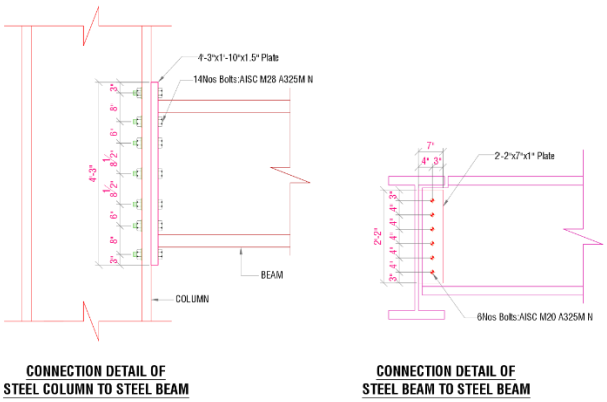


Figure 6.13 Connection Detail of Column to Beam and Beam to Beam

6.6 BEAM LAYOUT

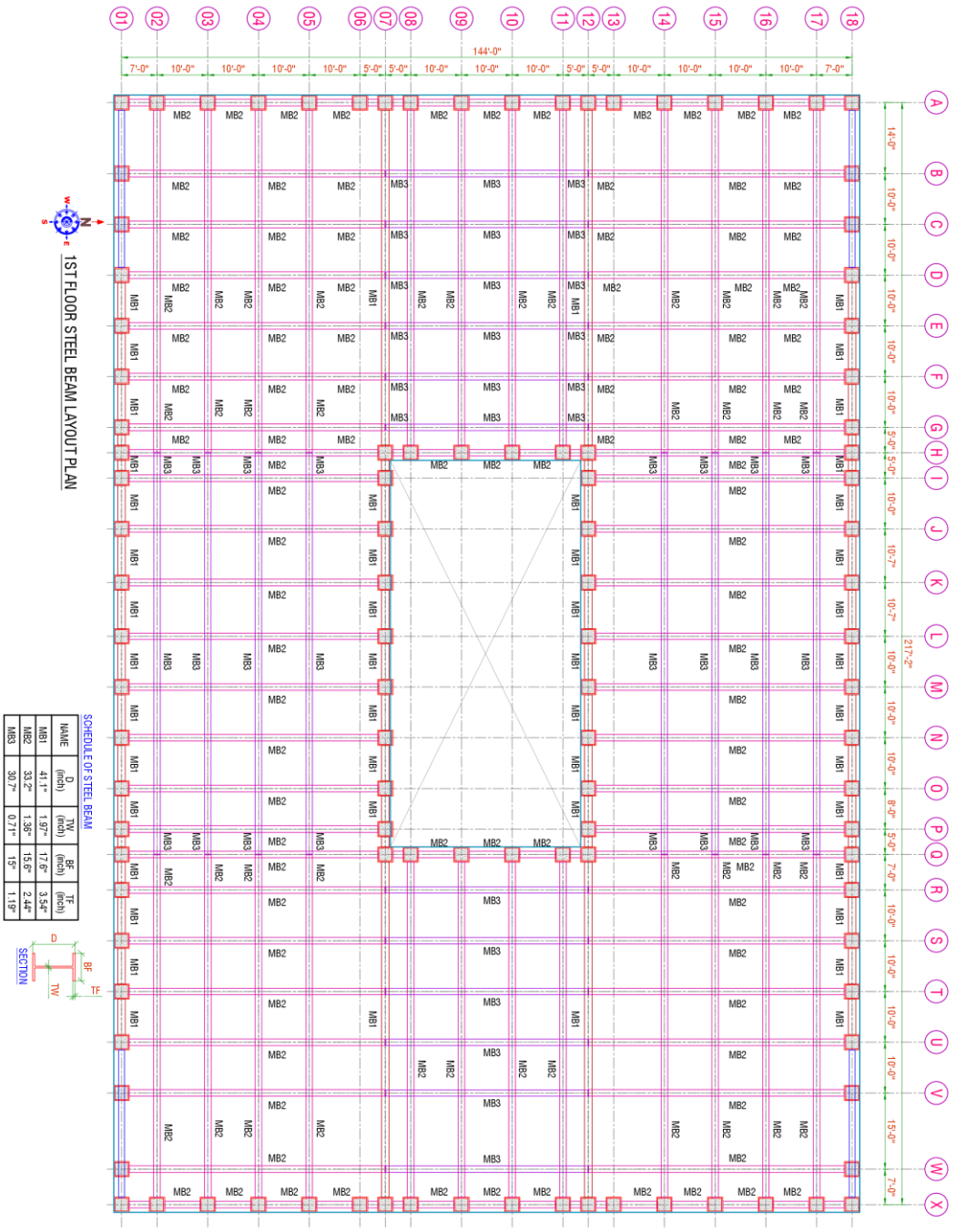


Figure 6.14 1st Floor Steel Beam Layout Plan

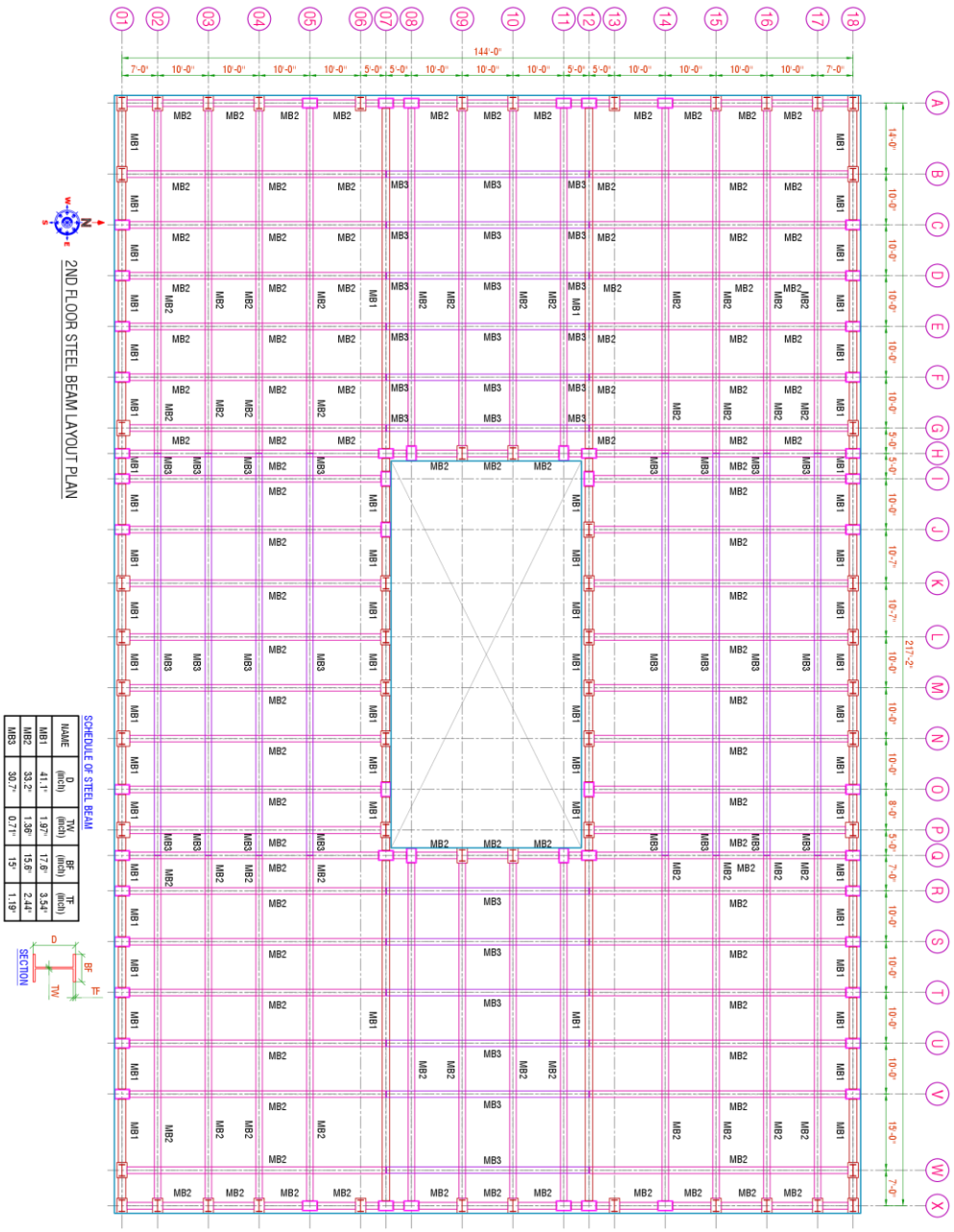


Figure 6.15 2nd Floor Steel Beam Layout Plan

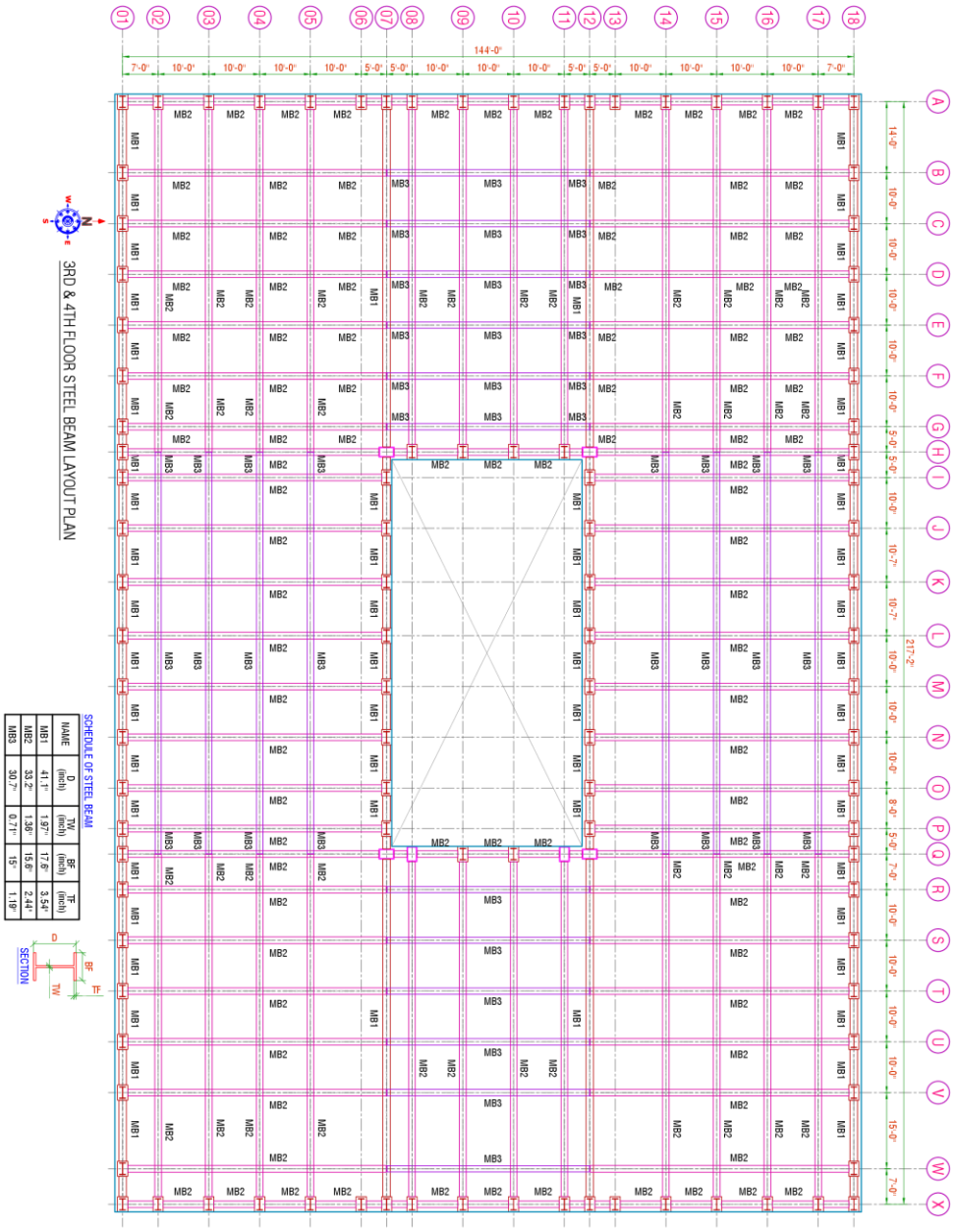


Figure 6.16 3rd & 4th Floor Steel Beam Layout Plan

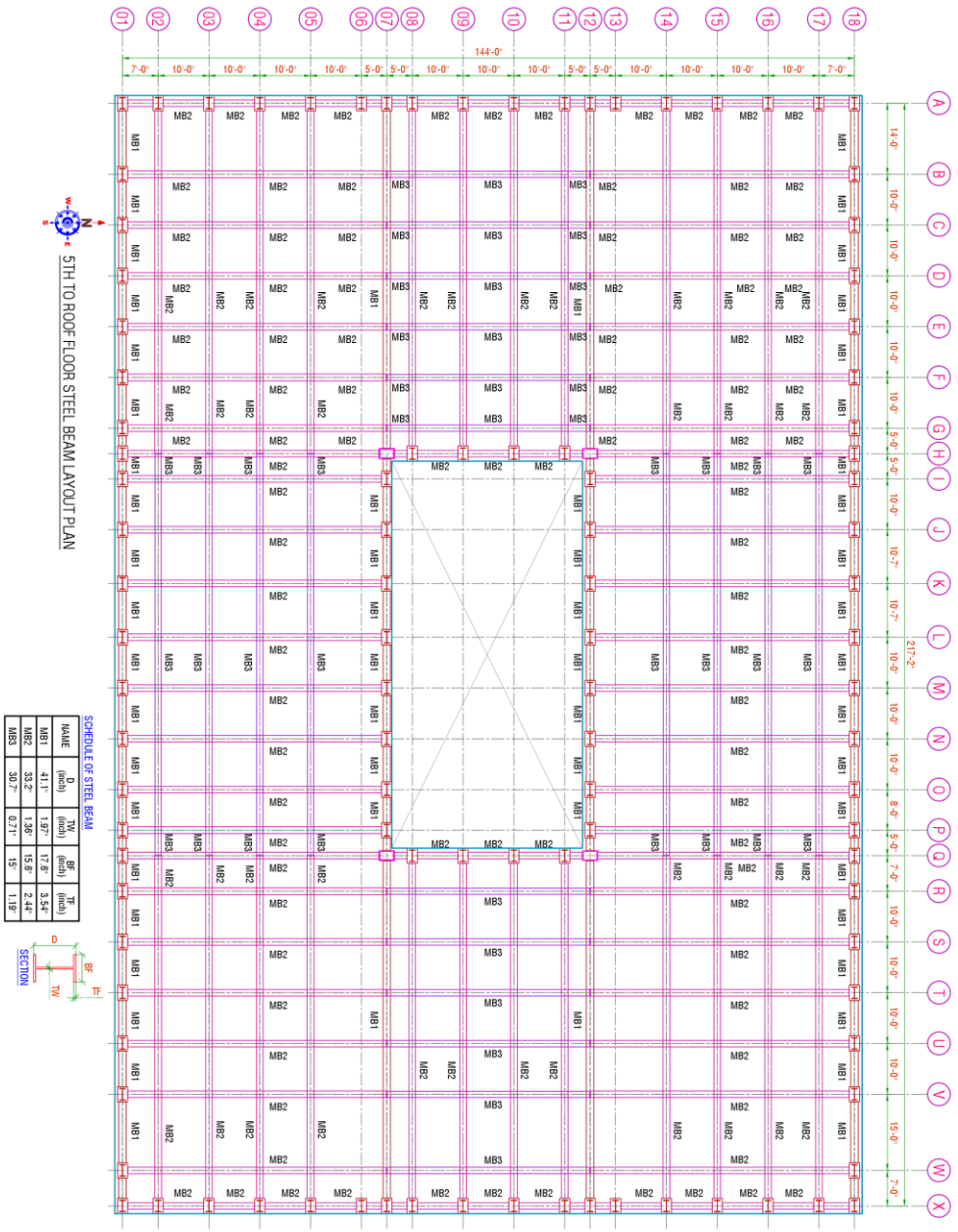


Figure 6.17 5th to Roof Floor Steel Beam Layout Plan

SCHEDULE OF STEEL BEAM

NAME	D (inch)	TW (inch)	BF (inch)	TF (inch)
MB1	41.1"	1.97"	17.6"	3.54"
MB2	33.2"	1.36"	15.6"	2.44"
MB3	30.7"	0.71"	15"	1.19"

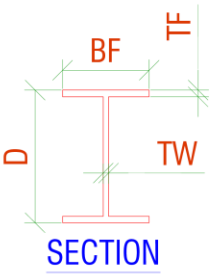


Figure 6.18 Schedule of Steel Beam

6.7 DECK SLAB LAYOUT

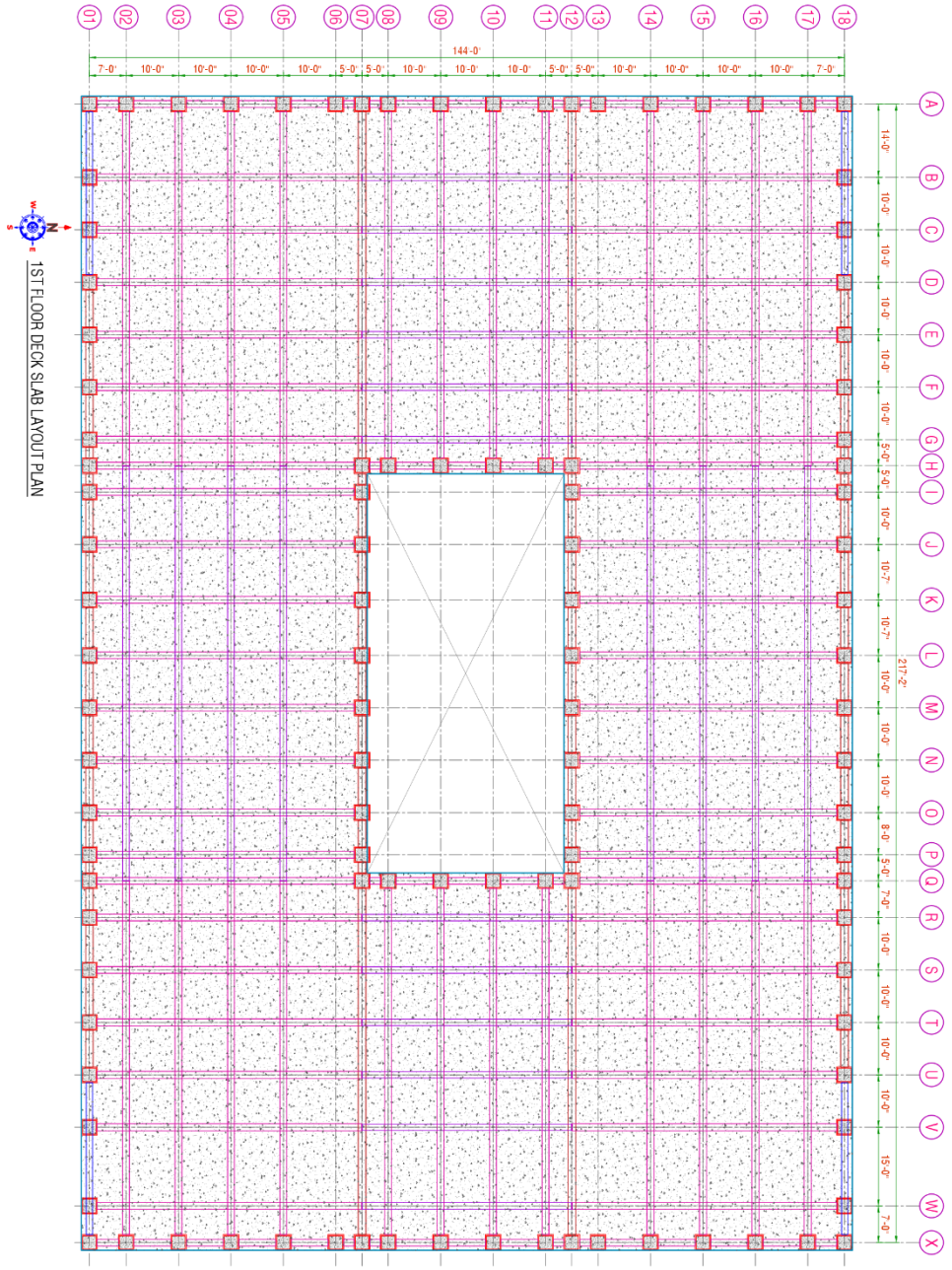


Figure 6.19 1st Floor Deck Slab Layout Plan

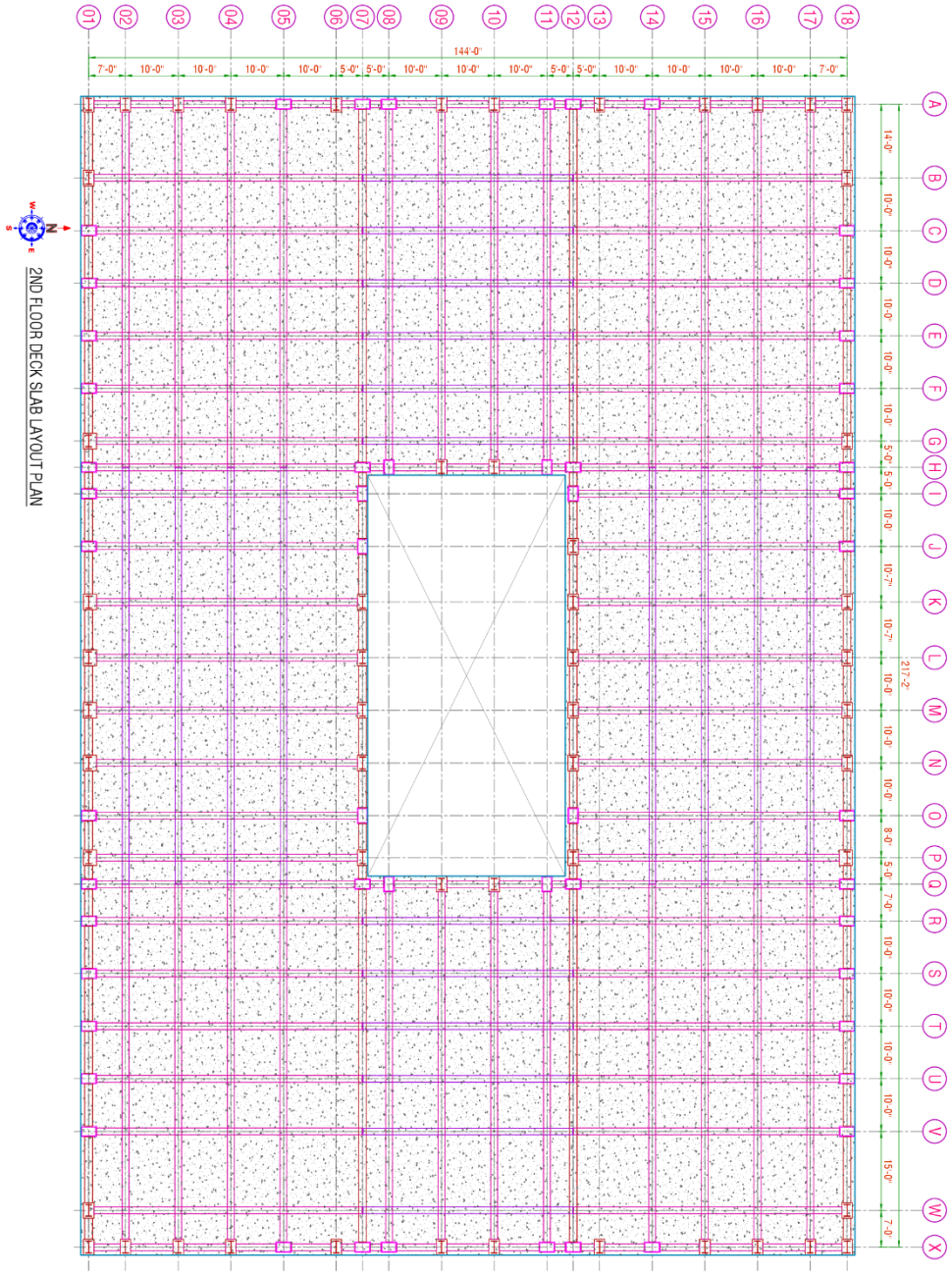


Figure 6.20 2nd Floor Deck Slab Layout Plan

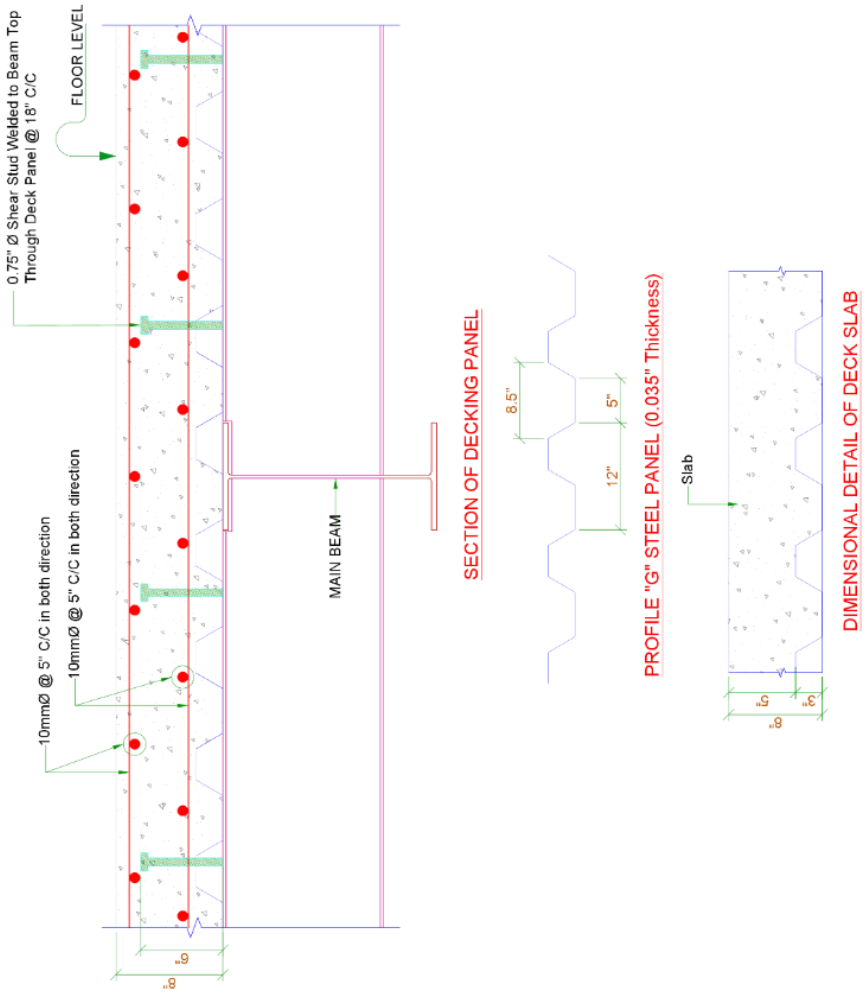


Figure 6.21 Deck Slab Detail

6.8 ELEVATION SECTION

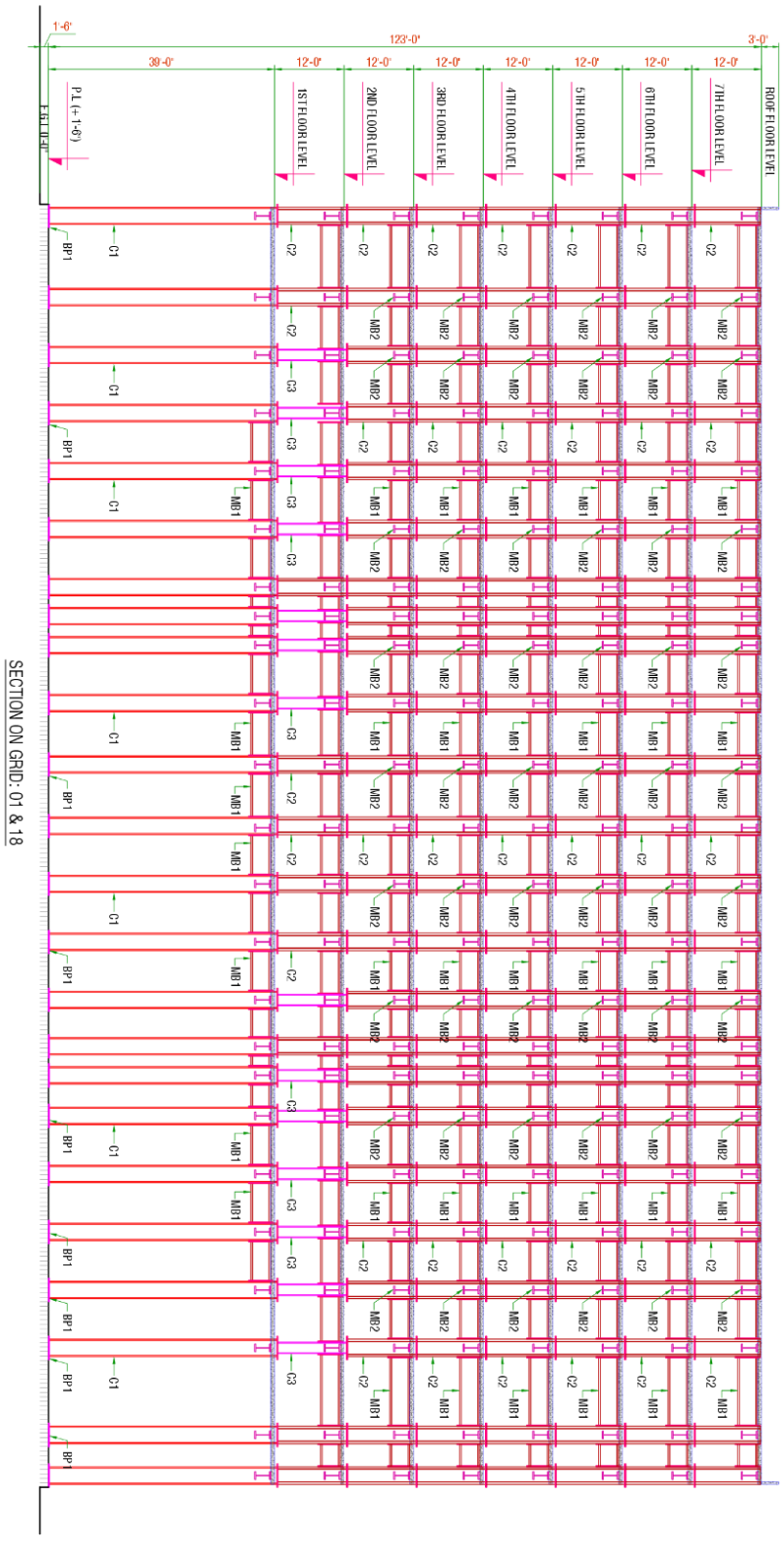


Figure 6.22 Section on Grid 01 & 18

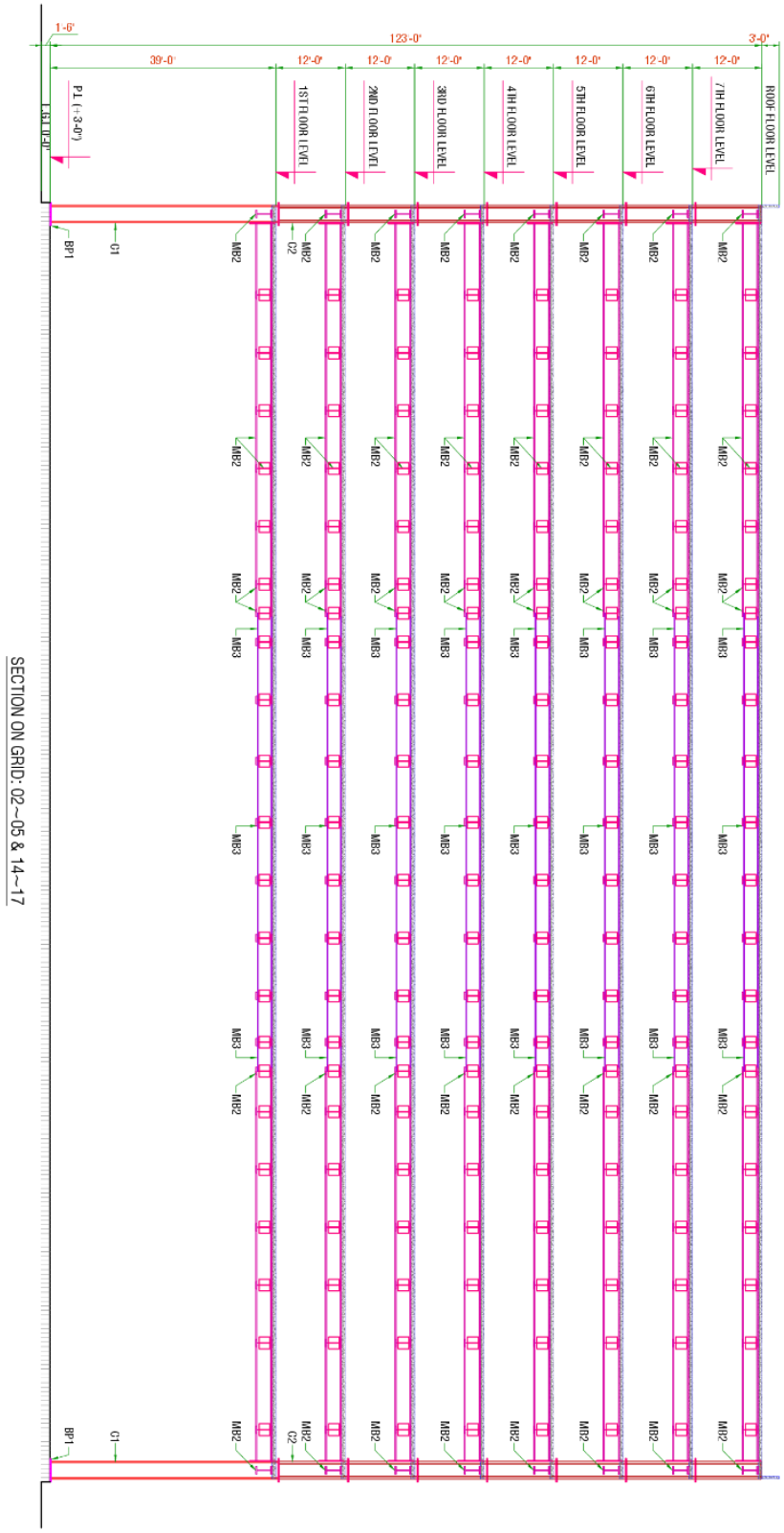


Figure 6.23 Section on Grid 02-05 & 14-17

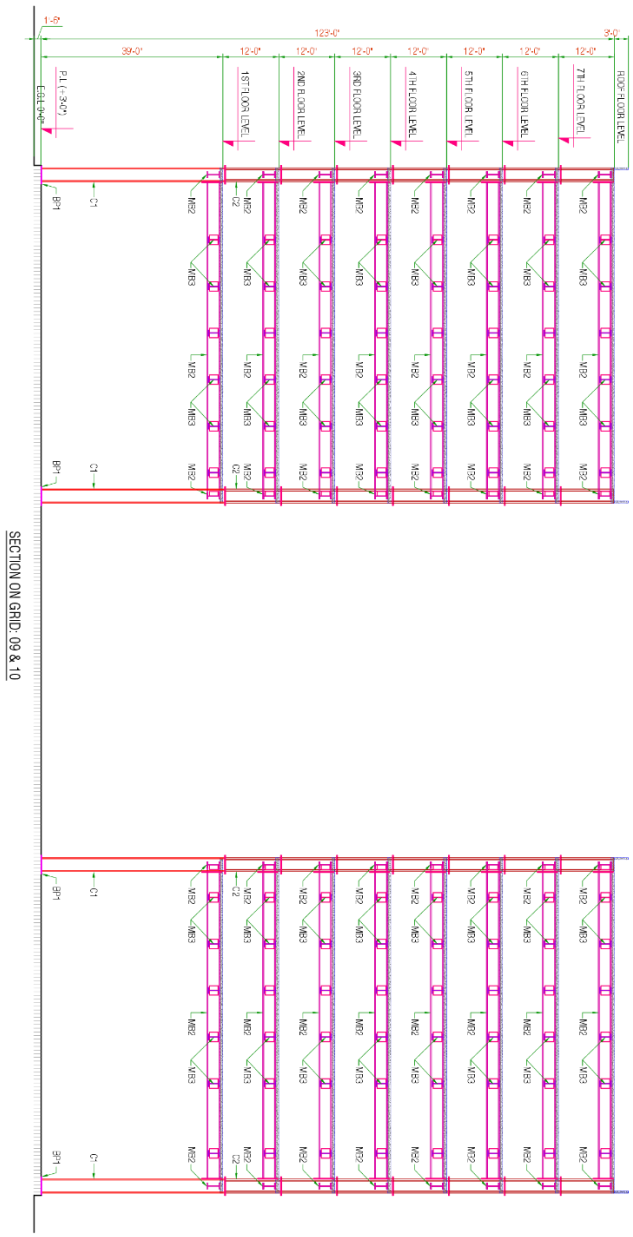


Figure 6.24 Section on Grid 09 & 10

Chapter 7: RESULT AND DISCUSSION

7.1 GENERAL

This chapter presents the results obtained from the structural analysis and design of the proposed seven-story composite steel building constructed above the existing RCC Civil Engineering building at CUET. The design process, carried out using ETABS 18 and supported by AutoCAD and IDEA StatiCa, generated detailed outputs for various structural components of the new building.

The results include reinforcement layouts, section properties, and stress behavior of key structural elements such as composite steel columns, beams, and floor systems. Specific data on member sizes, axial load ratios (PM ratios), and connection performance are derived directly from the software analysis. Additionally, details regarding shear wall design preferences (if applicable), vertical and longitudinal reinforcement areas, and spacing are summarized in tables and diagrams.

Key graphical representations, such as elevations and framing layouts, are included to illustrate how the loads are distributed and resisted by the new structural system. These results ensure that the proposed structure complies with the design requirements set by BNBC 2020 and validates the building's structural safety and performance under all applicable load conditions.

7.2 SHEAR WALL DESIGN RESULTS

The results for the shear wall design are presented in this section, focusing specifically on the structural components such as piers, spandrels, and in-plane reinforcement details. The analysis and design were carried out using ETABS, and the detailed reinforcement requirements—both vertical and horizontal—are provided based on the program's output.

It is important to note that the shear wall design has been implemented only in one principal direction of the building, specifically the longer (major) direction, to ensure adequate lateral load resistance. The placement of piers and spandrels was optimized in this direction to control drift and enhance structural stability.

Complete reinforcement detailing and references for each shear wall component are included in the subsequent figures and tables.

7.3 SHEAR WALL DESIGN PREFERENCES

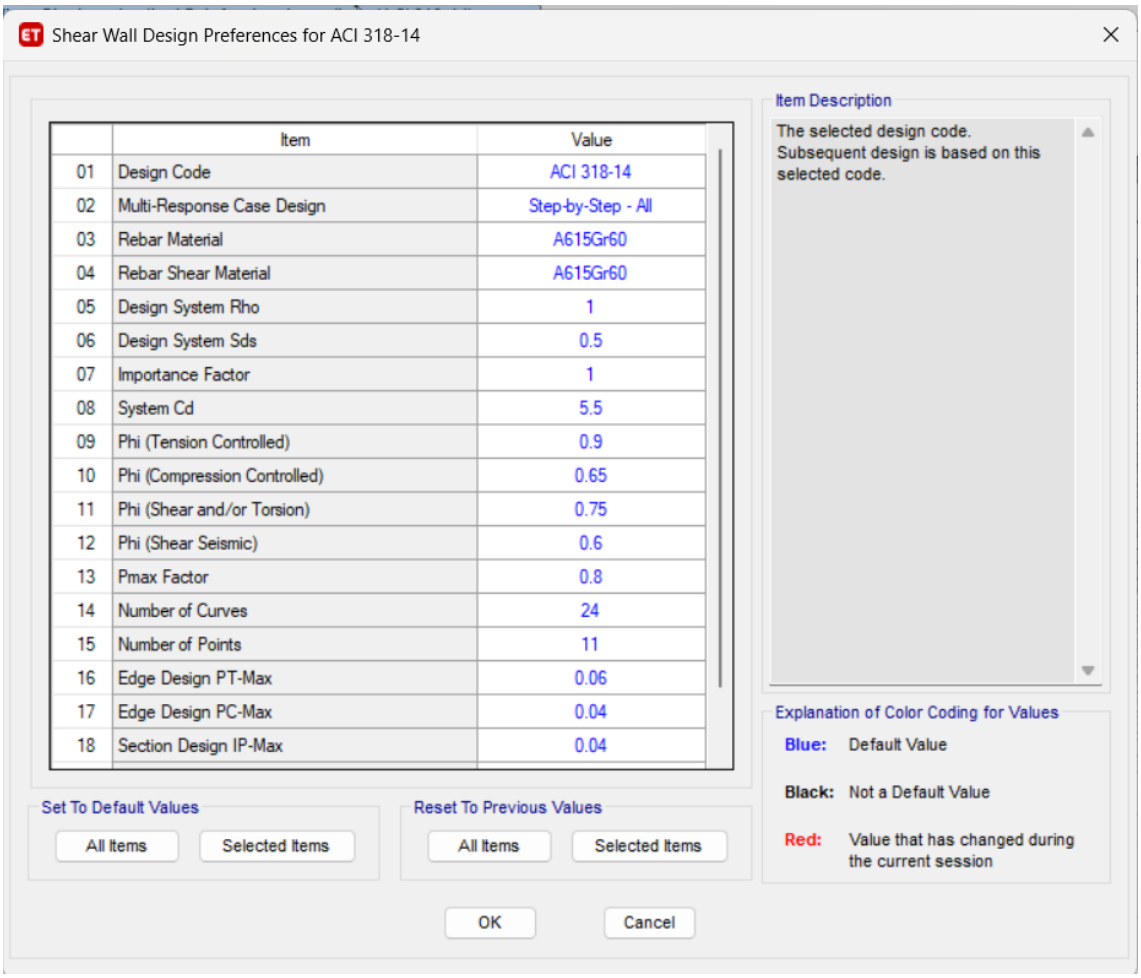


Figure 7.1 Shear Wall Design Preferences

This shows ETABS 18's settings for designing concrete shear walls according to ACI 318-14. It specifies ASTM A615 Grade 60 rebar for main and shear reinforcement. Key parameters include strength reduction factors (Phi: 0.9 tension, 0.65 compression, 0.75 shear, 0.6 seismic shear), seismic design factors (Rho=1, Sds=0.5, I=1, Cd=5.5), and maximum reinforcement ratios

(Edge Tension=0.06, Edge Compression=0.04, Wall Section=0.04). It also defines analysis methods and uses color coding (blue/default, black/modified, red/recent change) to track settings. These preferences govern how ETABS calculates wall strength and reinforcement.

7.4 SPANDREL DESIGN CHART FROM ETABS

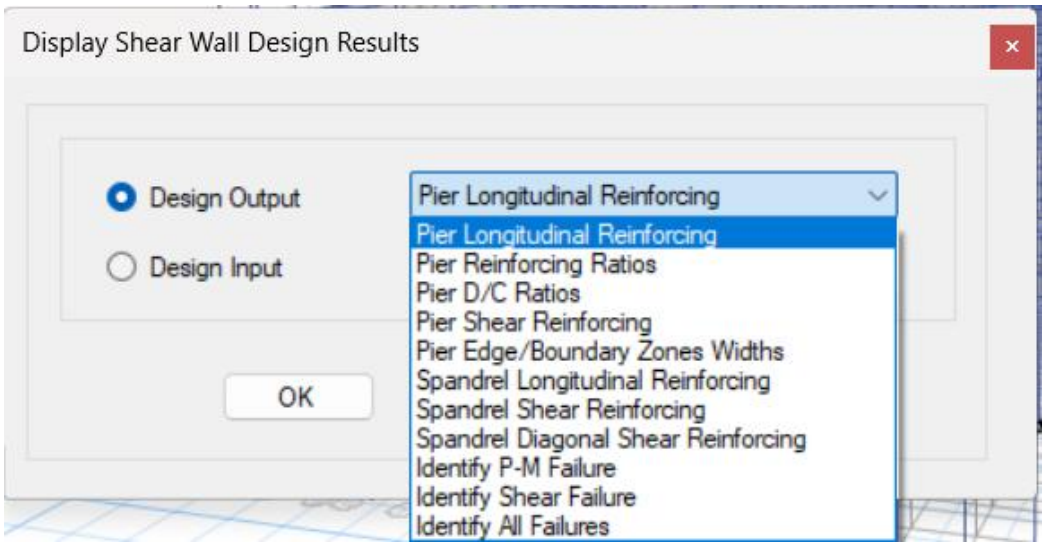


Figure 7.2 Shear Wall Design Result Criteria

This image shows ETABS 18's result display settings for shear wall spandrel design. It allows engineers to visualize reinforcement details (longitudinal, shear, diagonal shear) either by area, ratio, or D/C ratio. It also controls the display of boundary zone widths. Crucially, it includes checkboxes to visually identify structural failures on the model - flexural (P-M), shear, or all failures - enabling quick assessment of critical elements. The selected "Design Output" and "Per" filters determine what specific results are shown for the designed spandrels.

7.5 PIER LONGITUDINAL REINFORCING RESULT

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	LLRF
1st Floor	P1	84	0	168	14	0.784

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
3604996.5	4000	1	60000	60000

Design Code Parameters

Φ_t	Φ_c	Φ_v	Φ_v (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in
Top	Leg 1	0	0	168	0	168	14
Bottom	Leg 1	0	0	168	0	168	14

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (in²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft	Pier A_R in²
Top	18.9954	0.0081	0.0026	14) 1.3073D-Ex+0.3Ey+L	279.349	16.5594	7196.1733	2352
Bottom	17.6009	0.0075	0.0026	26) 0.793D-Ex+0.3Ey	276.319	-4.4921	-6767.2289	2352

Shear Design

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	ΦV_c kip	ΦV_u kip
Top	Leg 1	0.42	14) 1.3073D-Ex+0.3Ey+L	279.349	7196.1733	371.006	217.407	429.087
Bottom	Leg 1	0.42	14) 1.3073D-Ex+0.3Ey+L	404.262	-7264.9429	371.006	231.672	443.352

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (in)	Governing Combo	P_u kip	M_u kip-ft	Stress Comp lb/in²	Stress Limit lb/in²	C Depth in	C Limit in
Top-Left	Leg 1	4.7767	13) 1.3073D+Ex-0.3Ey+L	70.76	-4550.0573	859.18	800	9.5534	37.3333
Top-Right	Leg 1	7.7016	13) 1.3073D+Ex-0.3Ey+L	308.509	6929.05	1393.75	800	15.4031	37.3333
Bottom-Left	Leg 1	9.2383	15)1.3073D-Ex-0.3Ey+L	433.421	-7044.0517	1467.81	800	18.4765	37.3333
Bottom-Right	Leg 1	6.3134	15)1.3073D-Ex-0.3Ey+L	195.673	5072.7801	1007.53	800	12.6269	37.3333

Figure 7.3 ETABS Shear Wall Design

7.6 PIER LONGITUDINAL REINFORCING RESULT

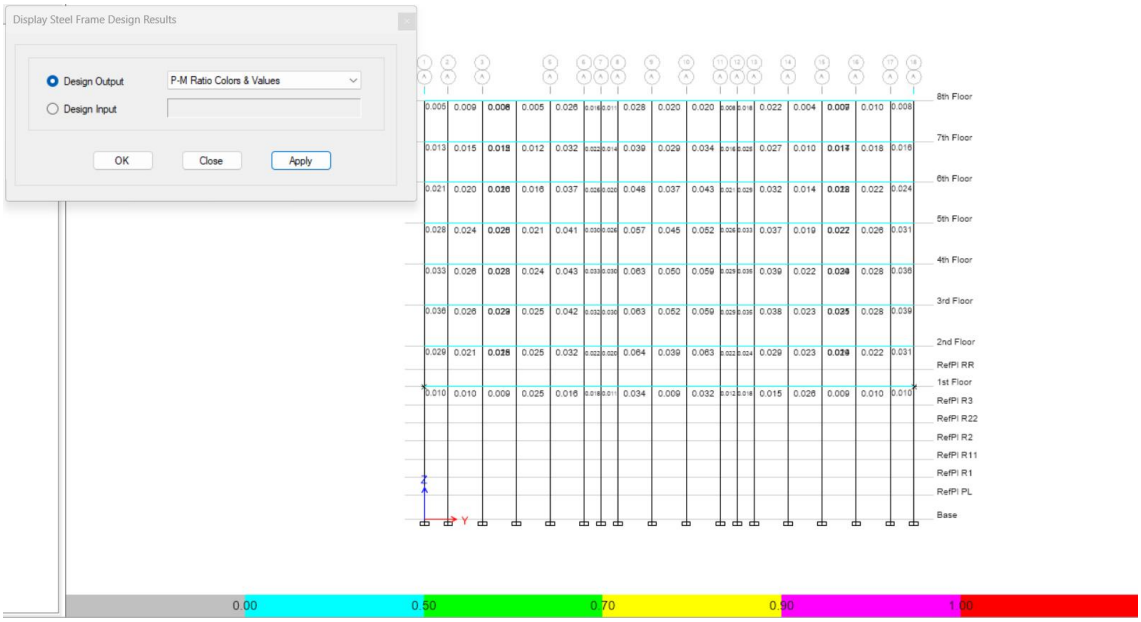


Figure 7.4 P-M Ratio value in elevation

This image shows ETABS 18's graphical output for shear wall pier design, specifically the P-M (Axial-Flexural) Demand/Capacity (D/C) ratio displayed in elevation. Colors indicate safety levels: blue (safe, low D/C), green/yellow (moderate), and red (critical, D/C ≥ 1.0). The marked value of 0.71 signifies demand is 71% of capacity at that section. This visualization allows engineers to instantly identify potential failure zones (red areas) where reinforcement may need adjustment to meet ACI 318-14 requirements.

7.7 P-M COLORS/SHEAR RATIO VALUES

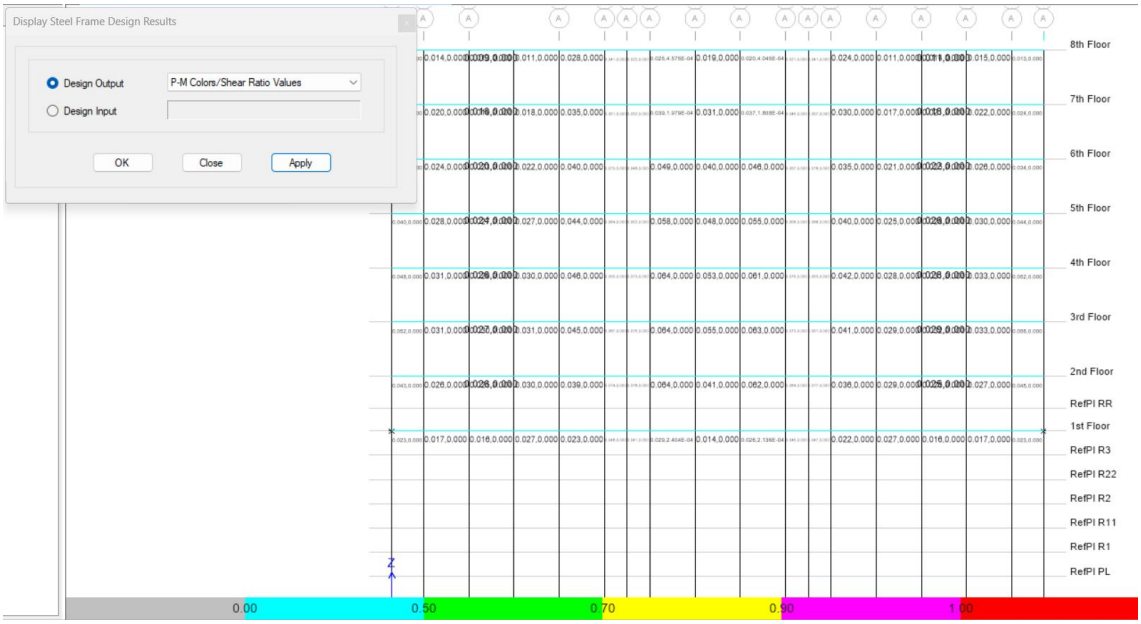


Figure 7.5 P-M Colors/Shear Ratio values

This image shows ETABS 18's combined visualization for shear wall design checks. Color bands indicate the P-M (Axial-Flexural) Demand/Capacity ratio (blue= safe, red= critical/failed). Numerical labels (like 0.58) show the Shear Demand/Capacity ratio at specific points. A value below 1.0 (e.g., 0.58) means shear demand is less than capacity (safe), while ≥1.0 indicates potential shear failure. This combined view enables engineers to quickly identify regions where the wall may be overstressed in flexure (based on color) or shear (based on numbers) according to ACI 318-14.

7.8 P-M RATIO COLOURS

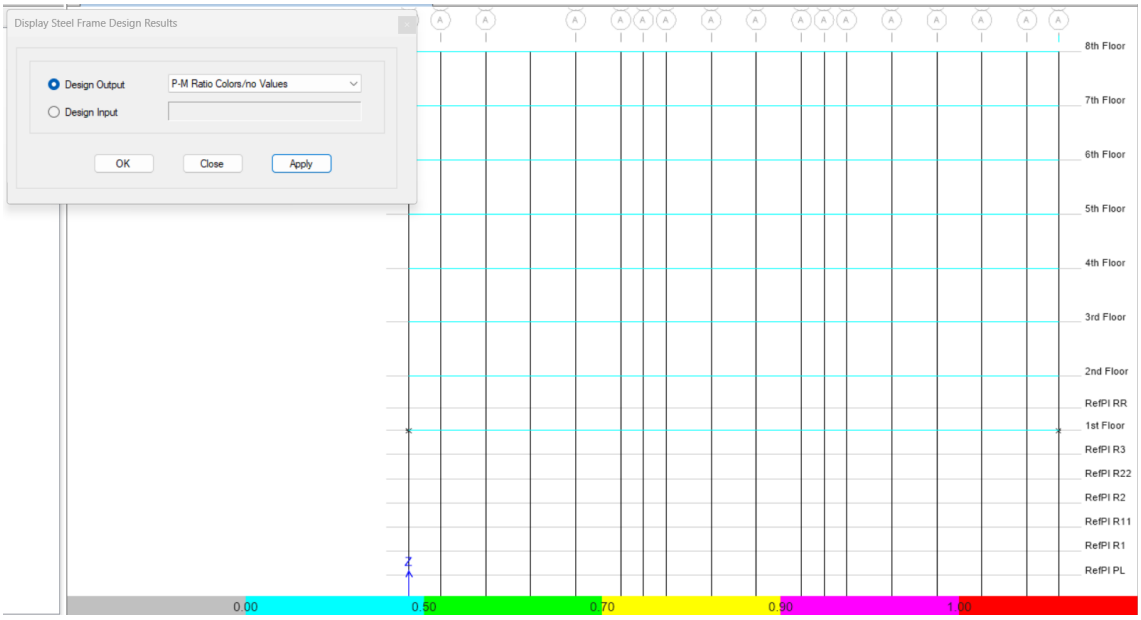


Figure 7.6 P-M Ratio Colors

This image visualizes P-M (Axial-Flexural) D/C ratios for a shear wall pier in ETABS 18. Colors indicate structural safety: blue (safe, low stress), green/yellow (moderate), and red (critical/failure, D/C ≥ 1.0). Unlike Figure 7.5, this view focuses only on flexural-axial performance without shear ratios. Engineers use this to pinpoint wall sections needing reinforcement adjustments (e.g., where red appears). It validates whether the design meets ACI 318-14 strength requirements under combined loading.

7.9 CONT. PLATE AREAS/DOUBLER PLATE THICKNESS

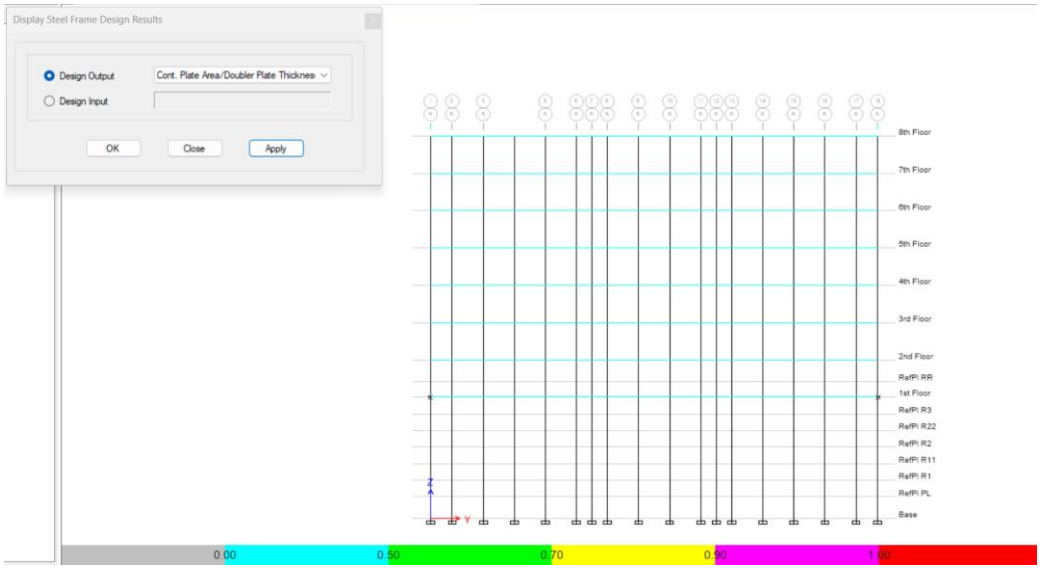


Figure 7.7 Cont. Plate Areas/DOUBLER Plate thickness in elevation

This shows ETABS 18's settings for visualizing steel joint reinforcement details in elevation. It controls the display of continuity plate areas (stiffeners at connections) and doubler plate thickness (shear reinforcement in panel zones). Tools like "Deep Outset" adjust viewing depth, while "Group Points" manages node groupings. The "Clear Rate" option resets display parameters. Engineers use this to verify joint reinforcement adequacy in steel frames, with "Apply" saving settings and "Close" exiting the dialog.

7.10 STEEL BEAM/COLUMN CAPACITY RATIOS

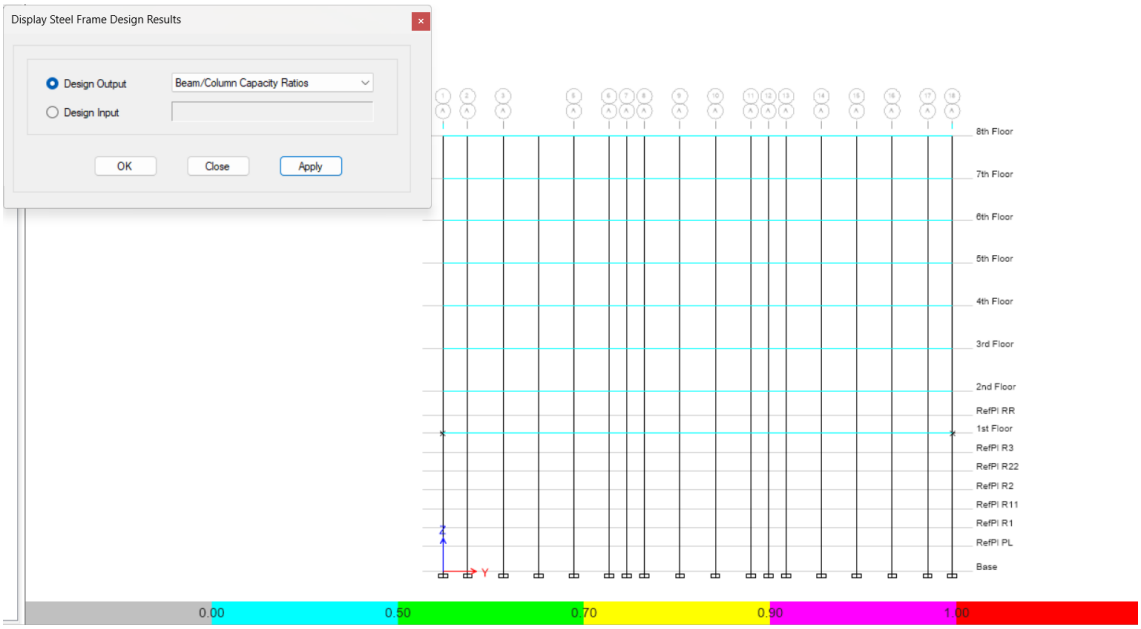


Figure 7.8 Steel Beam/Column Capacity Ratios

This shows ETABS 18's interface for reviewing steel member D/C ratios, indicating structural safety under loads. The color-coded scale (blue: safe, red: failed) visualizes ratios where ≥1.00 denotes failure. Though "Design Input" is selected (typically for setup), the focus is on capacity ratios across floors—e.g., "Soil Floor" and roof levels ("R1R1" etc.). Engineers use this to identify overstressed beams/columns (yellow/red zones) requiring redesign per steel codes (AISC 360). Buttons apply or close the dialog.

7.11 P-M COLORS/BEAM END SHEAR FORCE

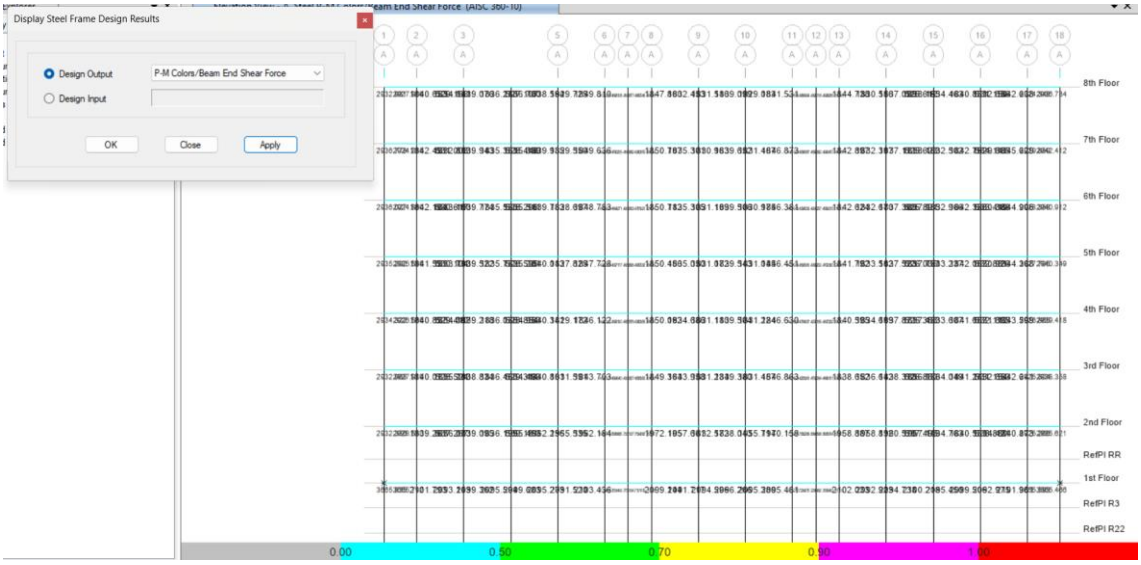


Figure 7.9 P-M Colors/Beam end Shear force

This image configures ETABS 18's visualization of steel beam/column results, combining P-M D/C ratios (color-coded) and beam-end shear forces. The color scale indicates structural safety (blue: safe, red: failure), while shear values inform connection design. Core functions allow engineers to simultaneously assess flexural-axial performance and critical shear forces. This aids in identifying overstressed members/joints per AISC 360.

7.12 STEEL FRAME DESIGN CHECK RESULT DETAILS

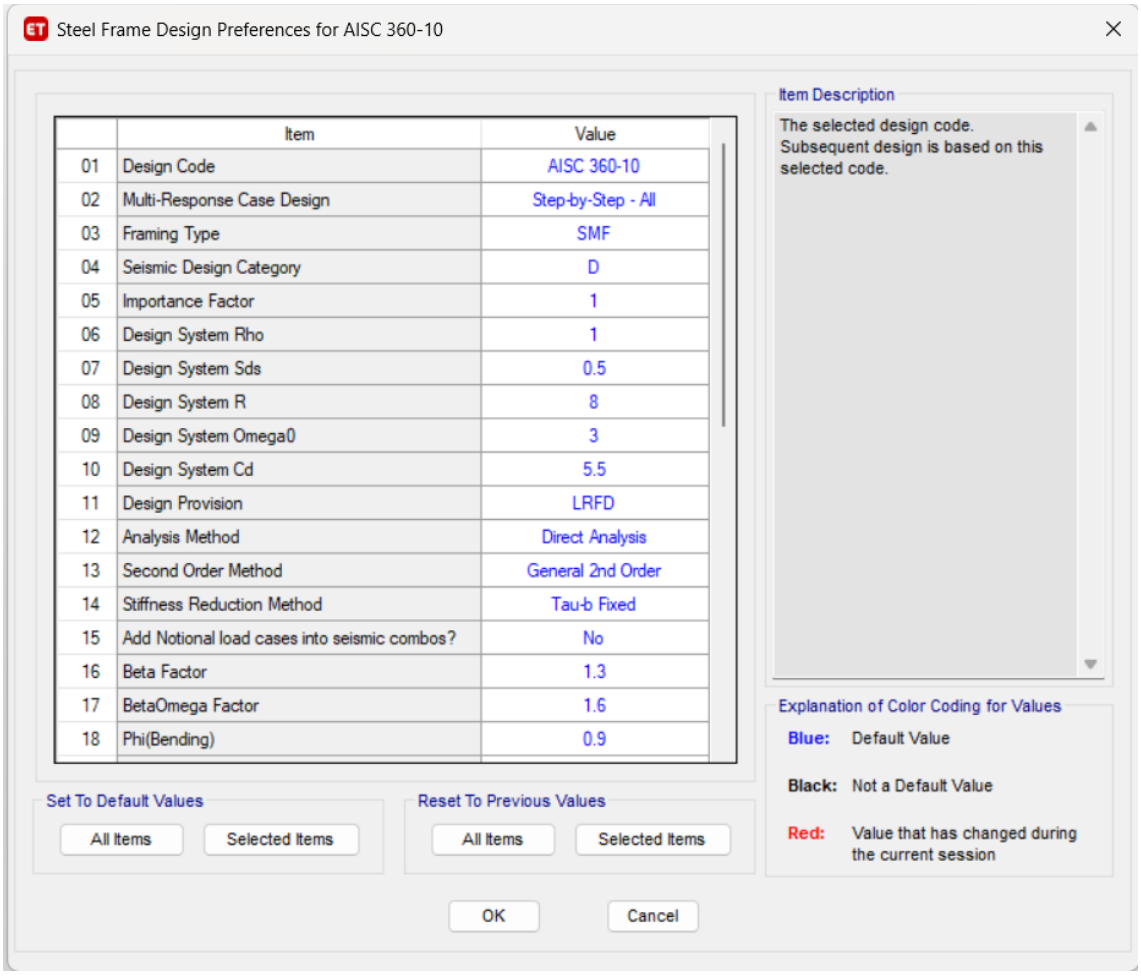


Figure 7.10 Values in preference

This image configures ETABS 18's steel design per AISC 360-10 LRFD for Special Moment Frames (SMF) in Seismic Category D. Key parameters include: seismic factors (R=8, Ω_0=3, Sds=0.5g), Direct Analysis method with stiffness reduction (Tau-b Fixed), and safety factors (Phi=0.9 for bending, Beta=1.3). The settings enforce stringent seismic detailing (e.g., SMF requirements) and advanced stability checks. Color coding (blue/default, black/modified, red/recent change) tracks customization. Buttons allow resetting defaults or applying changes.

ETABS Steel Frame Design

AISC 360-10 Steel Section Check (Deflection Details)

Element Details

Level	Element	Unique Name	Location (in)	Combo	Element Type	Section
8th Floor	B189	1691	60	DStlD2	Special Moment Frame	W30X391

LLRF and Demand/Capacity Ratio

L (in)	LLRF	Stress Ratio Limit
120.0000	1	0.95

DEFLECTION DESIGN (Combo DStlD2)

Type	Consider	Deflection in	Limit in	Ratio	Status
Dead Load	Yes	0.0002	1	2.344E-04	OK
Super DL + Live Load	Yes	0.0001	1	9.568E-05	OK
Live Load	Yes	0.0001	0.3333	2.418E-04	OK
Total Load	Yes	0.0003	0.5	0.001	OK
Total - Camber	Yes	0.0003	0.5	0.001	OK

Figure 7.11 Deflection Details

7.13 STRENGTH SUMMARY

ETABS Steel Frame Design

AISC 360-10 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (in)	Combo	Element Type	Section	Classification
8th Floor	B189	1691	12	DStlS14	Special Moment Frame	W30X391	Seismic HD

LLRF and Demand/Capacity Ratio

L (in)	LLRF	Stress Ratio Limit
120.0000	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0	0	1	0.8	0.8

Seismic Parameters

Ignore Seismic Code?	Ignore Special EQ Load?	Plug Welded?	SDC	I	Rho	S_{DS}	R	Ω_0	C_d
No	No	Yes	D	1	1	0.5	8	3	5.5

Design Code Parameters

Φ_b	Φ_c	Φ_{TY}	Φ_{TT}	Φ_V	Φ_{V-SI}	Φ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (in^2)	J (in^4)	I_{33} (in^4)	I_{22} (in^4)	A_{V2} (in^2)	A_{V3} (in^2)
115	173	20700	1550	76.13	45.15

Design Properties

S_{33} (in^3)	S_{22} (in^3)	Z_{33} (in^3)	Z_{22} (in^3)	r_{33} (in)	r_{22} (in)	C_w (in^6)
1246.99	198.72	1450	310	13.4164	3.6713	365195.18

Material Properties

E (lb/in^2)	f_y (lb/in^2)	R_y	α
29000000	50000	1.1	NA

Stress Check forces and Moments

Location (in)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
12	0	-47.5855	0	-18.522	0	0.0323

Axial Force & Biaxial Moment Design Factors (H1-1b)

	L Factor	K_1	K_2	B_1	B_2	C_m

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.8	1	1	1	1	1
Minor Bending	0.8	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{tb}	K_{tb}	C_b
0.8	1	2.414

Demand/Capacity (D/C) Ratio Eqn.(H1-1b)

D/C Ratio =	$(P_r/2P_c) + (M_{r33}/M_{c33}) + (M_{r22}/M_{c22})$
0.009 =	0 + 0.009 + 0

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
0	4922.636	5175

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	47.5855	5437.5	5437.5	5437.5
Minor Bending	0	1162.5		

Shear Design

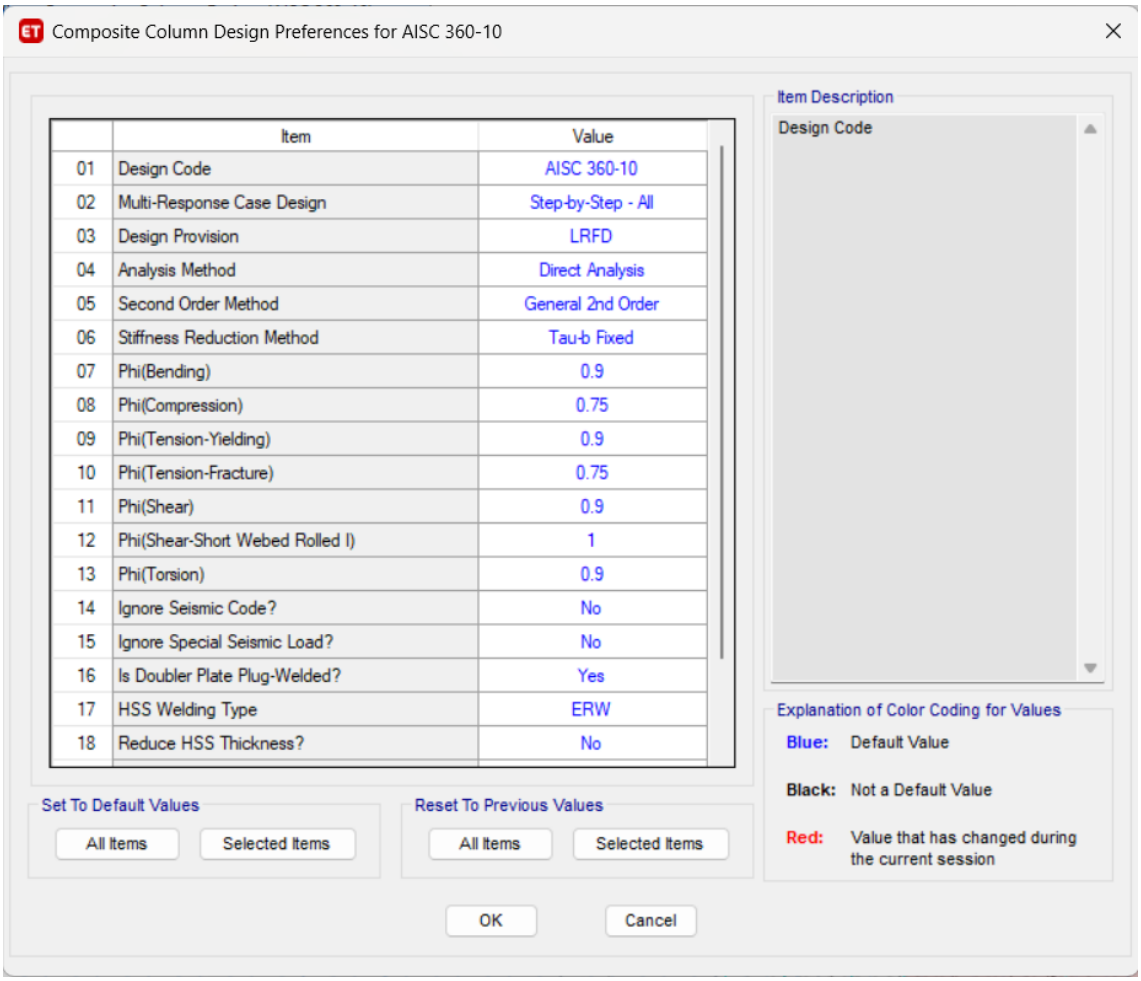
	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	18.522	1354.56	0.014
Minor Shear	0	2055.456	0

End Reaction Major Shear Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
1840.653	DStlS18	1834.641	DStlS18

Figure 7.12 Strength Summary

7.14 COMPOSITE COLUMN DESIGN CHECK RESULT DETAILS

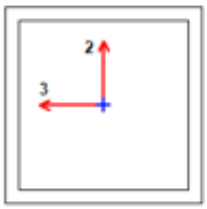


	Item	Value
01	Design Code	AISC 360-10
02	Multi-Response Case Design	Step-by-Step - All
03	Design Provision	LRFD
04	Analysis Method	Direct Analysis
05	Second Order Method	General 2nd Order
06	Stiffness Reduction Method	Tau-b Fixed
07	Phi(Bending)	0.9
08	Phi(Compression)	0.75
09	Phi(Tension-Yielding)	0.9
10	Phi(Tension-Fracture)	0.75
11	Phi(Shear)	0.9
12	Phi(Shear-Short Webed Rolled I)	1
13	Phi(Torsion)	0.9
14	Ignore Seismic Code?	No
15	Ignore Special Seismic Load?	No
16	Is Doubler Plate Plug-Welded?	Yes
17	HSS Welding Type	ERW
18	Reduce HSS Thickness?	No

Figure 7.13 Composite Column Design Preferences

This image shows ETABS 18's preferences for composite steel-concrete column design under AISC 360-10 LRFD. It specifies safety factors (Phi), seismic compliance and construction details like plug-welded doubler plates and ERW HSS welding. The Direct Analysis method with Fixed stiffness reduction ensures advanced stability checks. Settings enforce ductile failure modes (e.g., higher Phi for bending vs. compression). Color coding tracks customization, while buttons reset defaults or apply changes for seismic-resistant composite systems.

ETABS Composite Column Design



Element Details

Level	Element	Unique Name	Location (in)	Combo	Section	Classification
1st Floor	C163	430	0	DCon13	$tube_4$	Compact

LLRF and Demand/Capacity Ratio

L (in)	LLRF	Stress Ratio Limit
468.0000	0.424	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$
0.081	0.01

Seismic Parameters

Ignore Seismic Code?	Ignore Special EQ Load?	Plug Welded?
No	No	Yes

Design Code Parameters

Φ_b	Φ_c	Φ_{tY}	Φ_{tP}	Φ_v	$\Phi_{v,III}$	$\Phi_{v,I}$
0.9	0.75	0.9	0.75	0.9	1	1

Design Properties of Steel Section

A (in^2)	J (in^4)	I_{33} (in^4)	I_{22} (in^4)	A_{v2} (in^2)	A_{v2} (in^2)
335	93988.44	63007.92	63007.92	175.09	175.1

Material Properties

E_s (lb/in^2)	f'_c (lb/in^2)	F_y (lb/in^2)
29000000	4000	36000

Demand/Capacity (D/C) Ratio Eqn.(H1-1b)

D/C Ratio =	$(P_r/2P_c) + (M_{r33}/M_{c33}) + (M_{r22}/M_{c22})$

D/C Ratio =	$(P_r/2P_c) + (M_{r33}/M_{c33}) + (M_{r22}/M_{c22})$
0.098 =	0.045 + 0.027 + 0.026

Stress Check forces and Moments

Location (in)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
0	-973.031	327.869	314.4777	-14.515	-13.448	0

Axial Force & Biaxial Moment Design Factors (H1-1b)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.929	1	1	1	1	0.379
Minor Bending	0.929	1	1	1	1	0.38

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.929	0	2.184

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
973.031	10808.756	10854

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n Capacity (kip-ft)	ϕM_n (No LTB) (kip-ft)
Major Bending	327.869	12120.6485	12120.6485
Minor Bending	314.4777	12120.6485	

Torsion Moment and Capacities

T_u Moment (kip-ft)	T_n Capacity (kip-ft)	ϕT_n Capacity (kip-ft)
0	9991.6078	8992.447

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)
Major Shear	14.515	2770.2
Minor Shear	13.448	2770.2

Figure 7.14 ETABS Composite Column Design

7.15 MAXIMUM STORY DISPLACEMENT CHECK

Table 7.1 Maximum Story Displacement Check

Load combination	Maximum story displacement (in)	Story height above ground(ft)	Story height above ground (in)	Allowable story displacement (in)	Result
D+0.5L+0.7Wx	0.191	123	1476	2.952	Ok
D+0.5L-0.7Wx	0.143				Ok
D+0.5L+0.7Wy	0.208				Ok
D+0.5L-0.7Wy	0.083				Ok

The calculated maximum story displacements (ranging from 0.083 in to 0.208 in) under the specified load combinations (D+0.5L±0.7Wx/Wy) are all significantly below the BNBC 2020 (Page 503) allowable limit of 2,952 inches (h/600 for a story height of 1,476 in), confirming full compliance with sway limitations and indicating ample structural stiffness; consequently, all results are marked "Ok" and no corrective actions are required.

7.16 DRIFT RATIO CHECK

The drift ratio check, based on BNBC 2020, was performed for a building modeled in ETABS 18 under earthquake load in both X and Y directions. The drift ratios were amplified using a deflection amplification factor (Cd = 5.5) and importance factor (I = 1.25). All amplified drift values for each story were compared against the allowable drift limit of 0.015. The results show that drift ratios for all floors in both directions are well within the permissible limit, indicating that the lateral deflections due to seismic forces are structurally safe and compliant with BNBC 2020 requirements.

Table 7.2 Drift ration check for Ex in X direction

Drift ration check for Ex in X direction				
Story	Drift ratio	Amplified Drift ratio	Allowable Drift ratio limit	Result
8th Floor	0.00018	0.000792	0.015	ok
7th Floor	0.000288	0.0012672		ok
6th Floor	0.000391	0.0017204		ok
5th Floor	0.000477	0.0020988		ok
4th Floor	0.000542	0.0023848		ok
3rd Floor	0.000568	0.0024992		ok
2nd Floor	0.000448	0.0019712		ok
1st Floor	0.000145	0.000638		ok
Base	0	0		ok

Table 7.3 Drift ratio checks for Ex in Y direction

Drift ratio checks for Ex in Y direction				
Story	Drift ratio	Amplified Drift ratio	Allowable Drift ratio	Result
8th Floor	1.50E-05	6.60E-05	0.015	ok
7th Floor	2.50E-05	1.10E-04		ok
6th Floor	3.50E-05	1.54E-04		ok
5th Floor	4.20E-05	1.85E-04		ok
4th Floor	4.90E-05	2.16E-04		ok
3rd Floor	5.10E-05	2.24E-04		ok
2nd Floor	3.20E-05	1.41E-04		ok
1st Floor	3.16E-07	1.39E-06		ok
Base	0	0.00E+00		ok

7.17 TORSIONAL IRREGULARITY CHECK

Table 7.4 Torsional Irregularity Check

Torsional Irregularity Check according to BNBC -2020											
Load Case	1st End Displacement of upper Story	1st End Displacement of lower Story	2nd End displacement Of upper Story	2nd End Displacement of lower Story	1st End Drift	2nd End Drift	Maximum Drift	Average Drift	Maximum Drift /average Drift	Torsional irregularity check Drift	Ectreme Torsional Irregularit Check Result
Ex	0.46	0.436	0.459	0.435	0.024	0.024	0.024	0.024	1	Regular	Regular
Ey	0.833	0.777	0.816	0.761	0.056	0.055	0.056	0.0555	1.009009009	Regular	Regular

The torsional irregularity check per BNBC 2020 confirms that all stories comply with the code requirements, as the maximum story drift at any point does not exceed 1.2 times the average story drift for the same load case in either the X or Y direction. This indicates balanced structural resistance against twisting under lateral loads, with no torsional amplification required. The "Regular" classification across all levels validates the adequacy of the structural model's configuration and mass distribution, satisfying seismic and wind performance criteria without irregularities.

7.18 SOFT STORY AND EXTREME SOFT STORY CHECK

Stiffness irregularity of Soft story and Extreme soft story Checks for Ex in X direction

Story Name	Soft story Check						Extreme soft story Check			
	Stiffness ,X	70%of Stiffness	Result	Average of stiffness of 3 stories above	80%of Average stiffness of 3 stories avove	Result	60%of Stiffness	Result	70% of Average of Stiffness of 3 stories above	Result
8th Floor	25858.4012	18100.88084			0		15515.04072		0	
7th Floor	31613.8033	22129.66231	regular		0		18968.28198	regular	0	
6th Floor	32959.5933	23071.71531	regular	21100.75282	16880.60226		19775.75598	regular	14770.52697	
5th Floor	33670.9766	23569.68362	regular	22923.68708	18338.94966	regular	20202.58596	regular	16046.58096	regular
4th Floor	34418.8734	24093.21138	regular	23578.20344	18862.56275	regular	20651.32404	regular	16504.74241	regular
3rd Floor	36484.5378	25539.17646	regular	24400.69049	19520.55239	regular	21890.72268	regular	17080.48334	regular
2nd Floor	49462.51	34623.757	regular	28085.38161	22468.30529	regular	29677.506	regular	19659.76713	regular
1st Floor	47818.3578	33472.85046	regular	31211.92797	24969.54238	regular	28691.01468	regular	21848.34958	regular

Stiffness irregularity of Soft story and Extreme soft story Checks for Ey in Y direction

Story Name	Soft story Check						Extreme soft story Check			
	Stiffness , Y	70%of Stiffness	Result	Average of stiffness of 3 stories above	80%of Average stiffness of3 stories avove	Result	60%of Stiffness	Result	70% of Average of Stiffness of 3 stories above	Result
8th Floor	11565.54	8095.878			0		4857.5268			
7th Floor	13792.13	9654.491	regular		0		5792.6946	regular		
6th Floor	14232.1445	9962.50115	regular	9237.623383	7390.098707		5977.50069	regular	6466.336368	
5th Floor	14572.8149	10200.97043	regular	9939.32086	7951.456688	regular	6120.582258	regular	6957.524602	regular
4th Floor	15422.9825	10796.08775	regular	10319.85311	8255.882488	regular	6477.65265	regular	7223.897177	regular
3rd Floor	17830.0294	12481.02058	regular	11159.35959	8927.487669	regular	7488.612348	regular	7811.551711	regular
2nd Floor	35289.3613	24702.55291	regular	15993.22041	12794.57633	regular	14821.53175	regular	11195.25429	regular
1st Floor	28790.009	678021.5732	regular	238401.7156	190721.3724	regular	406812.9439	regular	166881.2009	regular

All stories (2nd to 8th floors) satisfy BNBC 2020 requirements for stiffness irregularity in the X direction, with no soft story or extreme soft story conditions detected. Each story's stiffness exceeds 80% of the average stiffness of the three stories above it (soft story check) and 70% of that same average (extreme soft story check), as confirmed by "regular" classifications across all results. The consistent compliance indicates uniform vertical stiffness distribution without abrupt changes, validating the structural model's performance against lateral deformation criteria per BNBC 2020.

7.19 P- DELTA CHECK

Ex

Story	height (ft)	height(in)	Gravity force /story force (P)	Story shear (Vx)	Displacement ,δ	Story drift,Δ	θ	result
8th Floor	12	144	9753.041	-636.099	0.460852	0.108284	0.002096	No P-delta
7th Floor	12	144	19501.009	-1233.468	0.436242	0.1720092	0.003434	No P-delta
6th Floor	12	144	29248.976	-1745.492	0.397149	0.2334288	0.004939	No P-delta
5th Floor	12	144	38996.944	-2175.835	0.344097	0.2847944	0.006445	No P-delta
4th Floor	12	144	48754.796	-2528.486	0.279371	0.3236332	0.007879	No P-delta
3rd Floor	12	144	58512.649	-2807.576	0.205818	0.3389408	0.008919	No P-delta
2nd Floor	12	144	68523.836	-3020.68	0.128786	0.2673132	0.007657	No P-delta
1st Floor	39	468	86655.489	-3234.538	0.068033	0.2993452	0.003116	No P-delta

Ey

Story	height (ft)	height(in)	Gravity force /storyforce (P)	Story shear (Vy)	Displacement ,δ	Story drift,Δ	θ	result
8th Floor	12	144	9753.041	-636.099	0.833203	0.2446972	0.004737	No P-delta
7th Floor	12	144	19501.009	-1233.468	0.77759	0.3956392	0.007898	No P-delta
6th Floor	12	144	29248.976	-1745.492	0.687672	0.5430876	0.01149	No P-delta
5th Floor	12	144	38996.944	-2175.835	0.564243	0.6622616	0.014987	No P-delta
4th Floor	12	144	48754.796	-2528.486	0.413729	0.7335548	0.017859	No P-delta
3rd Floor	12	144	58512.649	-2807.576	0.247012	0.70125	0.018453	No P-delta
2nd Floor	12	144	68523.836	-3020.68	0.087637	0.3778764	0.010823	No P-delta
1st Floor	39	468	86655.489	850.489	0.001756	0.0077264	0.000306	No P-delta

The P-Delta stability coefficient (θ) check per BNBC 2020 confirms that all stories (ranging from θ=0.002 to θ=0.009) are well below the critical threshold of 0.10, with maximum values at the 3rd Floor (θ=0.0089) and 2nd Floor (θ=0.0077). This indicates negligible second-order effects, validating that gravity-induced lateral deformations are controlled, and no P-Delta amplification of moments or drifts is required in the structural design. All stories are uniformly marked "No P-Delta," confirming global stability under combined gravity and lateral loads.

7.20 STRUCTURAL CHECKS AND STABILITY

- P-Delta effects: Passed
- Torsion check: Passed
- Stiffness irregularity: Passed
- Building is structurally stable and serviceable

7.21 DISCUSSION

7.21.1 Story Displacement and Drift Behaviour

The structural performance of the new seven-story steel building was evaluated under different lateral loads, including seismic and wind forces, using ETABS 18. Maximum story displacements occurred at the top story, as expected in high-rise structures. The displacements for seismic loading were recorded as 0.460 inches in the X-direction (Ex) and 0.833 inches in the Y-direction (Ey). Under wind loading, the displacements were 0.108 inches (Wx) and 0.297 inches (Wy) respectively. All displacement values were well within the allowable limit of 2.95 inches as per BNBC 2020 guidelines, confirming structural compliance and serviceability under lateral loads.

The story drift, which indicates relative displacement between adjacent floors, was also checked to ensure inter-story stability. The maximum drift under seismic loading occurred at the third floor in the X-direction (0.000535 inches) and the fourth floor in the Y-direction (0.001158 inches). These values remained well below the code-defined drift limits, ensuring adequate lateral stiffness and minimizing the risk of non-structural damage.

7.21.2 Structural Stability and Soft Story Control

Initial analysis showed a potential soft story behaviour in the building's longer direction, which could lead to structural vulnerability during seismic events. This issue was mitigated by the implementation of shear walls along the longer span, enhancing the building's lateral stiffness and eliminating irregularities. This intervention was critical to meeting BNBC 2020 drift and stiffness requirements, ensuring both safety and ductility of the system.

7.21.3 Code Compliance and Safety Verification

The model was subjected to a comprehensive set of structural checks, including:

- Story displacement and drift verification
- P-Delta analysis
- Torsional irregularity checks
- Stiffness irregularity checks

All results confirmed that the new composite steel structure passed the required thresholds with significant safety margins. The building's configuration, material properties, and design choices adhered to BNBC 2020 standards and Load and Resistance Factor Design (LRFD) methodology.

7.21.4 Seismic Safety Overview

Although the new building is structurally independent, a seismic evaluation of the site was performed for comprehensive safety. The seismic demand (Iso) was calculated as 0.234, and the seismic capacity of the structure was verified to be greater than this demand, confirming adequate performance under seismic excitation. The inclusion of a properly designed shear wall system further improved resilience, ensuring that the structure meets modern performance-based design expectations.

7.21.5 Summary of Structural Performance

Maximum story displacement: Within limits (Max = 0.833 in < 2.95 in).

Maximum story drift: Occurs at 3rd and 4th floors; both values within BNBC limits.
Shear walls:

- Effectively resolved soft story conditions and improved lateral stiffness.
- Structural checks: All passed—P-Delta, torsion, drift, and displacement.
- Foundation: Matt foundation designed to carry all loads independently.
- Seismic evaluation: Demand vs. capacity confirms site-wide safety.

This discussion validates that the new seven-story steel structure meets all structural and safety standards required for independent vertical expansion over an existing operational building. The findings demonstrate the effectiveness of the chosen design strategy and the applicability of composite steel systems in space-constrained institutional settings like CUET.

Chapter 8: CONCLUSIONS

8.1 GENERAL

This thesis has successfully demonstrated the structural feasibility, design methodology, and performance evaluation of a seven-story composite steel building constructed independently above an existing three-story RCC structure at Chittagong University of Engineering & Technology (CUET). The primary objective—to create an elevated extension without disturbing the operation or integrity of the existing building—was fully achieved through careful planning, advanced modelling, and code-compliant structural design.

Using ETABS 18, AutoCAD, and IDEA StatiCa, the proposed steel structure was modelled, analysed, and verified against key parameters such as story displacement, story drift, lateral stiffness, and seismic response. The results confirmed that all design outputs remained within the allowable limits set by BNBC 2020. The maximum story displacement under seismic loading was well below the permissible limit, and potential soft story irregularities were effectively resolved through the incorporation of shear walls in the longer direction of the building.

All essential structural checks—P-Delta effects, torsional irregularities, and inter-story drift—were validated, confirming the safety, serviceability, and stability of the system. Additionally, a site-wide seismic evaluation affirmed that the seismic capacity exceeds the demand (Iso = 0.234), ensuring resilience against potential earthquake forces.

The final design offers a reliable and sustainable model for vertical expansion in institutional settings where new construction space is limited and existing infrastructure must remain in continuous use. The approach not only preserves the functional and structural independence of the new extension but also enhances the long-term usability and value of the overall facility.

In conclusion, this thesis contributes a practical and well-engineered solution to the challenges of vertical expansion. It reinforces the potential of composite steel systems in modern construction and provides a reference framework for future projects of similar nature in academic or constrained environments.

8.2 LIMITATION OF THE WORK:

8.2.1 Foundation Construction Without Interference

Constructing a new mat foundation independently, without interfering with or damaging the existing building's shallow foundation, is structurally challenging. Maintaining complete separation while ensuring adequate load transfer for the new structure limits foundation layout and increases construction complexity.

8.2.2 Time-of-Day Restrictions

Due to the need for a quiet environment during morning academic sessions, equipment generating loud noise or strong vibrations (e.g., drilling rigs, compactors, heavy lifts) cannot be used at those times. This limits the working hours for critical operations, thereby slowing the overall construction timeline.

8.2.3 Logistical and Access Limitations

Working around an existing, occupied building introduces access constraints, limiting the use of cranes, scaffolding, and material storage areas. This necessitates careful planning and likely phased construction, increasing coordination demands and potential delays.

8.2.4 Cost Implications

The combination of independent foundation requirements, irregular design, constrained working hours, and complex logistics leads to a substantially higher construction cost than a typical vertical extension.

8.3 FUTURE SCOPE OF THE WORK

The current project presents a structurally sound and innovative solution for vertical expansion. However, several areas remain open for further research and development to enhance performance, cost-efficiency, and constructability. The future scope of this work includes:

8.3.1 Structural Framing System

Further study can be conducted on alternative structural framing systems to improve lateral stability and reduce material usage. Comparative analysis between different steel and composite framing systems (e.g., moment-resisting frames, braced frames, or modular frames) could help identify more efficient designs suited to similar vertical extensions in constrained urban environments.

8.3.2 Foundation

The assumed mat foundation, while effective, may not be the most economical or feasible solution in all soil conditions or urban settings. Future research may explore deep foundation options such as bored piles or micro-piles, which may offer better load-carrying capacity and reduced disturbance to the existing structure during installation.

8.3.3 Floor Systems

The composite deck slab used in the proposed design could be reassessed in future work to explore lighter or faster-installing alternatives. Systems like precast concrete planks, steel joist floors, or hollow-core slabs could potentially reduce dead load and accelerate construction, especially important when academic activities must continue uninterrupted.

8.3.4 Selection of structural materials

The project utilized standard material properties and analysis software; however, future investigations could incorporate high-performance materials such as ultra-high-performance concrete (UHPC) or advanced high-strength steel. Additionally, implementing more sophisticated analysis techniques, such as nonlinear time-history

analysis or performance-based seismic design, may provide deeper insight into the structure's behavior under extreme loading conditions.

Bibliography

[1] C. Gillott, “Potential for the Vertical Extension of Existing Buildings,” Ph.D. dissertation, University of Sheffield, 2022. Graduate School of Engineering. [Online]. Available: https://etheses.whiterose.ac.uk/id/eprint/32859/1/C.%20Gillott%20-%20Potential%20for%20the%20Vertical%20Extension%20of%20Existing%20Buildings.pdf

[2] M. Alam, M. H. Kabir, and M. A. Hossain, “Assessing Steel–Concrete Composite and RCC Structures in Bangladesh,” Innovative Infrastructure Solutions, vol. 9, no. 1, pp. 1–12, 2024. [Online]. Available: https://link.springer.com/article/10.1007/s44290-024-00121-8

[3] “ETABS vs Other Structural Analysis Software: A Comparison Guide,” Ask.com, Aug. 2024. [Online]. Available: https://www.ask.com/tv-movies/etabs-vs-structural-analysis-software-comparison-guide-downloaders

[4] P. Katsimpini, G. Papagiannopoulos, and G. Hatzigeorgiou, “An In-Depth Analysis of the Seismic Performance Characteristics of Steel–Concrete Composite Structures,” Applied Sciences, vol. 15, no. 7, p. 3715, 2025. [Online]. Available: https://doi.org/10.3390/app15073715

[5] Public Works Department (PWD), “Seismic Evaluation Manual: For Existing Reinforced Concrete Buildings in Bangladesh (2nd Edition),” The Project for Building Safety for Disaster Risk Reduction (BSPP), Japan International Cooperation Agency (JICA), Ministry of Housing and Public Works, Bangladesh, 2020.

[6] Housing and Building Research Institute (HBRI), “Bangladesh National Building Code (BNBC) 2020,” Ministry of Housing and Public Works, Government of the People’s Republic of Bangladesh, 2021.

Appendices

FINITE ELEMENT MODELLING (FEM)

Modelling of the whole structure which is a commercial steel building is done using ETABS software (Version 18.0.1)

1.1 INITIALIZATION OF COUNTRY CODES AND STANDARD CODES

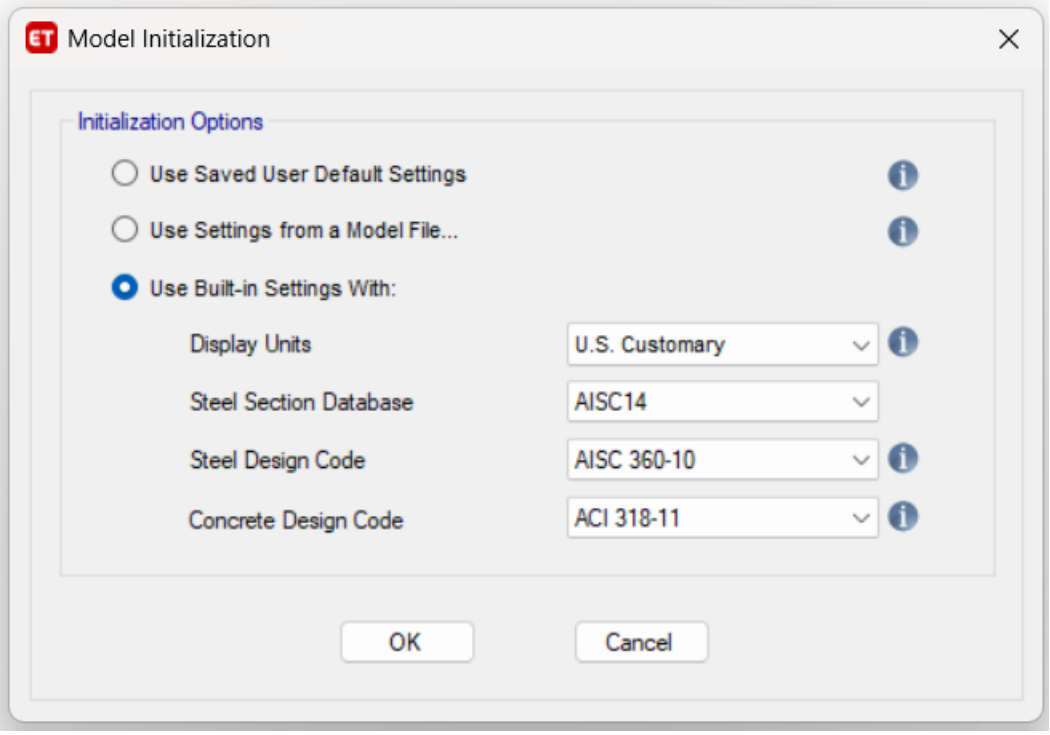


Figure 0.1 Required Code setup

1.2 GRID POINTS CREATION & STRUCTURAL GENERATION

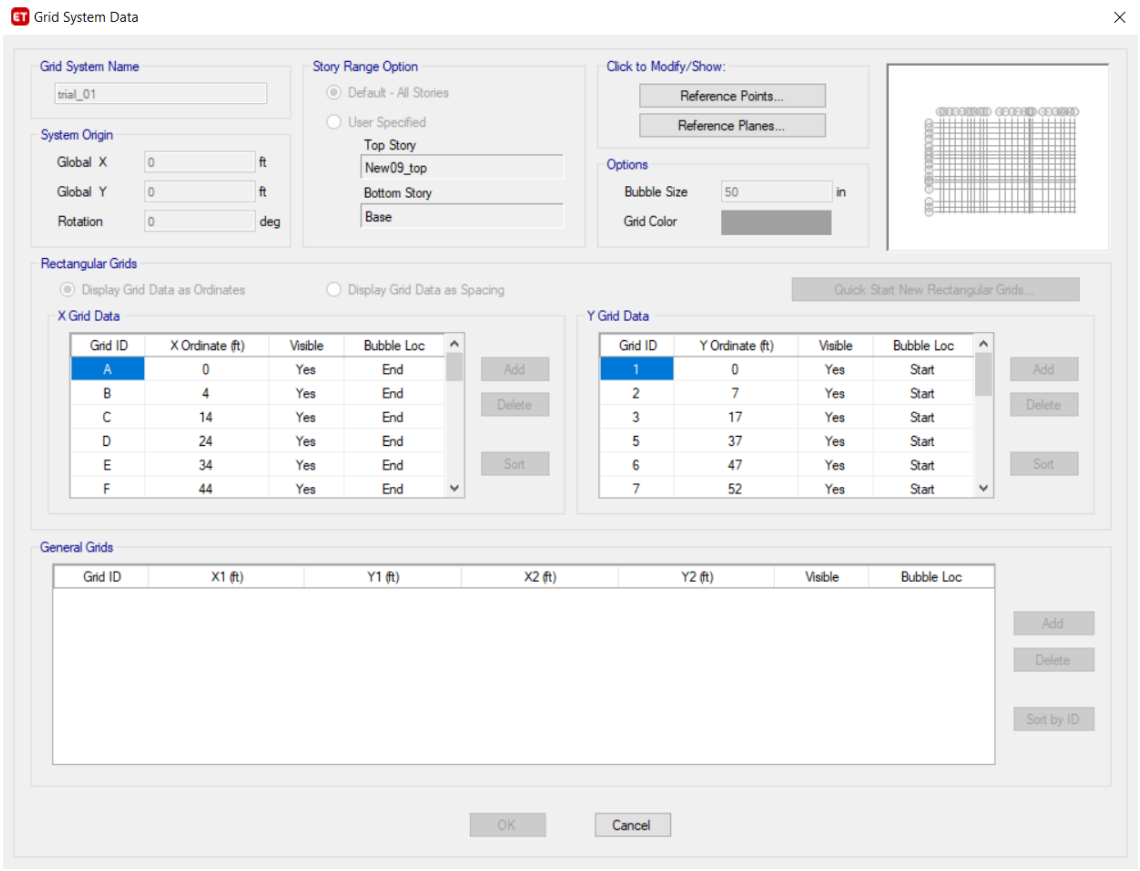


Figure 0.2 Grid System Chart

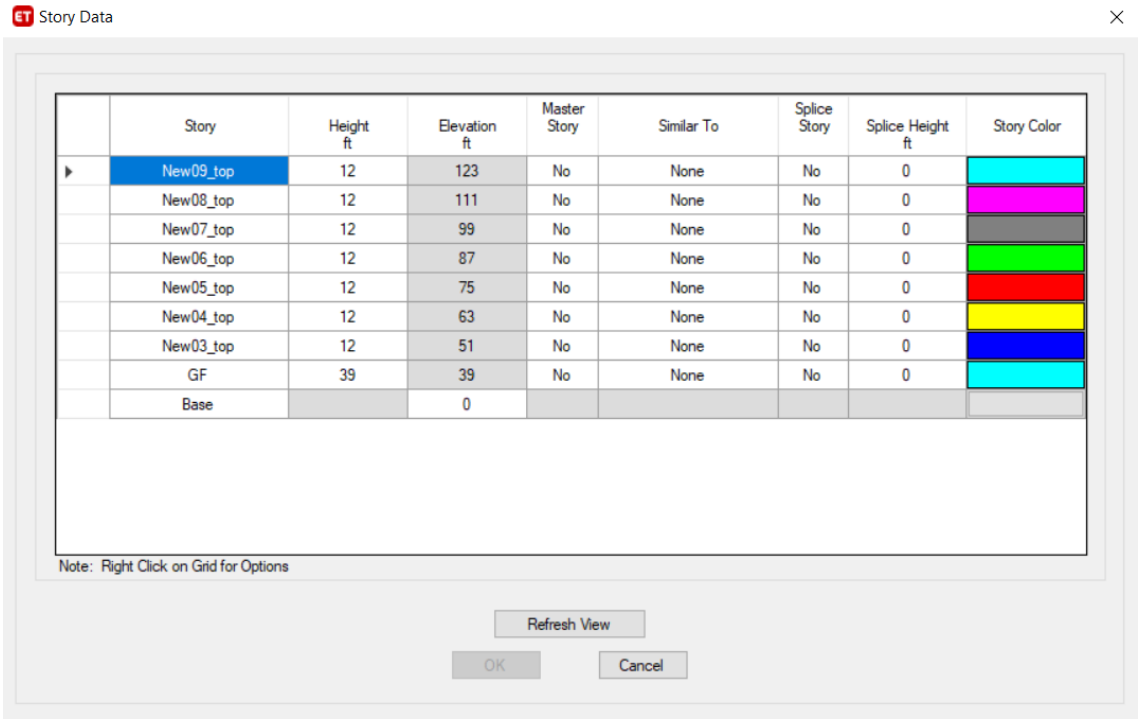


Figure 0.3 Elevation data of structure

1.3 DEFINING THE PROPERTIES

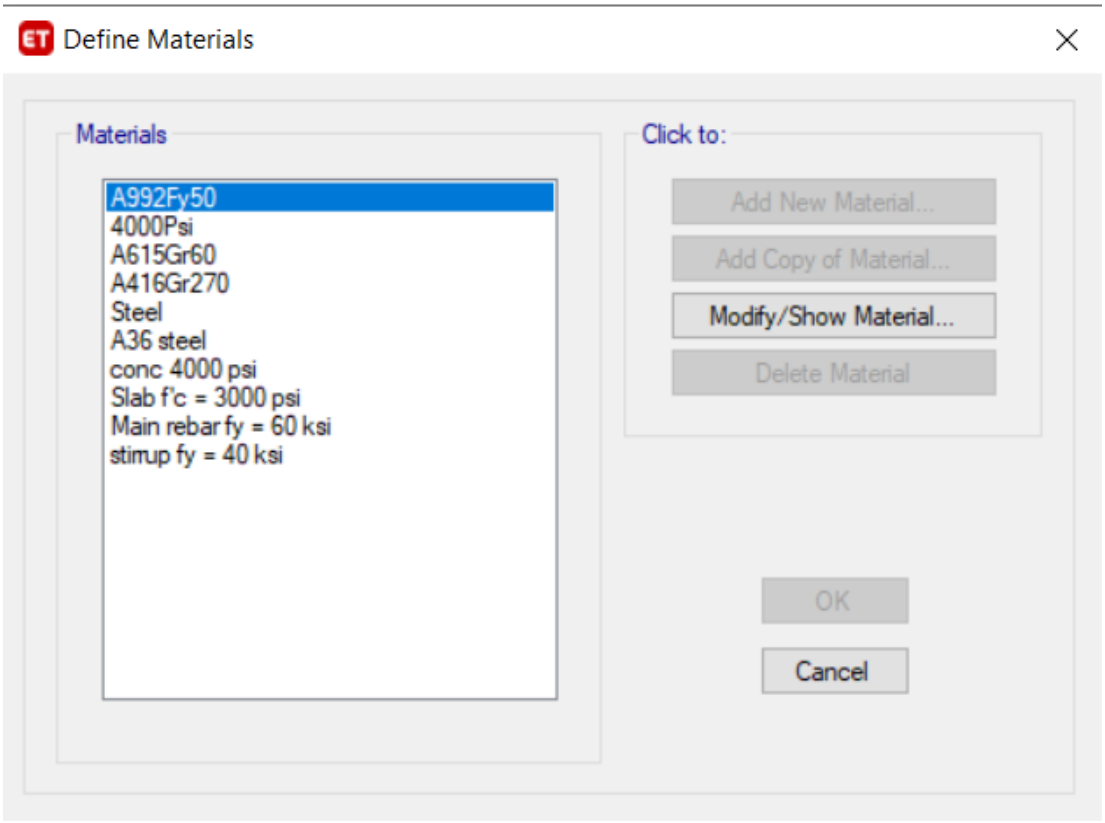


Figure 0.4 Defining material properties

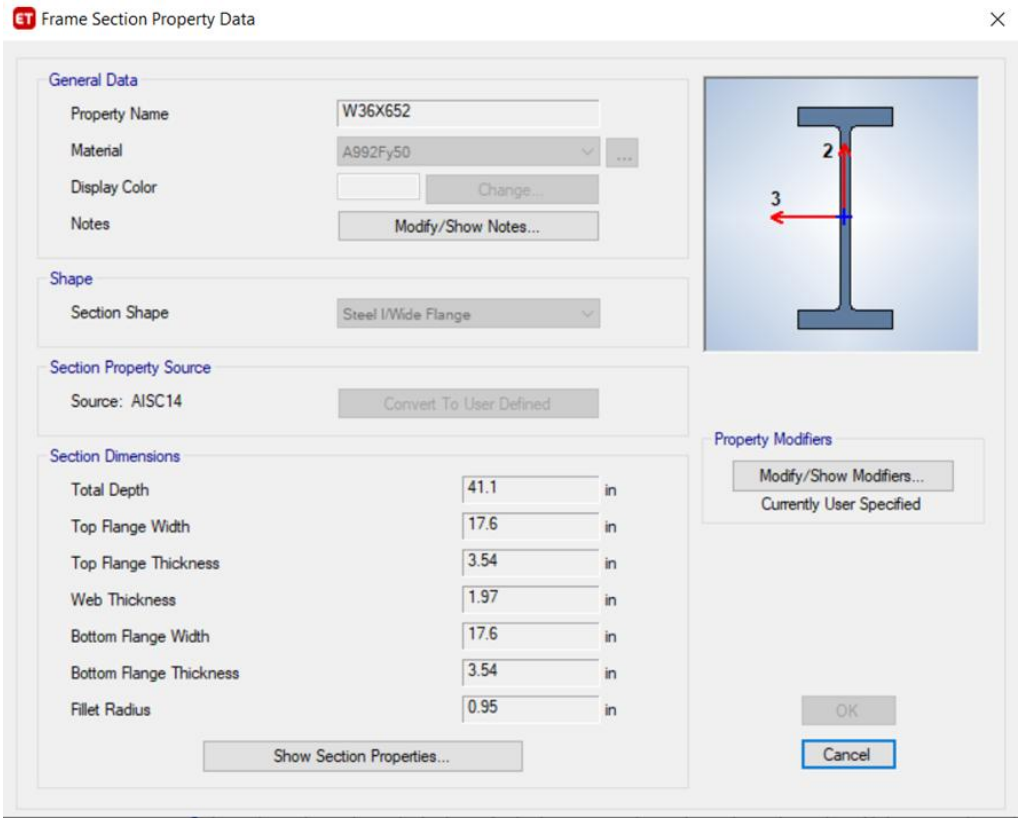


Figure 0.5 Selecting of Beam Section

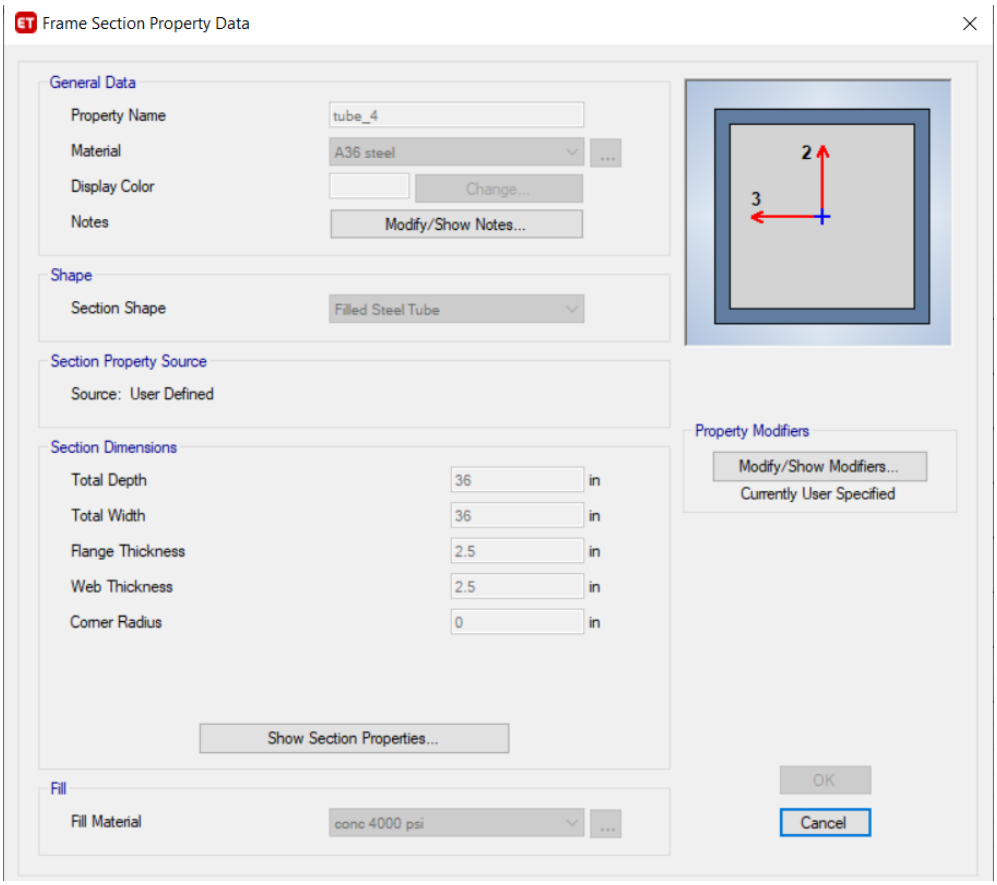


Figure 0.6 Selection of Column Section

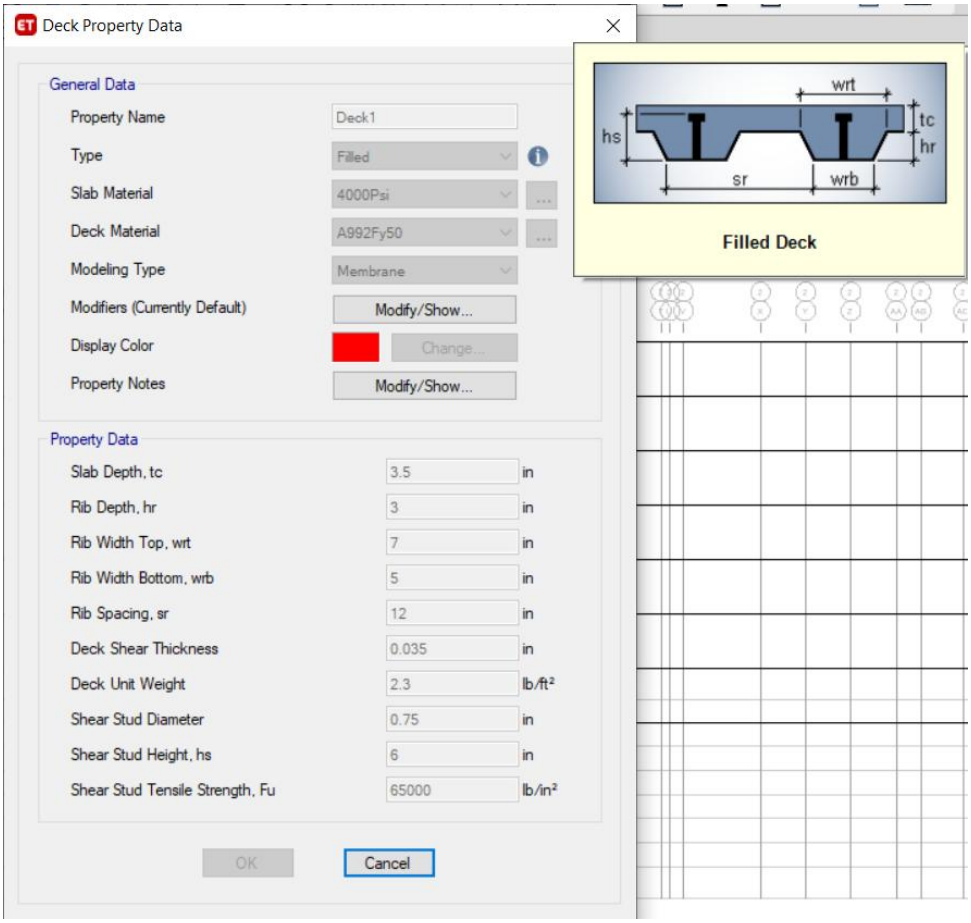


Figure 0.7 Selection of Deck Sectio

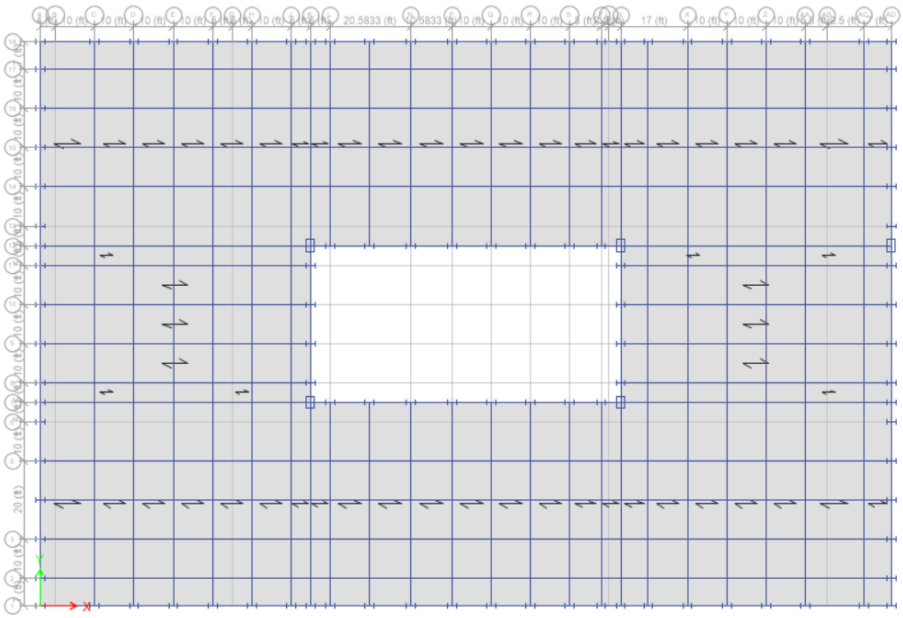


Figure 0.8 Assign Deck Section

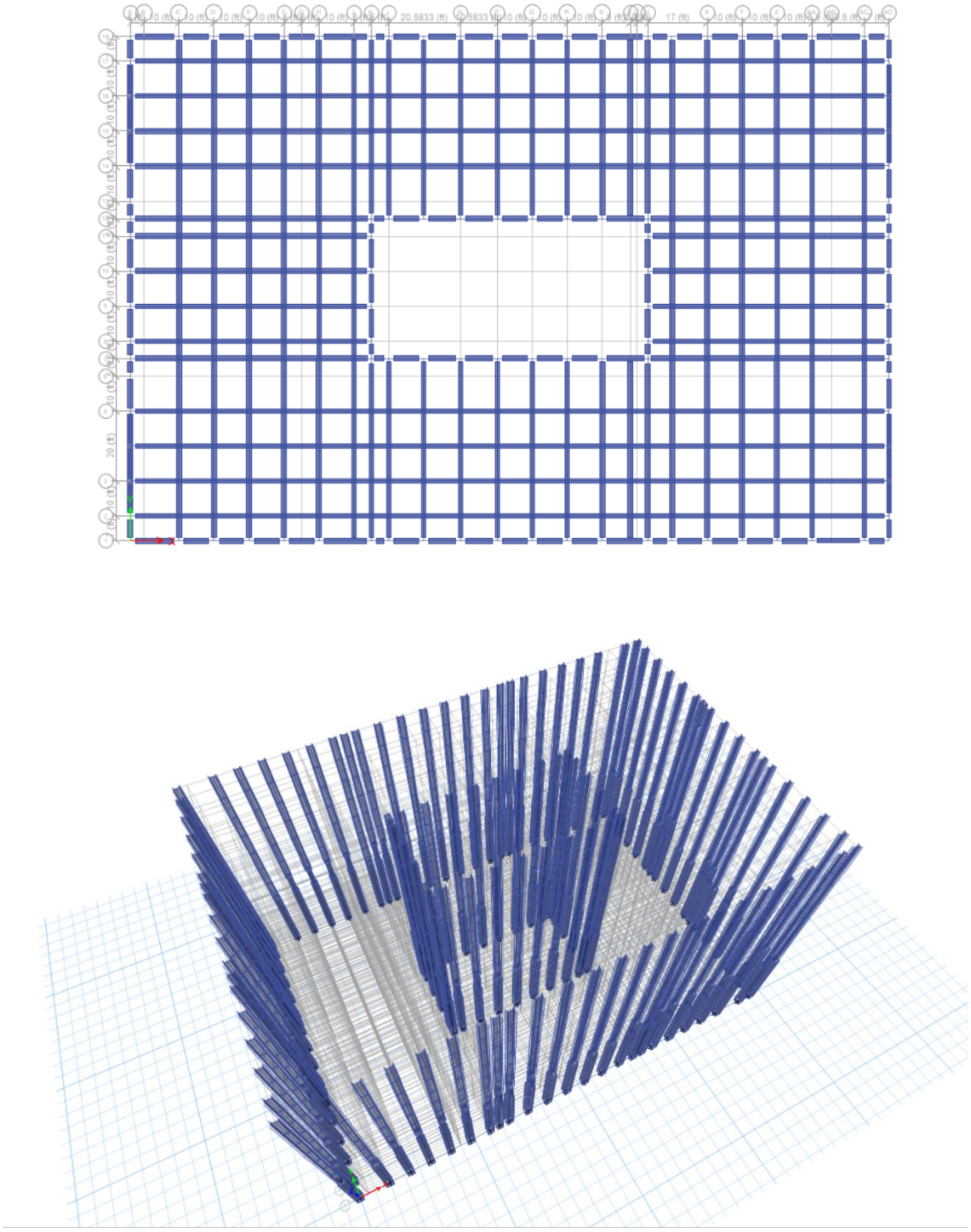


Figure 0.9 Assign Beam and Column Section

1.4 SUPPORTS ASSIGNING AND LOAD DEFINING

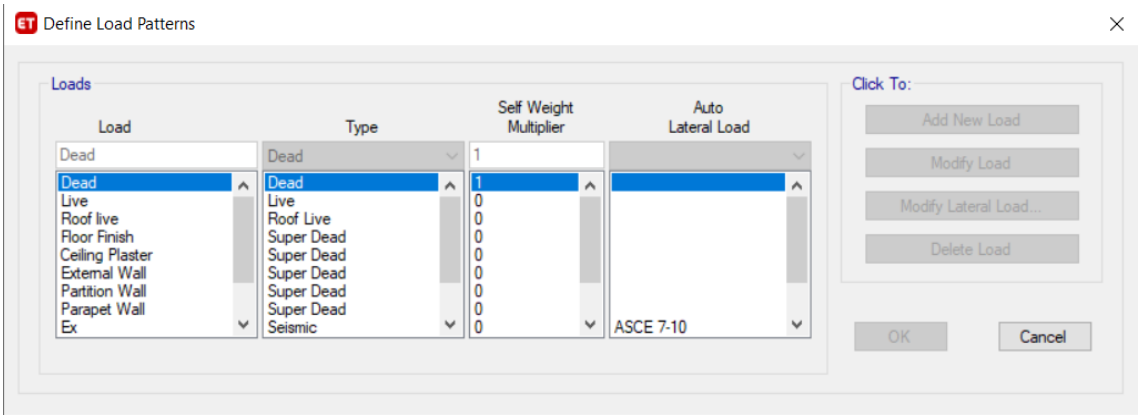


Figure 0.10 Defining Load Pattern

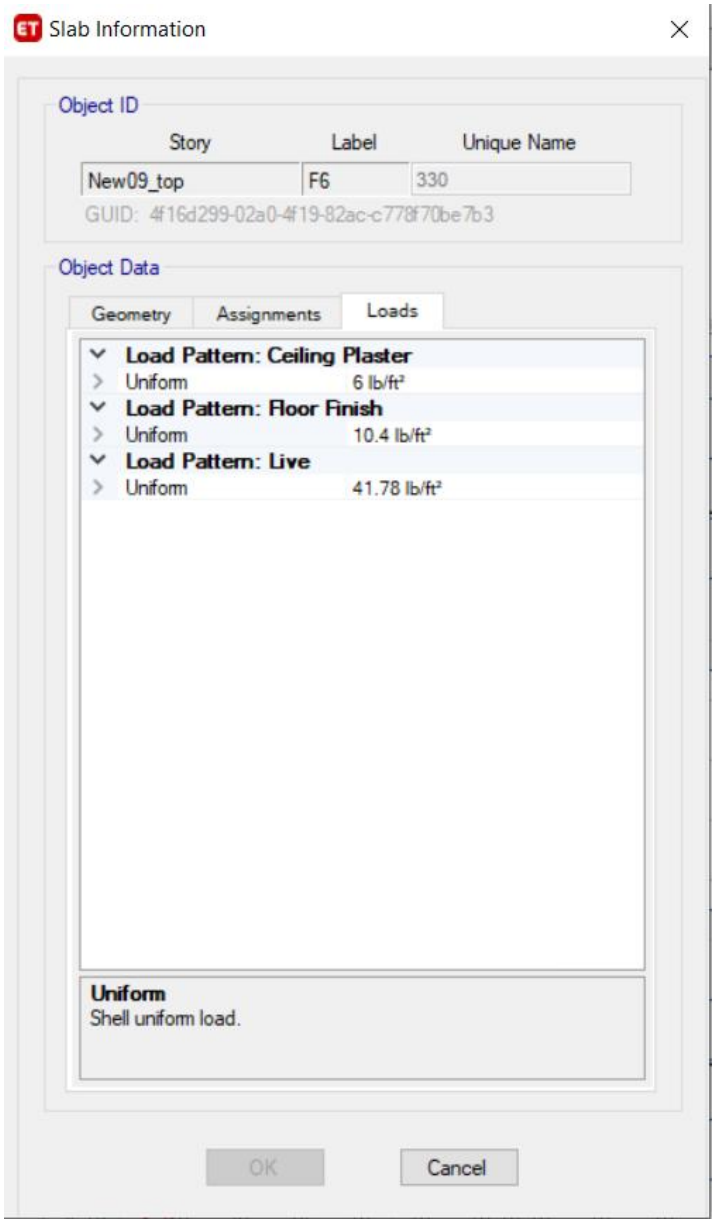


Figure 0.11 Assigning Loads

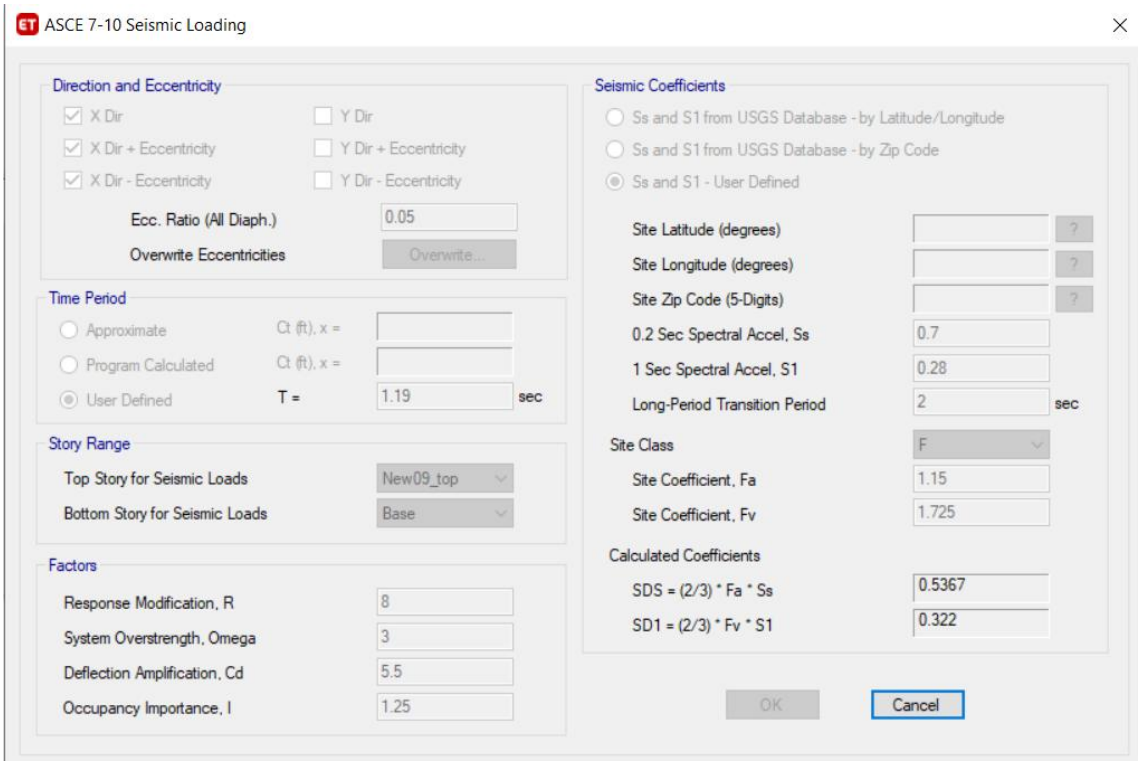


Figure 0.12 Defining Seismic Loads

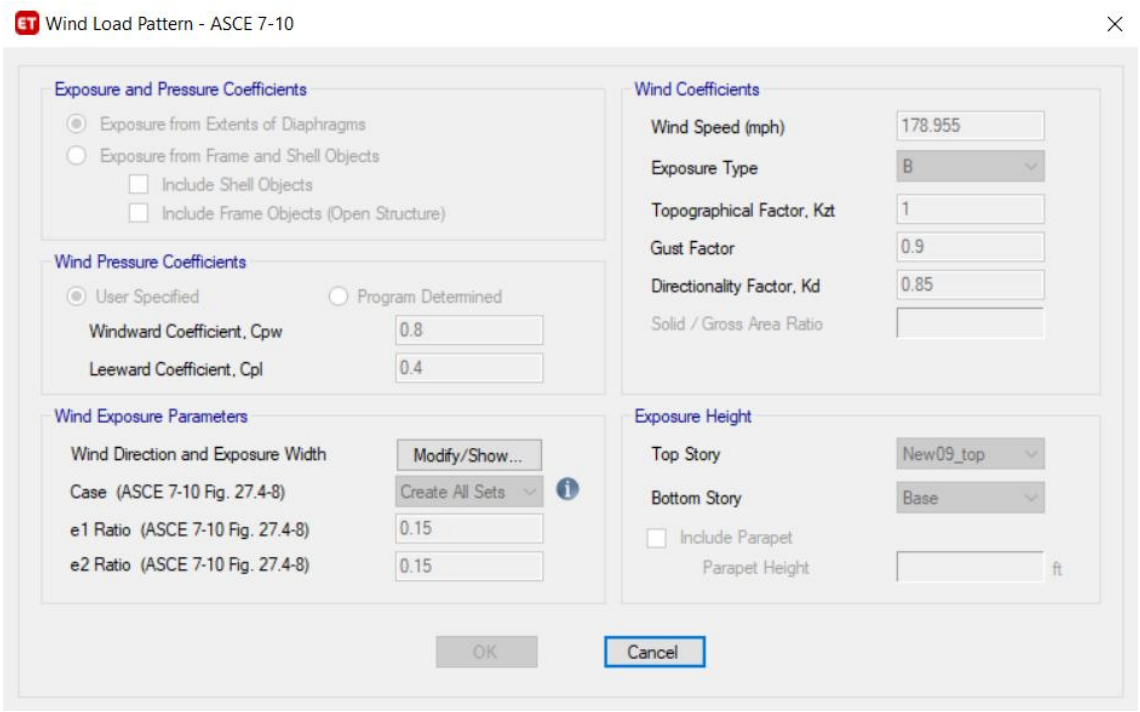


Figure 0.13 Defining Wind Loads

Table 0.1 Load Combinations

SL No	Load Combination	Combination Number
1	1)1.4D	1
2	2)1.2D+1.6L+0.5Lr	2
3	3)1.2D+1.6Lr+L	3
4	4)1.2D+1.6Lr+0.8Wx	3
5	5)1.2D+1.6Lr-0.8Wx	3
6	6)1.2D+1.6Lr+0.8Wy	3
7	7)1.2D+1.6Lr-0.8Wy	3
8	8)1.2D+1.6Wx+L+0.5Lr	4
9	9)1.2D-1.6Wx+L+0.5Lr	4
10	10)1.2D-1.6Wy+L+0.5Lr	4
11	11)1.2D+1.6Wy+L+0.5Lr	4
12	12)1.3073D+Ex+0.3Ey+L	5
13	13)1.3073D+Ex-0.3Ey+L	5
14	14)1.3073D-Ex+0.3Ey+L	5
15	15)1.3073D-Ex-0.3Ey+L	5
16	1.3073D+0.3Ex+Ey+L	5
17	1.3073D+0.3Ex-Ey+L	5
18	1.3073D-0.3Ex+Ey+L	5
19	1.3073D-0.3Ex-Ey+L	5
20	0.9D+1.6Wx	6
21	0.9D-1.6Wx	6
22	0.9D+1.6Wy	6
23	0.9D-1.6Wy	6
24	0.793D+Ex+0.3Ey	7
25	0.793D+Ex-0.3Ey	7
26	0.793D-Ex+0.3Ey	7
27	0.793D-0.3Ex-Ey	7
28	0.793D+0.3Ex+Ey	7
29	0.793D+0.3Ex-Ey	7
30	0.793D-0.3Ex+Ey	7
31	0.793D-Ex-0.3Ey	7