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DEPARTMENT OF CIVIL ENGINEERING

Response of High-rise Structures under Static and Dynamic Loadings

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Response of High-rise Structures under Static and Dynamic Loadings

A Project and Thesis

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DEPARTMENT OF CIVIL ENGINEERING

The project and thesis titled ***"Response of High-rise Structures under Static and Dynamic Loadings"***, submitted by **Shihabuddin Sardar,** ID No: CEN # 059 09052, Batch: 59, student of the Department of Civil Engineering at Stamford University Bangladesh has been examined thoroughly and accepted in partial fulfillment of the requirements for the degree of Bachelor of Science in Civil Engineering on September 12, 2020.

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DECLARATION

I am **Shihabuddin Sardar**, the undergraduate student of Civil Engineering program at Stamford University Bangladesh, hereby declare that the works presented in this **Project and Thesis** has been carried out by me and has not previously been out submitted to any other University/College/Organization for any academic qualification or certificate degree.

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ABSTRACT

This study was carried out in the Department of Civil Engineering of Stamford University Bangladesh with the objectives of the response of high-rise structures under static and dynamic loadings and the application of static and dynamic analysis of the structures.

High-rise buildings are exposed to both static and dynamic loads. Depending on the method used and how the structure is modelled in finite element software, the results can vary. Nowadays the world is going forward by the implementation of performance-based engineering analysis. Most of the multi-storied buildings in our country are still analyzed and designed without proper seismic consideration in the conventional way. Under strong earthquake, structures behaved unsatisfactory during major ground motions with large inelastic deformations, and dynamic analysis should be performed for the structure. Dynamic effects such as resonance frequencies and accelerations are considered. The variation in static results from reaction forces, overturning moments, deflections, critical buckling loads, forces between prefabricated elements and force distributions between concrete cores are investigated with different models. The structural dynamics is the direct application in design of high-rise building and structural analysis against earthquake and wind loading. Structural design for dynamic loading is primarily concerned with forces and their effects on motion.

All design against seismic loads must consider the dynamic nature of the load. However, for simple regular structures, analysis by equivalent linear static methods is often sufficient. This is permitted in most codes of practice for regular, low to medium-rise buildings. Equivalent static analysis can therefore work well for low to medium-rise buildings without significant coupled lateral-torsional modes, in which only the first mode in each direction is considered. Tall buildings (say, over 75 m or 25 storied), where second and higher modes can be important, or buildings with torsional effects, are much less suitable for the method, and require dynamic analysis of the buildings.

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Chapter 1

INTRODUCTION

CHAPTER ONE
INTRODUCTION

1.1 BACKGROUND OF THE STUDY

Reinforced Concrete (RC) Structure buildings are the most common type of constructions in urban-suburban areas of Bangladesh, which are subjected to several types of forces during their lifetime, such as static forces due to dead and live loads and dynamic forces due to wind and earthquakes. Unlike static forces, amplitude, direction and location of dynamic forces, especially due to earthquakes, vary significantly with time, causing considerable inertia effects on buildings. Behavior of buildings under dynamic forces depends upon the dynamic characteristics of buildings, which are controlled by both their mass and stiffness properties, whereas the static behavior is solely dependent upon the stiffness characteristics. To see the static and dynamic behavior of the structure especially high-rise structures, both the static and dynamic analysis should be performed for the study. The process of analyzing and designing of high-rise structures have changed over the past years. In the most recent years, it is usual to model full three-dimensional finite element models of the buildings. Architects challenging Engineers by proposing irregular shaped Buildings. Design and Analysis of irregular shaped building is more complex than the regular shaped building. This due to the increased computational power and more advanced software. However, these models produce huge amount of data and results where possible errors are easily overlooked, especially if the model is big and complex. If the engineer is not careful and have a lack of knowledge of structural behavior and finite element modelling, it is easy to just accept the results without critical thoughts. Furthermore, different ways of modelling have a big influence on the force and stress distribution. This can lead to time-consuming discussion and disagreements between engineers as they often have different results from calculations on the same building. In the present work, an analytical study is performed to evaluate the effect of plan irregularity on the seismic behavior of the conventional Reinforced Concrete Structures.

Historical seismic catalogues reveal that Bangladesh has been affected by earthquake since ancient times. Earthquakes occurred in 1664, 1828, 1852 and 1885 are shown to have Dhaka as epicentral area. Similarly, cities like Rangpur, Sylhet, Mymensing, Chittagong, Saidpur, Sirajgong, Pabna etc. have been shown to be the epicentral area of some of the major earthquakes in the past. Although the ancient record does not specify the earthquake epicenter by giving coordinates in terms of latitude and longitude. It is difficult to figure out whether these cities were directly hit by earthquakes. However, occurrence of earthquakes both inside and outside of the country and around major cities indicates that earthquake hazard exists for the country in general and the cities in particular. Consideration of earthquake forces in structural design, city planning and infrastructure development is therefore a prerequisite for future disaster mitigation. The seismicity data of Bangladesh and adjoining areas indicate that Bangladesh is vulnerable to earthquake hazards. As Bangladesh is the worlds' most densely populated area, any future earthquake shall affect more people per unit area than any other seismically active regions of the world. Both of the above factors call for evaluation of seismic hazard of Bangladesh so that proper hazard mitigation measure may be undertaken before it is too late.

1.2 OBJECTIVES OF THE STUDY

- To know ins and outs of static and dynamic analysis of high-rise structures.
- To know when and why, which method of analysis is required?
- To know and get the response of high-rise structures under static and dynamic loadings.
- To analyze different methods when performing calculations on high-rise buildings in regards to deflections, resonance frequencies, accelerations and stability.
- To understand and make a comparison of the behavior of high-rise structures under static and dynamic excitation.

1.3 SCOPES AND LIMITATIONS OF THE STUDY

- The study had been made based on high-rise (*Tall Building*) structural design concept with both Static and Dynamic Analysis.
- Considered Edge Supported floor system.
- Both Static and Dynamic Analysis of the structure was performed.
- ETABS 2018 and SAFE 2016 was used for analysis and design.

Chapter 2

LITERATURE REVIEW

CHAPTER TWO
LITERATURE REVIEW

2.1 HIGH-RISE STRUCTURES

A building is defined as high-rise when it is considerably higher than the surrounding buildings or its proportion is slender enough to give the appearance of a tall building. Now a day all over the world should be more concise and shorter to the performance-based engineering. In terms of high-rise or tall building cannot be defined in specific height or the number of floors. Tall building is a matter of person or community's circumstance and their consequent perception; therefore, a measurable definition of a tall building cannot be universal. From a structural engineer's point of view, tall building may be defined as that one, of its height, is affected by the lateral forces is subjected to dynamic loading which include people, wind, earthquake, waves, traffic, and blast. These actions play an important role in the structural design and must be considered from very beginning of the design process.

High-rise buildings are today iconic structures that have a purpose beyond housing people and offices. They often form the skyline and thus function as an image of the city itself. They are symbols of power and economic prosperity, as well as innovation. New advances in structural engineering has made it possible to adapt the architectural design to the local culture and expression. The construction of high-rise buildings started at the end of the 19th century in Chicago, with the evolution shown in Figure 2.1. This was made possible because of new inventions such as the safe elevator in 1853 and the telephone in 1876 that enabled transport of building materials and the ability to communicate to higher levels. In addition, the building materials changed as they went from wood and masonry to using steel frames with lighter masonry walls. Earlier buildings that were built with heavy masonry walls was limited to certain heights by its own self-weight. With steel frames, the masonry could be thinner and act only as facade for weather protection and taller buildings could be constructed.

High-rise structures pose particular design challenges for structural and geotechnical engineers, particularly if situated in a seismically active region or if the underlying soils have geotechnical risk factors such as high compressibility or bay mud. They also pose serious challenges to firefighters during emergencies in high-rise structures. New and old

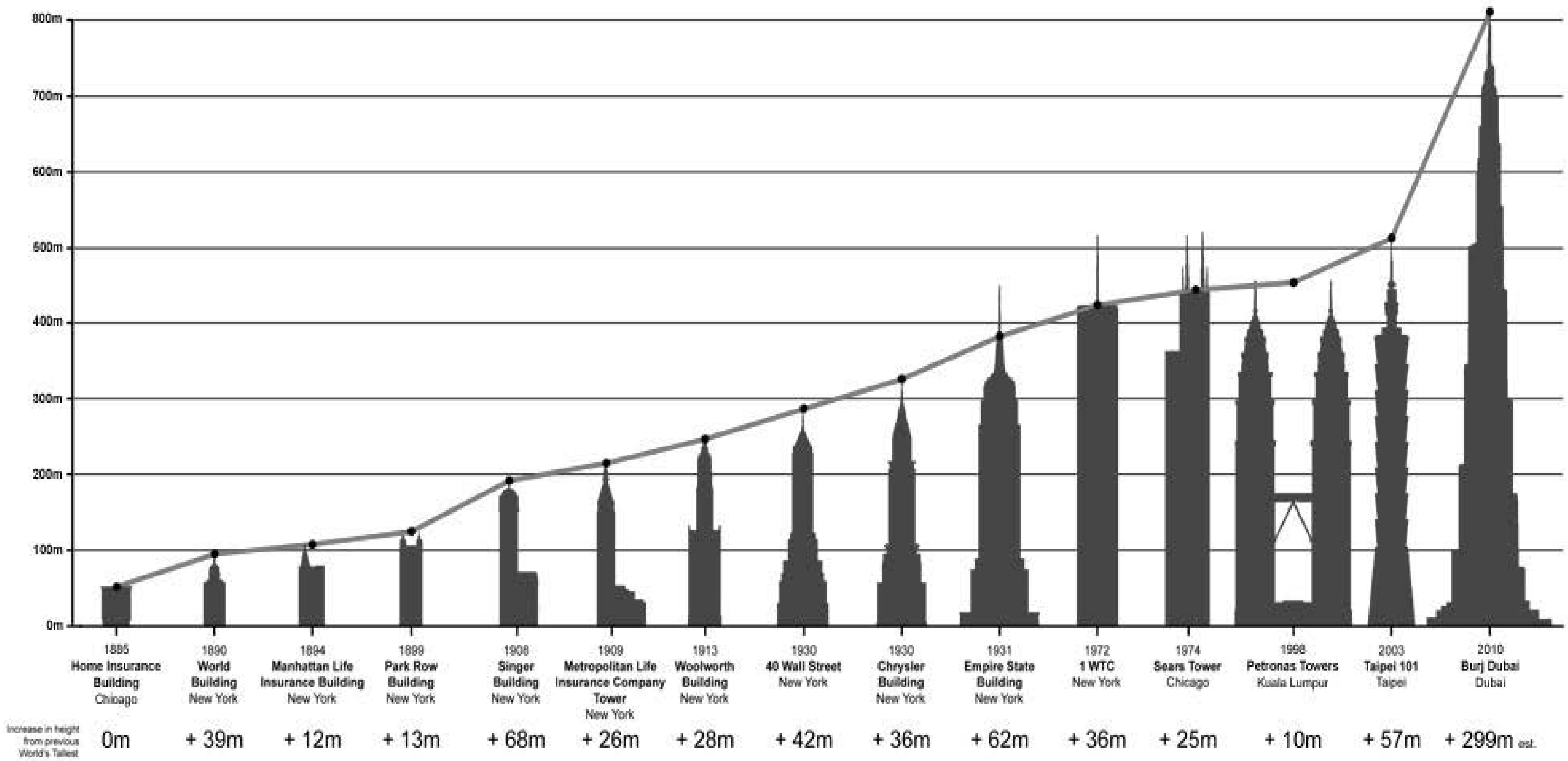


Figure 2.1: Diagram of buildings that have once claimed the title 'World's highest building

building design, building systems like the building standpipe system, HVAC systems (heating, ventilation and air conditioning), fire sprinkler system and other things like stairwell and elevator evacuations pose significant problems. Studies are often required to ensure that pedestrian wind comfort and wind danger concerns are addressed. In order to allow less wind exposure, to transmit more daylight to the ground and to appear slenderer, many high-rises have a design with setbacks.

2.1.1 High-rise Building Criteria

- As per *Bangladesh National Building Code (BNBC - 2017)*, any building which having mean roof height is greater than 60 feet (18.3 m).
- As per *Rajdhani Unnayan Kartripakkha (RAJUK - 2008)*, Dhaka, Bangladesh, any building which is more than 10 stores or 33 m in height.
- In *the U.S. The National Fire Protection Association* defines a high-rise as being higher than 75 feet (23 m) or 7 stories.
- According to *ASCE 7-16*, structure or building having mean roof height is greater than 60 feet (18 m).

2.2 OCCUPANCY CATEGORY

Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 2.1 for the purposes of applying flood, surge, wind and earthquake provisions. The occupancy categories range from I to IV, where Occupancy Category I represents buildings and other structures with a low hazard to human life in the event of failure and Occupancy Category IV represents essential facilities. Each building or other structure shall be assigned to the highest applicable occupancy category or categories. Assignment of the same structure to multiple occupancy categories based on use and the type of load condition being evaluated (e.g., wind or seismic) shall be permissible.

When buildings or other structures have multiple uses (occupancies), the relationship between the uses of various parts of the building or other structure and the independence of the structural systems for those various parts shall be examined. The classification for each independent structural system of a multiple-use building or other structure shall be that of the highest usage group in any part of the building or other structure that is dependent on that basic structural system.

Table 2.1: Occupancy category of Buildings and other Structures for Flood, Surge, Wind, and Earthquake Loads

Occupancy Category	Nature of Occupancy
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities.
II	All buildings and other structures except those listed in Occupancy Categories III, IV and I.
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area. • Buildings and other structures with day care facilities with a capacity greater than 150. • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250. • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities. • Healthcare facilities with a capacity of 50 or more resident patients, but not having surgery or emergency Treatment facilities. • Jails and detention facilities. Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous

	chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other healthcare facilities having surgery or emergency treatment facilities. • Fire, rescue, ambulance, and police stations and emergency vehicle garages. • Designated earthquake, hurricane, or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power generating stations and other public utility facilities required in an emergency. • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers. • Electrical substation structures, firewater storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency. • Aviation control towers, air traffic control centers, and emergency aircraft hangars. • Water storage facilities and pump structures required to maintain water pressure for fire suppression. • Buildings and other structures having critical national defense functions. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.
****Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.*	

2.3 STRUCTURAL CONFIGURATIONS

Based on the structural configuration, each structure shall be designated as a regular or irregular structure as defined below:

❖ **Regular Structures**

Regular structures have no significant physical discontinuities or irregularities in plan or vertical configuration or in their lateral force resisting systems.

❖ **Irregular Structures**

Buildings with irregularity in plan or elevation suffer much more damage in earthquakes than buildings with regular configuration. A building may be considered as irregular, if at least one of the conditions given below are applicable:

1. ***Plan Irregularity:*** Structures having one or more of the irregular features listed in Table 2.2 shall be designated as having a plan irregularity.
2. ***Vertical Irregularity:*** Structures having one or more of the irregular features listed in Table 2.3 shall be designated as having a vertical irregularity.

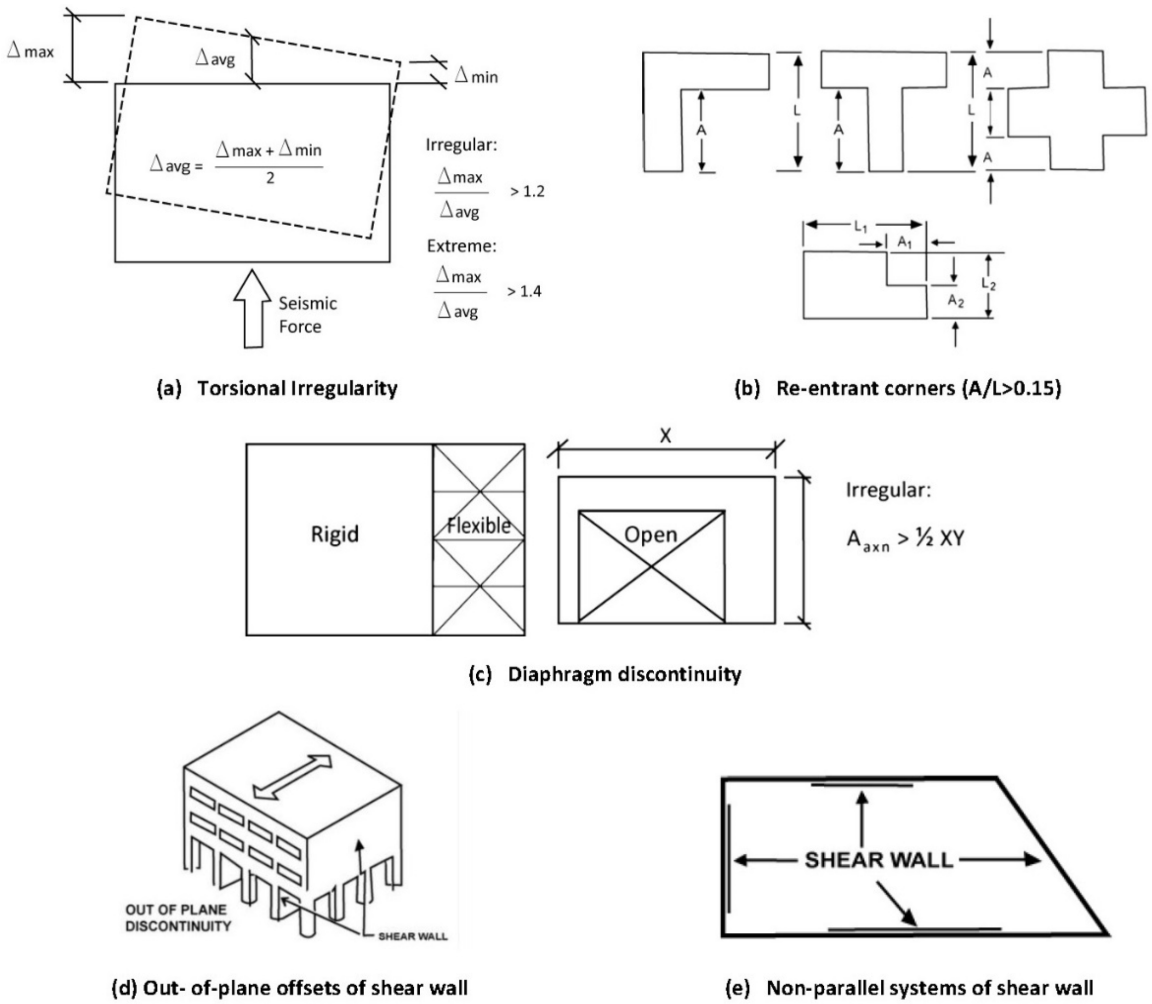


Figure 2.2(i): Different Types of Plan Irregularities of Building

Table 2.2: Plan (Horizontal) Irregularities of Structures

Plan Irregularity Type	Definition
I	**Torsional Irregularity (to be considered for diaphragms):** a. Torsional irregularity shall be considered to exist when the maximum storey drift (Δ_{max}), computed including accidental torsion, at one end of the structure is more than 1.2 times the average $\left(\Delta_{avg} = \frac{\Delta_{max} + \Delta_{min}}{2}\right)$ of the storey drifts at the two ends of the structure. b. Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts $\left(\Delta_{max} > 1.4\Delta_{avg}\right)$ at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semi rigid.
II	**Reentrant Corners:** Plan configurations of a structure and its lateral force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension in the given direction.
III	**Diaphragm Discontinuity:** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one storey to the next.
IV	**Out-of-plane Offsets:** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.
V	**Nonparallel Systems:** The vertical lateral load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.

Table 2.3: Vertical Irregularities of Structures

Vertical Irregularity Type	Definition
I	**a. Stiffness Irregularity (Soft Storey):** Soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average stiffness of the three stores above. **b. Stiffness Irregularity (Extreme Soft Storey):** Extreme soft storey irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.
II	**Mass Irregularity:** Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey. A roof, which is lighter than the floor below, need not be considered.
III	**Vertical Geometric Irregularity:** Vertical geometric irregularity shall be considered to exist where horizontal dimension of the lateral force-resisting system in any storey is more than 130% of that in an adjacent storey, one-storey penthouses need not be considered.
IV	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element:** An in-plane offset of the lateral load-resisting elements greater than the length of those elements.
V	**a. Discontinuity in Capacity (Weak Storey):** A weak storey is one in which the storey strength is less than 80% of that in the storey above. The storey strength is the total strength of all seismic-resisting elements sharing the storey shear for the direction under consideration. **b. Extreme Discontinuity in Capacity (Very Weak Storey):** A very weak storey is one in which the storey strength is less than 65% of that in the storey above.

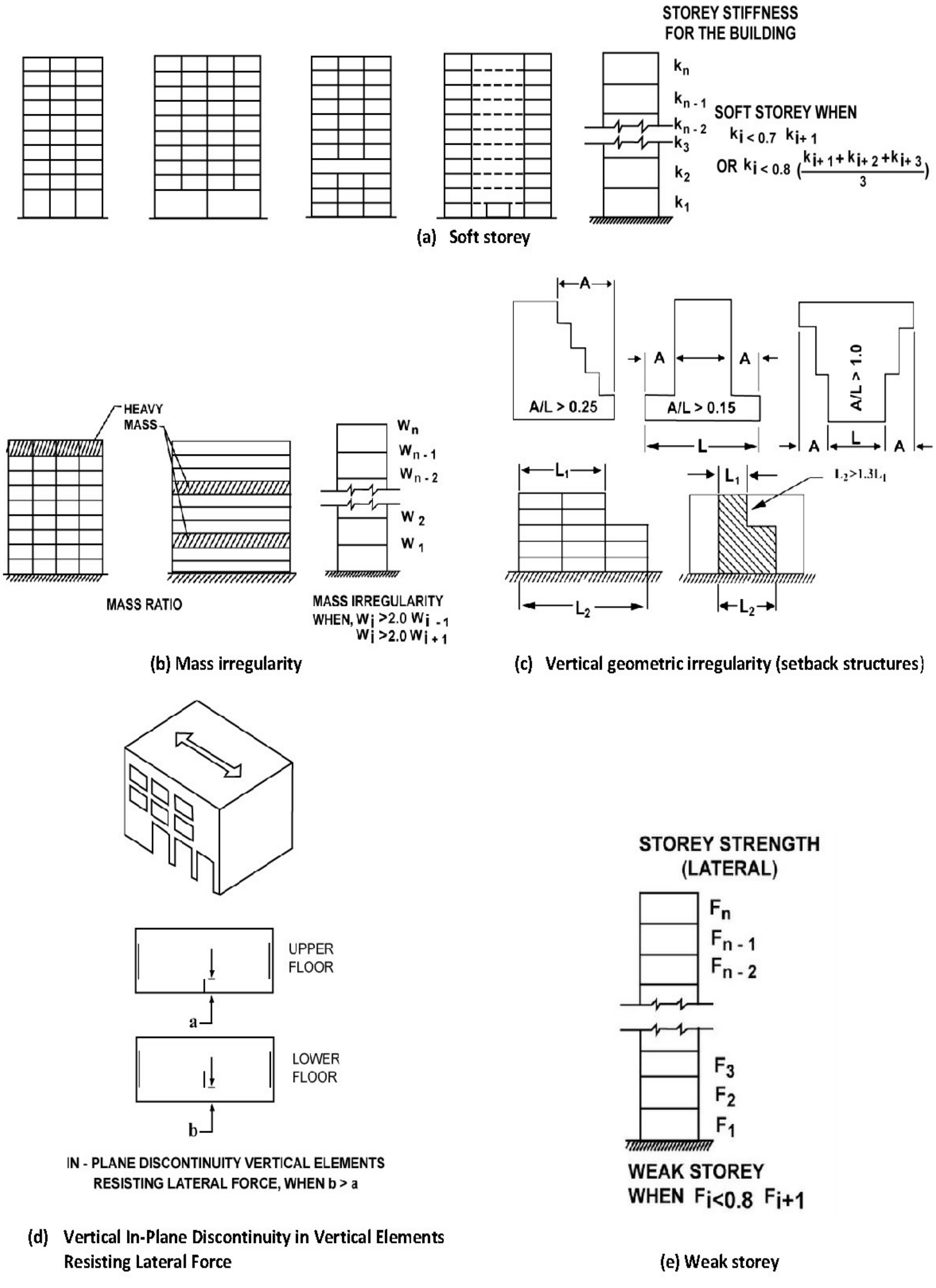


Figure 2.2(ii): Different Types of Vertical Irregularities of Building

2.4 STRUCTURAL SYSTEM

Every structure shall have one of the basic structural systems or a combination thereof. Structural systems for buildings and other structures shall be designated as one of the types A to G listed in Table 2.4. Each type is again classified as shown in the Table 2.4 by the types of vertical elements used to resist lateral forces. A brief description of different structural systems is presented in following sub-sections.

1. **Bearing Wall System**

   A structural system having bearing walls/bracing systems without a complete vertical load-carrying frame to support gravity loads. To resist the lateral loads is provided by shear walls or braced frames.

2. **Building Frame System**

   A structural system with an essentially complete space frame providing support for gravity loads. To resist the lateral loads is provided by shear walls or braced frames separately.

3. **Moment Resisting Frame (MRF) System**

   A structural system with an essentially complete space frame providing support for gravity loads. Moment resisting frames are also provide to resist lateral load primarily by flexural action of members, and may be classified as follows:

   - Special Moment Resisting Frames (SMRF)
   - Intermediate Moment Resisting Frames (IMRF)
   - Ordinary Moment Resisting Frames (OMRF)

4. **Dual System**

   A structural system having a combination of the following framing systems:

   - Moment resisting frames (SMRF, IMRF or steel OMRF), and
   - Shear walls or braced frames

   The two systems specified in above shall be designed to resist the total lateral force in proportion to their relative rigidities considering the interaction of the dual system at all levels. However, the moment resisting frames shall be capable of resisting at least 25% of the applicable total seismic lateral force, even when wind or any other lateral force governs the design.

5. **Special Structural System**

   A structural system not defined nor listed in Table and specially designed to carry the lateral loads, such as tube-in-tube, bundled tube, etc.

Table 2.4: Response Reduction Factor, Deflection Amplification Factor, and Height Limitations for Different Structural Systems

Seismic Force Resisting System	Response Reduction Factor, R	System Over Strength Factor, Ω_o	Deflection Amplification Factor, C_d	Height limit for Seismic Design Category (m)		
				B	C	D
A. BEARING WALL SYSTEMS (no frame)						
▪ Special reinforced concrete shear walls	5.0	2.5	5.0	NL	NL	50
▪ Ordinary reinforced concrete shear walls	4.0	2.5	4.0	NL	NL	NP
▪ Ordinary reinforced masonry shear walls	2.0	2.5	1.75	NL	50	NP
▪ Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)						
▪ Steel eccentrically braced frames, moment resisting connections at columns away from links	8.0	2.0	4.0	NL	NL	50
▪ Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	7.0	2.0	4.0	NL	NL	50
▪ Special steel concentrically braced frames	6.0	2.0	5.0	NL	NL	50
▪ Ordinary steel concentrically braced frames	3.25	2.0	3.25	NL	NL	11
▪ Special reinforced concrete shear walls	6.0	2.5	5.0	NL	NL	50
▪ Ordinary reinforced concrete shear walls	5.0	2.5	4.25	NL	NL	NP
▪ Ordinary reinforced masonry shear walls	2.0	2.5	2.0	NL	50	NP
▪ Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)						
▪ Special steel moment frames (SMRF)	8.0	3.0	5.5	NL	NL	NL
▪ Intermediate steel moment frames (IMRF)	4.5	3.0	4.0	NL	NL	35
▪ Ordinary steel moment frames (OMRF)	3.5	3.0	3.0	NL	NL	NP

▪ Special reinforced concrete moment frames (SMRF)	8.0	3.0	5.5	NL	NL	NL
▪ Intermediate reinforced concrete moment frames (IMRF)	5.0	3.0	4.5	NL	NL	NP
▪ Ordinary reinforced concrete moment frames (OMRF)	3.0	3.0	2.5	NL	NP	NP
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
▪ Steel eccentrically braced frames	8.0	2.5	4.0	NL	NL	NL
▪ Special steel concentrically braced frames	7.0	2.5	5.5	NL	NL	NL
▪ Special reinforced concrete shear walls	7.0	2.5	5.5	NL	NL	NL
▪ Ordinary reinforced concrete shear walls	6.0	2.5	5.0	NL	NL	NP
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
▪ Steel eccentrically braced frames	6.0	2.5	5.0	NL	NL	11
▪ Special reinforced concrete shear walls	6.5	2.5	5.0	NL	NL	50
▪ Ordinary reinforced masonry shear walls	3.0	3.0	3.0	NL	50	NP
▪ Ordinary reinforced concrete shear walls	5.5	2.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	2.5	4.0	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3.0	3.0	3.0	NL	NL	NP

➢ *Seismic design category, NL = No height restriction, NP = Not permitted. Number represents maximum allowable height (m).*

➢ *Dual Systems include buildings, which consist of both moment resisting frame and shear walls (or braced frame) where both systems resist the total design forces in proportion to their lateral stiffness.*

2.5 DEAD LOADS

Dead loads are loads of constant magnitude that remain in one position. They consist of the structural frame's own weight and other loads that are permanently attached to the frame. For a steel-frame building, the frame, walls, floors, roof, plumbing, and fixtures are dead loads. To design a structure, it is necessary for the weights, or dead loads, of the various parts to be estimated for use in the analysis. The exact sizes and weights of the parts are not known until the structural analysis is made and the members of the structure se-lected. The weights, as determined from the actual design, must be compared with the estimated weights. If large discrepancies are present, it will be necessary to repeat the analysis and design with better-estimated weights. Reasonable estimates of structure weights may be obtained by referring to similar types of structures or to various formulas and tables available in several publications. In estimating dead loads, the actual weights of materials and constructions shall be used, if in the absence of definite information, the weights given in *BNBC - 2017* shall be assumed for the purposes of design.

2.6 LIVE LOADS

Live loads are loads that may change in position and magnitude. They are caused when a structure is occupied, used, and maintained. Live loads that move under their own power, such as trucks, people, and cranes, are said to be *moving loads*. Those loads that may be moved are *movable loads*, such as furniture and warehouse materials. A great deal of information on the magnitudes of these various loads, along with specified minimum values, are presented in *BNBC*. The minimum gravity live loads to be used for building floors are clearly specified by the applicable building code. Unfortunately, however, the values given in these various codes vary from city to city, and the designer must be sure that his or her designs meet the requirements of the locality in question.

Table 2.5: Typical Minimum Uniform Live Loads for Design of Buildings

Occupancy or Use	Uniform (*psf*)	Concentrated (*lbs.*)
Apartment house except stairs & balconies	40.0	-
Balconies (exterior)	100.0	-
Dining rooms and Restaurants	100.0	-
Garages (passenger cars only)	40.0	-

Occupancy or Use	Uniform (*psf*)	Concentrated (*lbs.)*
Gymnasium, main floors & balconies	100.0	-
Assembly Areas:		
• Fixed seats	60.0	-
• Lobbies	100.0	-
• Movable seats	100.0	-
• Platforms	100.0	-
• Stage floor	150.0	-
Office building:		
• Lobbies and 1st floor corridors	100.0	-
• Offices	50.0	-
• Corridors above 1st floor	80.0	-
Stores:		
• 1st floor	100.0	1000.0
• Other's floor	75.0	1000.0
• Whole sales all floors	125.0	1000.0
Stair and Exit ways	100.0	-
Roof:		
• Used for promenade purpose	60	-
• Used for garden/assembly purpose	100	-
Office use	-	2000.0
Computer use	-	2000.0
Elevator machine room grating	-	300.0

2.7 EARTHQUAKE LOADS

Earthquakes are one of the most shocking forces in the nature its cause damage on infrastructure so, proper designs are necessary before construction. Heavier building is hit by more forces than the light building, so heavier building is more damage than light building. For earthquake resistance building construction each structural component of building should earthquake resistance from foundation level to completion of building. Earthquake or seismic load on a structure depends on the side of the structure, maximum earthquake intensity or string ground motion and the local soil, the stiffness design and construction pattern, and its orientation in relation to the incident seismic waves. Building experiences, the horizontal distortion when subjected to earthquake motion so building should be designed with lateral force resisting system. The dynamic analysis of the building is required under the action of the specified ground motion or design response spectra. Since the probable maximum earthquake occurrences are not so frequent, buildings are designed for such earthquakes to ensure that they remain elastic and damage-free. Instead, reliance is placed on kinetic energy dissipation in the structure through plastic deformation of elements and joints. The design forces are reduced accordingly. Thus, the main philosophy of seismic designs is to reduce collapse of structure rather than a damage free structure.

2.7.1 Characteristics of Earthquake Resistant Buildings

❖ **Structural Simplicity, Uniformity and Symmetry**

Structural simplicity, uniformity, and plan symmetry is characterized by an even distribution of mass and structural elements, which allows short and direct transmission of the inertia forces created in the distributed masses of the building to its foundation. A building configuration with symmetrical layout of structural elements of the lateral force resisting system, and well-distributed in-plan, is desirable. Uniformity along the height of the building is also important. Since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might cause premature collapse. Some basic guidelines are given below:

- With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes.
- Both the lateral stiffness and the mass of the individual stores shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.
- All structural elements of the lateral load resisting systems, such as cores, structural walls, or frames shall run without interruption from the foundations to the top of the building.
- An irregular building may be subdivided into dynamically independent regular units well separated against pounding of individual units to achieve uniformity.
- The length to breadth ratio ($\lambda = L_{max}/L_{min}$) of the building in plan shall not be higher than 4, where L_{max} and L_{min} are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions.

❖ Structural Redundancy

A high degree of redundancy accompanied by redistribution capacity through ductility is desirable, enabling a more widely spread energy dissipation across the entire structure and an increased total dissipated energy. The use of evenly distributed structural elements increases redundancy. Structural systems of higher static indeterminacy may result in higher response reduction factor R.

❖ Horizontal Bi-directional Resistance and Stiffness

Horizontal earthquake motion is a bi-directional phenomenon and thus the building structure needs to resist horizontal action in any direction. The structural elements of lateral force resisting system should be arranged in an orthogonal (in plan) pattern, ensuring similar resistance and stiffness characteristics in both main directions. The stiffness characteristics of the structure should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.

❖ Torsional Resistance and Stiffness

Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions, which tend to stress the different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

❖ Diaphragm Behavior

In buildings, floors (including the roof) act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located near the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure. The in-plane stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements.

2.7.2 Site Classification

Site will be classified as type SA, SB, SC, SD, SE, S_1, and S_2 based on the provisions of this Section. Classification will be done in accordance with Table 2.6 based on the soil properties of upper 30 meters of the site profile.

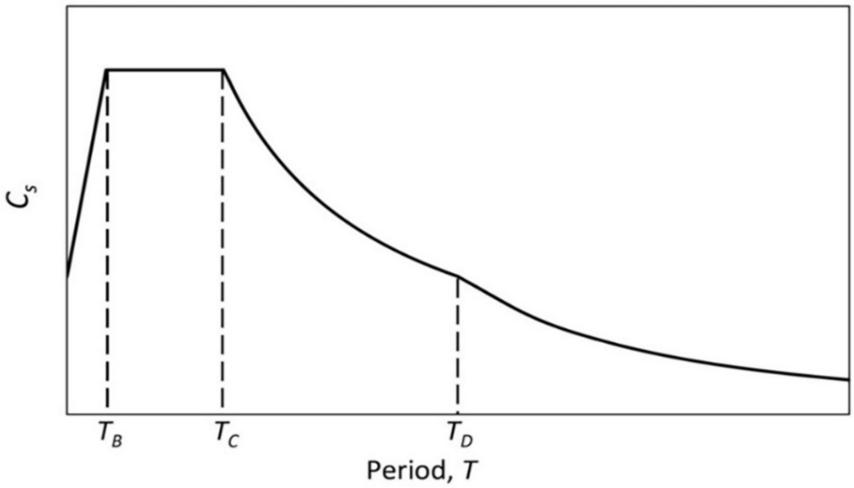


Figure 2.3: Typical shape of the elastic response spectrum coefficient C_s

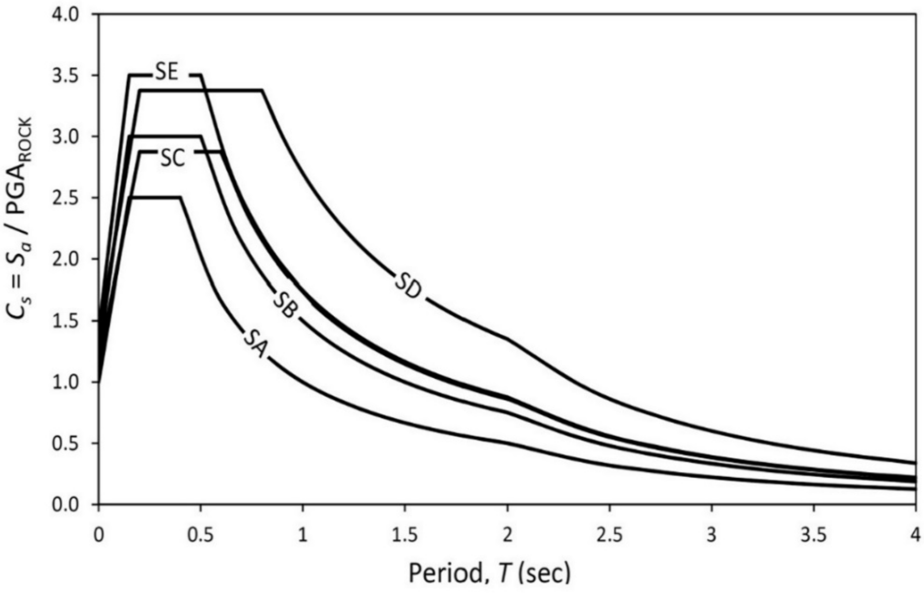


Figure 2.4: Normalized design acceleration response spectrum for different site classes

Table 2.6: Site Classification Based on Soil Properties

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, $\overline{V}_s$ (m/sec)	SPT Value, $\overline{N}$ (blows/30 cm)	Undrained Shear Strength, S_u (kPa)
SA	Rock or other rock-like geological formation including at most 5 m of weaker material at the surface.	> 800	–	–
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several 10s meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180 – 360	15 – 50	70 – 250
SD	Deposits of loose-to-medium cohesion less soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
SE	A soil profile consisting of a surface alluvium layer with V_S values of type SC or SD and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_S > 800 m/s.	–	–	–
$\boldsymbol{S_1}$	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high-water content.	< 100 (indicative)	–	10 – 20
$\boldsymbol{S_2}$	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1.	–	–	–

Table 2.7: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil Type	Soil Factor, S	T_B (sec)	T_C (sec)	T_D (sec)
SA	1.00	0.15	0.40	2.0
SB	1.20	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.40	0.15	0.50	2.0

2.7.3 Seismic Zones of Bangladesh

The intent of the seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country. In probabilistic terms, the maximum considered earthquake motion might be considered to correspond to having a 2% probability of exceedance within a period of 50 years. The country has been divided into four seismic zones with different levels of ground motion. Table 2.8 includes a description of the four seismic zones. Figure 2.5 presents a map of Bangladesh showing the boundaries of the four zones. Each zone has a seismic zone coefficient (Z) which represents the maximum considered peak ground acceleration (PGA) on very stiff soil/rock (site class SA) in units of g (acceleration due to gravity. Table 2.9 lists zone coefficients for some important towns of Bangladesh. The most severe earthquake prone zone, Zone 4 is in the northeast, which includes Sylhet and has a maximum PGA value of 0.36g.

Table 2.8: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient
1	Southwestern part with Barisal, Khulna, Jessore, Rajshahi.	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans.	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur.	Severe	0.28
4	Northeastern part with Sylhet, Mymensingh, Kurigram.	Very Severe	0.36

Table 2.9: Seismic Zone Coefficient Z for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Jamalpur	0.36	Natore	0.20
Bandarban	0.28	Jessore	0.12	Naogaon	0.20
Barguna	0.12	Jhalokati	0.12	Netrakona	0.36
Barisal	0.12	Jhenaidah	0.12	Nilphamari	0.12
Bhola	0.12	Khagrachari	0.28	Noakhali	0.20
Bogra	0.28	Khulna	0.12	Pabna	0.20
Brahmanbaria	0.28	Kishoreganj	0.36	Panchagarh	0.20
Chandpur	0.20	Kurigram	0.36	Patuakhali	0.12
Chapainababganj	0.12	Kushtia	0.20	Pirojpur	0.12
Chittagong	0.28	Lakshmipur	0.20	Rajbari	0.20
Chuadanga	0.12	Lalmanirhat	0.28	Rajshahi	0.12
Comilla	0.20	Madaripur	0.20	Rangamati	0.28
Cox's Bazar	0.28	Magura	0.12	Rangpur	0.28
Dhaka	0.20	Manikganj	0.20	Satkhira	0.12
Dinajpur	0.20	Maulvibazar	0.36	Shariatpur	0.20
Faridpur	0.20	Meherpur	0.12	Sherpur	0.36
Feni	0.20	Mongla	0.12	Sirajganj	0.28
Gaibandha	0.28	Munshiganj	0.20	Srimangal	0.36
Gazipur	0.20	Mymensingh	0.36	Sunamganj	0.36
Gopalganj	0.12	Narail	0.12	Sylhet	0.36
Habiganj	0.36	Narayanganj	0.20	Tangail	0.28
Jaipurhat	0.20	Narsingdi	0.28	Thakurgaon	0.20

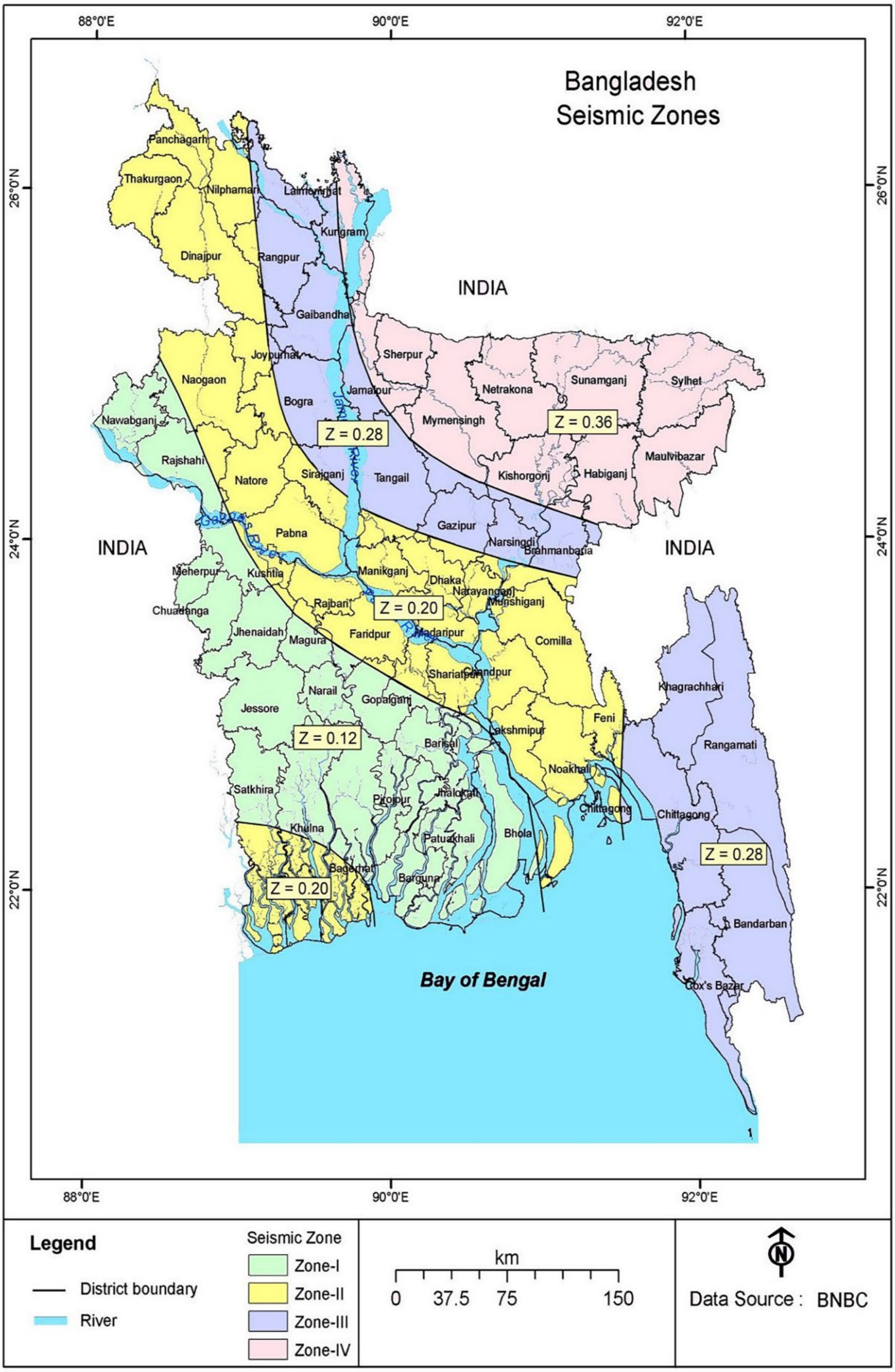


Figure 2.5: Seismic zoning maps of Bangladesh

2.7.4 Importance Factor

Buildings are classified in four occupancy categories depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. Depending on occupancy category, buildings may be designed for higher seismic forces using importance factor greater than one.

Table 2.10: Importance Factors for Buildings and Structures for Earthquake Design

Occupancy Category	Importance Factor, I
I & II	1.00
III	1.25
IV	1.50

2.7.5 Seismic Design Category (SDC)

Buildings shall be assigned a seismic design category among B, C, or D based on seismic zone, local site conditions, and importance class of building. Seismic design category D has the most stringent seismic design detailing, while seismic design category B has the least seismic design detailing requirements.

Table 2.11: Seismic Design Category of Buildings

Site Class	Occupancy Category I, II and III				Occupancy Category IV			
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

2.7.6 Seismic Design Parameters for Alternative Method for Calculation of Base Shear

Table 2.12: Spectral Response Acceleration Parameter S_s and S_1 for different Seismic Zone

Parameters	Zone 1	Zone 2	Zone 3	Zone 4
S_s	0.30	0.50	0.70	0.90
S_1	0.12	0.20	0.28	0.36

Table 2.13: Site Coefficient F_a for Different Seismic Zone and Soil Type

Soil Type	Zone 1	Zone 2	Zone 3	Zone 4
SA	1.00	1.00	1.00	1.00
SB	1.20	1.20	1.20	1.20
SC	1.15	1.15	1.15	1.15
SD	1.35	1.35	1.35	1.35
SE	1.40	1.40	1.40	1.40

Table 2.14: Site Coefficient F_v for Different Seismic Zone and Soil Type

Soil Type	Zone 1	Zone 2	Zone 3	Zone 4
SA	1.000	1.000	1.000	1.000
SB	1.500	1.500	1.500	1.500
SC	1.725	1.725	1.725	1.725
SD	2.700	2.700	2.700	2.700
SE	1.750	1.750	1.750	1.750

Table 2.15: Spectral Response Acceleration Parameter S_{DS} for Different Seismic Zone and Soil Type

Soil Type	Zone 1	Zone 2	Zone 3	Zone 4
SA	0.200	0.333	0.466	0.600
SB	0.240	0.400	0.560	0.720
SC	0.230	0.383	0.536	0.690
SD	0.270	0.450	0.630	0.810
SE	0.280	0.466	0.653	0.840

Table 2.16: Spectral Response Acceleration Parameter S_{D1} for Different Seismic Zone and Soil Type

Soil Type	Zone 1	Zone 2	Zone 3	Zone 4
SA	0.080	0.133	0.186	0.240
SB	0.120	0.200	0.280	0.360
SC	0.138	0.230	0.322	0.414
SD	0.216	0.360	0.504	0.648
SE	0.140	0.233	0.326	0.420

2.7.7 Structure/Building Period

The fundamental period T of the building in the horizontal direction under consideration shall be determined using the following guidelines:

(a) Structural dynamics procedures using structural properties and deformation characteristics of resisting elements, may be used to determine the fundamental period T of the building in the considered direction. This period shall not exceed the approximate fundamental period determined by the formula more than 40%.

(b) The building period T (in sec) may be approximated by the following formula:

$$T = C_t h_n^m$$

(c) For masonry or concrete shear wall structures, the approximate fundamental period, T in second may be determined as follows:

$$T = \frac{0.0062}{\sqrt{C_w}} h_n$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[\left(1 + 0.83\frac{h_i}{D_i}\right)^2\right]}$$

Where,

$h_n =$ Height of building in meters from foundation or from top of rigid basement. This excludes the basement stores, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement stores, when they are not so connected.

$A_B =$ Area of base of structure

$A_i =$ Web area of shear wall "i"

$D_i =$ Length of shear wall "i"

$h_i =$ Height of shear wall "i"

$x =$ Number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

Table 2.17: Values for Coefficients to Estimate Approximate Period

Structure Type	C_t	***m***
Concrete Moment Resisting Frames System	0.0466	0.90
Steel Moment Resisting Frames System	0.0724	0.80
Eccentrically Braced Steel Frames System	0.0731	0.75
All Other Structural System	0.0488	0.75

2.7.8 Design Response Spectrum

The earthquake ground motion for which the building has to be designed is represented by the design response spectrum. Both static and dynamic analysis methods are based on this response spectrum. This spectrum represents the spectral acceleration for which the building has to be designed as a function of the building period, taking into account the ground motion intensity. The spectrum is based on elastic analysis but in order to account for energy dissipation due to inelastic deformation and benefits of structural redundancy, the spectral accelerations are reduced by the response modification factor R. For important structures, the spectral accelerations are increased by the importance factor I. The design basis earthquake (DBE) ground motion is selected at a ground shaking level that is 2/3 of the maximum considered earthquake (MCE) ground motion. The effect of local soil conditions on the response spectrum is incorporated in the normalized acceleration response spectrum C_s. The spectral acceleration for the design earthquake is given by the following equation:

$$S_a = \frac{2ZIC_s}{3R} \geq 0.67\beta ZIS$$

Where,

S_a = Design spectral acceleration

β = Coefficient used to calculate lower bound for S_a. The Recommended value of β is 0.11.

Z = Seismic zone coefficient

I = Structure importance factor

R = Response reduction factor

C_s = Normalized acceleration response spectrum

S = Soil factor

<u>Normalized Acceleration Response Spectrum:</u>

The Normalized acceleration response spectrum, which is the function of structure (building) period and soil type (site class).

When, $T \leq T_B$; $\quad C_s = S\left(1 + \frac{T}{T_B}(2.5\eta - 1)\right)$

$T_B \leq T \leq T_C$; $\quad C_s = 2.5S\eta$

$T_C \leq T \leq T_D$; $\quad C_s = 2.5S\eta\left(\frac{T_C}{T}\right)$

$T_D \leq T \leq 4\ sec$; $\quad C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right)$

Where,

T = Structure (building) period

T_B = Lower limit of the period of the constant spectral acceleration

T_C = Upper limit of the period of the constant spectral acceleration

T_D = Lower limit of the period of the constant spectral displacement

η = Damping correction factor as a function of damping with a reference value of η is 1 for 5% viscous damping. It is given by the following expression:

$$\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55$$

Where, ξ is the viscous damping ratio of the structure, expressed as a percentage of critical damping.

<u>Design Response Spectrum for Elastic Analysis:</u>

For site classes SA to SE, the design acceleration response spectrum for elastic analysis methods is obtained using S_a (in units of g) as a function of period *T*. The design acceleration response spectrum represents the expected ground motion (Design Basis Earthquake) divided by the factor R/I.

<u>Design Response Spectrum for Inelastic Analysis:</u>

For inelastic analysis methods, the anticipated ground motion (Design Basis Earthquake) is directly used. Corresponding real design acceleration response spectrum is used, which is obtained by using S_a. R = 1.0 and I = 1.0 in S_a. The 'real design acceleration response spectrum' is equal to 'design acceleration response spectrum' multiplied by R/I.

<u>Site-Specific Design Response Spectrum:</u>

For site class S_1 and S_2, site-specific studies are needed to obtain design response spectrum. For important projects, site-specific studies may also be carried out to determine spectrum instead of using S_a. The objective of such site-specific ground-motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using simplified equations.

2.7.9 Drift and Deformation

Storey Drift Limitations:

Storey drift is the horizontal displacement of one level of a building or structure relative to the level above or below due to the design gravity (dead and live loads) or lateral forces (e.g. wind and earthquake loads). Calculated storey drift shall include both translational and torsional deflections and conform to the following requirements:

(a) Storey drift, Δ, for loads other than earthquake, shall be limited as follows:

$\Delta \leq 0.005h$ for $T < 0.7$ second

$\Delta \leq 0.004h$ for $T \geq 0.7$ second

$\Delta \leq 0.0025h$ for unreinforced masonry structures.

Where, h is the height of the building or structure. The building period T.

(b) The drift limits set out in (a) above may be exceeded where it can be demonstrated that greater drift can be tolerated by both structural and nonstructural elements without affecting life safety.

(c) For earthquake loads, the story drift, Δ shall be limited in accordance with:

The design storey drift (Δ) of each storey, as determined static and dynamic analysis procedure, shall not exceed the allowable storey drift (Δ_a) as obtained from Table 2.18 for any story.

For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C or D, having torsional irregularity, the design storey drift, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the storey under consideration. For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, the allowable storey drift for such linear elastic analysis procedures shall not exceed Δ_a/ρ where ρ is termed as a structural redundancy factor. The value of redundancy factor ρ may be considered as 1.0 with exception of structures of very low level of redundancy where ρ may be considered as 1.3.

For nonlinear time history analysis (NTHA), the storey drift obtained shall not exceed 1.25 times the storey drift limit specified above for linear elastic analysis procedures.

Table 2.18: Allowable Storey Drift Limit (Δ_a)

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

1. h_{sx} is the story height below Level x.
2. There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support, which are so constructed that moment transfer between shear walls (coupling) is negligible.

Diaphragm Deflection:

The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

2.7.10 Buildings with Soft Storey

Buildings with possible soft storey action at ground level for providing open parking spaces belong to structures with major vertical irregularity. Special arrangement is needed to increase the lateral strength and stiffness of the soft/open storey.

The following two approaches may be considered:

- Dynamic analysis of such building may be carried out incorporating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.
- Alternatively, when system over strength factor, Ω_o, is not included in determining seismic load effects, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other stores. Structural

elements (e.g. columns and beams) of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls. Shear walls placed symmetrically in both directions of the building as far away from the center of the building as feasible are to be designed exclusively for 1.5 times the lateral shear force calculated before.

2.8 STATIC ANALYSIS PROCEDURE

All design against seismic loads must consider the dynamic nature of the loads should theoretically require dynamic analysis procedures. For, certain type of building structures subjected to earthquake shaking, simplified static analysis procedures may also provide reasonably good results. The equivalent static force method is such a procedure for determining the seismic lateral forces acting on the structure with an estimation of base shear and distribute on each story level. This type of analysis may be applied to buildings whose seismic response is not significantly affected by contributions from modes higher than the fundamental mode in each direction. For simple regular structures, analysis by equivalent static methods is often sufficient. This is permitted in most codes of practice for regular, low to medium-rise buildings.

2.8.1 Requirement for Static Analysis

The requirement is deemed satisfied in buildings, which fulfill the following two conditions:

- The building period (T) in the two main horizontal directions is smaller than both $4T_C$ and 2 seconds.
- The building does not possess irregularity in elevation

2.8.2 Design Static Base Shear

The evaluation of the seismic loads starts with the calculation of the design base shear, which is derived from the design response spectrum. The seismic design base shear force in a given direction shall be determined from the following relation:

$$V = S_a W$$

Where,

S_a = Design spectral acceleration (in units of g)

W = Total seismic weight of the building.

Seismic Weight:

The Seismic weight (W) is the total dead load of a building or a structure, including partition walls, and applicable portions of other imposed loads listed below:

a) For live load up to and including 60 *psf*, a minimum of 25 percent of the live load shall be applicable. Then, $W = DL + 0.25LL$.

b) For live load above 60 *psf*, a minimum of 50 percent of the live load shall be applicable. Then, $W = DL + 0.50LL$.

c) Total weight (100 percent) of permanent heavy equipment or retained liquid or any imposed load sustained in nature shall be included.

Where the probable imposed loads (mass) at the time of earthquake are more correctly assessed, the designer may go for higher percentage of live load.

2.8.3 Vertical Distribution of Lateral Forces

In the absence of a more rigorous procedure, the total seismic lateral force at the base level, in other words the base shear V, shall be considered as the sum of lateral forces F_x induced at different floor levels, these forces may be calculated as –

$$F_x = V \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Where, n is the number of stories.

F_x = Part of base shear force induced at level x

w_i = Part of the total effective seismic weight of the structure (W) assigned to level "i"

w_x = Part of the total effective seismic weight of the structure (W) assigned to level "x"

h_i = The height from the base to level "i"

h_x = The height from the base to level "x"

k = 1.0 for structure period, T ≤ 0.5 seconds.

= 2.0 for structure period, T ≥ 2.5 seconds.

= linear interpolation between 1 and 2 for other periods.

2.8.4 Storey Shear and Its Horizontal Distribution

The design storey shear (V_x) at any storey x is the sum of the forces F_x in that storey and all other stories above it, given by the equation:

$$V_x = \sum_{i=x}^{n} F_i$$

Where, F_i is the portion of base shear induced at level 'i'.

If the floor diaphragms can be considered infinitely rigid in the horizontal plane, the shear V_x shall be distributed to the various elements of the lateral force resisting system in proportion to their relative lateral stiffness. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

2.8.5 Horizontal Torsional Moments

Design shall accommodate increase in storey shear forces resulting from probable horizontal torsional moments on rigid floor diaphragms. Computation of such moments shall be as follows:

- **In-Built Torsional Effects**

When there is in-built eccentricity between the center of mass and the center of rigidity (lateral resistance) at floor levels, rigid diaphragms at each level will be subject to the torsional moment M_t.

- **Accidental Torsional Effects**

In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, accidental torsional effects need to be always considered. The accidental moment (M_{ta}) is determined assuming the storey mass to be displaced from the calculated center of mass a distance equal to five percent (5%) of the building dimension at that level perpendicular to the direction of the force under consideration. The accidental torsional moment M_{tai} at level 'i' is given as:

$$M_{tai} = e_{ai}F_i$$

Where,

e_{ai} = accidental eccentricity of floor mass at level 'i' applied in the same direction at all floors = $\pm 0.05L_i$

L_i = floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists for Seismic Design Category C or D, the irregularity effects shall be accounted for by increasing the accidental torsion M_{ta} at each level by a torsional amplification factor, A_x as illustrated in Figure 1.6 determined from the following equation:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \leq 3.0$$

Here, δ_{max} = maximum displacement at level ‘x’ computed assuming A_x = 1. And δ_{avg} = average displacements at extreme points of the building at level ‘x’ computed assuming A_x = 1. The accidental torsional moment need not be amplified for structures of light-frame construction. The torsional amplification factor A_x should not exceed 3.0.

❖ **Design for Torsional Effects**

The torsional design moment at a given storey shall be equal to the accidental torsional moment M_{ta} plus the inbuilt torsional moment M_t (if any). Where earthquake forces are applied concurrently in two orthogonal directions, the required five percent displacement of the center of mass (for accidental torsion) need not be applied in both of the orthogonal directions at the same time, but shall be applied in only one direction that produces the greater effect.

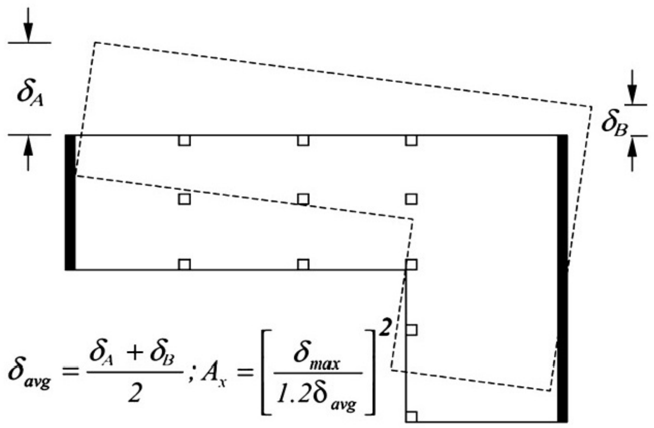


Figure 2.6: Torsional amplification factor A_x for plan irregularity

2.8.6 Deflection and Storey Drift

The deflections (δ_x) of level, ‘x’ at the center of the mass shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$

Here, C_d is the deflection amplification factor, I is the structure importance factor, and δ_{xe} is deflection determined by an elastic analysis.

The design storey drift at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration:

$$\Delta_x = \delta_x - \delta_{x-1}$$

2.8.7 Overturning Effects

The structure shall be designed to resist overturning effects caused by the seismic forces (Vertical). At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical force-resisting elements in the same proportion as the distribution of the horizontal shears to those elements. The overturning moments at level x, M_x shall be determined as follows:

$$M_x = \sum_{i=x}^{n} F_i(h_i - h_x)$$

Where,

F_i = Portion of the seismic base shear, V induced at level i

h_i = Height from the base to level i

h_x = Height from the base to level x

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_0 determined using above equation.

2.8.8 P-delta Effects

The P-Δ effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects aren't required to be considered if the stability coefficient (θ) determined by the following equation isn't more than 0.1.

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

Where,

P_x = Total vertical design load at and above level x; where computing Px, no individual load factor needs exceed 1.0.

Δ = Design story drift occurring simultaneously with V_x

V_x = Storey shear force acting between levels x and $x - 1$

h_{sx} = Storey height below level x

C_d = Deflection amplification factor

The stability coefficient θ shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

Where, β is the ratio of shear demand to shear capacity for the story between levels x and $x - 1$. This ratio is permitted to be conservatively taken as 1.0. Where, the stability coefficient θ is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $\frac{1}{(1-\theta)}$. Where, θ is greater than $\theta_{max}x$, the structure is potentially unstable and shall be redesigned.

Where, the P-delta effect is included in an automated analysis, the equation of θ_{max} shall still be satisfied, however, the value of θ computed from the equation above, using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking the equation of θ_{max}.

2.9 DYNAMIC ANALYSIS PROCEDURE

Dynamic analysis method involves applying principles of structural dynamics to compute the response of the structure to applied dynamic (earthquake) loads. Newton's second law of motion forms the basic principle of structural dynamics. This law states that the resultant force on a body is equal to its mass times the acceleration induced. Therefore, just as the 1st law of motion is a special case of the second law, static structural analysis is also a special case of structural dynamics.

A dynamic system is a simple representation of physical systems and is modeled by mass, damping and stiffness. Stiffness is the resistance it provides to deformations; mass is the matter it contains and damping represents its ability to decrease its own motion with time. This is simply the weight of the structure divided by the acceleration due to gravity. Mass contributes an inertia force in the dynamic equation of motion.

2.9.1 Requirement for Dynamic Analysis

Dynamic analysis should be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load-resisting elements, for the following buildings:

- Regular buildings with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.
- Irregular buildings with height greater than 12 m in Zones 2, 3, 4 and greater than 40 m in Zone 1.

For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended.

2.9.2 Methods of Analysis

Dynamic analysis may be carried out through the following two methods:

1. **Response Spectrum Analysis** method is a linear elastic analysis method using modal analysis procedures, where the structure is subjected to spectral accelerations corresponding to a design acceleration response spectrum. The design earthquake ground motion in this case is represented by its response spectrum.
2. **Time History Analysis** method is a numerical integration procedure where design ground motion time histories (acceleration record) are applied at the base of the structure. Time history analysis procedures can be two types: linear and non-linear. Several options determine the type of time-history analysis to be performed:
   - Linear vs. Nonlinear.
   - Modal vs. Direct-integration: These are two different solution methods, each with advantages and disadvantages. Under ideal circumstances, both methods should yield the same results to a given problem.
   - Transient vs. Periodic: Transient anal y sis considers the applied load as a onetime event, with a beginning and end. Periodic analysis considers the load to repeat in definitely, with all transient response damped out.
   - Periodic analysis is only available for linear modal time-history analysis. In a nonlinear analysis, the stiffness, damping, and load may all depend upon the displacements, velocities, and time. This re quires an iterative solution to the equations of motion.

2.9.3 Response Spectrum Analysis (RSA)

A response spectrum analysis shall consist of the analysis of a linear mathematical model of the structure to determine the maximum accelerations, forces, and displacements resulting from the dynamic response to ground shaking represented by the design acceleration response spectrum S_a. Response spectrum analysis is also called a modal analysis procedure because it considers different modes of vibration of the structure and combines effects of different modes.

Response spectrum is an elastic response spectrum for 5% equivalent viscous damping used to represent the dynamic effects of the design basis ground motion for the design of structures. Response spectrum analysis seeks the likely maximum response to these equations rather than the full-time history. The earthquake ground acceleration in each direction is given as a digitized response spectrum curve of pseudo spectral acceleration response versus period of the structure. Even though accelerations may be specified in three directions, only a single, positive result is produced for each response quantity. The response quantities include displacements, forces, and stresses.

Modeling (RSA):

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models are permitted to be constructed to represent each system. For irregular structures or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. The structure shall be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations.

In addition, the model shall comply with the following:

- Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.
- The contribution of panel zone deformations to overall story drift shall be included for steel moment frame resisting systems.

Number of Modes (RSA):

An analysis shall be conducted using the masses and elastic stiffness's of the seismic-force-resisting system to determine the natural modes of vibration for the structure including the period of each mode, the modal shape vector ϕ, the modal participation factor P and modal mass M. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions.

Modal Story Shears and Moments (RSA):

For each mode, the story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces shall be computed. The peak lateral force F_{ik} induced at level i in mode k is given by:

$$F_{ik} = A_k \phi_{ik} P_k W_i$$

Where,

A_k = Design horizontal spectral acceleration corresponding to period of vibration T_k of mode k obtained from design response spectrum (S_a).

ϕ_{ik} = Modal shape coefficient at level i in mode k

P_k = Modal participation factor of mode k

W_i = Weight of floor i

Structure Response (RSA):

In the response spectrum analysis method, the base shear, V_{rs}; each of the story shear, moment, and drift quantities; and the deflection at each level shall be determined by combining their modal values. The combination shall be carried out by taking the square root of the sum of the squares (SRSS) of each of the modal values or by the complete quadratic combination (CQC) technique. The complete quadratic combination shall be used where closely spaced periods in the translational and torsional modes result in cross-correlation of the modes.

The distribution of horizontal shear shall be in accordance with the requirements of static storey shear and its horizontal distribution. It should be noted that amplification of accidental torsion as per static procedure is not required where accidental torsional effects are included in the dynamic analysis model by offsetting the center of mass in each story by the required amount.

A base shear, V shall also be calculated using the equivalent static force procedure. Where the base shear, V_{rs} is less than 85% of V all the forces but not the drifts obtained by response spectrum analysis shall be multiplied by the ratio $\frac{0.85V}{V_{rs}}$.

The displacements and drifts obtained by response spectrum analysis shall be multiplied by C_d/I to obtain design displacements and drifts, as done in equivalent static analysis procedure. The P-delta effects shall be determined in accordance with static analysis procedure.

2.10 WIND LOADS

The actual intensity wind pressure depends on a number of factors like angle of incidence of the wind, roughness of surrounding area, effects of architectural features, shape of the structure etc. and lateral resistance of the structure. Apart from these, the maximum design wind load pressure depends on the duration and amplitude of the gusts and the probability of occurrence of an exceptional wind in the lifetime of building. It is possible to take into account the above factors in determining the wind pressure.

The lateral load due to wind is the major factor that causes the design of high-rise buildings to differ from those of low rise to medium rise buildings. For buildings of up to about 10 storied, typical properties and design is rarely affected by the wind loads. Above this height, however, the increase in size of the structural members, and the possible rearrangement of the structure to account for wind load, incurs a cost premium that increases progressively with height. Innovations in architectural treatment increase in the strengths of materials, and advances in method of analysis-tall building structures become more efficient and lighter and, consequently, more prone to deflect and even to sway under wind load. Buildings and other structures, including the Main Wind Force Resisting System (MWFRS) and Components and Cladding (C&C), shall be designed and constructed to resist wind loads as specified herein.

The design wind load buildings and other structures, including the MWFRS and C&C elements thereof, shall be determined using one of the following procedures:

- Method – 1: Simplified Procedure.
- Method – 2: Analytical Procedure.
- Method – 3: Wind Tunnel Procedure.

Wind Pressures:

Acting on opposite faces of each building surface. In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

2.10.1 Building, Open

A building having each wall at least 80% open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$.

Where,

A_o = total area of openings in a wall that receives positive external pressure (m^2).

A_g = the gross area of that wall in which A_o is identified (m^2).

2.10.2 Building, Partially Enclosed

A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%.
2. The total area of openings in a wall that receives positive external pressure exceeds 0.37 m^2 or 1% of the area of that wall, whichever is smaller, and the percentage of openings in the balance of building envelope doesn't exceed 20 %.

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$
2. $A_o > 0.37\ m^2\ or > 0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$.

Where,

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2.

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in m^2.

A_o and A_g are as defined for open building.

2.10.3 Building/Other Structure, Flexible/Dynamically Sensitive

Slender buildings or other structures that have a fundamental natural frequency less than 1 Hz, or natural period greater than 1.0 second.

2.10.4 Building/Other Structure, Rigid

A building or other structure whose fundamental frequency is greater than or equal to 1 Hz, or natural period less than or equal to 1.0 second.

2.10.5 Importance Factor

An importance factor, I for the building or other structure shall be determined from Table 2.19 based on building and structure categories.

Table 2.19: Importance Factor for Wind Loads

Occupancy Category	Non-Cyclone Prone Regions and Cyclone Prone Regions with V = 38 – 44 m/sec	Cyclone Prone Regions with V > 44 m/sec
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

2.10.6 Exposure

For each wind direction considered, the upwind exposure category shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

Surface Roughness Categories:

Surface Roughness A: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness B: Open terrain with scattered obstructions having heights generally less than 9.1 m. This category includes flat open country, grasslands, and all water surfaces in hurricane prone regions.

Surface Roughness C: Flat, unobstructed areas and water surfaces outside hurricane prone regions. This category includes smooth mud flats and salt flats.

Exposure Categories:

Exposure A: Exposure A shall apply where the ground surface roughness condition, as defined by Surface Roughness A, prevails in the upwind direction for a distance of at least 792 m or 20 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 9.1 m, the upwind distance may be reduced to 457 m.

Exposure B: Exposure B shall apply for all cases where Exposures A or C do not apply.

Exposure C: Exposure C shall apply where the ground surface roughness, as defined by Surface Roughness C, prevails in the upwind direction for a distance greater than 1,524 m or 20 times the building height, whichever is greater. Exposure C shall extend into

downwind areas of Surface Roughness A or B for a distance of 200 m or 20 times the height of the building, whichever is greater.
For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

Exception: An intermediate exposure between the preceding categories is permitted in a transition zone if it is determined by a rational analysis method defined in the recognized literature.

Exposure Category for Main Wind-Force Resisting System:

- ***Buildings and Other Structures:*** For each wind direction considered, wind loads for design of MWFRS determined from Figure 2.10 shall be based on exposure categories.
- ***Low-Rise Buildings:*** Wind loads for the design of the MWFRSs for low-rise buildings shall be determined using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site where external pressure coefficients GC_{pf} given in Figure 2.14 are used.

Exposure Category for Components and Cladding:
Components & cladding design pressures for all buildings and other structures shall be based on the exposure resulting in the highest wind loads for any direction at the site.

Velocity Pressure Exposure Coefficient:
Based on the exposure category, a velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined from Table 2.21. For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h, between those shown in Table 2.21, are permitted, if they are determined by a rational analysis method defined in the recognized literature.

Table 2.20: Terrain Exposure Constants

Exposure	α	z_g (m)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (m)	$\bar{\in}$	z_{min} (m)
A	7.0	365.76	1/7	0.84	1/4	0.45	0.30	97.54	1/3	9.14
B	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5	4.57
C	11.5	213.36	1/11.5	1.07	1/9	0.80	0.15	198.12	1/8	2.13

z_{min} = minimum height used to ensure the equivalent height z is greater of *0.6h* or z_{min}.
For building with $h \le z_{min}$, $\bar{z}$ shall be taken as z_{min}.

Table 2.21: Velocity Pressure Exposure Coefficients, K_z or K_h

Height above ground level (z)	Exposure Category			
	A		B	C
(m)	*Case 1*	*Case 2*	*Case 1 & 2*	*Case 1 & 2*
0 – 4.6	0.70	0.57	0.85	1.03
6.1	0.70	0.62	0.90	1.08
7.6	0.70	0.66	0.94	1.12
9.1	0.70	0.70	0.98	1.16
12.2	0.76	0.76	1.04	1.22
15.2	0.81	0.81	1.09	1.27
18.0	0.85	0.85	1.13	1.31
21.3	0.89	0.89	1.17	1.34
24.4	0.93	0.93	1.21	1.38
27.41	0.96	0.96	1.24	1.40
30.5	0.99	0.99	1.26	1.43
36.6	1.04	1.04	1.31	1.48
42.7	1.09	1.09	1.36	1.52
48.8	1.13	1.13	1.39	1.55
54.9	1.17	1.17	1.43	1.58
61.0	1.20	1.20	1.46	1.61
76.2	1.28	1.28	1.53	1.68
91.4	1.35	1.35	1.59	1.73
106.7	1.41	1.41	1.64	1.78
121.9	1.47	1.47	1.69	1.82
137.2	1.52	1.52	1.73	1.86
152.4	1.56	1.56	1.77	1.89

1. *Case 1:*
   - All components and cladding.
   - The MWFRS in low-rise buildings designed using Figure 2.14.

   Case 2:
   - All MWFRS in buildings except those in low-rise buildings designed using Figure 2.14.
   - All main wind force resisting systems in other structures.
2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For $4.57\text{ m} \le z \le z_g$; $\quad K_z = 2.01(z/z_g)^{2/\alpha}$

   For $4.57\text{ m} > z$; $\quad K_z = 2.01(4.57/z_g)^{\frac{2}{\alpha}}$

   Note: z shall not be taken less than 9.1 m for Case 1 in exposure A.
3. α and z_g are tabulated in Table 2.20.
4. Linear interpolation for intermediate values of height z is acceptable.

2.10.7 Topographic Effects

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

- The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (*100H*) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height *H* of the hill, ridge, or escarpment is determined.
- The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 3.22 km radius in any quadrant by a factor of two or more.
- The building or other structure is located as shown in Figure 2.8 in the upper one-half of a hill or ridge or near the crest of an escarpment.
- $H/L_h \geq 0.2$.
- H is greater or equal to 4.5 m for Exposure B and C and 18.3 m for Exposure A.

Topographic Factor: The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

Here, K_1, K_2, and K_3 are given in Figure 2.8.

If site conditions and locations of structures do not meet all the conditions then, K_{zt} = 1.0.

2.10.8 Basic Wind Speed

The basic wind speed, V used in the determination of design wind loads on buildings and other structures shall be as given in Figure 2.7 except special wind regions. The wind shall be assumed to come from any horizontal direction. Basic wind speed in different places of Bangladesh as describe in the Table 2.23.

Special Wind Regions:

The basic wind speed shall be increased where records or experience indicate that wind speeds are higher than those reflected in Figure 2.7. Mountainous terrain, gorges and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Figure 2.7 to account for higher local wind speeds. Such adjustment shall be based on adequate meteorological information and other necessary data.

Wind Directionality Factor:

The wind directionality factor, K_d shall be determined from Table 2.22. This factor shall only be applied when used in conjunction with load combinations.

Table 2.22: Wind Directionality Factor

Structure Type	Directional Factor, K_d
Buildings:	
Main Wind Force Resisting System (MWFRS)	0.85
Components and Cladding (C&C)	0.85
Arched Roofs	0.85
Chimneys, Tanks and Similar Structures:	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers:	
Triangular, Square, Rectangular	0.85
All Other Cross-section	0.95

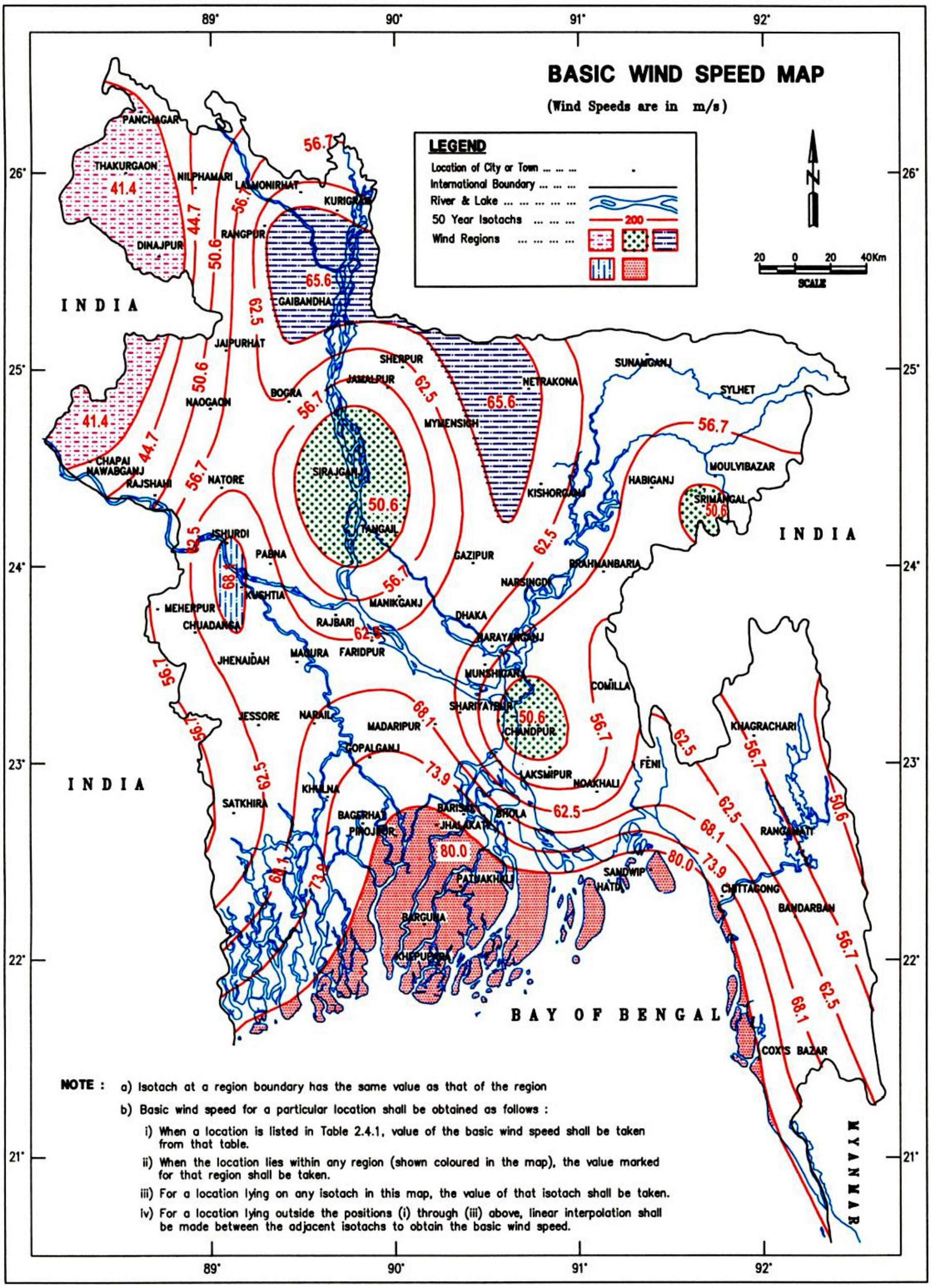


Figure 2.7: Basic Wind Speed (V in m/sec) Map of Bangladesh

Table 2.23: Basic Wind Speeds (V in m/sec.) for Selected Locations in Bangladesh

Location	Basic Wind Speed (m/s)	Location	Basic Wind Speed (m/s)
Angarpota	47.8	Lalmonirhat	63.7
Bagerhat	77.5	Madaripur	68.1
Bandarban	62.5	Magura	65.0
Barguna	80.0	Manikganj	58.2
Barisal	78.7	Meherpur	58.2
Bhola	69.5	Maheshkhali	80.0
Bogra	61.9	Moulvibazar	53.0
Brahmanbaria	56.7	Munshiganj	57.1
Chandpur	50.6	Mymensingh	67.4
Chapai Nawabganj	41.4	Naogaon	55.2
Chittagong	80.0	Narail	68.6
Chuadanga	61.9	Narayanganj	61.1
Comilla	61.4	Narsinghdi	59.7
Cox's Bazar	80.0	Natore	61.9
Dahagram	47.8	Netrokona	65.6
Dhaka	65.7	Nilphamari	44.7
Dinajpur	41.4	Noakhali	57.1
Faridpur	63.1	Pabna	63.1
Feni	64.1	Panchagarh	41.4
Gaibandha	65.6	Patuakhali	80.0
Gazipur	66.5	Pirojpur	80.0
Gopalganj	74.5	Rajbari	59.1
Habiganj	54.2	Rajshahi	49.2
Hatiya	80.0	Rangamati	56.7
Ishurdi	69.5	Rangpur	65.3
Joypurhat	56.7	Satkhira	57.6
Jamalpur	56.7	Shariatpur	61.9
Jessore	64.1	Sherpur	62.5
Jhalakati	80.0	Sirajganj	50.6
Jhenaidah	65.0	Srimangal	50.6
Khagrachhari	56.7	St. Martin's Island	80.0
Khulna	73.3	Sunamganj	61.1
Kutubdia	80.0	Sylhet	61.1
Kishoreganj	64.7	Sandwip	80.0
Kurigram	65.6	Tangail	50.6
Kushtia	66.9	Teknaf	80.0
Lakshmipur	51.2	Thakurgaon	41.4

2.10.9 Minimum Design Wind Loading

The design wind load, determined by any one of the procedures, shall be not less than specified in this Section.

- **Main Wind Force Resisting System (MWFRS)**

The wind load to be used in the design of the Main Wind Force Resisting System for an enclosed or partially enclosed building or other structure shall not be less than $0.5\ kN/m^2$ multiplied by the area of building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than $0.5\ kN/m^2$) multiplied by A_f.

- **Components and Cladding (C&C)**

The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.5 kN/m^2 acting in either direction normal to the surface.

Sway Limitation:

The overall sway (horizontal deflection) at top level of the building or structure due to wind loading shall not exceed $\frac{1}{500}$ times of the total height of the building above the ground level.

2.11 ANALYTICAL PROCEDURE

A building or other structure whose design wind loads are determined in accordance with this Section shall meet all of the following conditions:

1. The building or other structure is a regular-shaped.
2. The building or other structure does not have response characteristics making it subject to a cross wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

The provisions of this Section take into consideration the load magnification effect caused by gusts in resonance with along wind vibrations of flexible buildings or other structures. Buildings or other structures not meeting the requirements of simplified procedure, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure.

2.11.1 Design Procedure

- Determine the basic wind speed V and wind directionality factor K_d.
- An importance factor I shall be determined.
- An exposure category or exposure categories and velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined for each wind direction.
- A topographic factor K_{zt} shall be determined.
- A gust effect factor G or G_f, as applicable, shall be determined.
- An enclosure classification shall be determined.
- Internal pressure coefficient GC_{pi} shall be determined.
- External pressure coefficients C_p or GC_{pf}, or force coefficients C_f.
- Velocity pressure q_z or q_h.
- Design wind load P or F shall be determined

2.11.2 Gust Effects

The gust-effect factor for a rigid building or other structure is permitted to be taken as 0.85. To determine whether a building or other structure is rigid or flexible. The fundamental natural frequency n_1, shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. Low-rise buildings are permitted to be considered rigid.

- ***For rigid buildings or other structures,*** the gust-effect factor shall be taken as 0.85 or calculated by this formula:

$$G = 0.925 \frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}$$

$$I_{\bar{z}} = c \left(\frac{10}{\bar{z}}\right)^{1/6}$$

Where, $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$. $\bar{z}$ is the equivalent height of the structure defined as *0.6h*, but not less than z_{min} for all building heights, (*h*). z_{min} and *c* are listed for each exposure in Table 2.20. The value of g_Q and g_v shall be taken as 3.4. The background response Q is given by –

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B + h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{10}\right)^{\bar{\varepsilon}}$$

Where,

B = horizontal dimension of building measured normal to wind direction, in m.

h = mean roof height of the building or other structure, except that eve height shall be used for roof angle θ is less than or equal to 10^0, in m.

In which l and $\bar{\epsilon}$ are constants listed in Table 2.20.

- ***For flexible or dynamically sensitive structures,*** the gust effect factor shall be calculated by –

$$G_f = 0.925\left(\frac{1 + 1.7I_{\bar{z}}\sqrt{g_Q^2Q^2 + g_R^2R^2}}{1 + 1.7g_vI_{\bar{z}}}\right)$$

The value of g_Q and g_v shall be taken as 3.4, and g_R is given by –

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}}$$

R, the resonant response factor, is given by –

$$R = \sqrt{\frac{1}{\beta}R_nR_hR_B(0.53 + 0.47R_L)}$$

$$R_n = \frac{7.47N_1}{(1 + 10.3N_1)^{5/3}}$$

$$N_1 = \frac{n_1L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})\text{; For } \eta > 0$$

$$R_\ell = 1 \text{ for } \eta = 0$$

Here, the subscript l in the above R_l equation shall be taken as h, B, and L, respectively.

$R_h = R_l$; setting $\eta = 4.6n_1\,h/\bar{V}_{\bar{z}}$

$R_B = R_l$; setting $\eta = 4.6n_1\,B/\bar{V}_{\bar{z}}$

$R_L = R_l$; setting $\eta = 15.4n_1\,L/\bar{V}_{\bar{z}}$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V$$

Where,

β = damping ratio, percent of critical

n_1 = building natural frequency, in Hz.

B = horizontal dimension of building measured normal to wind direction in m.

h = mean roof height of the building or other structure, except that eve height shall be used for roof angle θ is less than or equal to 10^0, in m.

L = horizontal dimension of building measured parallel to the wind direction in m

V = basic wind speed in m/sec.

$\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 2.20.

2.11.3 Enclosure Classifications

For, the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open.

Openings:

A determination shall be made of the number of openings in the building envelope to determine the enclosure classification as defined below.

Wind-Borne Debris:

Glazing in buildings located in wind-borne debris regions shall be protected with an impact-resistant covering or be impact-resistant glazing according to the requirements specified in ASTM E1886 and ASTM E1996 or other approved test methods and performance criteria. The levels of impact resistance shall be a function of Missile Levels and Wind Zones specified in ASTM E1886 and ASTM E1996.

Exceptions:

- Glazing in Category II, III, or IV buildings located over 18.3 m above the ground and over 9.2 m above aggregate surface roofs located within 458 m of the building shall be permitted to be unprotected.
- Glazing in Category I buildings shall be permitted to be unprotected.

Multiple Classifications:

If a building by definition complies with both the “open” and “partially enclosed” definitions, it shall be classified as an “open” building. A building that does not comply with either the “open” or “partially enclosed” definitions shall be classified as an “enclosed” building.

Velocity Pressure:

Velocity pressure, q_z evaluated at height z shall be calculated by the following equation:

$$q_z = 0.000613 K_z K_{zt} K_d V^2 I \qquad [kN/m^2]$$

Where,

K_d = wind directionality factor

K_z = velocity pressure exposure coefficient

K_{zt} = topographic factor

V = basic wind speed in m/sec.

I = importance factor

2.11.4 Pressure and Force Coefficients

Internal Pressure Coefficient:

Internal pressure coefficients, GC_{pi} shall be determined from Figure 2.9 based on building enclosure classifications.

Reduction Factor for Large Volume Buildings, R_i. For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi} shall be multiplied by the following reduction factor, R_i:

$$R_i = 1.0 \; or, \qquad R_i = 0.5\left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{6951 A_{og}}}}\right) \leq 1.0$$

Where,

A_{og} = total area of openings in the building envelope (walls and roof, in m^2)

V_i = unpartitioned internal volume, in m^3

External Pressure Coefficients:

❖ ***Main Wind-Force Resisting Systems***

External pressure coefficients for MWFRSs C_p are given in Figures 2.10 to 2.12. Combined gust effect factor and external pressure coefficients, GC_{pf} are given in Figure 2.14 for low-rise buildings. The pressure coefficient values and gust effect factor in Figure 2.14 shall not be separated.

❖ ***Components and Cladding***

Combined gust effect factor and external pressure coefficients for components and cladding GC_p are given in Figures 2.15 to 2.21. The pressure coefficient values and gust-effect factor shall not be separated.

Force Coefficient:

Force coefficients C_f are given in Figures 2.22 to 2.25.

2.11.5 Design Wind Loads on Enclosed and Partially Enclosed Buildings

- ***Sign Convention:*** Positive pressure acts toward the surface and negative pressure acts away from the surface.
- ***Critical Load Condition:*** Values of external and internal pressures shall be combined algebraically to determine the most critical load.
- ***Tributary Areas Greater than 65 m²:*** Component and cladding elements with tributary areas greater than 65 m^2 shall be permitted to be designed using the provisions for main wind force resisting systems.

Main Wind-Force Resisting Systems:

❖ ***Rigid Buildings of All Heights***

Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}); \qquad kN/m^2$$

Where,

$q = q_z$ for windward walls evaluated at height z above the ground level.

$q = q_h$ for leeward walls, sidewalls, and roofs, evaluated at height h.

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings.

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering, shall be treated as an opening. For positive internal pressure evaluation, q_i may conservatively be evaluated at height $h = (q_i - q_h)$.

G = gust effect factor.

C_p= external pressure coefficient from Figures 2.10 or 2.12.

GC_{pi} = internal pressure coefficient from Figure 2.9.

q and q_i shall be evaluated using exposure. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in the Figures 2.10 & 2.12.

❖ ***Flexible Buildings***

Design wind pressures for the MWFRS of flexible buildings shall be determined from the following equation:

$$p = qG_f C_p - q_i(GC_{pi}); \qquad kN/m^2$$

Where,

q, q_i, C_p and GC_{pi} are defined in previous.

G_f is the gust factor determined by the flexible or dynamically sensitive structure formula.

<u>Design Wind Load Cases:</u>

The MWFRS of buildings of all heights, whose wind loads have been determined under the previous section, shall be designed for the wind load cases as defined in Figure 2.13. The eccentricity *e* for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (ex, ey).

The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (ex, ey):

$$e = \frac{e_Q + 1.7 I_{\bar{z}} \sqrt{\left(g_Q Q e_Q\right)^2 + (g_R R e_R)^2}}{1 + 1.7 I_{\bar{z}} \sqrt{\left(g_Q Q\right)^2 + (g_R R)^2}}$$

Where,

e_Q = Eccentricity e as determined for rigid structures in Figure 2.13.

e_R = Distance between the elastic shear center and center of mass of each floor.

$I_{\bar{z}}, g_Q, Q, g_R$ and R shall be defined as gust factor determination section.

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

Exception:

One-story buildings with h less than or equal to 9.1 m, buildings two stories or less framed with light-frame construction, and buildings two stories or less designed with flexible diaphragms need only be designed for Load Case 1 and Load Case 3 in Figure 2.13.

Components and Cladding:

❖ ***Buildings with h > 18.3 m***

Design wind pressures on components and cladding for all buildings with h > 18.3 m shall be determined from the following equation:

$$p = q(GC_p) - q_i(GC_{pi}); \qquad kN/m^2$$

Where,

$q = q_z$ for windward walls calculated at height z above the ground level

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

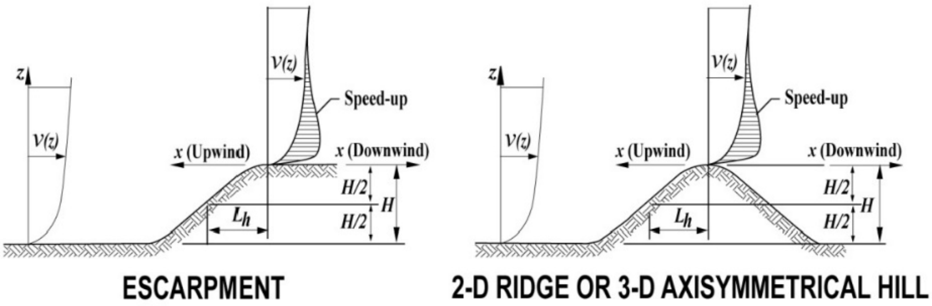
$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings.

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering, shall be treated as an opening. For positive internal pressure evaluation, q_i may conservatively be evaluated at height $h = (q_i - q_h)$.

GC_p = external pressure coefficient from Figure 2.21.

GC_{pi} = internal pressure coefficient given in Figure 2.9.

q and q_i shall be evaluated using exposure.



Topographic Multipliers for Exposure B										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute $2H$ for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:

   H: Height of hill or escarpment relative to the upwind terrain, in meters.
   L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in meters.
   K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   K_3: Factor to account for reduction in speed-up with height above local terrain.
   x: Distance (upwind or downwind) from the crest to the building site, in meters.
   z: Height above local ground level, in meters.
   W: Horizontal attenuation factor.
   γ: Height attenuation factor.

   Equation:

   $K_{zt} = (1 + K_1 K_2 K_3)^2$; K_1 determined from Table below; $K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$; $K_3 = e^{-\gamma z/L_h}$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$			γ	μ	
	Exposure A	Exposure B	Exposure C		Upwind of crest	Downwind of Crest
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. Hill	0.95	1.05	1.15	4	1.5	1.5

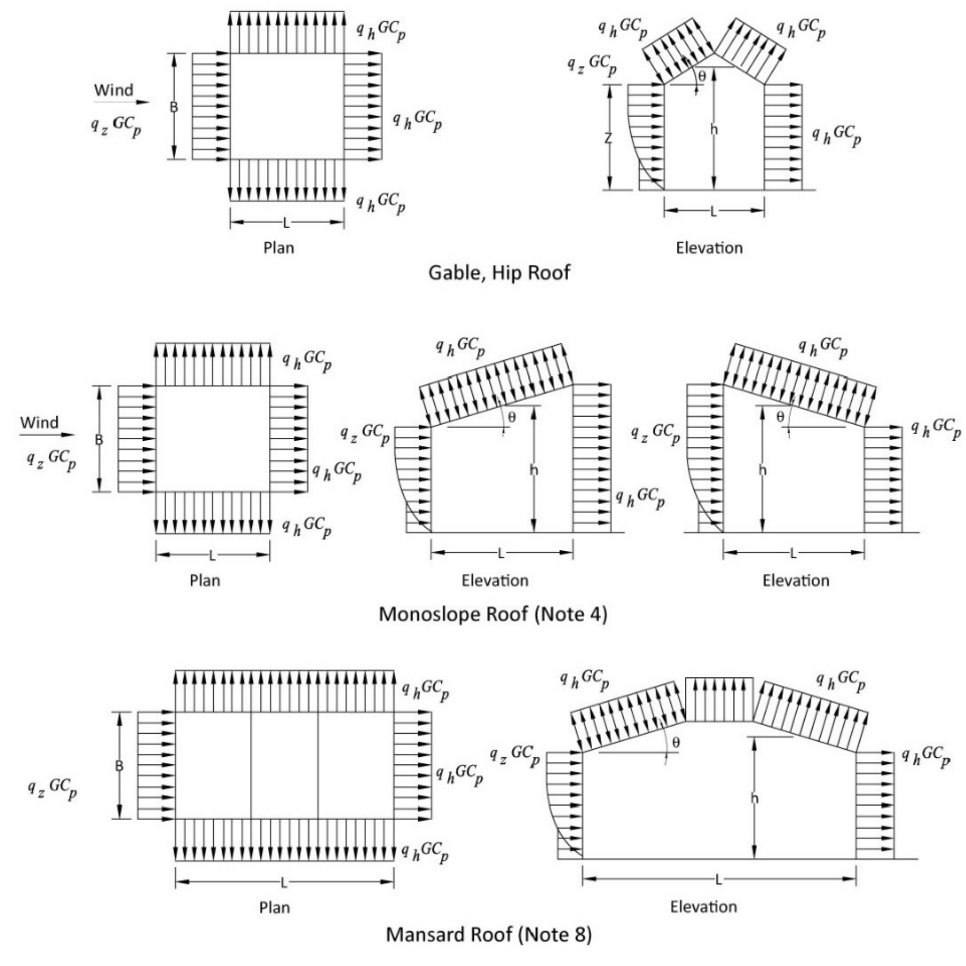
Figure 2.8: Topographic Factor, K_{zt} – Method 2

Enclosed, Partially Enclosed, and Open Buildings: Walls & Roofs

Enclosure Classification	GC_{pi}	Notes:
Open Building	0.00	1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
Partially Enclosed Building	+0.55 -0.55	2. Values of GC_{pi} shall be used with q_z or q_h as specified. 3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
Enclosed Building	+0.18 -0.18	(i) a positive value of GC_{pi} applied to all internal surfaces (ii) a negative value of GC_{pi} applied to all internal surfaces.

Figure 2.9: Internal Pressure Coefficient, GC_{pi} MWFRS and C&C – Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings: Walls & Roofs



Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All values	-0.7	q_h

Figure 2.10: External Pressure Coefficients, C_p MWFRS – Method 2 (All Heights)

Roof Pressure Coefficients, C_p, for use with q_h												
Wind Direction	**Windward**									**Leeward**		
	Angle, θ (degrees)									**Angle, θ (degrees)**		
	h/L	10	15	20	25	30	35	45	>60#	10	15	>20
Normal To ridge for θ $\geq 10^0$	$\leq$0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01θ	-0.5	-0.5	-0.6
	$\geq$1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horizontal distance from Windward edge	C_p		
Normal To ridge for θ <10° and Parallel to ridge for all θ	$\leq$0.5	0 to $h/2$	-0.9, -0.18		
		$h/2$ to h	-0.9, -0.18		
		h to $2h$	-0.5, -0.18		
		> 2h	-0.3, -0.18		
	$\geq$ 1.0	0 to $h/2$	-1.3**,-0.18	Area (m²)	Reduction Factor
				$\leq$ 9.3	1.0
		> $h/2$	-0.7, -0.18	23,2	0.9
				$\geq$ 92.9	0.8

* Value is provided for interpolation purposes

** Value can be reduced linearly with area over which it is applicable as follows

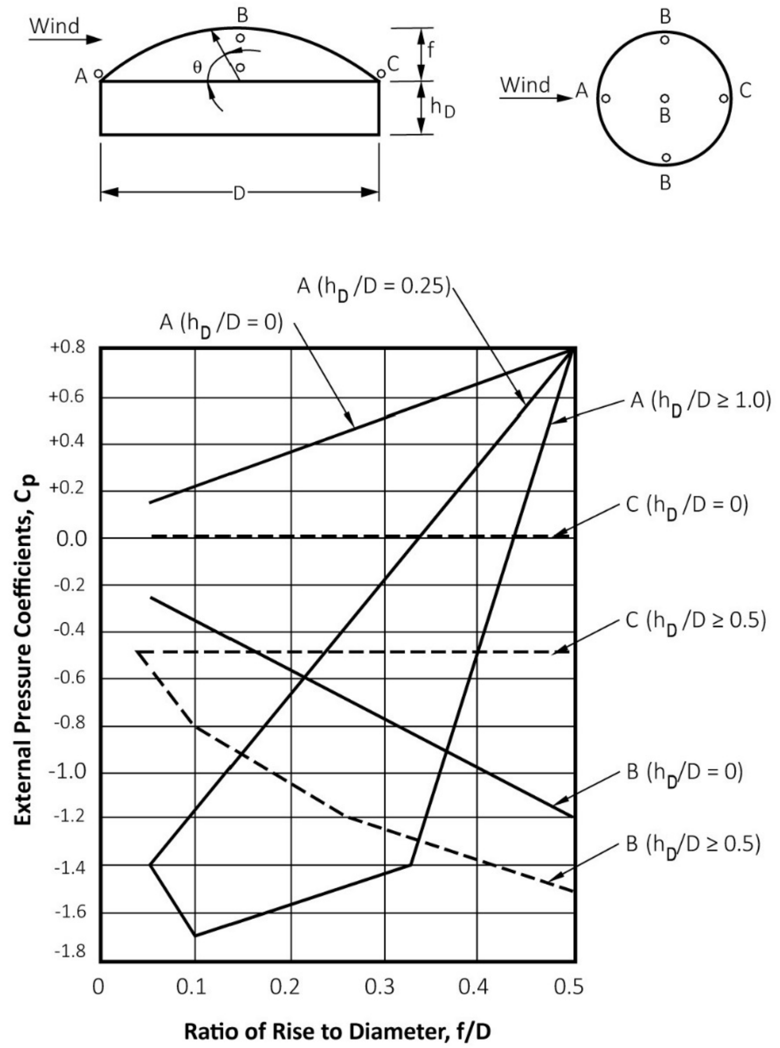
Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of L/B, h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined.
6. Refer to Figure 2.11 for domes and Figure 2.12 for arched roofs.
7. Notation:
   - B: Horizontal dimension of building, in meter, measured normal to wind direction.
   - L: Horizontal dimension of building, in meter, measured parallel to wind direction.
   - h: Mean roof height in meters, except that eave height shall be used for e 10 degrees.
   - z: Height above ground, in meters.
   - G: Gust effect factor.
   - q_z, q_h: Velocity pressure, in N/m², evaluated at respective height.
   - θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Figure 2.10(Contd.): External Pressure Coefficients, C_p MWFRS – Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings and Structures: Domed Roofs



External Pressure Coefficients for Domes With a Circular Base.
(Adapted from Eurocode, 1995)

Notes:

1. Two load cases shall be considered:
   Case A. C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;
   Case B. C_p shall be the constant value of A for θ ≤ 25 degrees, and shall be determined by linear interpolation from 25 degrees to B and from B to C.
2. Values denote C_p to be used with q_{h_D+f} where ($h_D + f$) is the height at the top of the dome.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
5. For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
6. ϑ=0 degrees on dome springline, θ=90 degrees at dome center top point. f is measured from springline to top.
7. The total horizontal shear shall not be less than that determined by neglecting wind forces roof surfaces.
8. For f/D values less than 0.05, use Figure 2.10.

Figure 2.11: External Pressure Coefficients, C_p MWFRS – Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings and Structures: Arched Roofs

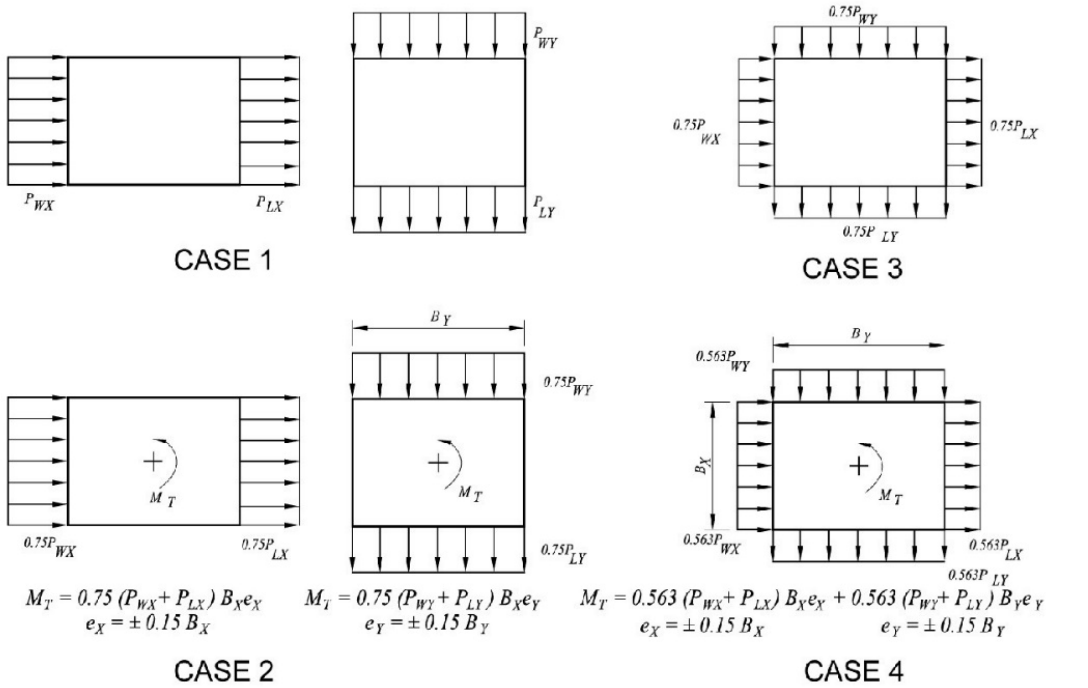
Condition	Rise-to-span ratio, r	C_p		
		Windward quarter	Center half	Leeward quarter
Roof on elevated structure	$0 < r < 0.2$	-0.9	$-0.7 - r$	-0.5
	$0.2 \le r < 0.3$*	$1.5r - 0.3$	$-0.7 - r$	-0.5
	$0.3 \le r \le 0.6$	$2.75r - 0.7$	$-0.7 - r$	-0.5
Roof springing from ground level	$0 < r \le 0.6$	$1.4r$	$-0.7 - r$	-0.5

Notes:

* When the rise-to-span ratio is $0.2 \le r \le 0.3$, alternate coefficients given by $(6r - 2.1)$ shall also be used for the windward quarter.

1. Values listed are for the determination of average load on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Figure 2.10 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Figure 2.15 with e based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this Table multiplied by 0.87.

Figure 2.12: External Pressure Coefficients, C_p MWFRS and C&C – Method 2 (All Heights)



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with Method 2, Design Wind Loads on Enclosed and Partially Enclosed Buildings as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.

   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.

   $e(e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.

   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Figure 2.13: Design Wind Load Cases for MWFRS – Method 2 (All Heights)

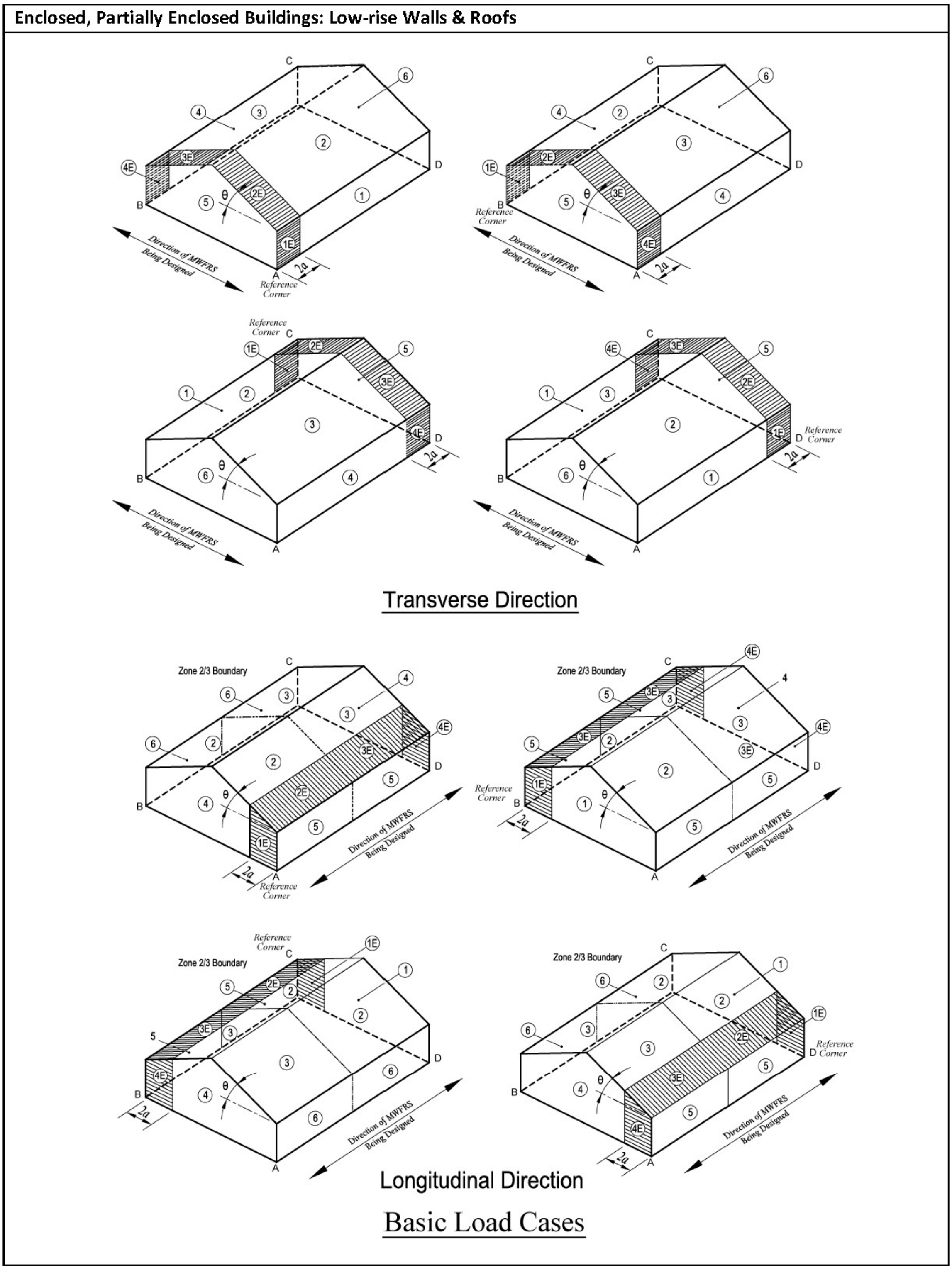


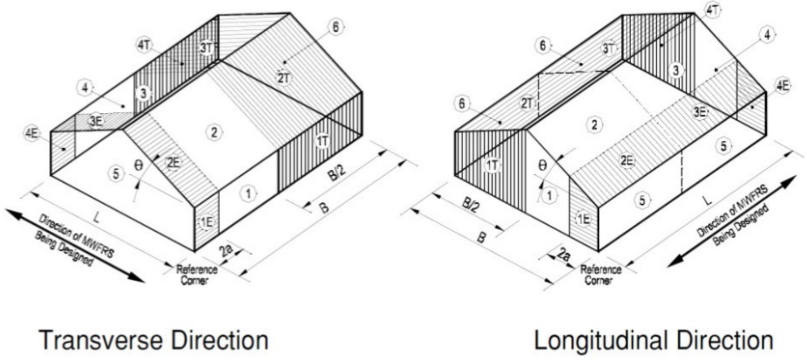
Figure 2.14: External Pressure Coefficients, GC_{pf} for MWFRS – Method 2 (h $\leq$ 18.3 m)

Enclosed, Partially Enclosed Buildings: Low-rise Walls & Roofs

Roof Angle θ (degrees)	Building Surface									
	1	2	3	4	5	6	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	-0.45	-0.45	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	-0.45	-0.45	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	-0.45	-0.45	0.69	0.69	-0.48	-0.48

Notes:

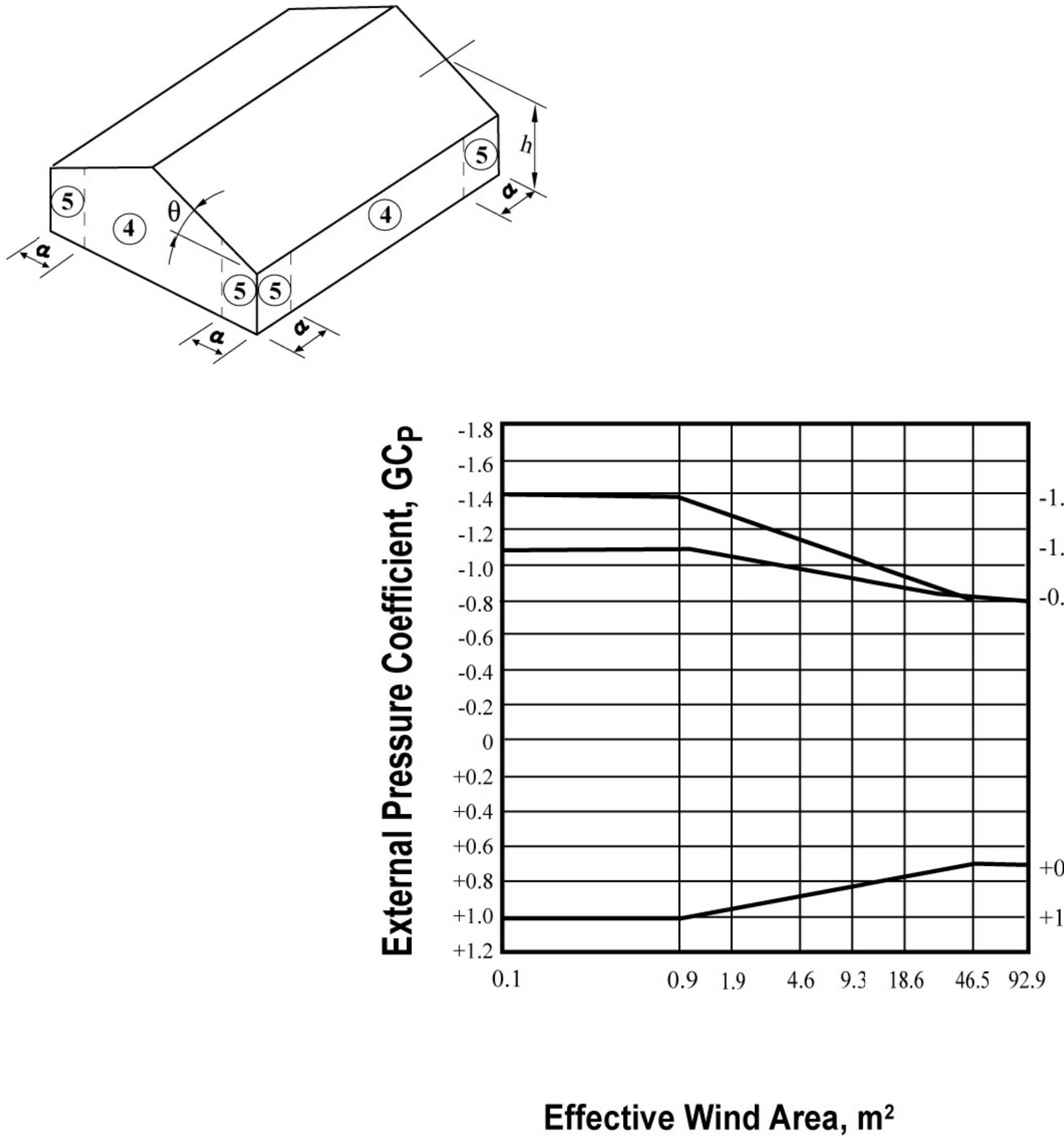
1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of ϑ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
4. Combinations of external and internal pressures (see Figure 2.9) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4).
   Exception: One story buildings with h less than or equal to 9.1m, buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.
   Torsional loading shall apply to all eight basic load patterns using the figures below applied at each reference corner.
6. Except for moment-resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
7. For the design of the MWFRS providing lateral resistance in a direction parallel to a ridge line or for flat roofs, use θ = 0° and locate the zone 2/3 boundary at the mid-length of the building.
8. The roof pressure coefficient GC_{pf}, when negative in Zone 2 or *2E*, shall be applied in Zone *2/2E* for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height, he, at the windward wall, whichever is less; the remainder of Zone *2/2E* extending to the ridge line shall use the pressure coefficient GC_{pf} for Zone *3/3E*.
9. Notation:
   a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.
   h: Mean roof height, in meters, except that eave height shall be used for $\vartheta \leq 10°$.
   ϑ: Angle of plane of roof from horizontal, in degrees.



Torsional Load Cases

Figure 2.14(Contd.): External Pressure Coefficients, GC_{pf} for MWFRS – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Walls



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_P for walls shall be reduced by 10% when $\theta \leq 10^0$.
6. Notation:

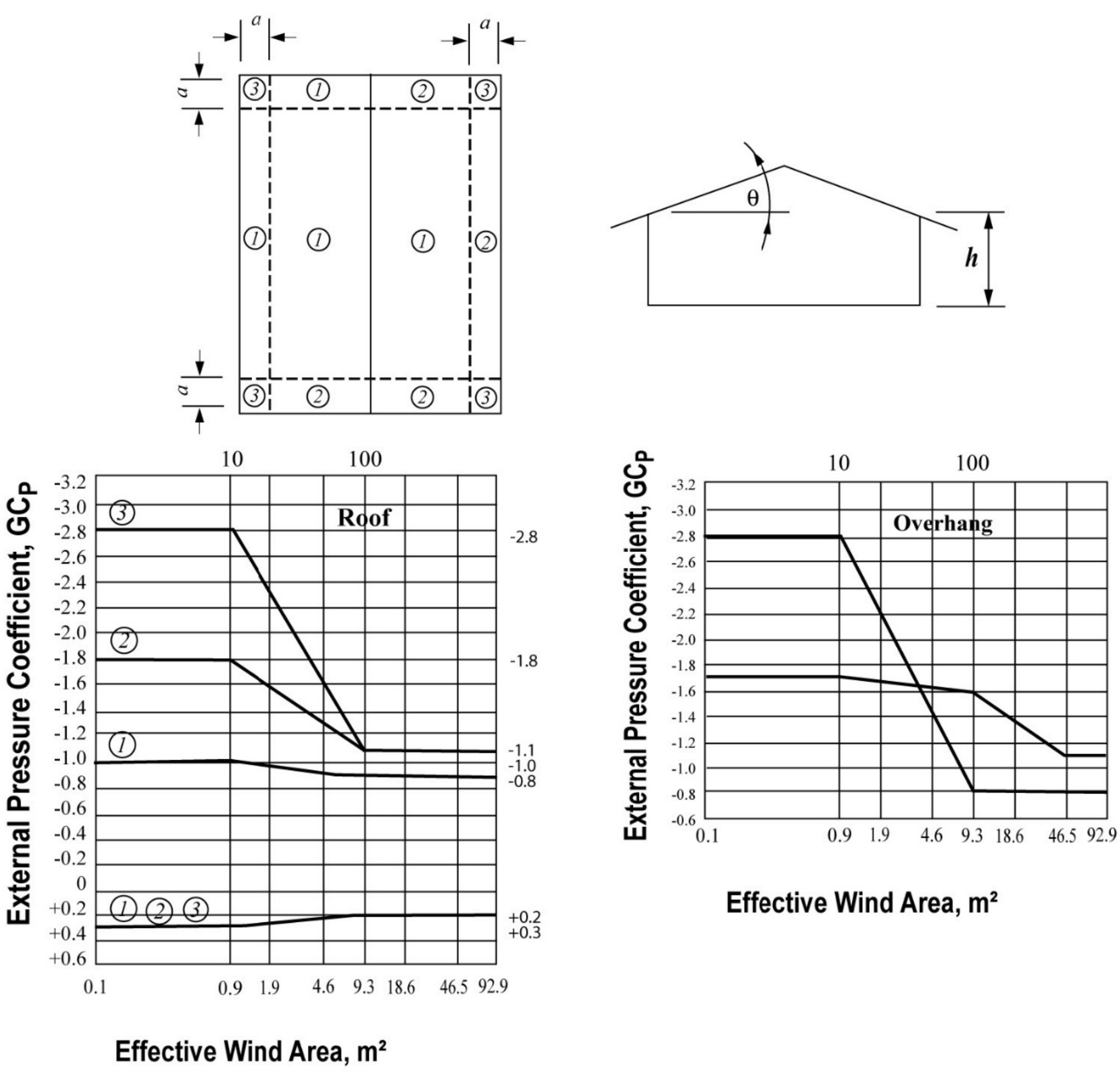
   a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9m.

   h: Mean roof height, in meters, except that eave height shall be used for $\theta \leq 10^0$.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.15(a): External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Gable Roofs θ ≤ 70



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with θ ≤ 7⁰, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_P in Zones 2 and 4 shall be set equal to those for wall Zones 4 and 5 respectively in Figure 2.15(a).
6. Values of GC_P for roof overhangs include pressure contributions from both upper and lower surfaces.
7. Notation:

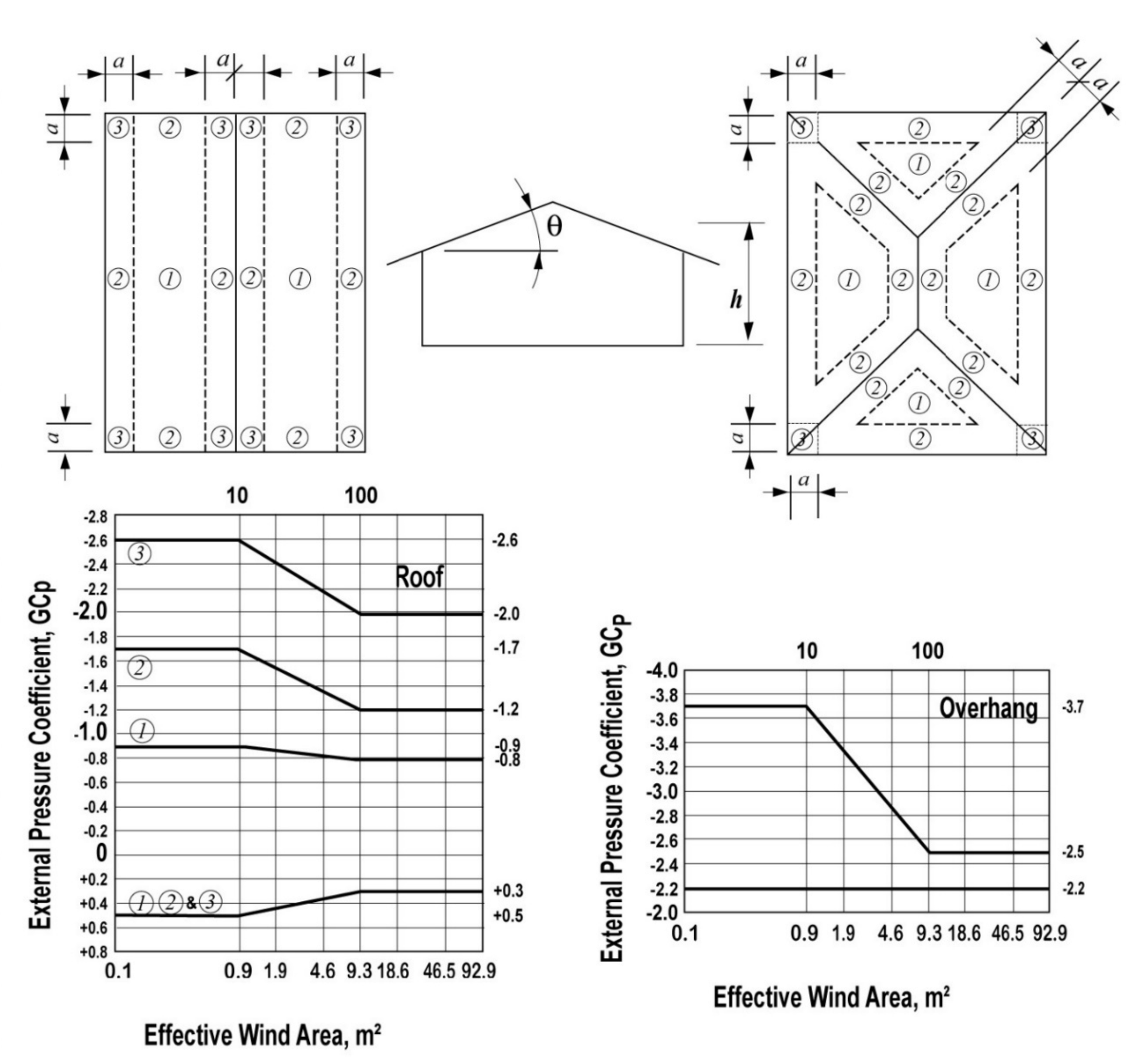
   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

   h: Eave height shall be used for θ ≤ 10^0.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.15(b): External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Gable/Hip Roofs $7^0 < \theta \leq 27^0$



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7^0 < \theta \leq 27^0$, edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $7^0 < \theta \leq 25^0$, Zone 3 shall be treated as Zone 2.
8. Notation:

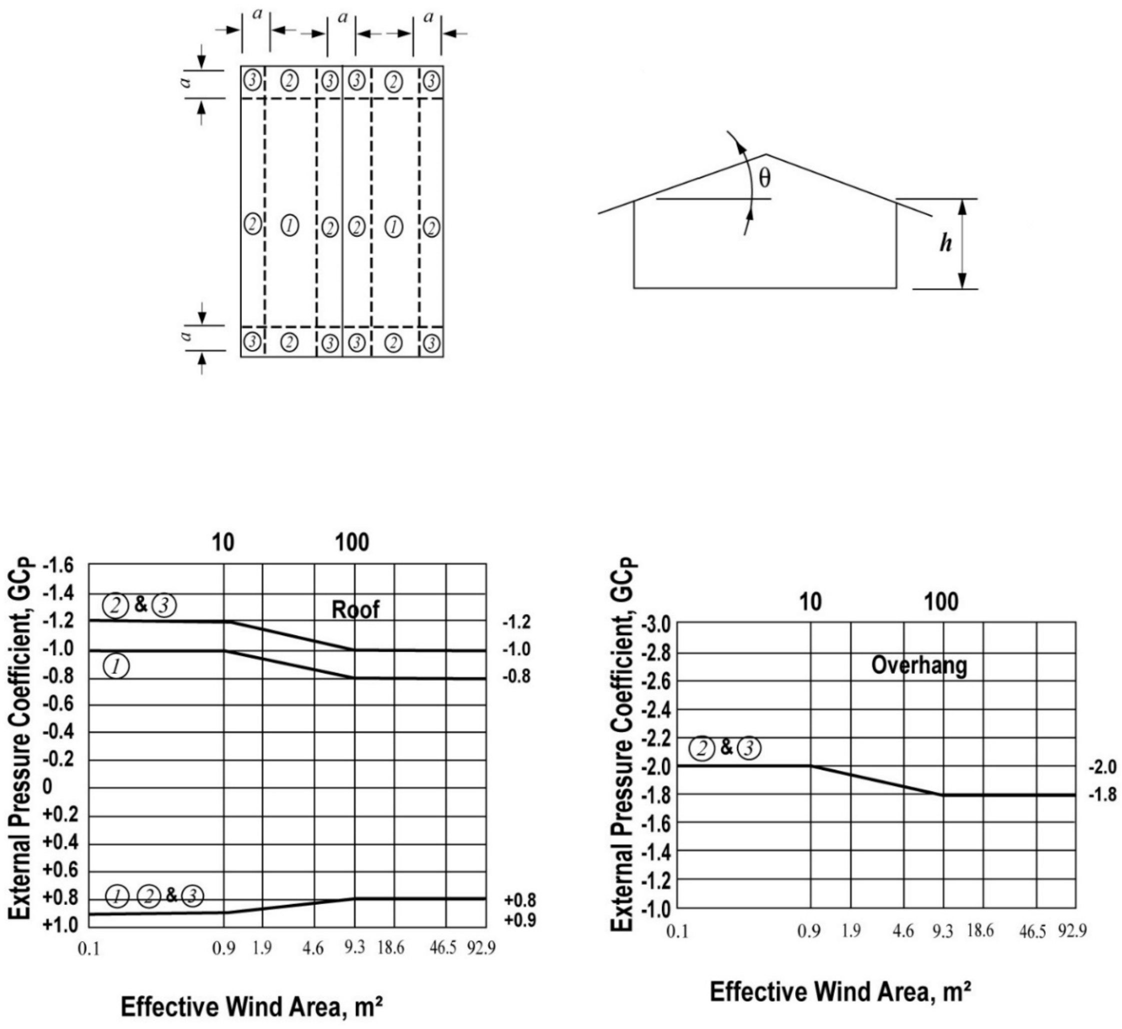
   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

   h: Mean roof height, in meters, except that eave height shall be used for $\theta \leq 10^0$.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.15(c): External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Gable Roofs $27^0 < \theta \le 45^0$



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_P for roof overhangs include pressure contributions from both upper and lower surfaces.
6. Notation:

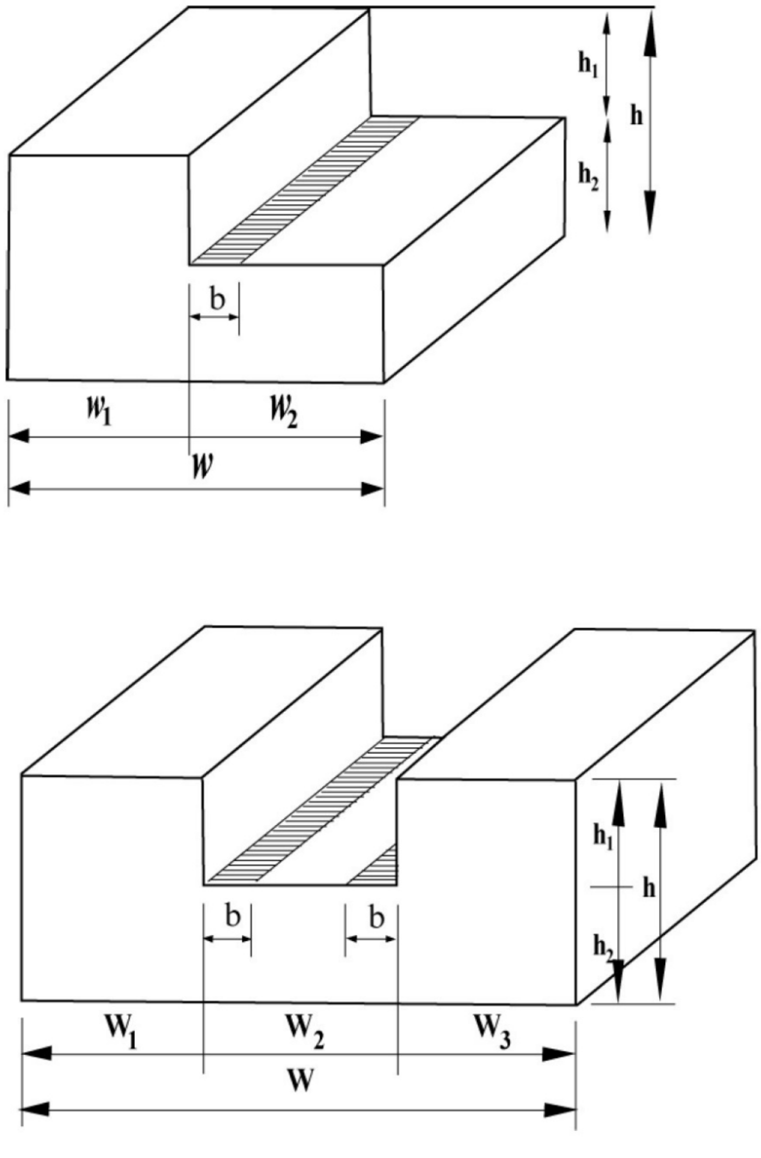
   a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9m.

   h: Mean roof height, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.15(d): External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Stepped Roofs



Notes:

On the lower level of flat, stepped roofs shown in Figure 2.16, the zone designations and pressure coefficients shown in Figure 2.15(b) shall apply, except that at the roof-upper wall intersection(s), Zone 3 shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in Figure 2.15(a) shall apply on the cross-hatched areas shown in Figure 2.16.

Notation:

b: $1.5h_1$ in Figure 2.16, but not greater than 30.5 m.

h: Mean roof height, in meters.

h_i: h_1 or h_2 in Figure 2.16; $h = h_1 + h_2$; $h_1 \geq 3.1$ m; h_i/h = 0.3 to 0.7.

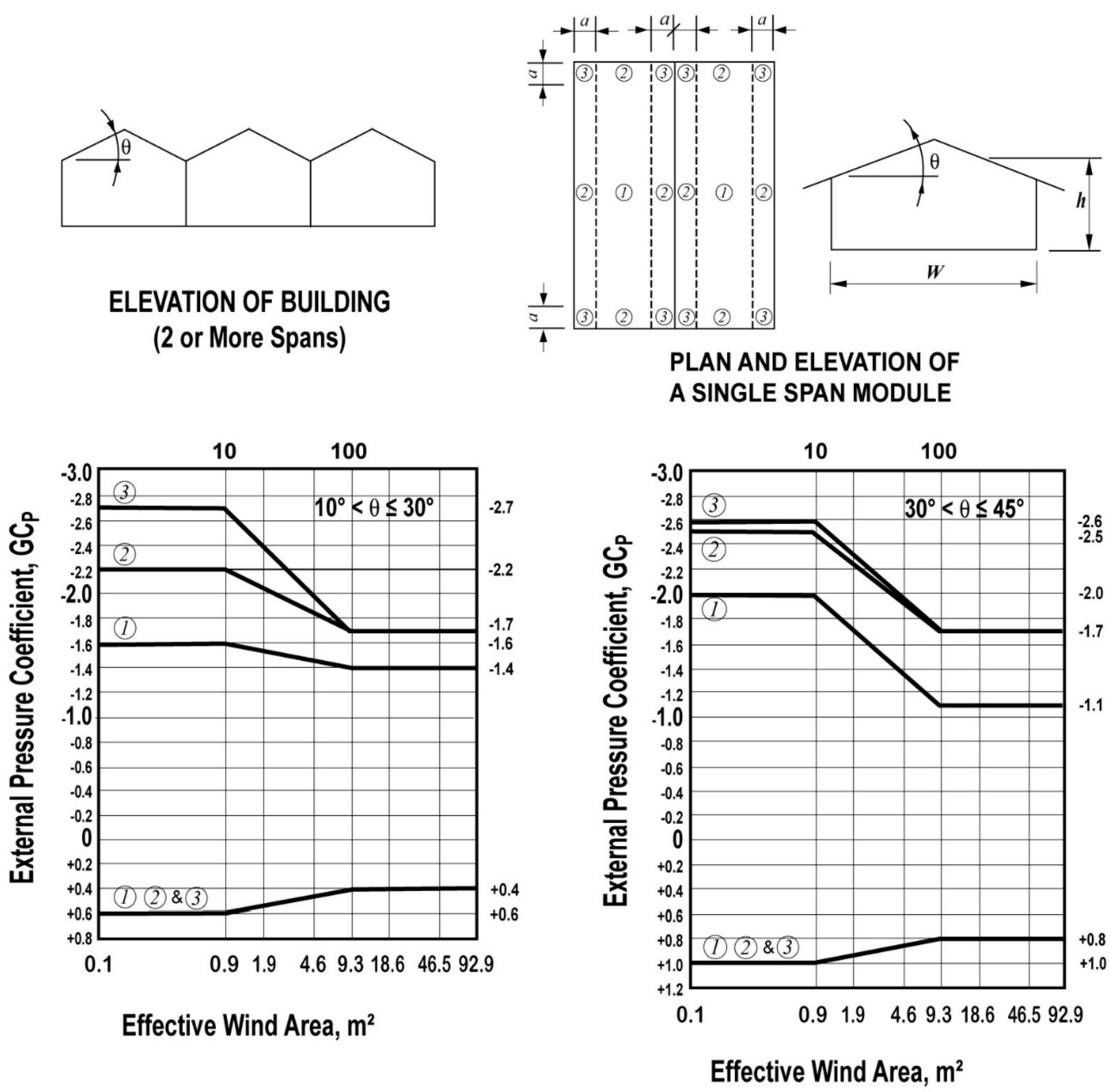
W: Building width in Figure 2.16.

W_i: W_1 or W_2 or W_3 in Figure 2.16. $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; W_i/W = 0.25 to 0.75.

e: Angle of plane of roof from horizontal, in degrees.

Figure 2.16: External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Multispan Gable Roofs



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For θ ≤ 10⁰ Values of GC_p from Figure 2.15 shall be used.
6. Notation:

   a: 10 percent of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

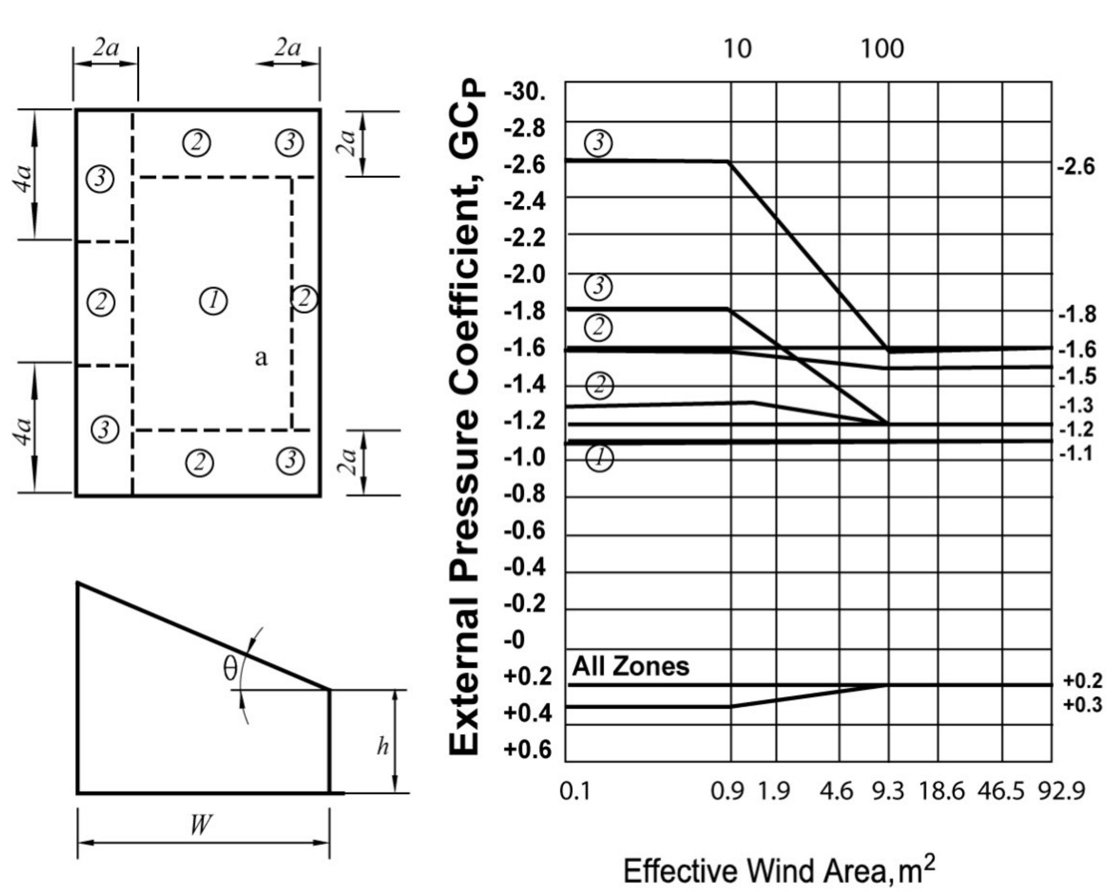
   h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10^0$.

   W: Building module width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.17: External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Monoslope Roofs $3^0 < \theta \leq 10^0$



Notes:

1. Vertical scale denotes GC_P to be used with q_h
2. Horizontal scale denotes effective wind area *A,* in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For θ ≤ 30 Values of GC_P from Figure 2.15(b) shall be used.
6. Notation:

   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

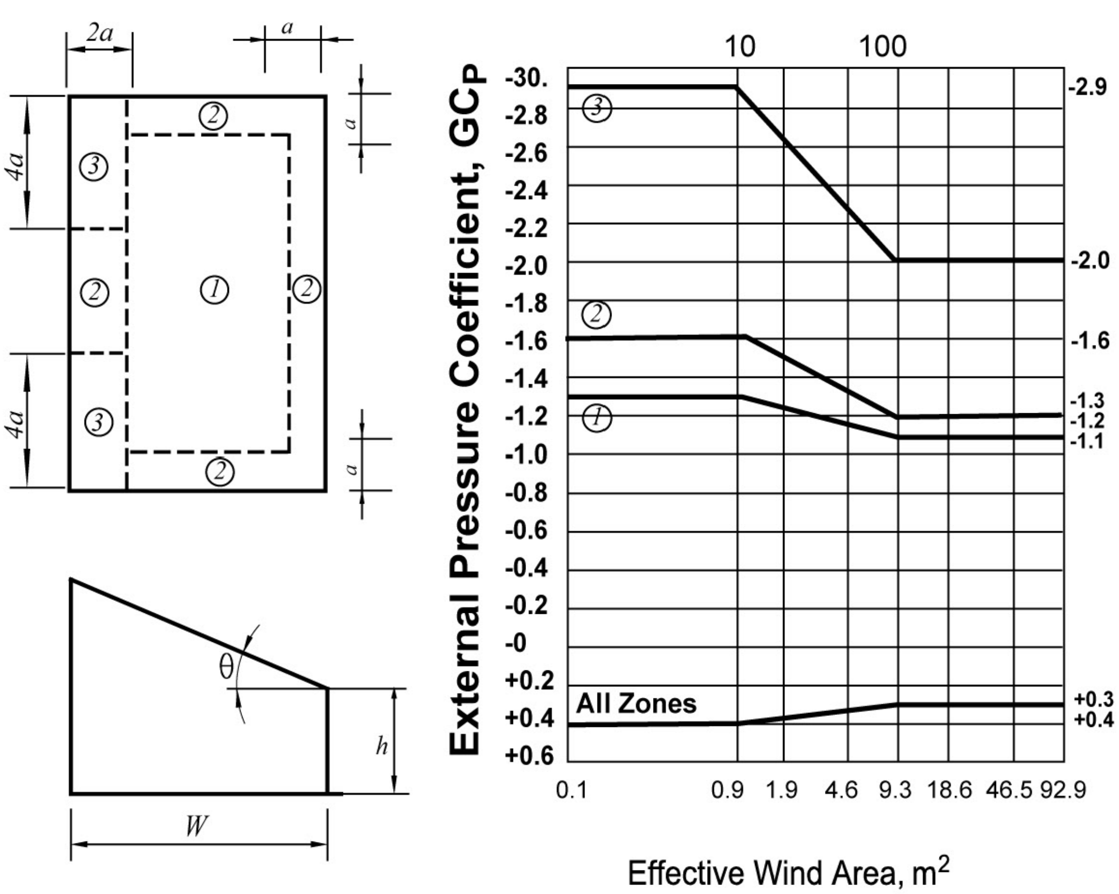
   h: Eave height shall be used for $\theta \leq 10^0$.

   W: Building width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.18(a): External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Monoslope Roofs $10^0 < \theta \leq 30^0$



Notes:

1. Vertical scale denotes GC_P to be used with q_h
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Notation:

   a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

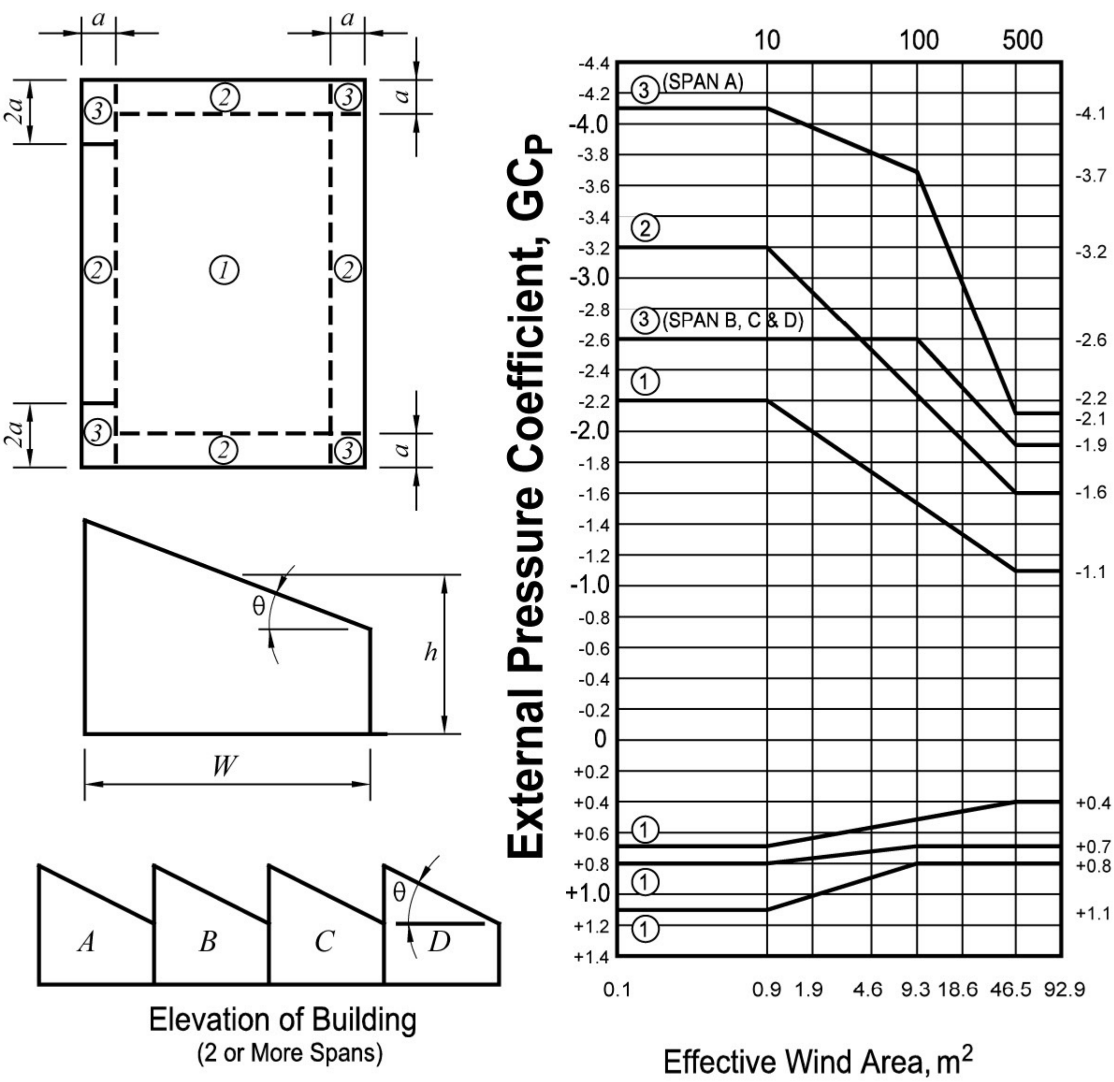
   h: Mean roof height in meters.

   W: Building width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.18(b): External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Sawtooth Roofs



Notes:

1. Vertical scale denotes GC_P to be used with q_h
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\vartheta \leq 10^0$ Values of GC_P from Figure 2.15 shall be used.
6. Notation:

   a: 10 percent of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

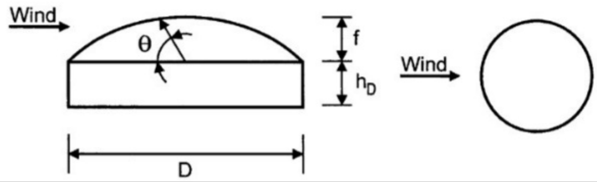
   h: Mean roof height in meters except that eave height shall be used for $\theta \leq 10^0$.

   W: Building width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.19: External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Domed Roofs



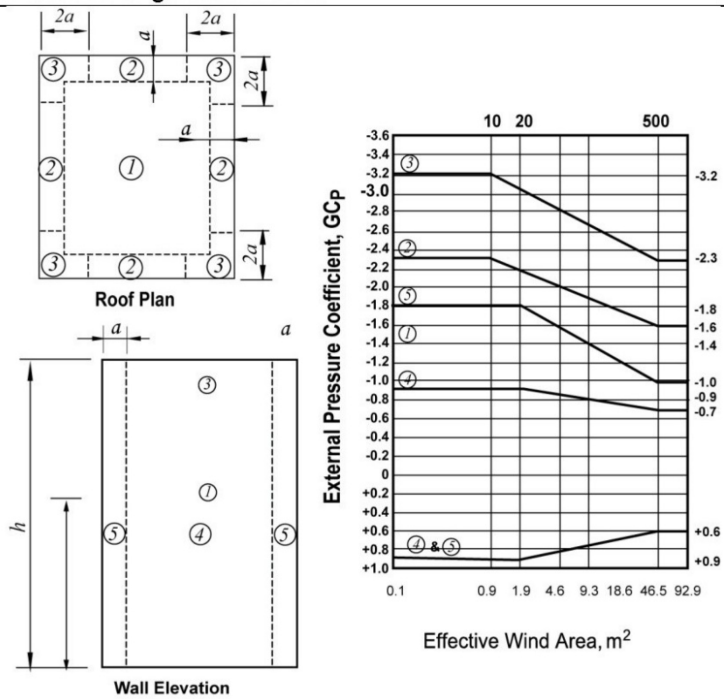
External Pressure Coefficients for Domes with a circular Base			
θ, degrees	Negative Pressures	Positive Pressures	Positive Pressures
	0 – 90	0 – 60	61 – 90
GC_p	-0.9	+0.9	+0.5

Notes:

1. Values denote C_p to be used with $q_{(hD+f)}$ where h_D+f is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for maximum positive and negative pressures.
4. Values apply to $\theta \leq h_D D \leq 0.5$, $0.2 \leq f/D \leq 0.5$.
5. θ =0° on dome springline, θ = 90° at dome center top point. f is measured from springline to top.

Figure 2.20: External Pressure Coefficients, GC_p for C&C – Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings: Walls & Roofs



Notes:

1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle θ≤10°. For other roof angles and geometry, use GC_p values from Figure 2.15 and attendant q_h based on exposure definition.
7. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \leq 10°$, Zone 3 shall be treated as Zone 2.
8. Notation:

   a: 10 percent of least horizontal dimension, but not less than 0.9 m.

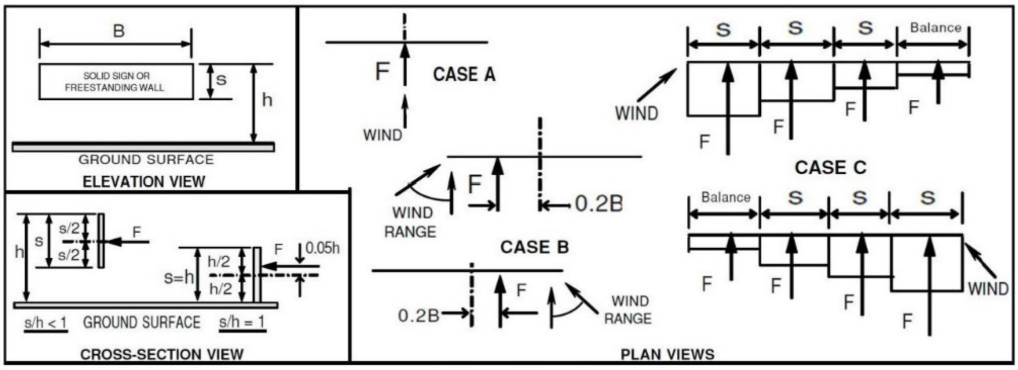
   h: Mean roof height, in meters, except that eave height shall be used for θ≤10°.

   z: height above ground, in (meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 2.21: External Pressure Coefficients, GC_p for C&C – Method 2 (h ≤ 18.3 m)

Solid Freestanding Walls & Solid Signs



Clearance Ratio, s/h	C_f, CASE A & CASE B — Aspect Ratio, B/s											
	≤0.05	0.1	0.2	0.5	1	2	4	5	10	20	30	≥45
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40
0.7	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55
0.5	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75
0.3	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85
0.2	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95
≤0.16	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95

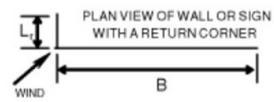
C_f, CASE C

Region (horizontal distance from windward edge)	Aspect Ratio, B/s								
	2	3	4	5	6	7	8	9	10
0 to s	2.25	2.60	2.90	3.10*	3.30*	3.40*	3.55*	3.65*	3.75*
s to $2s$	1.50	1.70	1.90	2.00	2.15	2.25	2.30	2.35	2.45
$2s$ to $3s$		1.15	1.30	1.45	1.55	1.65	1.70	1.75	1.85
$3s$ to $10s$			1.10	1.05	1.05	1.05	1.05	1.00	0.95

Region (horizontal distance from windward edge)	Aspect Ratio, B/s	
	13	≥45
0 to s	4.00*	4.30*
s to $2s$	2.60	2.55
$2s$ to $3s$	2.00	1.95
$3s$ to $4s$	1.50	1.85
4s to 5s	1.35	1.85
5s to 10s	0.90	1.10
>10s	0.55	0.55

*Values shall be multiplied by the following reduction factor when a return corner is present:

L_r/s	Reduction Factor
0.3	0.9
1.0	0.75
≥2	0.60



Notes:

1. The term "signs" in notes below also applies to "freestanding walls".
2. Signs with openings comprising less than 30% of the gross area are classified as solid signs. Force coefficients for solid signs with openings shall be permitted to be multiplied by the reduction factor $(1 - (1 - \varepsilon)^{1.5})$.
3. To allow for both normal and oblique wind directions, the following cases shall be considered:
   For $s/h < 1$:
   CASE A: resultant force acts normal to the face of the sign through the geometric center.
   CASE B: resultant force acts normal to the face of the sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign.
   For $B/s \geq 2$, CASE C must also be considered:
   CASE C: resultant forces act normal to the face of the sign through the geometric centers of each region.
   For $s/h = 1$:
   The same cases as above except that the vertical locations of the resultant forces occur at a distance above the geometric center equal to 0.05 times the average height of the sign.
4. For CASE C where $s/h > 0.8$, force coefficients shall be multiplied by the reduction factor $(1.8 - s/h)$.
5. Linear interpolation is permitted for values of s/h, B/s and L_r/s other than shown.
6. Notation: B: horizontal dimension of sign, in meters;
   h: height of the sign, in meters;
   s: vertical dimension of the sign, in meters;
   ε: ratio of solid area to gross area;
   L_r: horizontal dimension of return corner, in meters

Figure 2.22: Force Coefficient, C_f for Other Structures – Method 2 (All Heights)

Chimneys, Tanks, Rooftop Equipment, & Similar Structures

Cross-Section	Type of Surface	h/D		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagomal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round $D\sqrt{q_z} > 5.3, D$ in m, q_z in $\mathrm{N/m^2}$	Moderately smooth	0.5	0.6	0.7
	Rough (D'/D=0.02)	0.7	0.8	0.9
	Very rough (D'/D=0.08)	0.8	1.0	0.2
Round $D\sqrt{q_z} \leq 5.3, D$ in m, q_z in $\mathrm{N/m^2}$	All	0.7	0.8	1.2

Notes:
1. The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
2. Linear interpolation is permitted for *h/D* values other than shown.
3. Notation:
   D: diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-section at elevation under consideration, in meters;
   D': depth of protruding element such as ribs and spoilers, in meters;
   H: height of structure, meters and
   q_z: velocity pressure evaluated at height z above ground, in $\mathrm{N/m^2}$

Figure 2.23: Force Coefficient, C_f for Other Structures – Method 2 (All Heights)

Open Signs & Lattice Frameworks

∈	Flat-Sided Members	Rounded Members	
		$(D\sqrt{q_z} \leq 5.3,)$	$(D\sqrt{q_z} > 5.3,)$
<0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

Notes:
1. Signs with openings comprising 30% or more of the gross area are classified as open signs.
2. The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind.
3. The area A_f consistent with these force coefficients is the solid area projected normal the wind direction.
4. Notation:
   $\in$: ratio of solid area to gross area;
   D: diameter of a typical round number, in meters
   q_z: velocity pressure evaluated at height z above ground in $\mathrm{N/m^2}$.

Figure 2.24: Force Coefficient, C_f for Other Structures – Method 2 (All Heights)

Open Structures: Trussed Tower

Tower Cross Section	C_f
Square	$4.0 \in^2 - 5.9 \in + 4.0$
Triangle	$3.4 \in^2 - 4.7 \in + 3.4$

Notes:
1. For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
2. The specified force coefficients are for towers with structural angles or similar flat-sided members.
3. For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members: $0.51 \in^2 + 0.57 \leq 1.0$
4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:
   $1 + 0.75 \in \leq 1.2$
5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
6. Notation:
   $\in$: ratio of solid area to gross area of one tower face for the segment under consideration.

Figure 2.25: Force Coefficient, C_f for Other Structures – Method 2 (All Heights)

Chapter 3

DESCRIPTION OF THE STRUCTURE

CHAPTER THREE

DESCRIPTION OF THE STRUCTURE

3.1 INTRODUCTION

This chapter provides detailed description of the structure. 41 storied high-rise structures having flat plate and edge supported floor system with beam-column and shear wall had been considered. The structure has three parts to purposes of the Commercial, Official, and Residential users.

The main features of the structure can be summarized as below:

- **Location** : Dhaka
- **Height of the building** : 411.5 *ft.* (125.42 m)
- **Typical floor height** : 10 *ft.* (3.05 m)
- **Height of the Basement** : 12 *ft.* (3.66 m)
- **Number of Basement floors** : 2 *nos.*
- **Number Garage Floor** : 3 *nos.* (Car Parking)
- **Number of Residential floors** : 40 *nos.*
- **Modern Amenities** :

  There are eight high-speed simplex elevators having capacity 19 persons and four stairs as emergency exits. 24 hours electricity supply with generator facility; to fulfill the demand of water supply for all users, covers by huge capacity of underground water reservoir; large parking lot for the users. Total passenger car parking capacity of the building is 744 nos. and huge space for bike parking.

Different Views of the Structure are shown in the Figures 3.1 ~ 3.3.

Figure 3.1: 3D View of the Structure

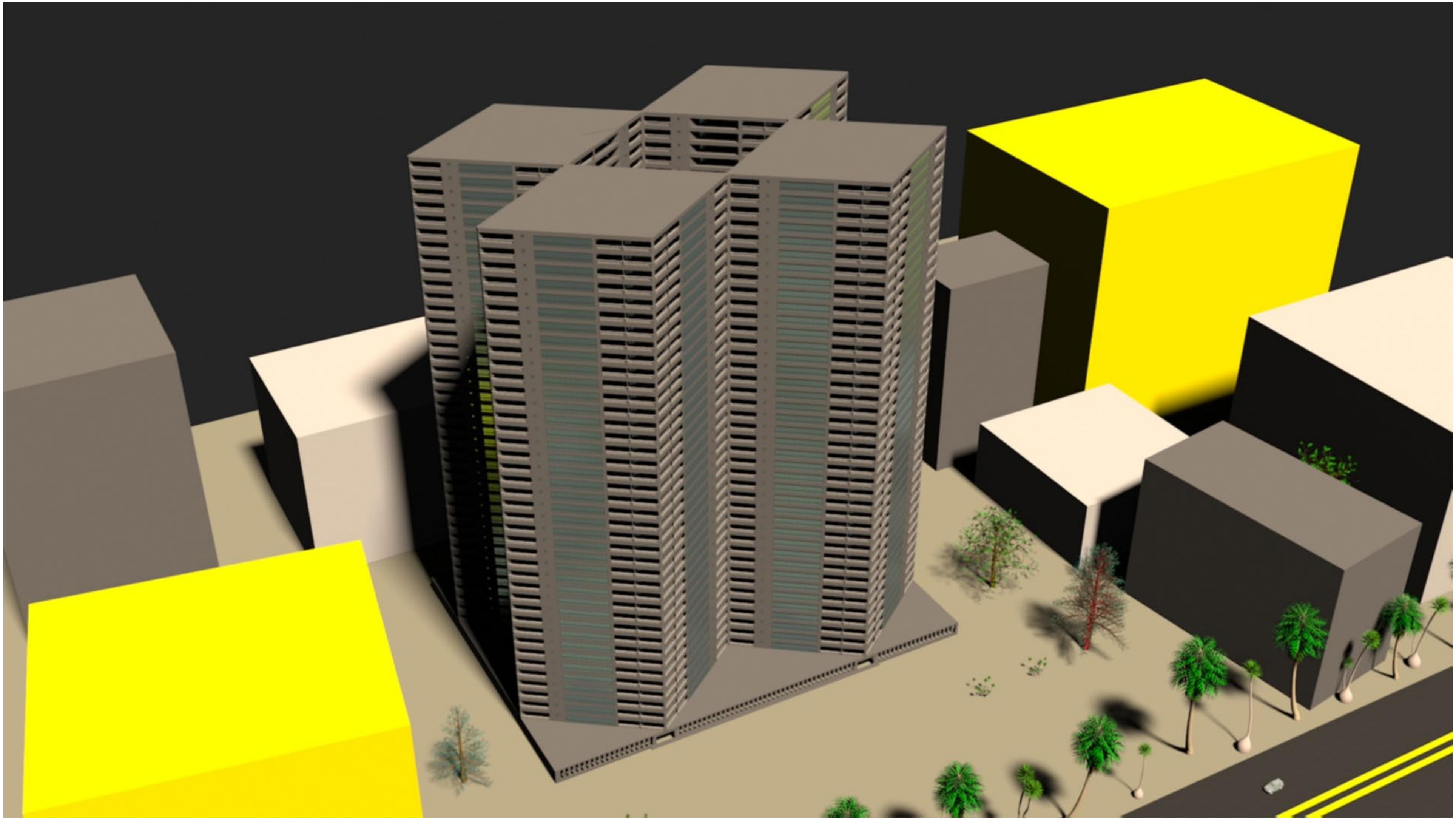


Figure 3.2: 3D View of the Structure from Top

3.2 DESCRIPTION OF THE DIFFERENT FLOORS

3.2.1 Garage (2 Basement + Ground Floor)

There are two basement floors in the building, which are used for garage. The 2nd basement is facilitated with 252 cars parking and 4 generator rooms. There are six ramp, four ramps for in and out to the main way and other two are connect with 2nd basement to 1st basement floor. The parking specification is same for all the floors as follows:

- Parking width is 8.0 *ft.*
- Parking length is 17.0 *ft.*
- Bay width 14.0 *ft.* for two-way direction, and 12.0 *ft.* for one-way direction.

<u>2nd Basement Floor:</u>

22.5*ft.* down from the road level and connected with eight simplex passenger elevators & four stairs. The entry &exit are connected by two interior ramps with 1st basement floor.

- The floor is on existing ground level.
- Area of the floor is 90,668.15 *square ft.*
- Parking capacity is 252 *nos.* passenger car and space for bike.
- There are four generator rooms.
- There are two ramps (interior)
  - Width of ramp : 15.0 *ft.*
  - Length of ramp : 37 *ft.*
  - Angle of ramp with horizontal : 18.92 *degrees*

The plan view of the 2nd basement floor with all facilities is shown in Figure 3.5.

<u>1st Basement Floor:</u>

10.5 *ft.* down from the road level and connected with eight simplex passenger elevators & four stairs. The main entry & exit are connected by two exterior ramps with road level.

- Height of the floor is 12 *ft.*
- Area of the floor is 90,668.15 *square ft.*
- Parking capacity is 234 *nos.* passenger car and space for bike.
- There are two-driver waiting room with toilets.
- There are two ramps (exterior).
  - Width of ramp : 15.0 *ft.*
  - Length of ramp : 32.38 *ft.*
  - Angle of ramp with horizontal : 18.92 *degrees*

The plan view of the 1st basement floor with all facilities is shown in Figure 3.6.

<u>Ground Floor:</u>

1.5 *ft.* high from the road level and entry & exit are separately for resident and cars with road level. The floor is connected with eight simplex passenger elevators & four stairs for the resident.

- Height of the floor is 12 *ft.*
- Area of the floor is 90,668.15 *square ft.*
- Parking capacity is 258 *nos.* passenger car and space for bike.
- There are two-driver waiting room with toilets and two security rooms.

The plan view of the ground floor with all facilities is shown in Figure 3.7.

3.2.2 Residential Floors (1st – 40th floor)

Connected with eight simplex passenger elevators & four stairs for the residential users. The building is divided as four parts, and each part has same configuration. Each part having area of 11,658.38 square ft. and each part contains four units.

- Height of the floor 10 *ft.*
- **Unit-A, E, I & M:** area of 2801.88 *square ft.* consists of 3 master beds with veranda and one bed, specious drawing & dining space, family living, kitchen, servant room and a common bath.
- **Unit-B, F, J & N:** area of 2775.90 *square ft.* consists of 3 master beds with veranda and one bed, specious drawing & dining space, family living, kitchen, servant room and a common bath.
- **Unit-C, G, K & O:** area of 2662.77 *square ft.* consists of 3 master beds with veranda and one bed, specious drawing & dining space, family living, kitchen, servant room and a common bath.
- **Unit-D, H, L & P:** area of 2679.30 *square ft.* consists of 3 master beds with veranda and one bed, specious drawing & dining space, family living, kitchen, servant room and a common bath.

The plan view of the ground floor with all facilities is shown in Figure 3.8.

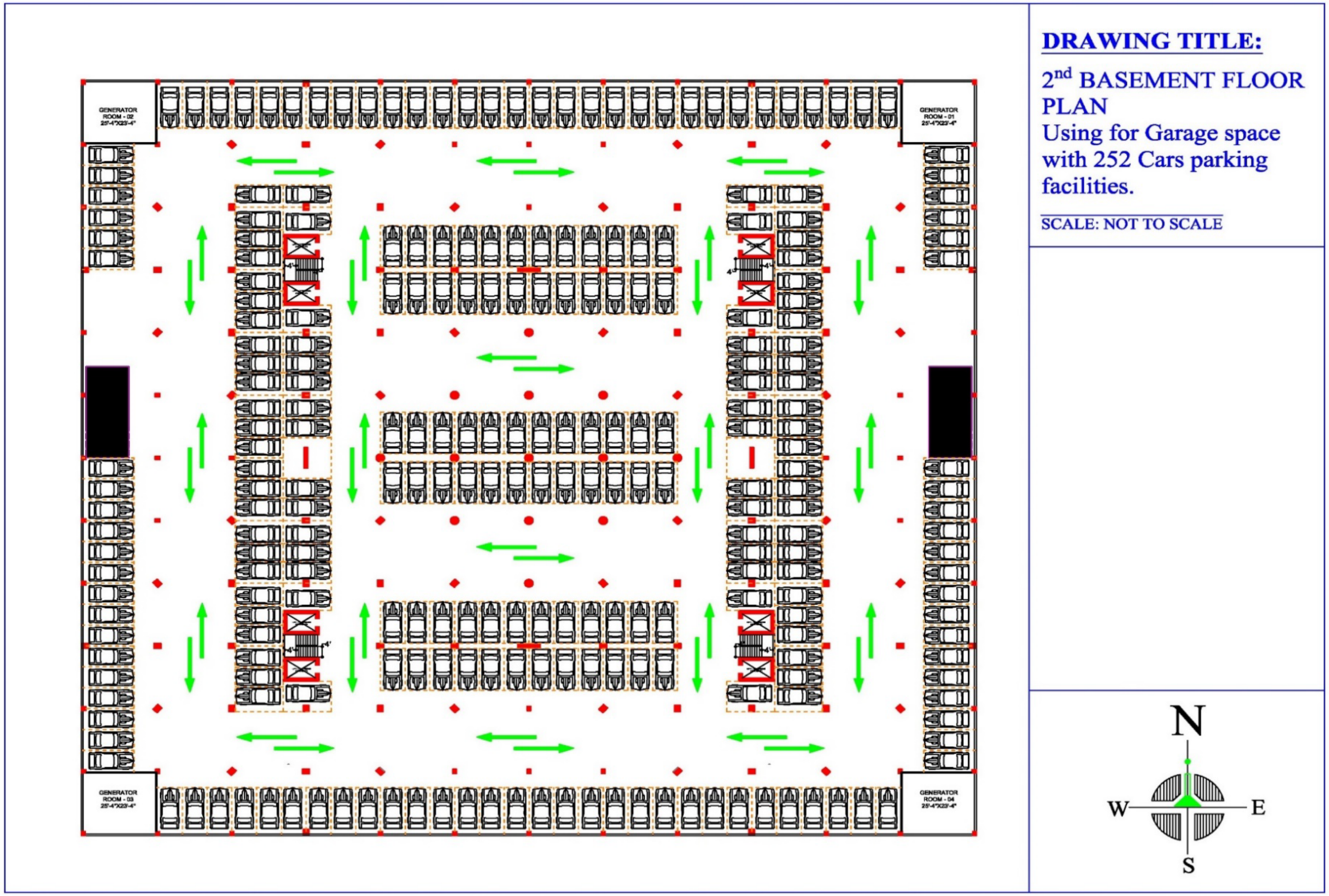


Figure 3.3: 2nd Basement Floor Plan (Garage)

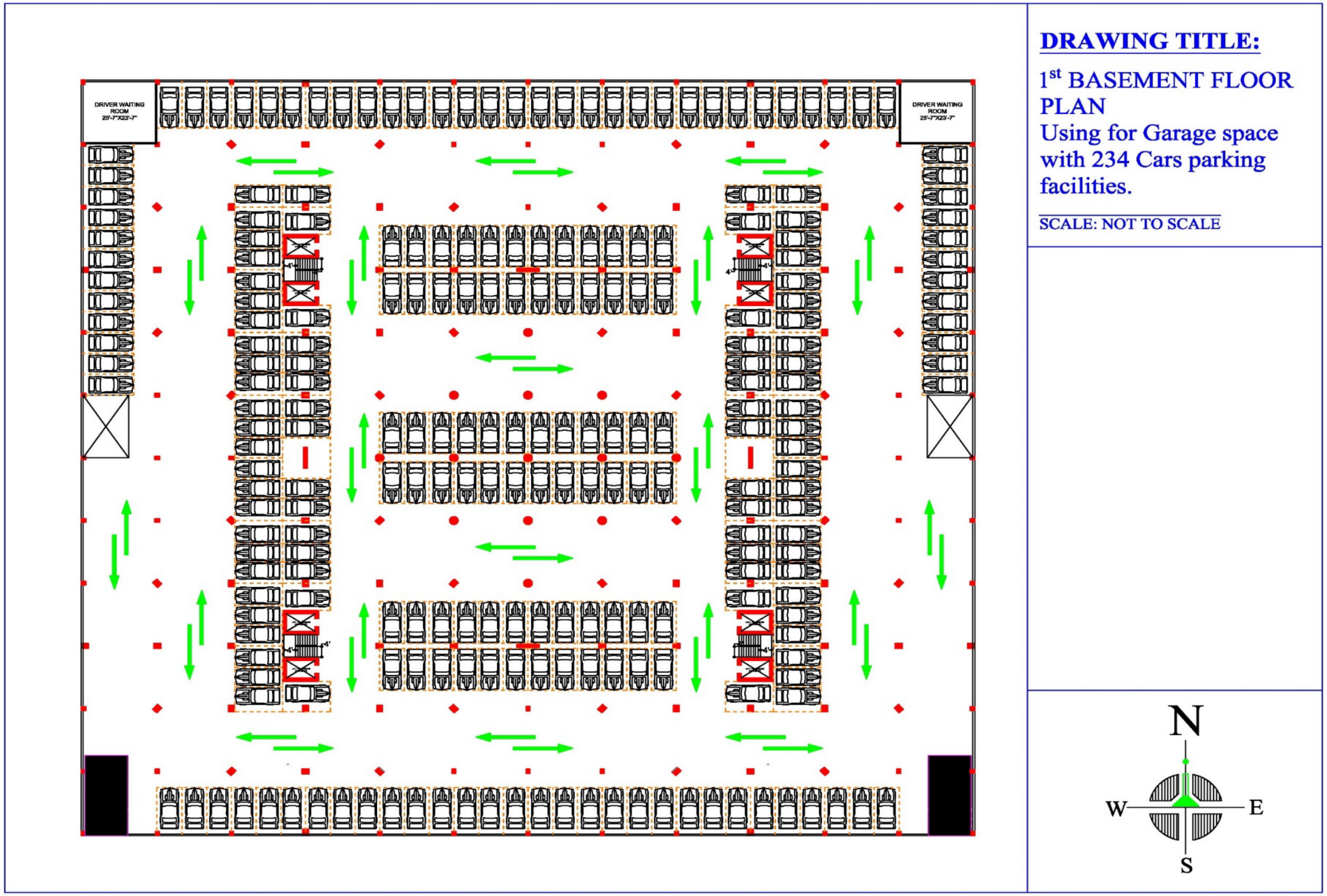


Figure 3.4: 1st Basement Floor Plan (Garage)

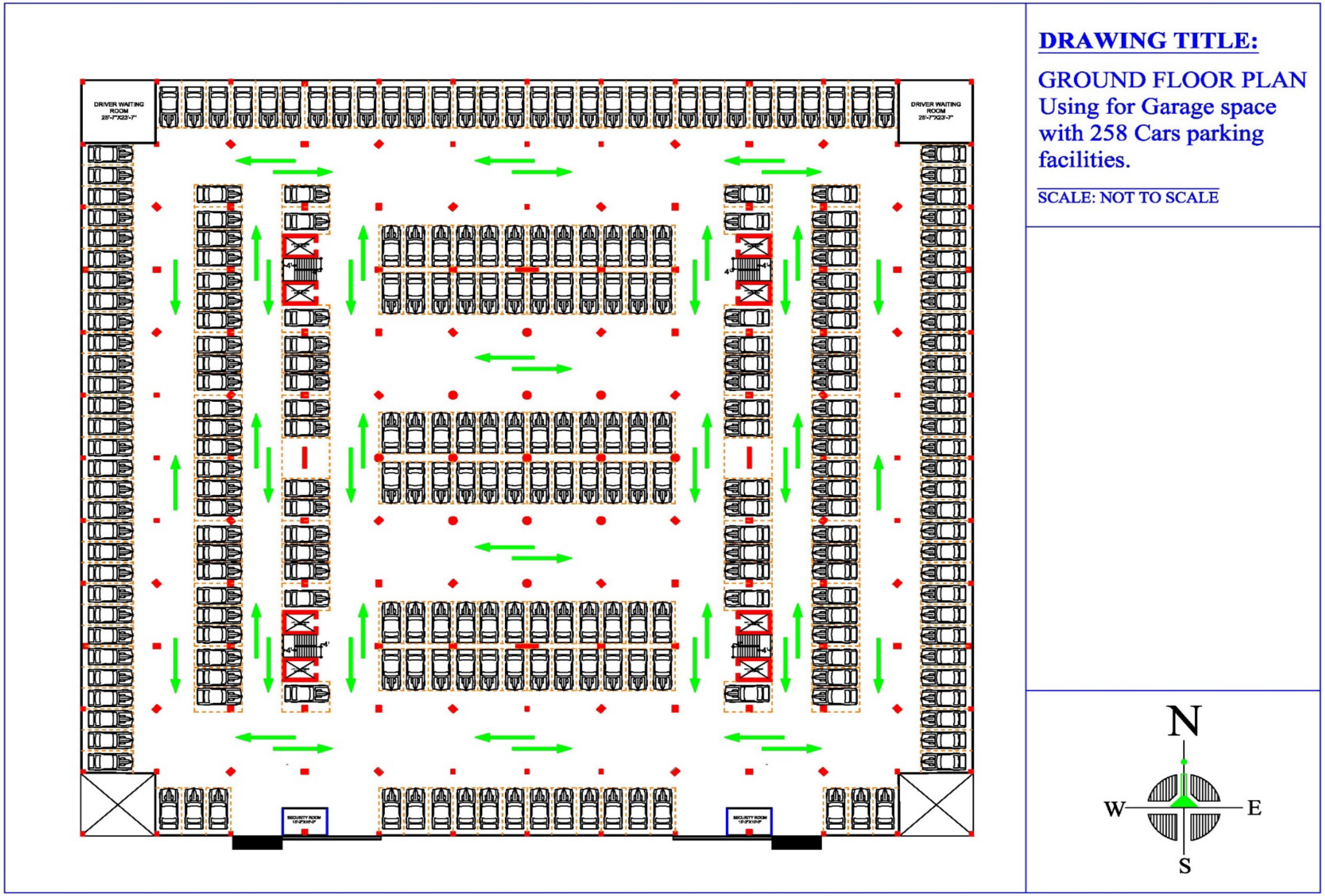


Figure 3.5: Ground Floor Plan (Garage)

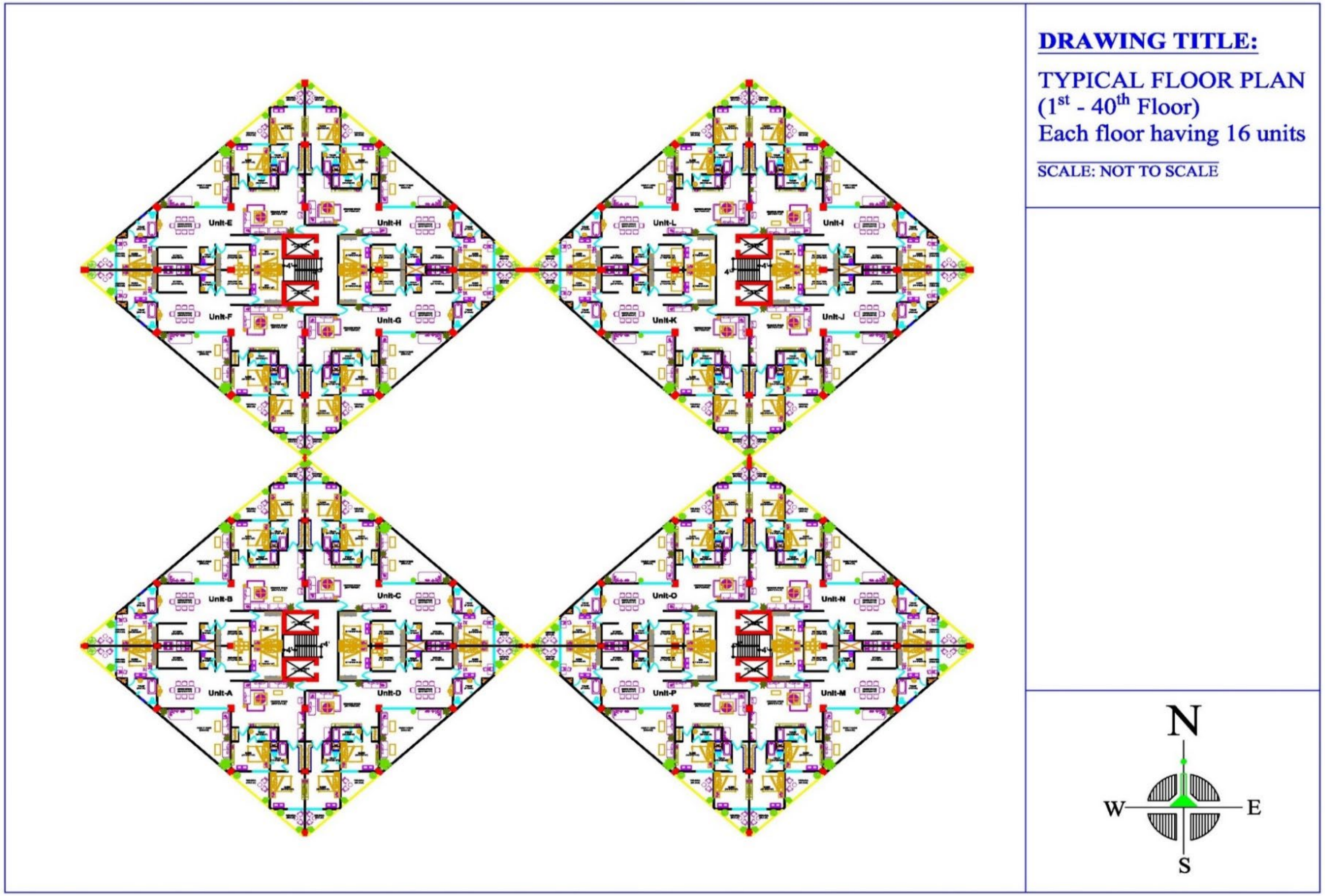


Figure 3.6: Typical Floor Plan (Residential 1st to 40th Floor)

Chapter 4

ANALYSIS OF THE STRUCTURE

CHAPTER FOUR

ANALYSIS OF THE STRUCTURE

4.1 CONSIDERATIONS FOR THE STUDY

Structure should be designed such that it can withstand each and every force that is likely to occur. It is of paramount importance that the structural form is sound. The architect achieves the structural configuration and the structural engineer proportions the member sizes. There are certain principles to be borne in mind. The whole study was carried out based on few considerations and specifications, which are summarized as below:

Items	Description
Design Code	• Bangladesh National Building Code (*BNBC*), 2017. • ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures. • ACI 318-19: Building Code Requirements for Structural Concrete.
Structural System	Dual Structural System
Floor System	Two-way Edge Supported Floor System
Building Components	• Column type = Tied (rectangular and circular) • Footing type = Pile Foundation (2 basements) • Thickness of all partition walls = 5-inch • Thickness of shear-wall=16-inch • Thickness of two-way edge supported slab = 7-inch • Thickness of overhead tank bottom slab = 8-inch • Thickness of overhead tank top slab = 5-inch • Thickness of overhead tank wall = 10-inch • Thickness of boundary wall = 8-inch • Ramp thickness = 21-inch and 24-inch • Angle of ramp = 18.92-degree • Thickness of waist slab = 6-inch • Trade 12-*inch* and Riser 6-inch

Material Properties	• Yield strength of reinforcing bars, f_y = 60,000 psi • Compressive strength Concrete, f_c' = 5,000 psi • Unit weight of concrete, w_c = 150 pcf • Unit weight of brick, w_b = 120 pcf • Unit weight of soil, γ_s = 120 pcf • Unit weight of water, γ_w = 62.5 pcf

4.2 LOADINGS AND ASSESMENTS

4.2.1 Calculation of Partition Wall Load (PW)

A. Residential Floor:

- Thickness of partition wall = 5" = 0.42 *ft.*
- Height of partition wall = 9'-4" = 9.33 *ft.*
- Total length of partition wall = 3,968 *ft.*
- Area of the floor = 45,995 *square ft.*

So, Partition Wall Load = $\frac{thickness\ of\ wall * height\ of\ wall * length\ of\ wall}{floor\ area} * w_b$

$= \frac{0.42*9.33*3968}{45995} * 120$

= 40.56 psf < 45 psf

≡ 45 psf

B. Exterior Beam Line Load (all floor except basement floors):

- Thickness of the wall = 5" = 0.42 ft.
- Height of the wall = 8.17 ft.

So, load from the wall = thickness of wall $*$ height of wall $*$ w_b

= 0.42*8.17*120 = 411.77 lb/ft. **= 0.412 kip/ft.**

C. Loads on Beam from Overhead Tank Wall:

- Thickness of the wall = 10" = 0.83 ft.
- Height of the wall = 7 ft.

So, load from the wall = thickness of wall $*$ height of wall $*$ w_c

= 0.83*7*150 = 871.5 lb/ft. **= 0.87 kip/ft.**

4.2.2 Floor Finish Load (FF)

Category	Loads (*psf.*)
Residential Floor	20.0
Stair (all)	25.0
Ramp & Basement Floor	15.0
Overhead Water Tank	10.0

4.2.3 Live Load (LL)

Category	Loads (*psf.*)
• Floor • Stair	40.0 100.0
Roof (Promenade Purpose)	60.0
Ramp & Basement Floor: • Live load • Passenger Car	 10.0 40.0
Overhead Water Tank: • Top and bottom slab • Water pressure ($\gamma_W H$)	 10.0 62.5∗6.5 = 406.25
Boundary Wall: • Earth Pressure on wall ($\gamma_s H$)	 120∗12 = 1440.0

4.2.4 Earthquake Load

Occupancy Category : II

Location : Dhaka

Earthquake Zone : Zone 2

Site Class : SC (medium dense sand/stiff clay)

Structural System : Dual System (IMRF)

Height of the building : 434 *ft.* (Base to Roof)

Input Data:

- Importance Factor, I = 1.0
- Seismic Zone Coefficient, Z = 0.20
- Soil Factor, S = 1.15
- Lower limit of the period of the constant spectral acceleration, T_B = 0.20
- Upper limit of the period of the constant spectral acceleration, T_C = 0.60

- Lower limit of the period of the constant spectral displacement, T_D = 2.0
- Response Reduction Factor, R = 6.5
- System Over Strength Factor, Ω_o = 2.5
- Deflection Amplification Factor, C_d = 5.0
- Viscous Damping Ratio, ξ= 5%
- β = 0.11
- C_t = 0.0466
- m = 0.75
- h_n = 446 *ft.* = 132.28 *m*

Damping Correction Factor, $\eta = \sqrt{10/(5+\xi)} \geq 0.55$

$= \sqrt{10/(5+5)} \geq 0.55$

$= 1.0 \geq 0.55$ (Okay)

Structure (building) Period, $T = C_t h_n^m$

$= 0.0488 * 132.28^{0.75}$

= 1.90 *seconds*

Hence, $T_C < T < T_D$

So, Normalized Acceleration Response Spectrum, $C_s = 2.5 S\eta \left(\frac{T_C}{T}\right)$

$= 2.5 * 1.15 * 1 * \left(\frac{0.60}{1.90}\right)$

= 0.9079

Design Spectral Acceleration,

$$S_a = \frac{2}{3}\frac{ZIC_s}{R} \geq 0.67\beta ZIS = \frac{2}{3} * \frac{0.20 * 1.0 * 0.9079}{6.5} \geq 0.67 * 0.11 * 0.20 * 1.0 * 1.15$$

= 0.0186 > 0.0170 (Okay).

<u>Design Parameter for Alternative Method of Base Shear Calculation (ETABS):</u>

- Spectral Response Acceleration, S_s = 0.50
- Spectral Response Acceleration, S_1 = 0.20
- Site Coefficient, F_a = 1.15
- Site Coefficient, F_v = 1.725
- Spectral Response Acceleration, S_{DS} = 0.3833
- Spectral Response Acceleration, S_{D1} = 0.23

4.2.5 Wind Load

Occupancy Category : II

Location : Dhaka

Height of the building : 415 *ft.* (Ground Surface to Roof)

Roughness Category : A

Exposure Category : A

Diaphragm Rigidity : Rigid

Input Data:

- Importance Factor, I = 1.0
- Basic Wind Speed, V = 65.7 *m/sec* (146.97 *mph*)
- Wind Directional Factor, K_d = 0.85
- Ground Elevation Factor, K_e = 1.0
- Topographic Factor, K_{zt} = 1.0
- Gust Factor, G or G_f = 0.85
- Height above ground level, z = 415 *ft.* = 126.49 *m*
- Velocity Pressure Exposure Coefficient, K_z = 1.485

So, the Velocity Pressure, $q_z = 0.000613K_zK_{zt}K_dV^2I$; kN/m^2, *V in m/sec*

$$= 0.000613 * 1.485 * 1.0 * 0.85 * 65.7^2 * 1.0$$

$$= 3.34\ kN/m^2$$

$$= 69.76\ psf$$

4.2.6 Load Cases

1. Dead Load
2. Live Load
3. Earthquake-X (+Eccentricity)
4. Earthquake-X (−Eccentricity)
5. Earthquake-Y (+Eccentricity)
6. Earthquake-Y (−Eccentricity)
7. Wind-X
8. Wind-Y
9. Response Spectrum (For Dynamic Analysis)

4.2.7 Load Combinations

Applying load combinations according to the *BNBC – 2017* code

1. 1.4D
2. 1.2D + 1.6L
3. 1.2D + 1.6W_X + 1.0L
4. 1.2D – 1.6W_X + 1.0L
5. 1.2D + 1.6W_Y + 1.0L
6. 1.2D – 1.6W_Y + 1.0L
7. 1.28D + 1.0E_X + 1.0L
8. 1.28D – 1.0E_X + 1.0L
9. 1.28D + 1.0E_Y + 1.0L
10. 1.28D – 1.0E_Y + 1.0L
11. 0.9D + 1.6W_X
12. 0.9D – 1.6W_X
13. 0.9D + 1.6W_Y
14. 0.9D – 1.6W_Y
15. 0.82D + 1.0E_X
16. 0.82D – 1.0E_X
17. 0.82D + 1.0E_Y
18. 0.82D – 1.0E_Y

4.3 STRUCTURAL ARRANGEMENT PLAN

Beam, Column, and Shear Wall layout of the structure are shown in the Figures 4.1 to 4.7.

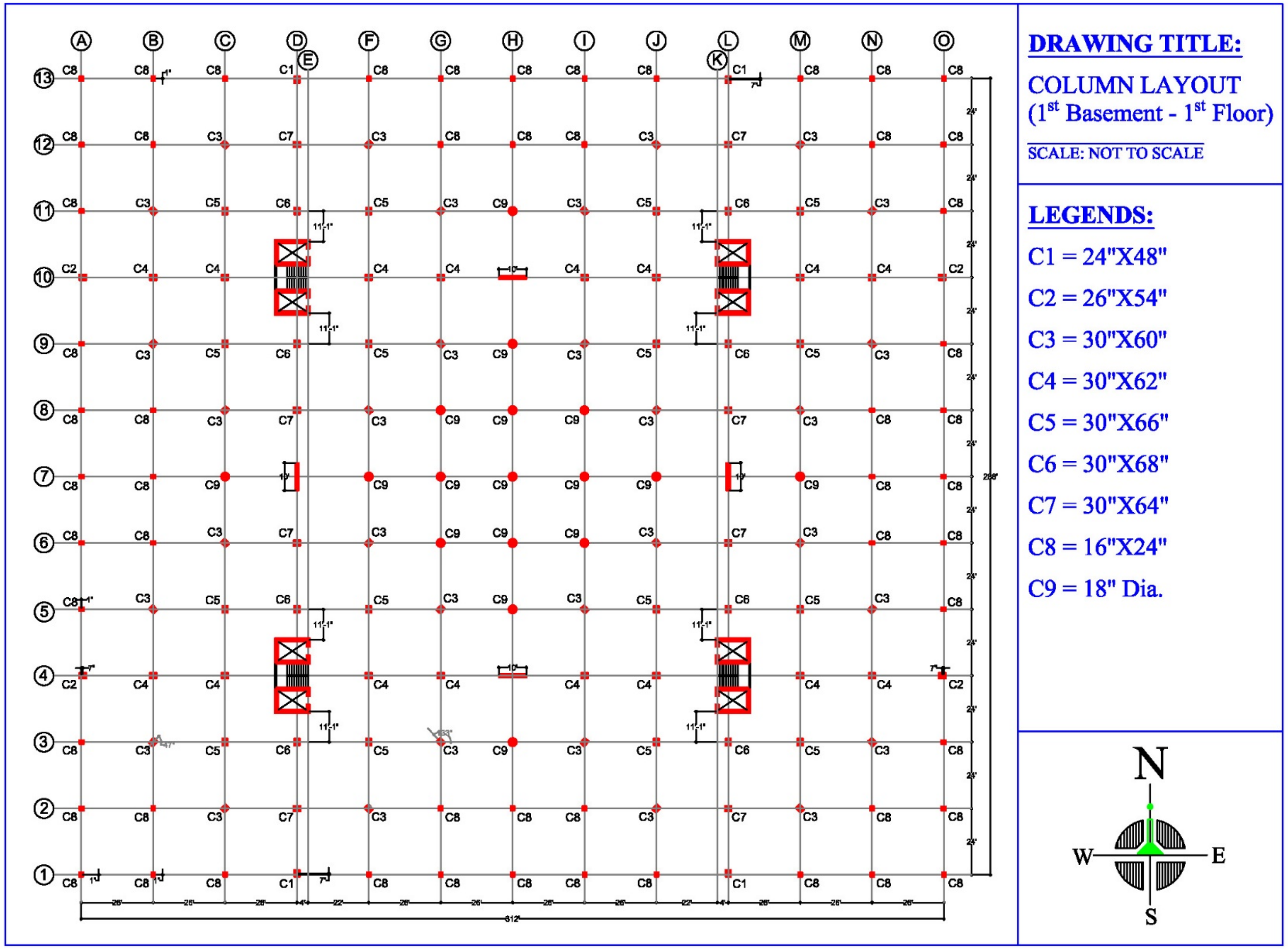


Figure 4.1: Column Layout of 1st Basement to 1st Floor

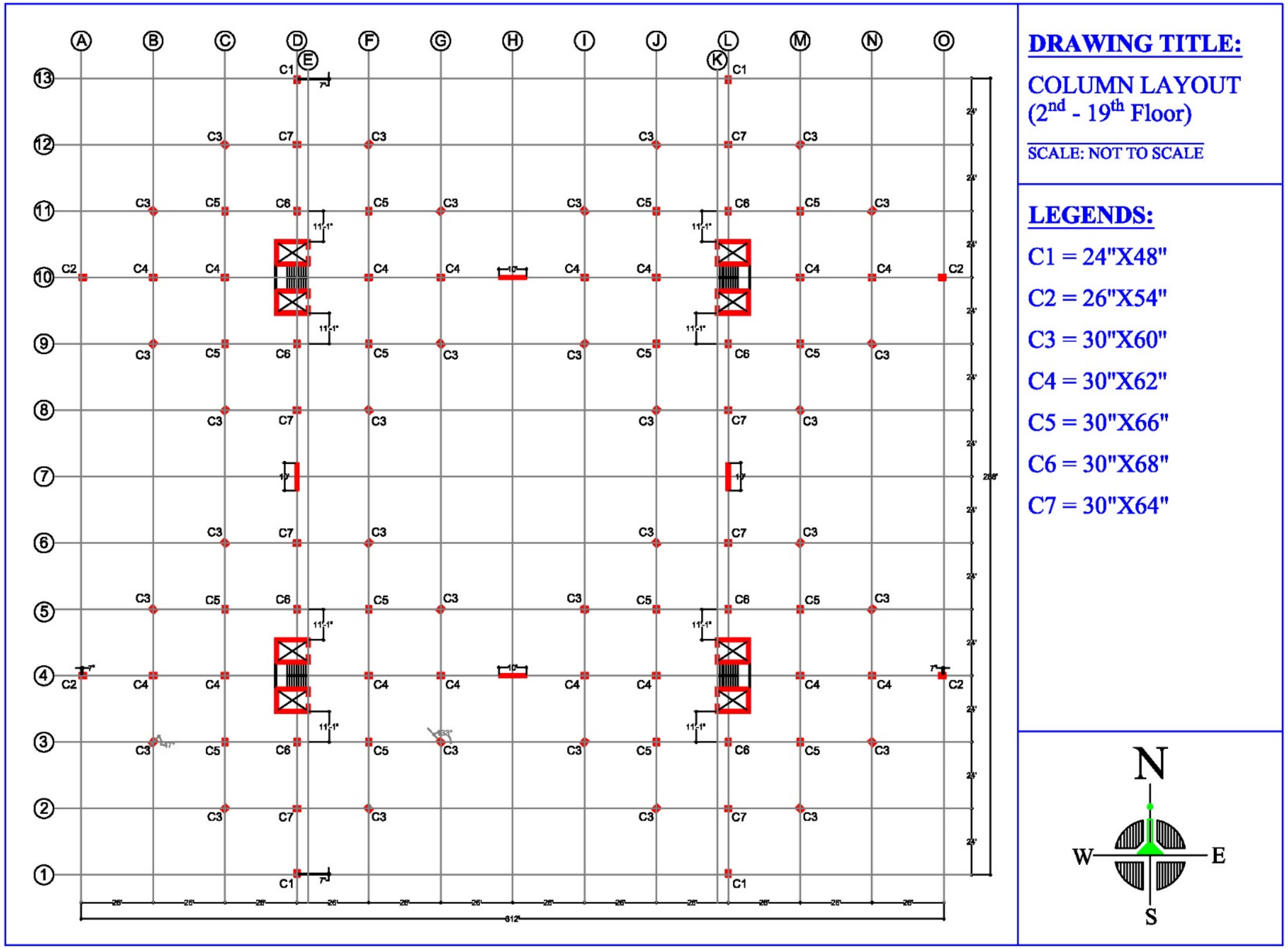


Figure 4.2: Column Layout of 2nd Floor to 19th Floor

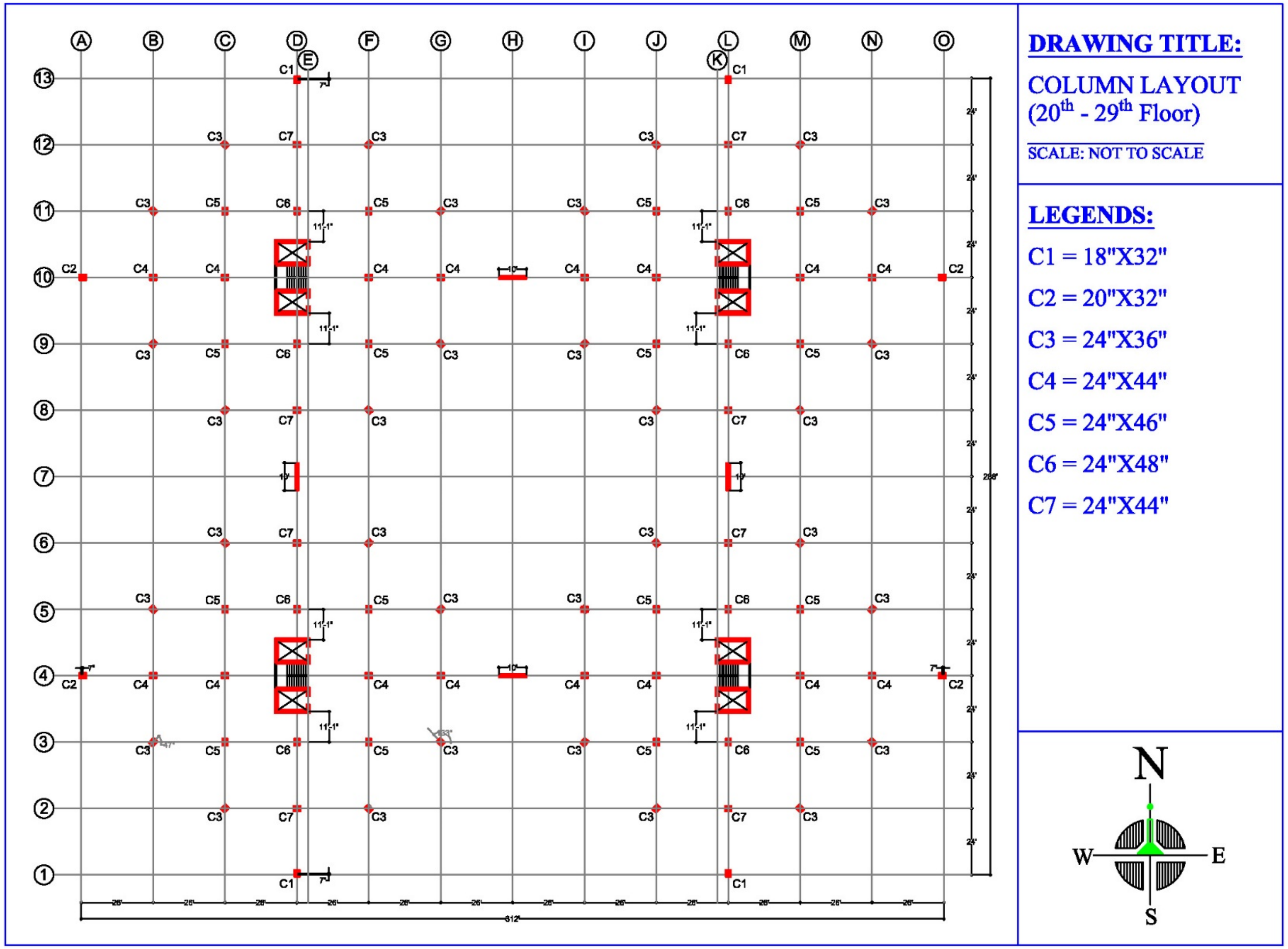


Figure 4.3: Column Layout of 20th Floor to 29th Floor

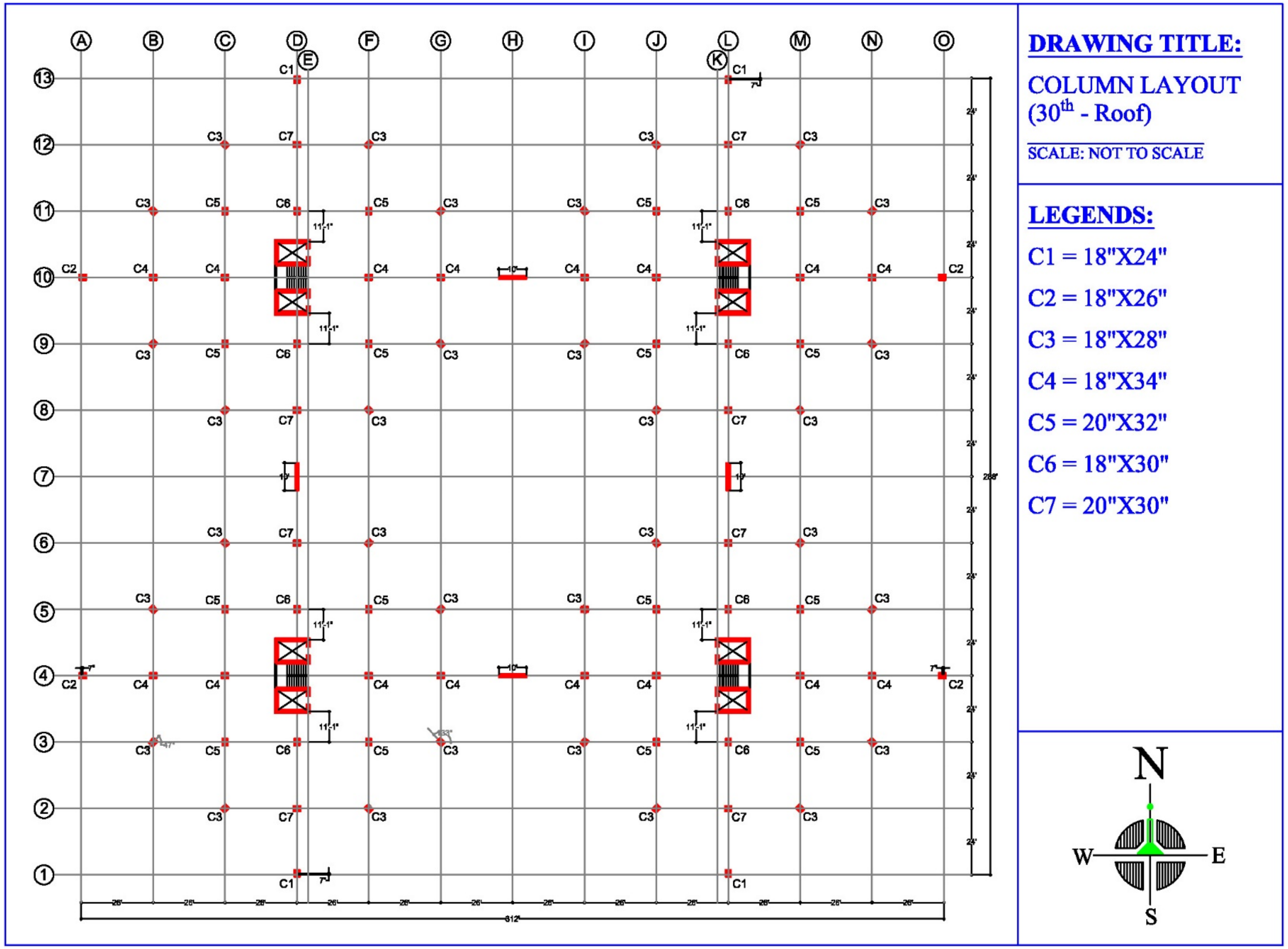


Figure 4.4: Column Layout of 30th Floor to Roof

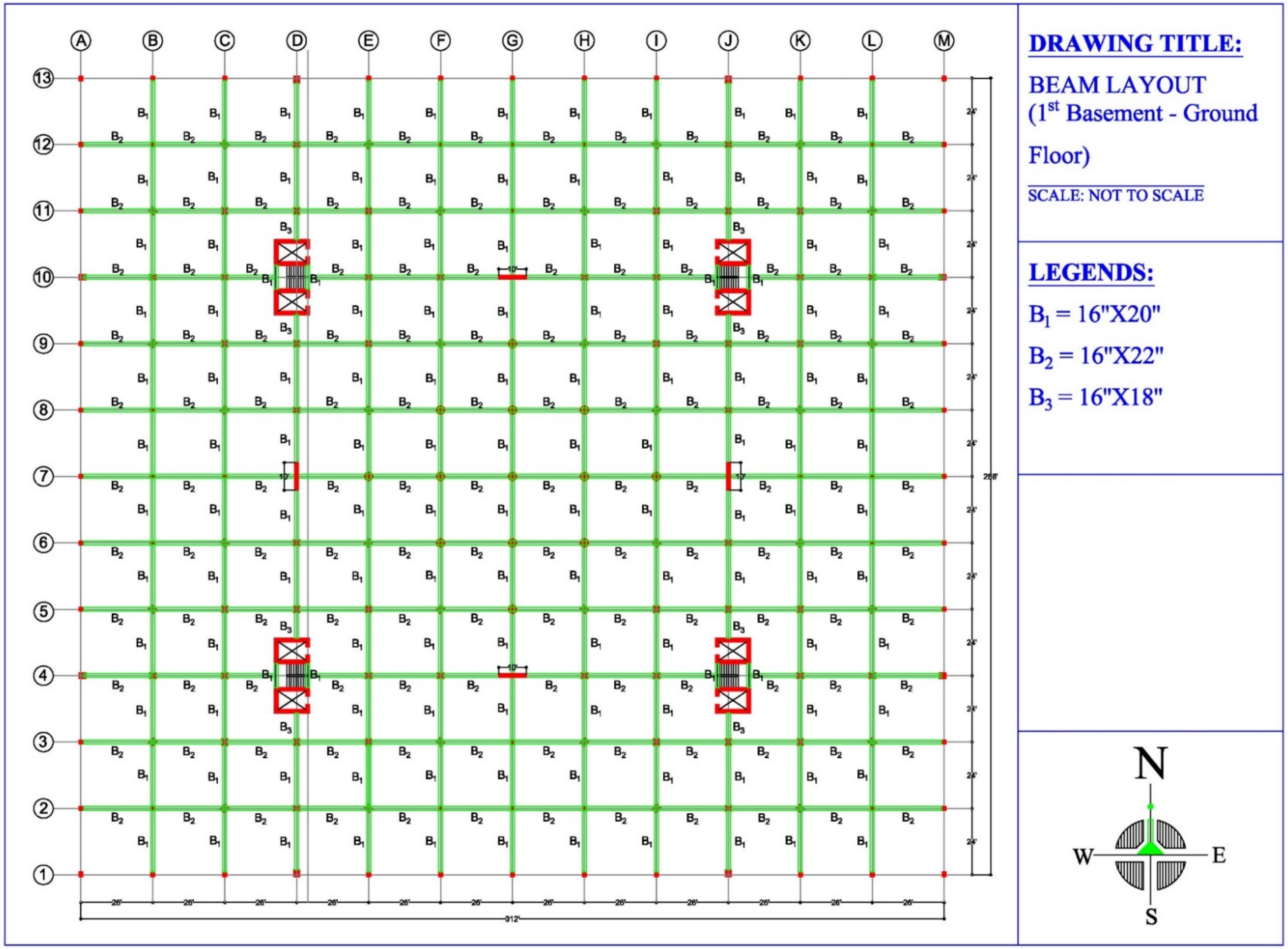


Figure 4.5: Beam Layout of 1st Basement Floor to Ground Floor

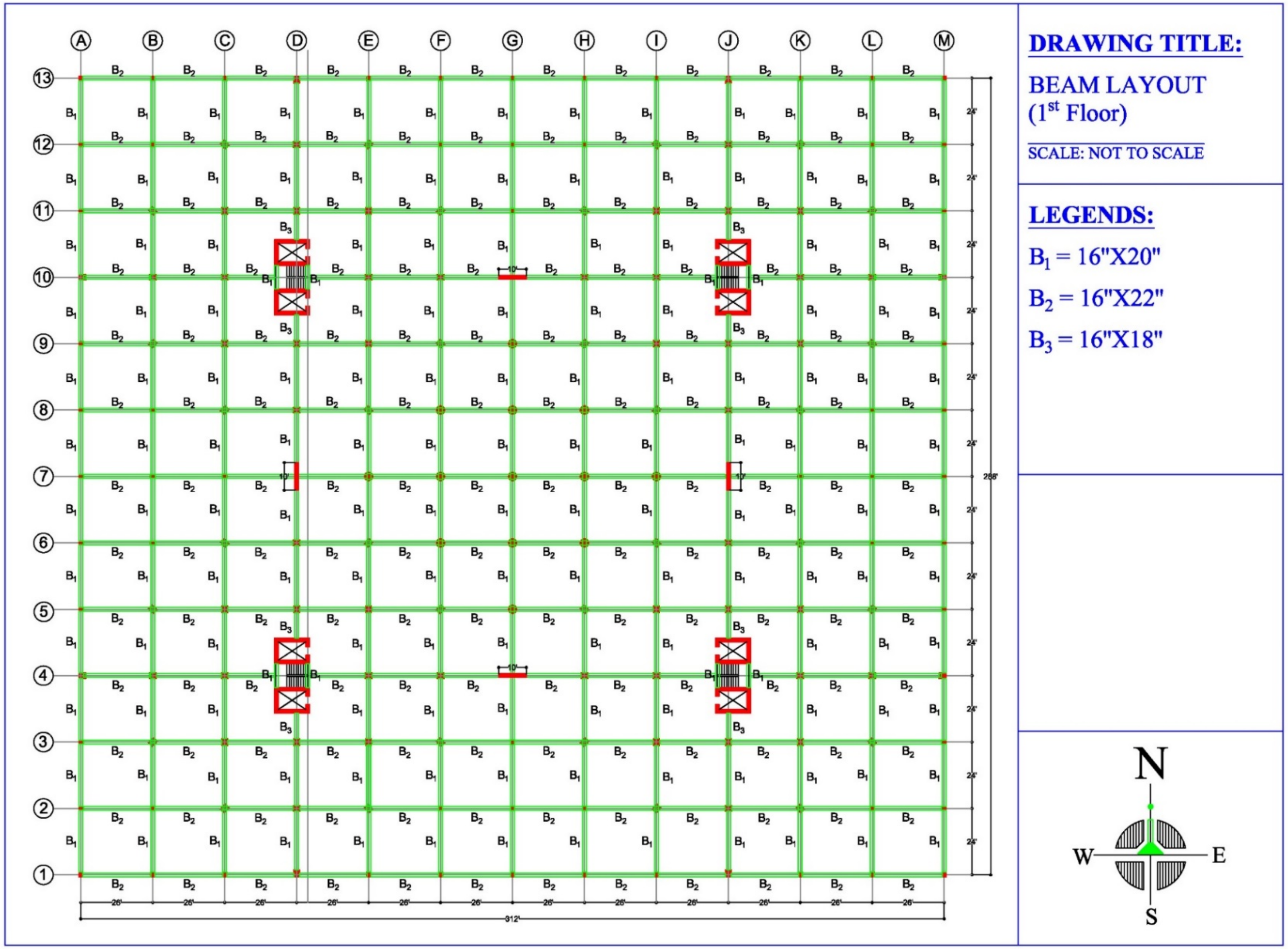


Figure 4.6: Beam Layout of 1st Floor

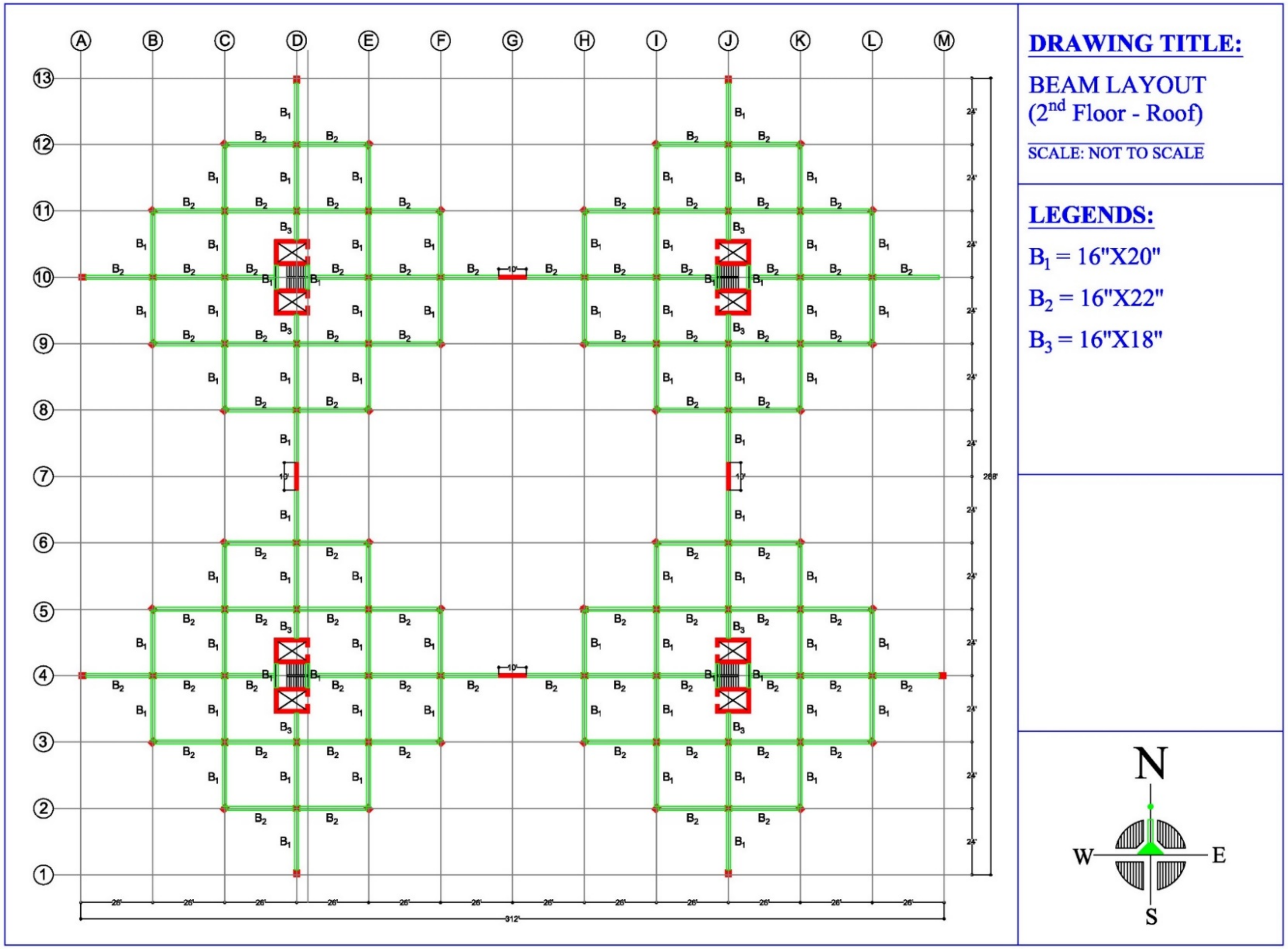


Figure 4.7: Beam Layout of 2nd Floor to Roof

4.4 DIAPHRAGM

4.4.1 Diaphragm Analysis under Earthquake Load

- ***Rigid Diaphragm Condition:*** Diaphragms of concrete slabs or concrete-filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.
- ***Flexible Diaphragm Condition:*** Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:
  a. In structures where the vertical elements are steel braced frames; steel and concrete composite braced frames; or concrete, masonry, steel, or steel and concrete composite shear walls.
  b. In one- and two-family dwellings.
  c. In structures of light-frame where all of the following conditions are met:
     - Topping of concrete/similar materials is not placed over wood structural panel diaphragms except for non-structural topping no greater than 1.5 inch thick.
     - Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift.

4.4.2 Diaphragm Analysis under Wind Load

Roof, floor, or other membrane or bracing system acting to transfer lateral forces to the vertical main wind force resisting system. For analysis under wind loads, diaphragms constructed of untopped steel decks, concrete-filled steel decks, and concrete slabs, each having a span-to-depth ratio of 2 or less, shall be permitted to be idealized as ***rigid***. Diaphragm constructed of wood structural panels are permitted to be idealized as ***flexible***.

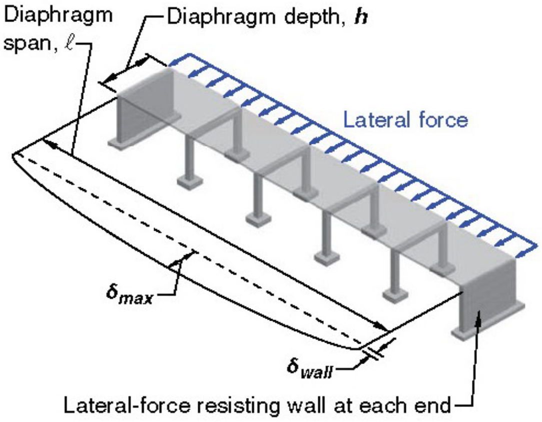


Figure 4.8: Span and Depth of Diaphragm

Now in this study model, all the slab panels having span-to-depth ratio is less than two. Where the largest panel length and depth is respectively 26 ft. and 24 ft.

Span-to-depth ratio of the panel = 26/24 = 1.08

- For earthquake consideration, span-to-depth ratio = 1.08 < 3.0…Rigid diaphragm
- For wind load consideration, span-to-depth ratio = 1.08 < 2.0…..Rigid diaphragm

4.5 MASS SOURCE

Mass values are calculated for structural elements according to volume and material density. Mass is then automatically concentrated at joint locations. During dynamic analysis, mass translation along each of the three joint displacement degrees-of-freedom (DOF) correlates with Acceleration to create inertial forces. Additional concentrated mass may be assigned to joint locations. Further, mass moment of inertia may be applied to each of the three joint rotational DOF to account for rotational inertia. Mass is uncoupled between different joints, and between DOF at a joint location.

In this study, mass source is defined as from self-weight and additional masses, includes lateral and vertical mass, and is not lumped at stories. The snapshot as shown in the figure describe the Mass Source input in ETABS of our model.

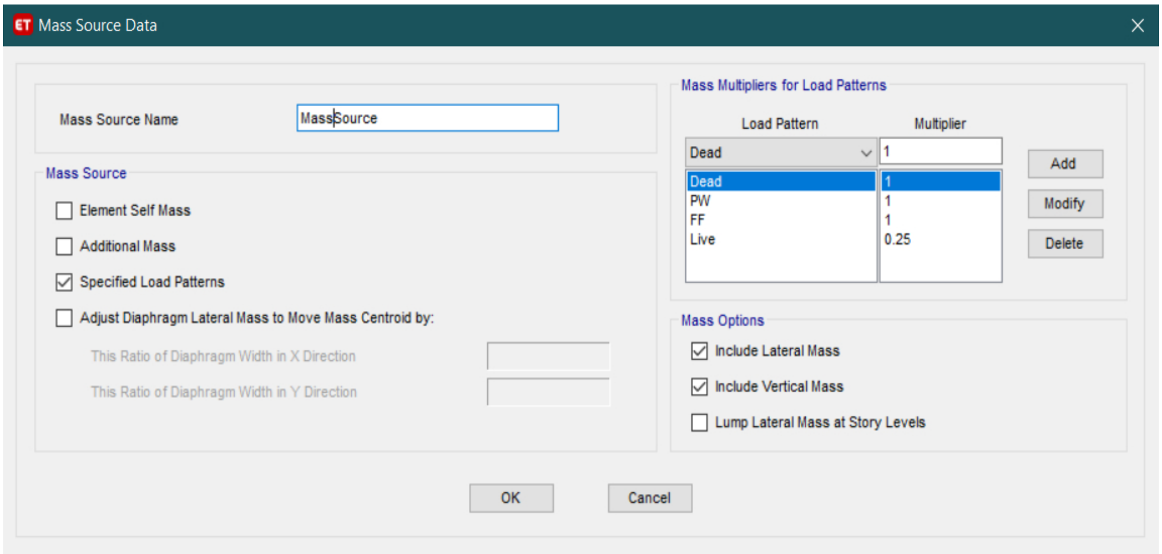


Figure 4.9: Mass Source

4.6 DEFINE DYNAMIC CONSIDERATION

Dynamic analysis may be carried out through the (i) Response Spectrum and (ii) Time History analysis methods. In our study we consider the Response Spectrum method.

4.6.1 Define Response Spectrum Function

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

The design earthquake ground motion in this case is represented by response spectrum function according to BNBC 2017 (ASCE 7-16 in ETABS).

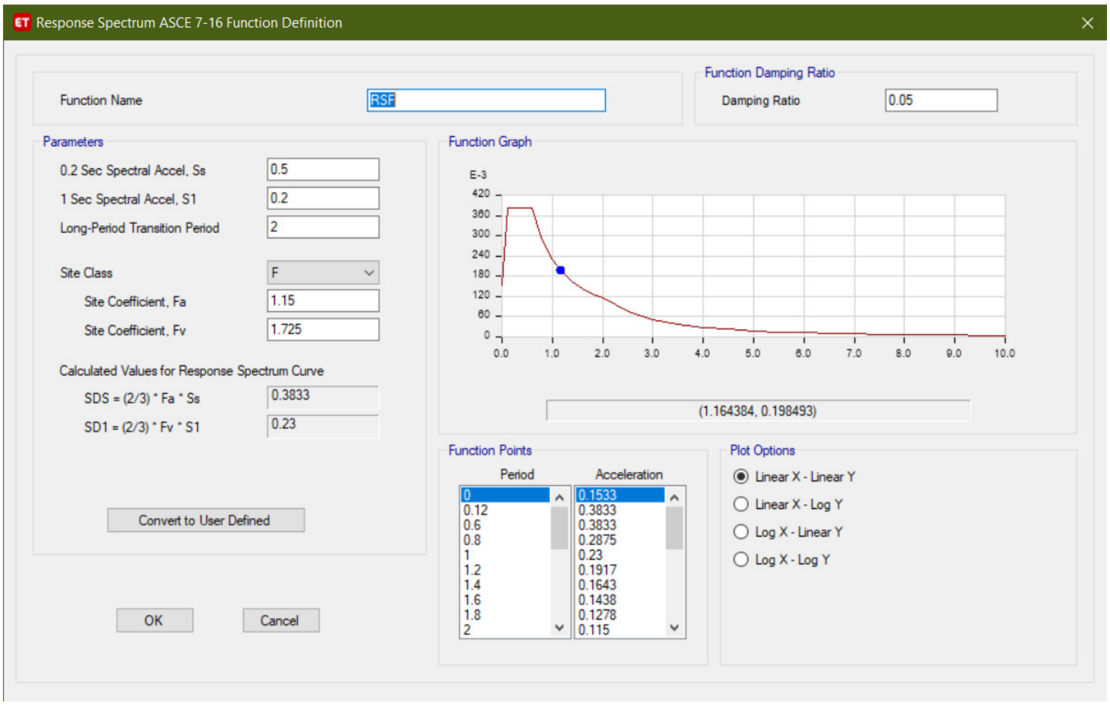


Figure 4.10: Response Spectrum Function

4.6.2 Define P-Delta

P-Delta effect, which is associated with local deformation relative to the chord between member ends. P-Delta Options and using either of the following two methods:

1. ***Non-iterative Based on Mass***, in which load is automatically computed from the mass at each level. This is an approximate method which does not require an iterative solution, providing for faster computation. P-Delta is considered by treating the structure as a simplified stick model, a process which is most effective with a single rigid diaphragm at each level. Local buckling is not captured as effectively.
2. ***Iterative Based on Load Cases***, in which load is computed from a specified combination of static load cases, then known as the P-Delta load combination. This is an iterative method which considers P-Delta on an element-by-element basis. Local buckling is captured more effectively. An example application may be when load includes the dead load case and a fraction of a live load case.

In this study, 'Non-iterative Based on Mass' has been chosen. The benefit of this non-iterative method is that P-δ may be considered in load cases which do not specify the gravity loads.

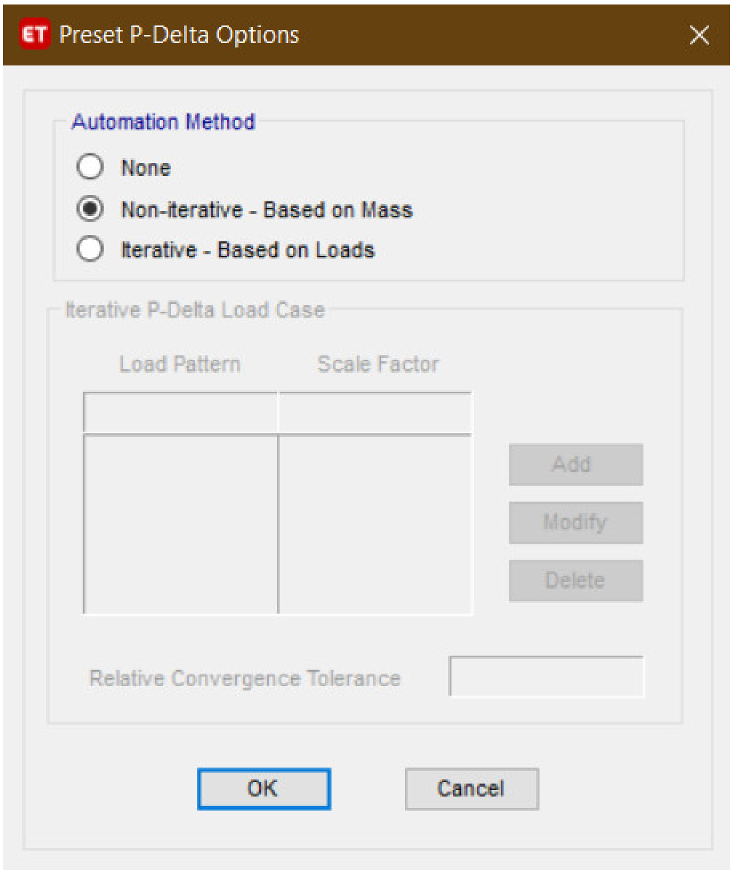


Figure 4.11: P-Delta Options

4.6.3 Define Modal Cases

The analysis shall be conducted using the masses and elastic stiffnesses of the seismic-force-resisting system to determine the natural modes of vibration for the structure including the period of each mode, the modal shape vector ϕ, the modal participation factor P and modal mass M. Modal cases should be carried out by (i) Eigen vector or (ii) Ritz vector.

Eigen Vectors:

Eigen modes are most suitable for determining response from horizontal ground acceleration, though a missing-mass (residual-mass) mode may need to be included to account for missing high-frequency effects. Mass participation is a common measure for determining whether or not there are enough modes, though it does not provide information about localized response.

Eigen analysis is useful for checking behavior and locating problems within the model. Another benefit is that natural frequencies indicate when resonance should be expected under different loading conditions. Users may control the convergence tolerance. Orthogonality is strictly maintained to within the accuracy of the machine (15 decimal digits). Sturm sequence checks are performed and reported to avoid missing Eigen vectors when using shifts. Internal accuracy checks are performed and used to automatically control the solution. Ill-conditioned systems are detected and reported, then still produce Eigen vectors which may be used to trace the source of the modeling problem.

Ritz Vectors:

Load-dependent Ritz vectors are most suitable for analyses involving vertical ground acceleration, localized machine vibration, and the nonlinear FNA method. Ritz vectors are also efficient and widely used for dynamic analyses involving horizontal ground motion. Their benefit here is that, for the same number of modes, Ritz vectors provide a better participation factor, which enables the analysis to run faster, with the same level of accuracy.

Further, missing-mass modes are automatically included, there is no need to determine whether or not there are enough modes, and when determining convergence of localized response with respect to the number of modes, Ritz vectors converge much faster and more uniformly than do Eigen vectors. Ritz vectors are not subject to convergence questions, though strict orthogonality of vectors is maintained, similar to Eigen vectors.

In this study, the Eigen vectors and starting with 12 number of maximum modes have been taken, and allow the Auto frequency Shifting. Further the maximum number of modes should be increased to getting Modal Participating Mass Ratio at least 90 percent.

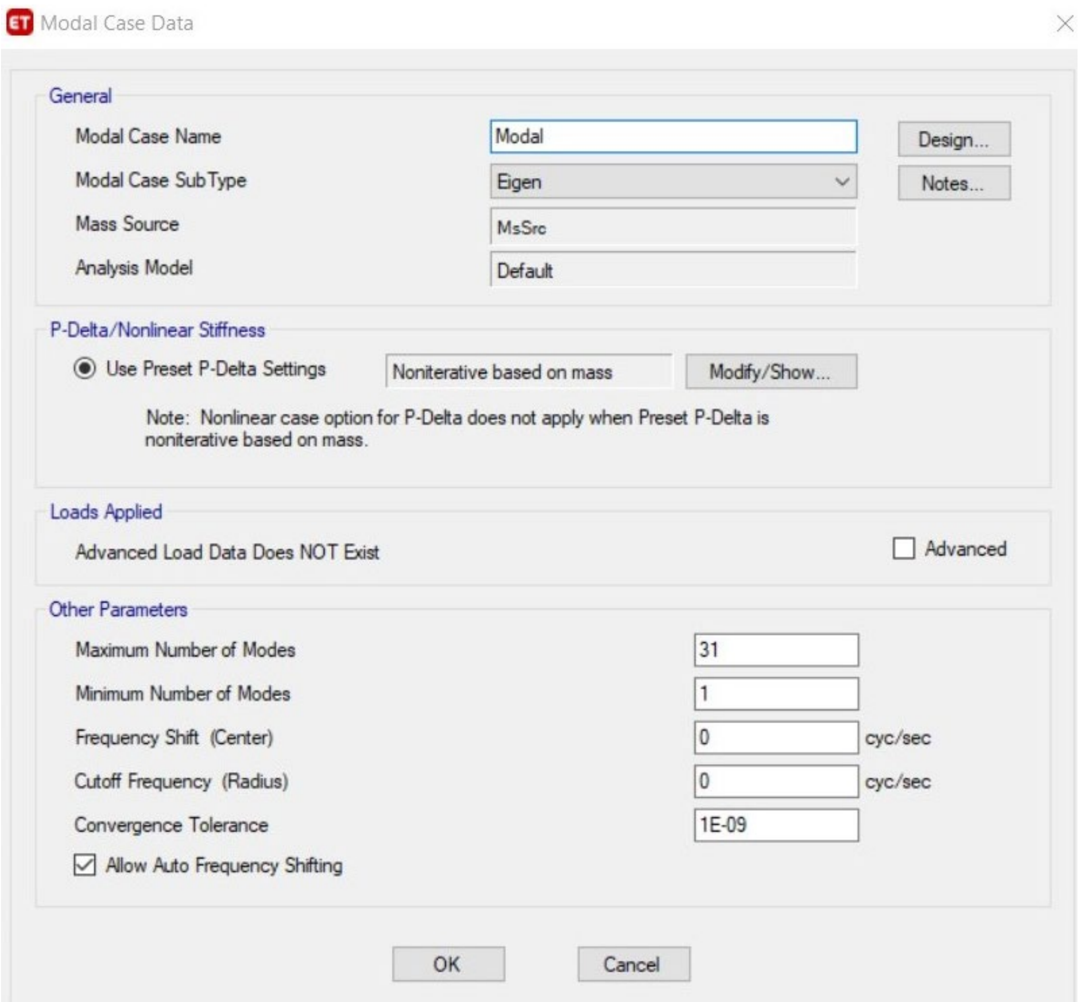


Figure 4.12: Modal Case Data

4.6.4 Define Response Spectrum Load Case

The loads applied in the response spectrum load case as acceleration U1 & U2 for two orthogonal directions with response spectrum function as defined before. The scale factor is determined by the equation ($I*g/R$), but will find the actual value after 1st analysis trial. Where, 'I' is the occupancy importance factor, 'g' is the acceleration due to gravity, and 'R' is the response modification factor.

The combination shall be carried out by taking the square root of the sum of the squares (SRSS) of each of the modal values or by the complete quadratic combination (CQC) technique. The complete quadratic combination shall be used where closely spaced periods in the translational and torsional modes result in cross-correlation of the modes.

To set the damping value 5 percent for all modes, and diaphragm eccentricity is also set to be 5 percent.

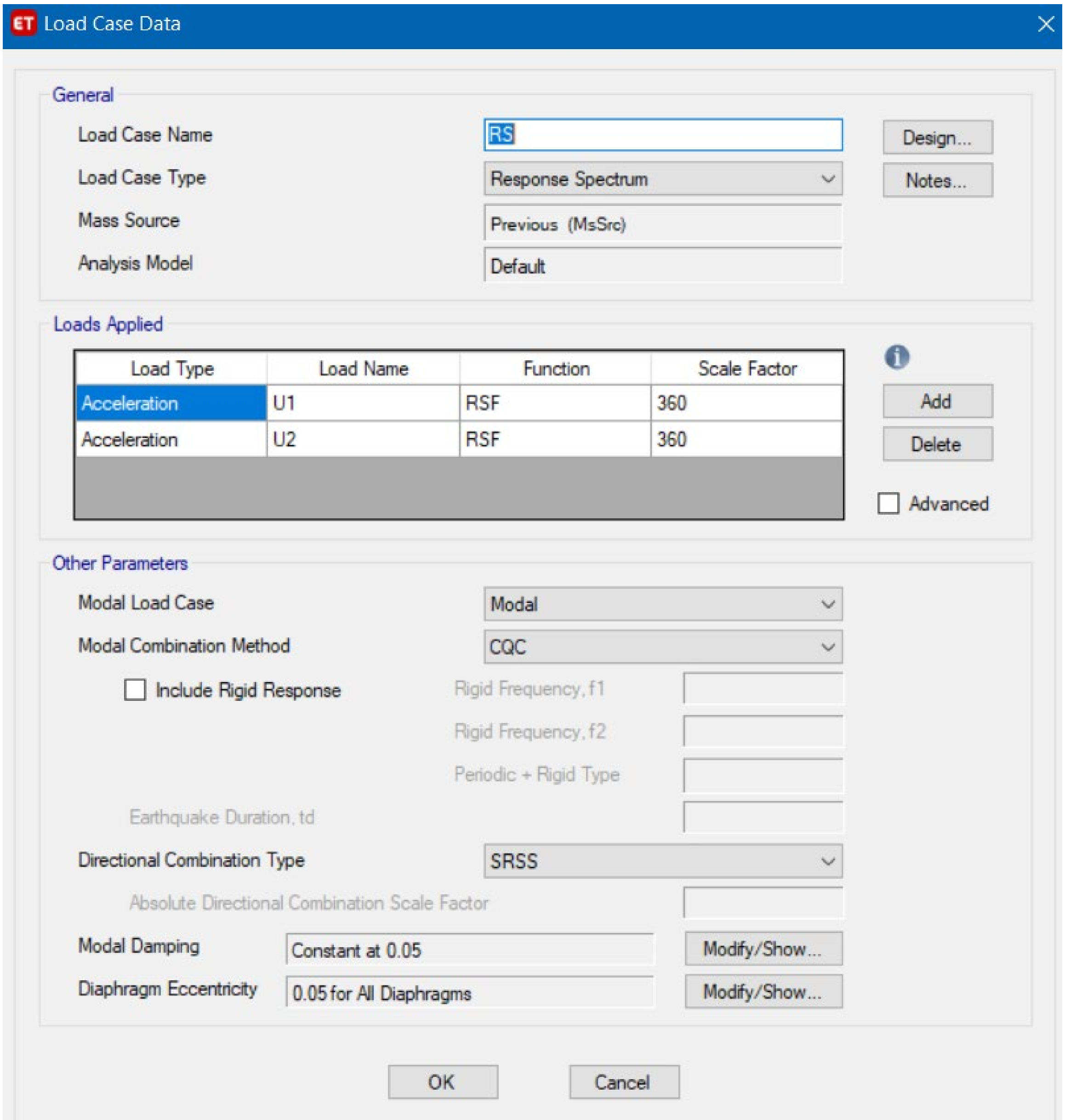


Figure 4.13: Response Spectrum Load Case

4.7 CHECK THE MODEL UNDER DYNAMIC ANALYSIS

4.7.1 Check for Stability

For stability, the number of negative stiffness should be zero and the number of negative eigenvalues should also be zero.

The snapshot as shown in the figure 4.14 has taken from ETABS analysis run log of the study model. It is observed that the negative stiffness and number of negative eigenvalues is zero. So, the model is stable.

```
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BASIC STABILITY CHECK FOR LINEAR LOAD CASES:
   NUMBER OF NEGATIVE STIFFNESS EIGENVALUES SHOULD BE ZERO FOR STABILITY.
   (NOTE: FURTHER CHECKS SHOULD BE CONSIDERED AS DEEMED NECESSARY,
    SUCH AS REVIEWING EIGEN MODES FOR MECHANISMS AND RIGID-BODY MOTION)

   NUMBER OF NEGATIVE EIGENVALUES         =           0,  OK.
--------------------------------
```

Figure 4.14: Snapshot from ETABS (Dynamic Analysis)

4.7.2 Check for Modal Participating Mass Ratio

The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions. After 34 number of modes, the study gives -

- The Modal Participating Mass Ratio at X-direction, SumUX = **96.28%** > 90%....Ok
- The Modal Participating Mass Ratio at Y-direction, SumUY = **96.46%** > 90%.....Ok
- The Modal Participating Mass Ratio at X-direction, SumRX = **91.54%** > 90%....Ok
- The Modal Participating Mass Ratio at Y-direction, SumRY = **91.11%** > 90%.....Ok

4.7.3 Check for Base Shear

The dynamic base shear (V_{rs}) must be greater than 85% of static base shear (V) in each direction. Where the dynamic base shear (V_{rs}) is less than 85% of static base shear (V), rescaling factor by $\left[\frac{I*g}{R} * 0.85 * \frac{V}{V_{rs}}\right]$ in response spectrum load case. Check it again.

The study model has –

- Dynamic base shear at X-direction = 1667.347 kips
- Dynamic base shear at Y-direction = 1697.857 kips
- Static base shear at X-direction = 10108.964 kips
- Static base shear at Y-direction = 10108.317 kips
- The ratio at X-direction = V_{rs}/V = 1667.347/9842.594 = 0.165 < 0.85…Not Ok
- The ratio at Y-direction = V_{rs}/V = 1697.857/9842.599 = 0.168 < 0.85…..Not Ok

Hence, the dynamic base shear is less than 85% of the static base shear in each direction.

So, the rescaling factor in response spectrum load case

$= \frac{I*g}{R} * \left[0.85 * \frac{V}{V_{rs}}\right] = \frac{1.0*386.4}{6.5} * \left[0.85 * \frac{10108.964}{1667.347}\right] = 306.353 \equiv \mathbf{310.0}$

Re-checking-

- Dynamic base shear at X-direction = 9326.079 kips
- Dynamic base shear at Y-direction = 9395.161 kips
- Static base shear at X-direction = 10108.964 kips
- Static base shear at Y-direction = 10108.317 kips
- The ratio at X-direction = V_{rs}/V = 9326.079/10108.964 = **0.92** > 0.85…Ok
- The ratio at Y-direction = V_{rs}/V = 9395.161/10108.317 = **0.93** > 0.85…Ok

4.7.4 Check for Torsional Irregularity

a. Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure is more than 1.2 times the average of the storey drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid.

b. Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.

- Irregularity occurs when $\frac{\Delta_{max}}{\Delta_{avg}} > 1.2$
- Extreme Irregularity occurs when $\frac{\Delta_{max}}{\Delta_{avg}} > 1.4$

- **In X-direction (Figure 4.15),**

  Δ_{max} = 61.62 inch, and $\Delta_{avg} = \frac{61.62+55.60}{2}$ = 58.61 inch

  Hence, $\frac{\Delta_{max}}{\Delta_{avg}} = \frac{61.62}{58.61}$ = 1.05 < 1.2……Ok.

  So, no torsional irregularity exists in X-direction.

- **In Y-direction (Figure 4.16),**

  Δ_{max} = 38.85 inch and $\Delta_{avg} = \frac{38.85+31.76}{2}$ = 35.305 inch

  Hence, $\frac{\Delta_{max}}{\Delta_{avg}} = \frac{38.85}{35.305}$ = 1.10 < 1.2………Ok.

  So, no torsional irregularity exists in Y-direction.

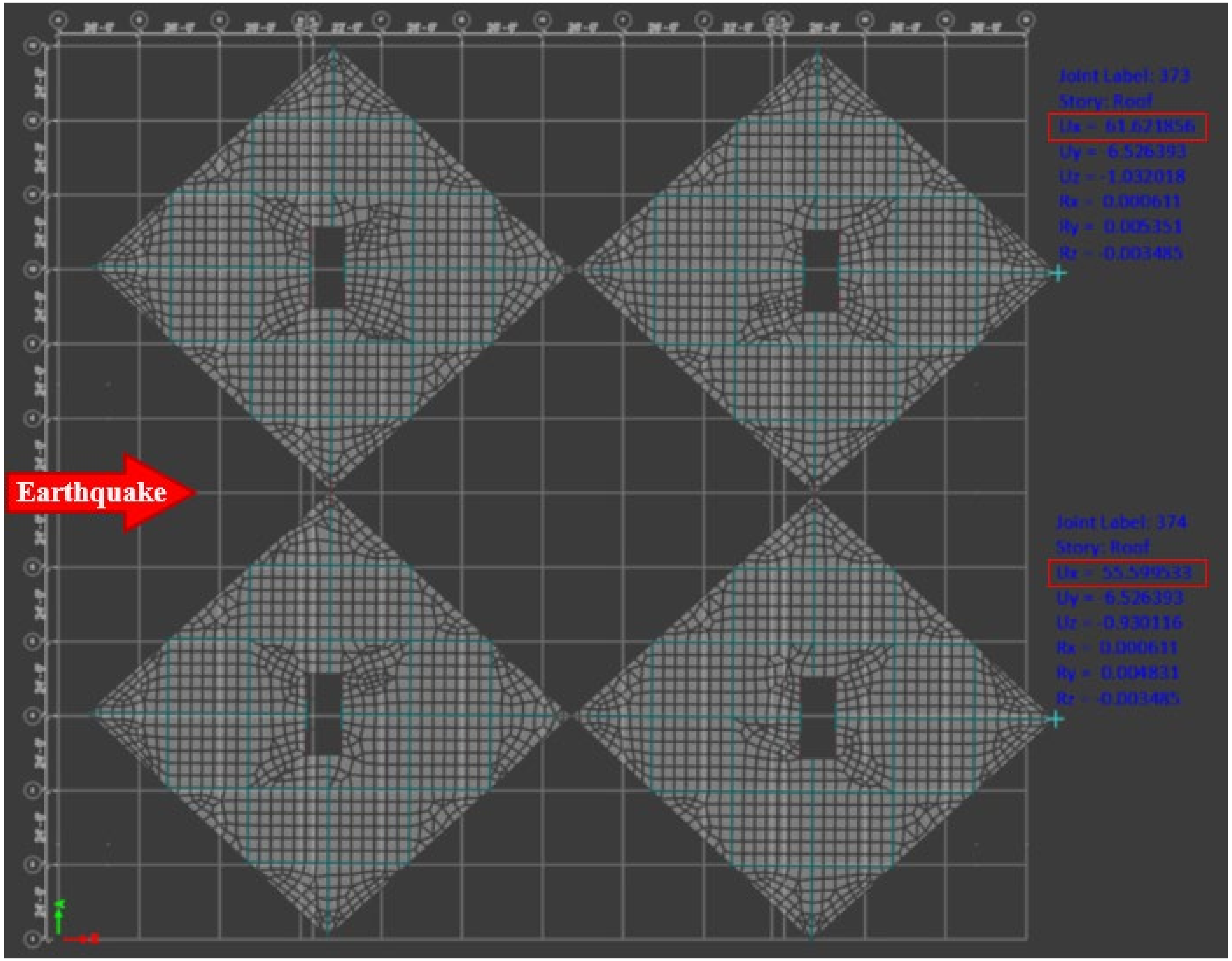

Figure 4.15: Displacement due to Seismic Force (Dynamic) at Roof in X-direction

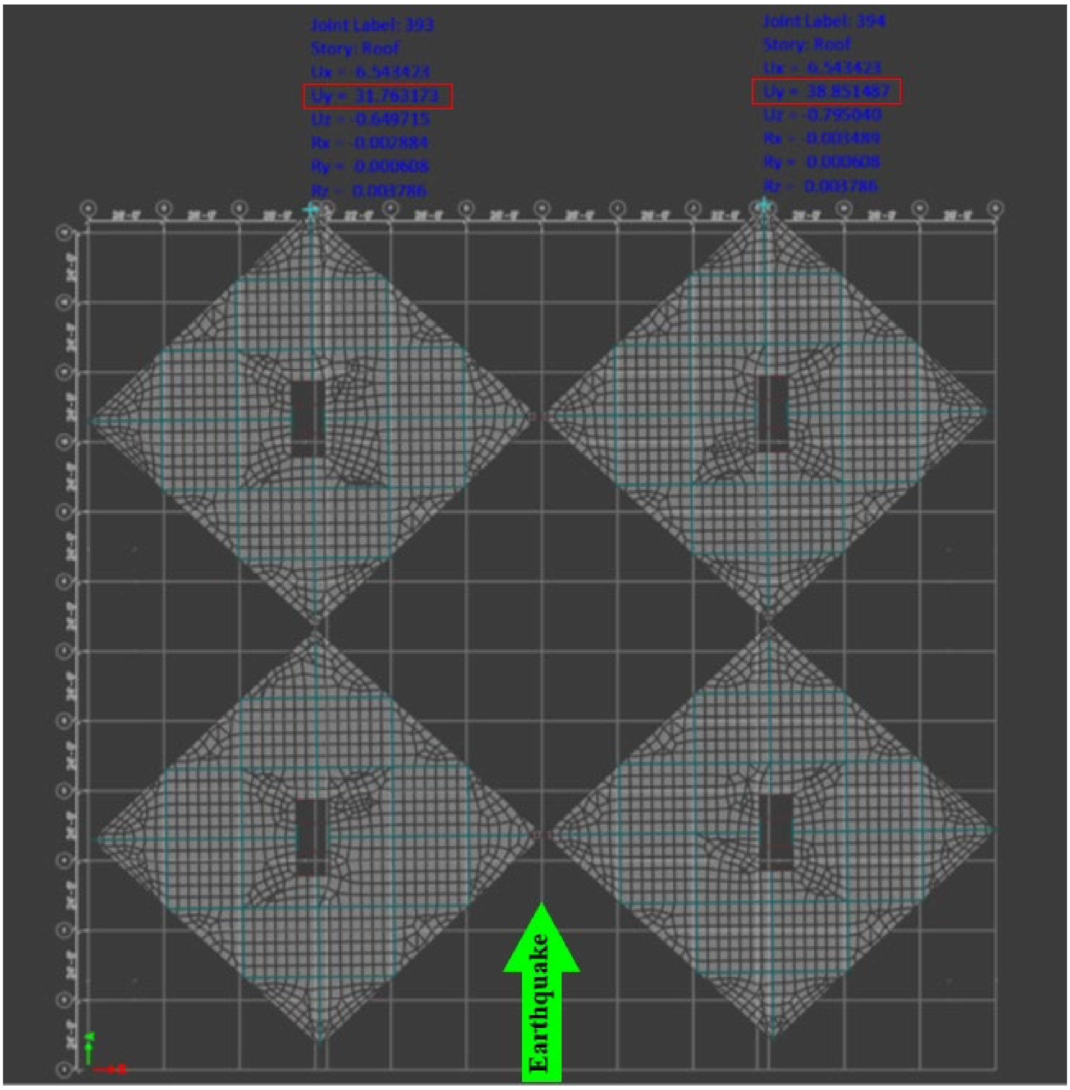

Figure 4.16: Displacement due to Seismic Force (Dynamic) at Roof in Y-direction

4.7.5 Check for Mass Irregularity

a. Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150 percent of the effective mass of an adjacent storey. A roof which is lighter than the floor below need not be considered.

b. Extreme Mass Irregularity is defined to exist where the seismic weight of any storey is more than twice of that of its adjacent; need not be considered in case of roofs.

- Irregular, when $Mass_i > 1.5Mass_{i+1}\ or, Mass_i > 1.5Mass_{i-1}$
- Extreme Irregular, when $Mass_i > 2.0Mass_{i+1}\ or, Mass_i > 2.0Mass_{i-1}$

In this model, the major changes of mass observed at ground floor to first floor and first floor to second floor in both X and Y directions. Where the mass are same at both directions,

- The mass at ground floor = 580502.76 lb-s^2/ft.
- The mass at first floor = 691337.85 lb-s^2/ft.
- The mass at second floor = 376397.55 lb-s^2/ft.

Considering the first floor,

- 1.5∗Mass$_{2nd\ floor}$ = 1.5∗376397.55 = 564596.325 lb-s^2/ft. < Mass$_{1st\ floor}$….Irregularity exists.
- 1.5∗Mass$_{ground\ floor}$ = 1.5∗580502.76 = 870754.14 lb-s^2/ft. > Mass$_{1stfloor}$……No Irregularity exists.
- 2.0∗Mass$_{2nd\ floor}$ = 2.0∗376397.55 = 752795.10 lb-s^2/ft. > Mass$_{1st\ floor}$….No Extreme Irregularity exists.

Hence, the first floor has mass irregularity due to mass at 2nd floor, but hasn't extreme irregularity in both X and Y directions. In the model, other storeys has no mass irregularity in both directions as shown in the Table 4.A.1 & 4.A.2 in the Appendix.

4.7.6 Check for Stiffness Irregularity (Soft Storey)

a. Soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average stiffness of the three storeys above.
b. Extreme soft storey irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.

- Irregular, when $K_i < 0.7K_{i+1}\ or, K_i < \frac{0.8}{3}(K_{i+1} + K_{i+2} + K_{i+3})$
- Extreme Irregular, when $K_i < 0.6K_{i+1}\ or, K_i < \frac{0.7}{3}(K_{i+1} + K_{i+2} + K_{i+3})$

In this model, the major changes of storey stiffness observed at basement floor to ground floor and ground floor to first floor in both X and Y directions.

- **Where the stiffness at X-direction under Earthquake,**
  - The stiffness at basement floor = 147517.07 kip/in
  - The stiffness at ground floor = 97654.36 kip/in
  - The stiffness at 1st floor = 45908.38 kip/in
  - The stiffness at 2nd floor = 35211.83 kip/in
  - The stiffness at 3rd floor = 27998.02 kip/in

Considering the ground floor,

$0.7*K_{1st\ floor} = 0.7*45908.38 = 32135.866$ kip/in < $K_{ground\ floor}$....No Irregularity.

Or, $0.8/3(K_{1st\ floor}+ K_{2nd\ floor}+ K_{3rd\ floor}) = 0.8/3(45908.38+35211.83+27998.02)$

$= 29098.195$ kip/in < $K_{ground\ floor}$......No Irregularity.

- **Where the stiffness at Y-direction under Earthquake,**
  - The stiffness at basement floor = 177237.62 kip/in
  - The stiffness at ground floor = 99185.14 kip/in
  - The stiffness at 1st floor = 65134.63 kip/in
  - The stiffness at 2nd floor = 45240.99 kip/in
  - The stiffness at 3rd floor = 36862.65 kip/in

Considering the ground floor,

$0.7*K_{1st\ floor} = 0.7*65134.63 = 45594.241$ kip/in < $K_{ground\ floor}$....No Irregularity.

Or, $0.8/3(K_{1st\ floor}+ K_{2nd\ floor}+ K_{3rd\ floor}) = 0.8/3(65134.63+45240.99+36862.65)$

$= 39263.54$ kip/in < $K_{ground\ floor}$...No Irregularity.

Hence, the ground floor has no stiffness irregularity (soft storey) in both X and Y directions. Other storeys has also no stiffness irregularity (soft storey) in both directions as shown in the Table 4.A.3 & 4.A.4 in the Appendix.

4.7.7 Check for Discontinuity in Capacity (Weak Storey)

a. A weak storey is one in which the storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear for the direction under consideration.
b. An extreme weak storey is one in which the storey lateral strength is less than 65% of that in the storey above.

- Irregular, when $Str_i < 0.8Str_{i+1}$
- Extreme Irregular, when $Str_i < 0.65K_{i+1}$

In this model, the major changes of storey shear observed at first floor to second floor in both X and Y directions.

- **Where the storey shear at X-direction under Earthquake,**
  - The storey shear at 1st floor = 15245.401 kip
  - The storey shear at 2nd floor = 15173.658 kip

Considering the 1st floor,

$0.8*Str_{2nd\ floor} = 0.8*15173.658 = 12138.9264$ kip < $Str_{1st\ floor}$……No Irregularity.

- **Where the storey shear at Y-direction under Earthquake,**
  - The storey shear at 1st floor = 13216.68 kip
  - The storey shear at 2nd floor = 13162.084 kip

Considering the 1st floor,

$0.8*Str_{2nd\ floor} = 0.8*13162.084 = 10529.6672$ kip < $Str_{1st\ floor}$……No Irregularity.

Hence, the first floor has no discontinuity in capacity irregularity (weak storey) in both X and Y directions. Other storeys has also no discontinuity in capacity irregularity (weak storey) in both directions as shown in the Table 4.A.7 in the Appendix.

4.7.8 Check for Vertical Geometric Irregularity

Vertical geometric irregularity [Figure 2.2(ii) (c)] shall be exist where horizontal dimension of the lateral force-resisting system in any storey is more than 130% of that in an adjacent storey.

- Hence, there is no vertical geometric irregularity exist in the model.

4.7.9 Check for In-Plane Discontinuity in Vertical Lateral Force-Resisting Element

An in-plane offset [Figure 2.2(ii) (d)] of the lateral load-resisting elements greater than the length of those elements.

- Hence, there is no in-plane offset of the lateral force resisting elements in the model.

4.7.10 Check for Re-entrant Corners

Both projections of the structure beyond a re-entrant comer [Figure 2.2(i) (b)] are greater than 15% of its plan dimension in the given direction.

- Hence, there is no re-entrant corners exist in our model.

4.7.11 Check for Diaphragm Discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out [Figure 2.2(i) (c)] or open areas greater than 50% of gross enclosed diaphragm area.

- Irregular, when $A_{open} > 0.5A_{gross}$

In this model,

- Considering the 1st basement and ground floor. Each floor having,

Gross area, A_{gross} = 90668 square ft. and Open area, A_{open} = 1692.36 square ft.

So, 0.5A_{gross} = 0.5∗90668 = 45334 square ft. > A_{open}; …………..No Irregularity Exist.

- Considering the 1st floor, having

Gross area, A_{gross} = 90668 square ft. and Open area, A_{open} = 924.36 square ft.

So, 0.5A_{gross} = 0.5∗90668 = 45334 square ft. > A_{open}; …………..No Irregularity Exist.

- Considering the 2nd floor to roof, having

Gross area, A_{gross} = 46695.12 square ft. and Open area, A_{open} = 924.36 square ft.

So, 0.5A_{gross} = 0.5∗46695.12 = 23347.56 square ft. > A_{open}; …….No Irregularity Exist.

Hence, there is no diaphragm discontinuity exist in any floor of the model.

4.7.12 Check for Out- of-Plane Offsets

Discontinuities in a lateral force resistance path, such as out of-plane offsets [Figure 2.2(i) (d)] of vertical elements.

- Hence, there is no out-of-plane offset of shear wall in the model.

4.7.13 Check for Non-parallel Systems

The vertical elements resisting the lateral force are not parallel to or symmetric [Figure 2.2 (i) (e)] about the major orthogonal axes of the lateral force resisting elements.

- Hence, the shear walls provided in the model are parallel and symmetric in both axes.

4.7.14 Check for Storey Drift

The design storey drift of each storey shall not exceed the allowable storey drift.

- ***For Earthquake Loads***

  The allowable storey drift is 0.020h_{sx}. Where h_{sx} is the height of the structures below level-x. From the model we found maximum drift at 14th floor under both X and Y direction as shown in the Table 4.A.9 in the Appendix.

  Considering the 15th floor allowable drift, Δ_a 0.20h_{sx} = $0.020*120 = 2.40$ inch

In our model,

- Drift at 15^{th} floor under X-direction = 1.051560 inch < Δ_a……Ok.
- Drift at 15^{th} floor under Y-direction = 0.659845 inch < Δ_a……….Ok.

➢ ***For Loads Other than Earthquake Loads***

The allowable storey drift is 0.004h for T ≥ 0.7 seconds. Where h is the height of the structures.

Considering the roof level, h = 411.5 ft. = 4938.0 inch

So, total allowable drift, Δ_a = 0.004∗4938 = 19.75 inch

4.7.15 Check for Sway Limitation

The overall sway (horizontal deflection) at the top level of the building or structure due to wind loading shall not exceed $\frac{1}{500}$ times of the total height of the building above the ground level.

Height of the building above ground level = 411.5 ft. = 4938.0 inch

So, the maximum sway due to wind load = $\frac{1}{500}$ ∗4938 = 9.88 inch

In the model, it is found the maximum storey displacement of top level (Roof) of the building is 32.61 inch under wind load in X-direction.

➢ Hence, the sway = 32.61 inch > allowable sway = 9.88 inch [Not ok].

So, before design, set the target lateral displacement as 9.5 inch under wind load.

4.8 CHECK THE MODEL UNDER STATIC ANALYSIS

4.8.1 Check for Stability

For stability, the number of negative stiffness should be zero and the number of negative eigenvalues should also be zero.

The snapshot as shown in the figure 4.17 has taken from ETABS analysis run log of the study model. It is observed that the negative stiffness and number of negative eigenvalues is zero. So, the model is stable.

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BASIC STABILITY CHECK FOR LINEAR LOAD CASES:
   NUMBER OF NEGATIVE STIFFNESS EIGENVALUES SHOULD BE ZERO FOR STABILITY.
   (NOTE: FURTHER CHECKS SHOULD BE CONSIDERED AS DEEMED NECESSARY,
    SUCH AS REVIEWING EIGEN MODES FOR MECHANISMS AND RIGID-BODY MOTION)

   NUMBER OF NEGATIVE EIGENVALUES          =            0,  OK.
---------------------------------
```

Figure 4.17: Snapshot from ETABS (Static Analysis)

4.8.2 Check for Torsional Irregularity

- Irregularity occurs when $\frac{\Delta_{max}}{\Delta_{avg}} > 1.2$
- Extreme Irregularity occurs when $\frac{\Delta_{max}}{\Delta_{avg}} > 1.4$

- **In X-direction (Figure 4.18),**

  Δ_{max} = 44.93 inch and $\Delta_{avg} = \frac{44.93+40.12}{2}$ = 42.525 inch

  Hence, $\frac{\Delta_{max}}{\Delta_{avg}} = \frac{44.93}{42.525}$ = 1.06 < 1.2……Ok.

  So, no torsional irregularity exists in X-direction.

- **In Y-direction (Figure 4.19),**

  Δ_{max} = 31.59 inch and $\Delta_{avg} = \frac{31.59+25.95}{2}$ = 28.77 inch

  Hence, $\frac{\Delta_{max}}{\Delta_{avg}} = \frac{31.59}{28.77}$ = 1.10 < 1.2………Ok.

  So, no torsional irregularity exists in Y-direction.

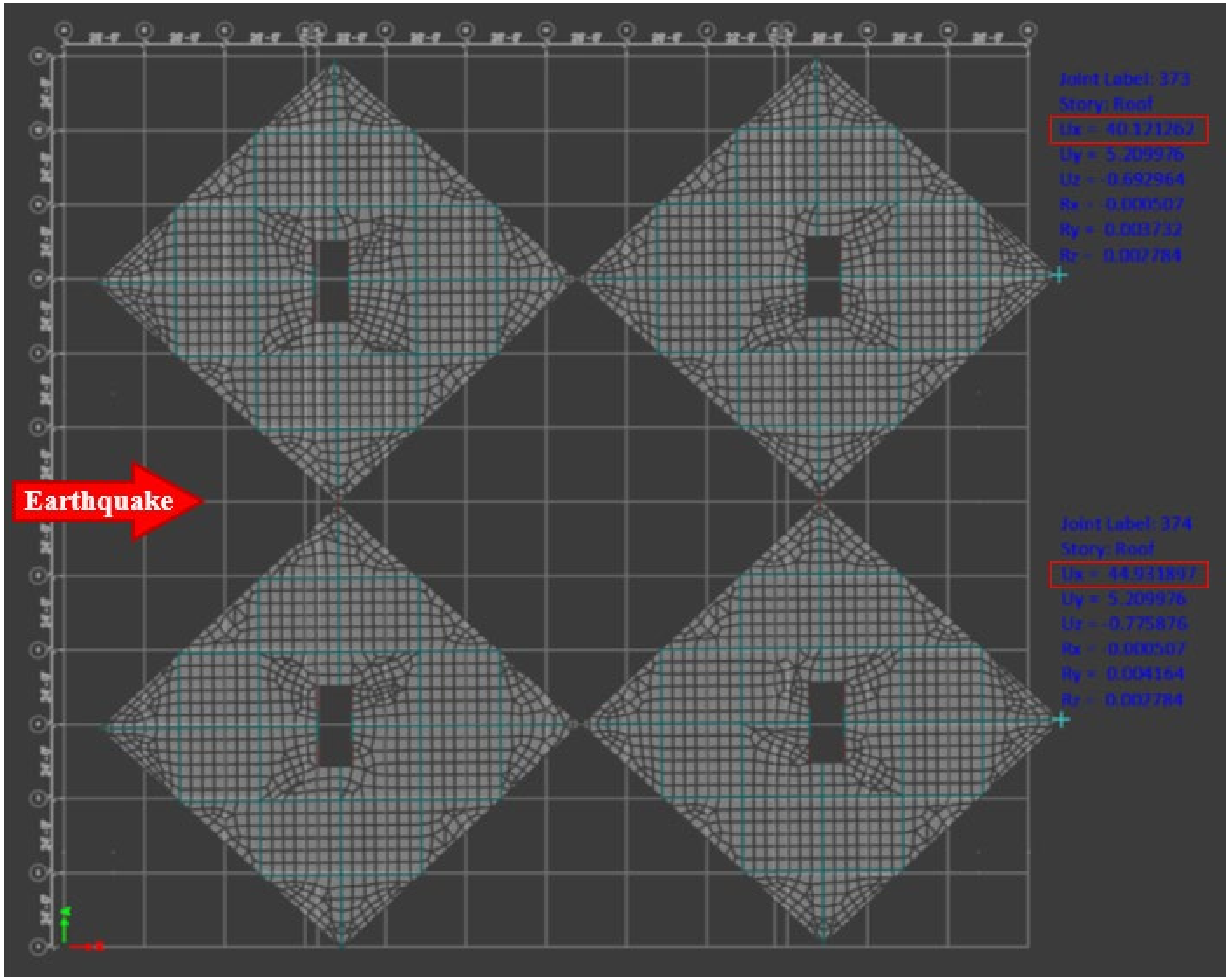


Figure 4.18: Displacement due to Seismic Force (Static) at Roof in X-direction

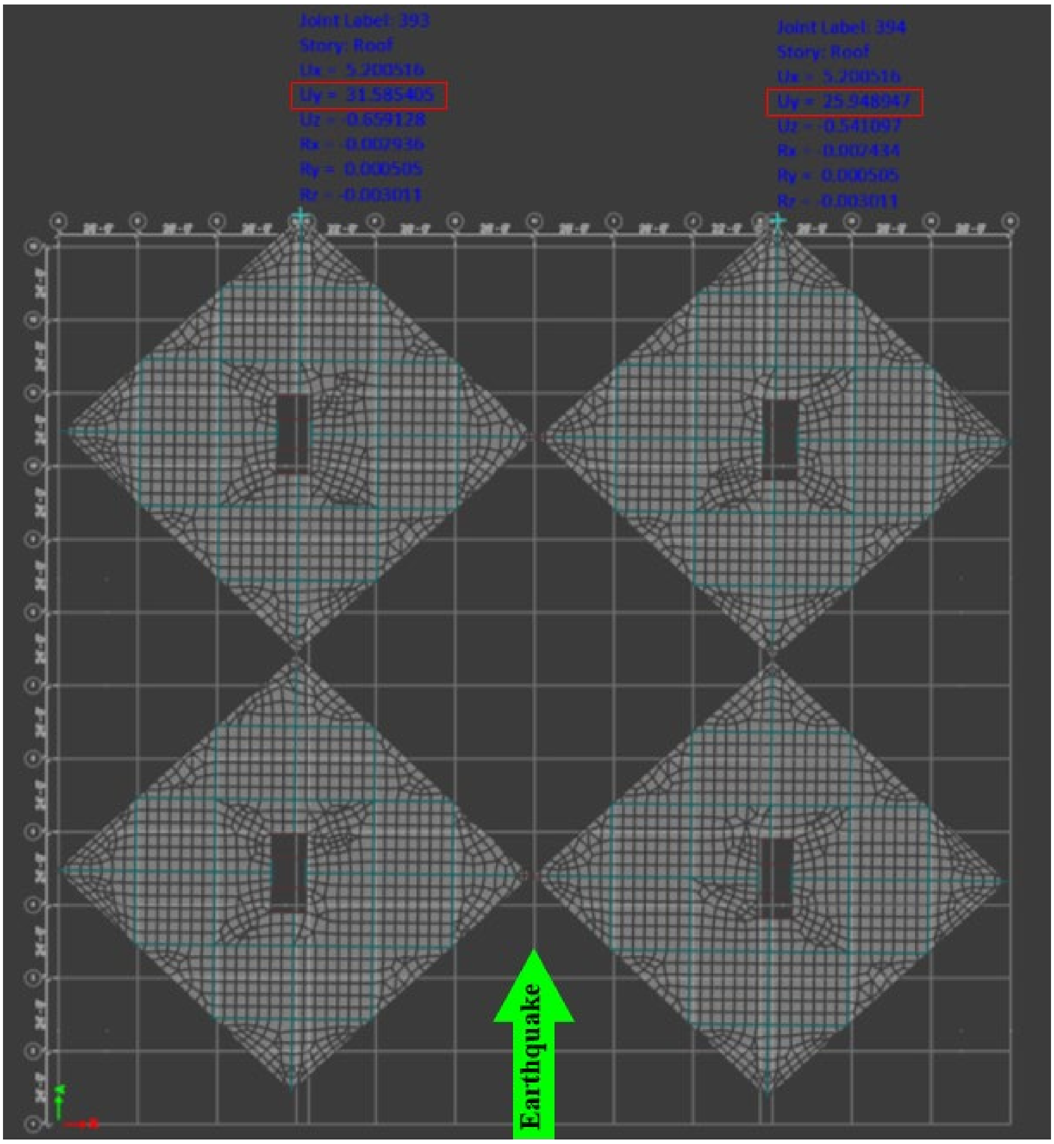


Figure 4.19: Displacement due to Seismic Force (Static) at Roof in Y-direction

4.8.3 Check for Mass Irregularity

- Irregular, when $Mass_i > 1.5Mass_{i+1}\ or, Mass_i > 1.5Mass_{i-1}$
- Extreme Irregular, when $Mass_i > 2.0Mass_{i+1}\ or, Mass_i > 2.0Mass_{i-1}$

In this model, the major changes of mass observed at ground floor to first floor and first floor to second floor in both X and Y directions. Where the mass are same at both directions,

- The mass at ground floor = 580502.76 lb-s^2/ft.
- The mass at first floor = 691337.85 lb-s^2/ft.
- The mass at second floor = 376397.55 lb-s^2/ft.

Considering the first floor,

- 1.5∗Mass$_{2nd\ floor}$ = 1.5∗376397.55 = 564596.325 lb-s^2/ft. < Mass$_{1st\ floor}$…Irregularity exists.
- 1.5∗Mass$_{ground\ floor}$ = 1.5∗580502.76 = 870754.14 lb-s^2/ft. > Mass$_{1stfloor}$......No Irregularity exists.
- 2.0∗Mass$_{2nd\ floor}$ = 2.0∗376397.55 = 752795.10 lb-s^2/ft. > Mass$_{1st\ floor}$....No Extreme Irregularity exists.

Hence, the first floor has mass irregularity due to mass at 2nd floor, but hasn't extreme irregularity in both X and Y directions. In the model, other storeys has no mass irregularity in both directions as shown in the Table 4.A.1 & 4.A.2 in the Appendix.

4.8.4 Check for Stiffness Irregularity (Soft Storey)

- Irregular, when $K_i < 0.7K_{i+1}\ or, K_i < \frac{0.8}{3}(K_{i+1} + K_{i+2} + K_{i+3})$
- Extreme Irregular, when $K_i < 0.6K_{i+1}\ or, K_i < \frac{0.7}{3}(K_{i+1} + K_{i+2} + K_{i+3})$

In this model, the major changes of storey stiffness observed at basement floor to ground floor and ground floor to first floor in both X and Y directions.

- **Where the stiffness at X-direction under Earthquake,**
  - The stiffness at basement floor = 145413.2568 kip/in
  - The stiffness at ground floor = 91001.0122 kip/in
  - The stiffness at 1st floor = 41247.5864 kip/in
  - The stiffness at 2nd floor = 31006.8858 kip/in
  - The stiffness at 3rd floor = 24585.3526 kip/in

Considering the ground floor,

0.7∗K$_{1st\ floor}$ = 0.7∗41247.5864 = 28873.31 kip/in < K$_{ground\ floor}$....No Irregularity.

Or, 0.8/3(K$_{1st\ floor}$+ K$_{2nd\ floor}$+ K$_{3rd\ floor}$) = 0.8/3(41247.5864+31006.8858+24585.3526)

= 25823.95 kip/in < K$_{ground\ floor}$......No Irregularity.

- **Where the stiffness at Y-direction under Earthquake,**
  - The stiffness at basement floor = 176130.0789 kip/in
  - The stiffness at ground floor = 88277.4716 kip/in
  - The stiffness at 1st floor = 55826.4976 kip/in
  - The stiffness at 2nd floor = 38132.5578 kip/in
  - The stiffness at 3rd floor = 30826.5205 kip/in

Considering the ground floor,

$0.7*K_{1st\ floor} = 0.7*55826.4976 = 39078.55$ kip/in < $K_{ground\ floor}$....No Irregularity.

Or, $0.8/3(K_{1st\ floor}+ K_{2nd\ floor}+ K_{3rd\ floor}) = 0.8/3(55826.4976+38132.5578+30826.5205)$

$= 33276.15$ kip/in < $K_{ground\ floor}$...No Irregularity.

Hence, the ground floor has no stiffness irregularity (soft storey) in both X and Y directions. Other storeys has also no stiffness irregularity (soft storey) in both directions as shown in the Table 4.A.5 & 4.A.6 in the Appendix.

4.8.5 Check for Discontinuity in Capacity (Weak Storey)

- Irregular, when $Str_i < 0.8Str_{i+1}$
- Extreme Irregular, when $Str_i < 0.65K_{i+1}$

In this model, the major changes of storey shear observed at first floor to second floor in both X and Y directions.

- **Where the storey shear at X-direction under Earthquake,**
  - The storey shear at 1st floor = 10098.224 kip
  - The storey shear at 2nd floor = 10082.396 kip

Considering the 1st floor,

$0.8*Str_{2nd\ floor} = 0.8*10082.396 = 8065.92$ kip < $Str_{1st\ floor}$……No Irregularity.

- **Where the storey shear at Y-direction under Earthquake,**
  - The storey shear at 1st floor = 10097.709 kip
  - The storey shear at 2nd floor = 10081.882 kip

Considering the 1st floor,

$0.8*Str_{2nd\ floor} = 0.8*10081.882 = 8065.51$ kip < $Str_{1st\ floor}$…….No Irregularity.

Hence, the first floor has no discontinuity in capacity irregularity (weak storey) in both X and Y directions. Other storeys has also no discontinuity in capacity irregularity (weak storey) in both directions as shown in the Table 4.A.8 in the Appendix.

4.8.6 Check for Vertical Geometric Irregularity

Vertical geometric irregularity [Figure 2.2(ii) (c)] shall be exist where horizontal dimension of the lateral force-resisting system in any storey is more than 130% of that in an adjacent storey.

- Hence, there is no vertical geometric irregularity exist in the model.

4.8.7 Check for In-Plane Discontinuity in Vertical Lateral Force-Resisting Element

An in-plane offset [Figure 2.2(ii) (d)] of the lateral load-resisting elements greater than the length of those elements.

- Hence, there is no in-plane offset of the lateral force resisting elements in the model.

4.8.8 Check for Re-entrant Corners

Both projections of the structure beyond a re-entrant comer [Figure 2.2(i) (b)] are greater than 15% of its plan dimension in the given direction.

- Hence, there is no re-entrant corners exist in our model.

4.8.9 Check for Diaphragm Discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out [Figure 2.2(i) (c)] or open areas greater than 50% of gross enclosed diaphragm area.

◆ Irregular, when $A_{open} > 0.5A_{gross}$

In this model,

- Considering the 1st basement and ground floor. Each floor having,

Gross area, A_{gross} = 90668 square ft. and Open area, A_{open} = 1692.36 square ft.

So, $0.5A_{gross}$ = 0.5∗90668 = 45334 square ft. > A_{open}; …………..No Irregularity Exist.

- Considering the 1st floor, having

Gross area, A_{gross} = 90668 square ft. and Open area, A_{open} = 924.36 square ft.

So, $0.5A_{gross}$ = 0.5∗90668 = 45334 square ft. > A_{open}; …………..No Irregularity Exist.

- Considering the 2nd floor to roof, having

Gross area, A_{gross} = 46695.12 square ft. and Open area, A_{open} = 924.36 square ft.

So, $0.5A_{gross}$ = 0.5∗46695.12 = 23347.56 square ft. > A_{open}; …….No Irregularity Exist.

Hence, there is no diaphragm discontinuity exist in any floor of the model.

4.8.10 Check for Out- of-Plane Offsets

Discontinuities in a lateral force resistance path, such as out of-plane offsets [Figure 2.2(i) (d)] of vertical elements.

- Hence, there is no out-of-plane offset of shear wall in the model.

4.8.11 Check for Non-parallel Systems

The vertical elements resisting the lateral force are not parallel to or symmetric [Figure 2.2 (i) (e)] about the major orthogonal axes of the lateral force resisting elements.

- Hence, the shear walls provided in the model are parallel and symmetric in both axes.

4.8.12 Check for Storey Drift

The design storey drift of each storey shall not exceed the allowable storey drift.

- ***For Earthquake Loads***

  The allowable storey drift is $0.020h_{sx}$. Where h_{sx} is the height of the structures below level-x. From the model we found maximum drift at 15th floor under both X and Y direction as shown in the Table 4.A.10 in the Appendix.

  Considering the 15th floor allowable drift, Δ_a $0.20h_{sx} = 0.020*120 = 2.40$ inch

In our model,

- Drift at 15th floor under X-direction = 0.753225 inch < Δ_a……Ok.
- Drift at 15th floor under Y-direction = 0.527292 inch < Δ_a………..Ok.

- ***For Loads Other than Earthquake Loads***

  The allowable storey drift is 0.004h for $T \geq 0.7$ seconds. Where h is the height of the structures.

  Considering the roof level, h = 411.5 ft. = 4938.0 inch

  So, total allowable drift, $\Delta_a = 0.004*4938 = 19.75$ inch

4.8.13 Check for Sway Limitation

The overall sway (horizontal deflection) at the top level of the building or structure due to wind load shall not exceed $\frac{1}{500}$ times of the total height of building above the ground level.

Height of the building above ground level = 411.5 ft. = 4938.0 inch

So, the maximum sway due to wind load = $\frac{1}{500}*4938 = 9.88$ inch

In the model, it is found the maximum storey displacement of top level (Roof) of the building is 24.74 inch under wind load in X-direction.

- Hence, the sway = 24.74 inch > allowable sway = 9.88 inch [Not ok].

So, before design, set the target lateral displacement as 9.5 inch under wind load.

Chapter 5

RESULT AND DISCUSSIONS

CHAPTER FIVE

RESULT AND DISCUSSIONS

5.1 INTRODUCTION

This chapter provides findings of the study and discussion of the obtained results as per references used. This study focuses on the responses by analyzing the effects of static and dynamic load on a 41 storied high-rise structure. All results are summarized in several tabular forms and presented in graphical forms in order to make comparative analyses. Also, few explanations were made based on data from ETABS. Finally, presents a comparative result to identify best method for analyzing the high-rise structure against lateral loadings.

5.2 COMPARISON BETWEEN STATIC & DYNAMIC ANALYSIS

Items under Check	Dynamic Analysis	Static Analysis
Basic Stability	Stable	Stable
Modal Participating Mass Ratio	92.08% in X-direction and 92.19% Y-direction	Not applicable
Base Shear Check	The ratio of V_{rs} to V in X and Y direction is respectively 0.91 and 0.86	Not applicable
Torsional Irregularity	None	None
Mass Irregularity	1st floor has irregularity and other's floor has no irregularity in both directions	1st floor has irregularity and other's floor has no irregularity in both directions
Stiffness Irregularity (Soft Storey)	No Soft Storey	No Soft Storey
Discontinuity in Capacity (Weak Storey)	No Weak Storey	No Weak Storey

Vertical Geometric Irregularity	None	None
In-Plane Discontinuity in Vertical Lateral Force-Resisting Element	None	None
Re-entrant Corners	None	None
Diaphragm Discontinuity	None	None
Out- of-Plane Offsets	None	None
Non-parallel Systems	None	None
Maximum Storey Drift	For earthquake – 1.05156 in	For earthquake – 0.75323 in
Sway Limitation	Due to wind – 32.61 in	Due to wind – 24.74 in

5.3 COMPARATIVE DISCUSSIONS

According to the main objective of this study, it is required to find out the response of structure under static and dynamic loadings, having similar floor plan and areas and make a comparison among these structural responses. To obtain this goal, the whole comparative study is divided into several sub topics so that a clear picture can be obtained and complete discussions are possible. Also following points are considered:

1. The structure are divided into several grids in ETABS plan: **1~13 in horizontal grids** and **A~O in vertical grids** as shown in the Figure 5.1 & 5.2.
2. Analyses data are taken for Lateral loads to Stories; Maximum stories Displacement; Maximum stories Drifts; Stories Shear, Stories Overturning Moments and Stories Stiffness in case of both structures.

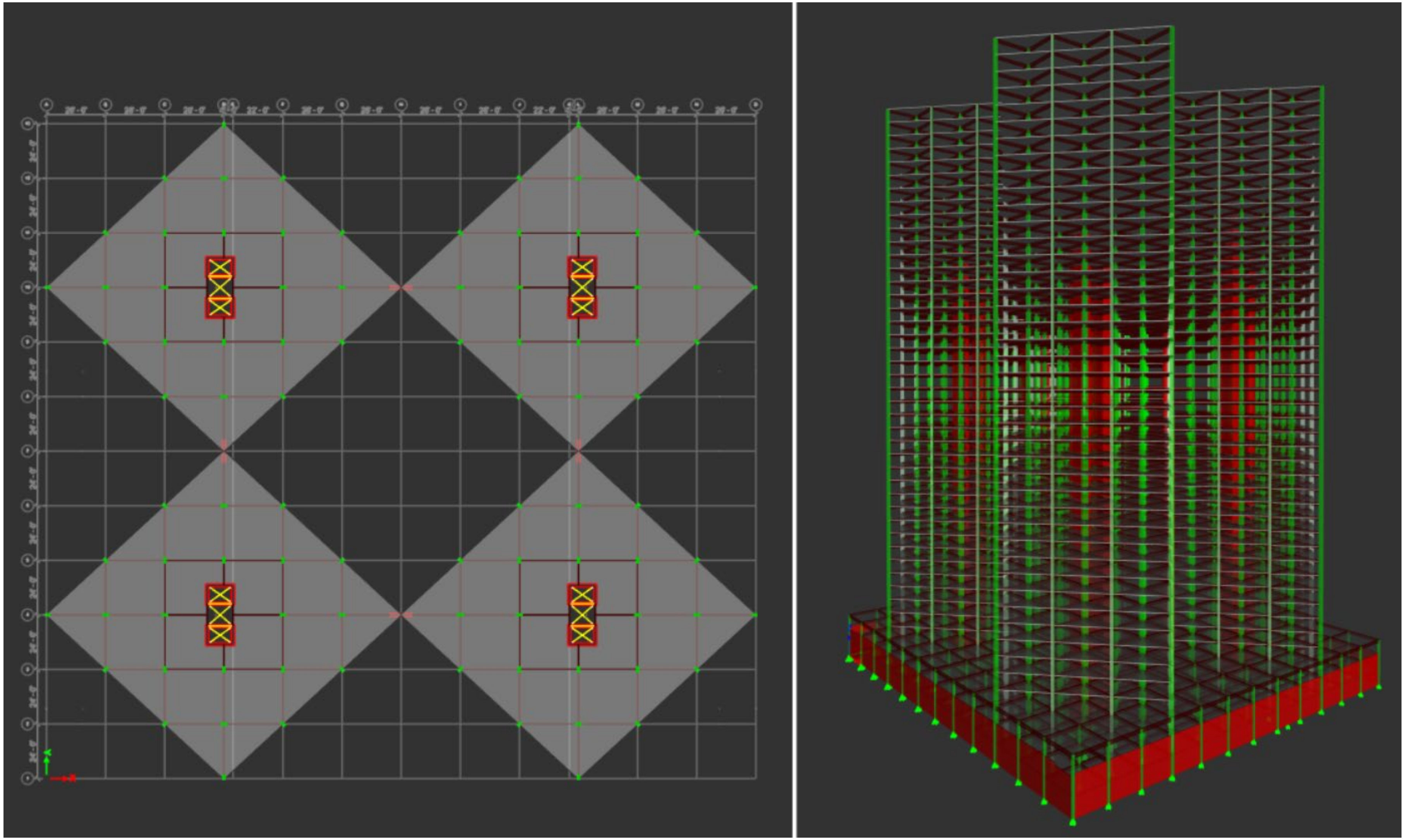


Figure 5.1: Gridlines and 3D view in ETABS

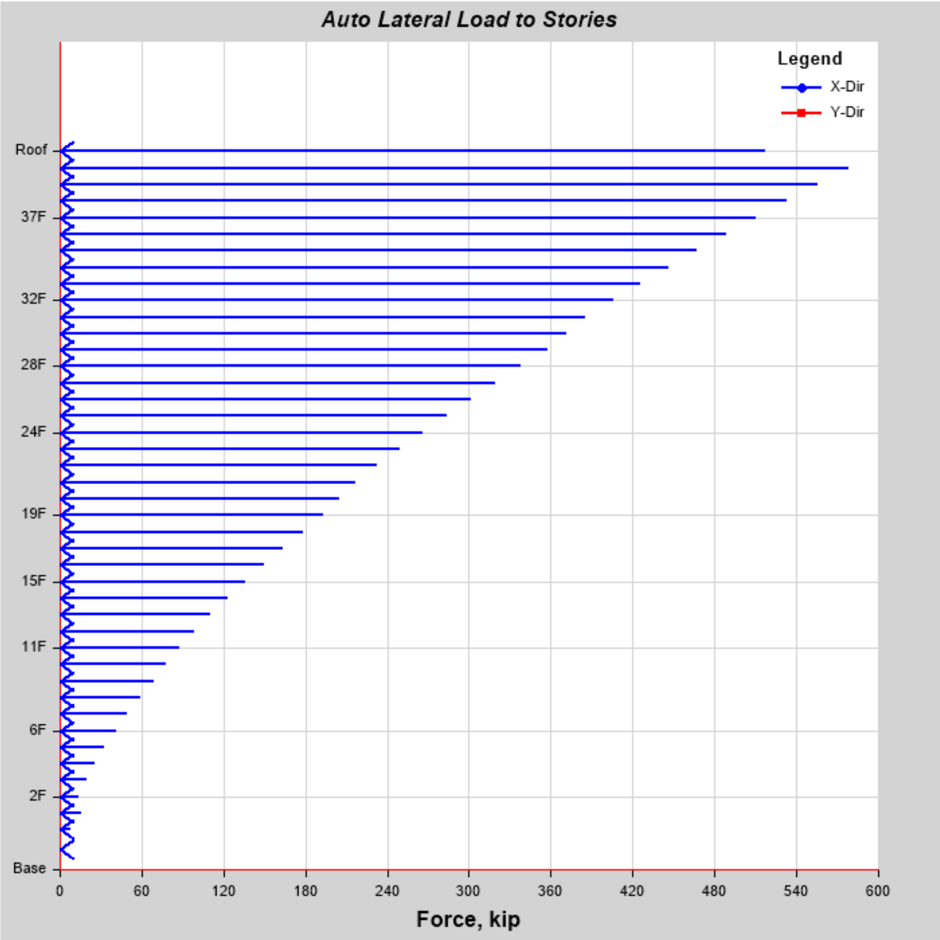
5.3.1 Response due to Earthquake

➢ **Auto Lateral Load to Stories**

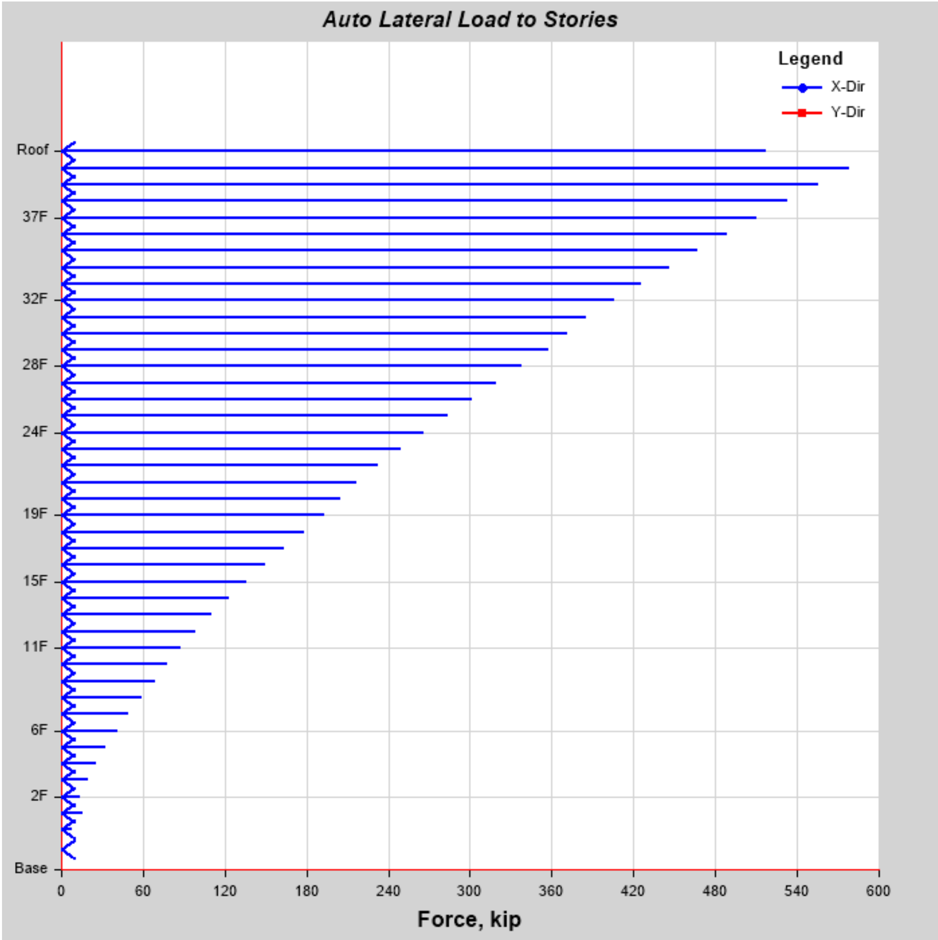
Figures 5.2 and 5.3 illustrated auto lateral loads to stories, where the horizontal axis represent lateral load in kips and vertical axis represent the stories; blue curve state the response due to lateral loads implying in X-direction and red curve in Y-direction. Also comparison between the responses about lateral load to stories under static and dynamic loadings in X and Y direction are clearly shown in Tables 5.1 and 5.2 respectively.

From figures, it is clearly seen that the response curves are same under both dynamic and static loadings while the value changes gradually in each story up to 40th floor and the value is decreased suddenly at roof. It shows that the value of lateral load due to earthquake in both X and Y direction increases gradually from base to 40th floor. Because the earthquake hit at base but the effect of earthquake increases as it goes up; at roof top, mass is decreases due to no partition wall load that's why the lateral force of this story decreases. It means the seismic force at story level varies with respect to height and mass of the story.

It is found the maximum lateral force due to earthquake at 40th floor under both static & dynamic loading in both X and Y direction 578.58 kips.

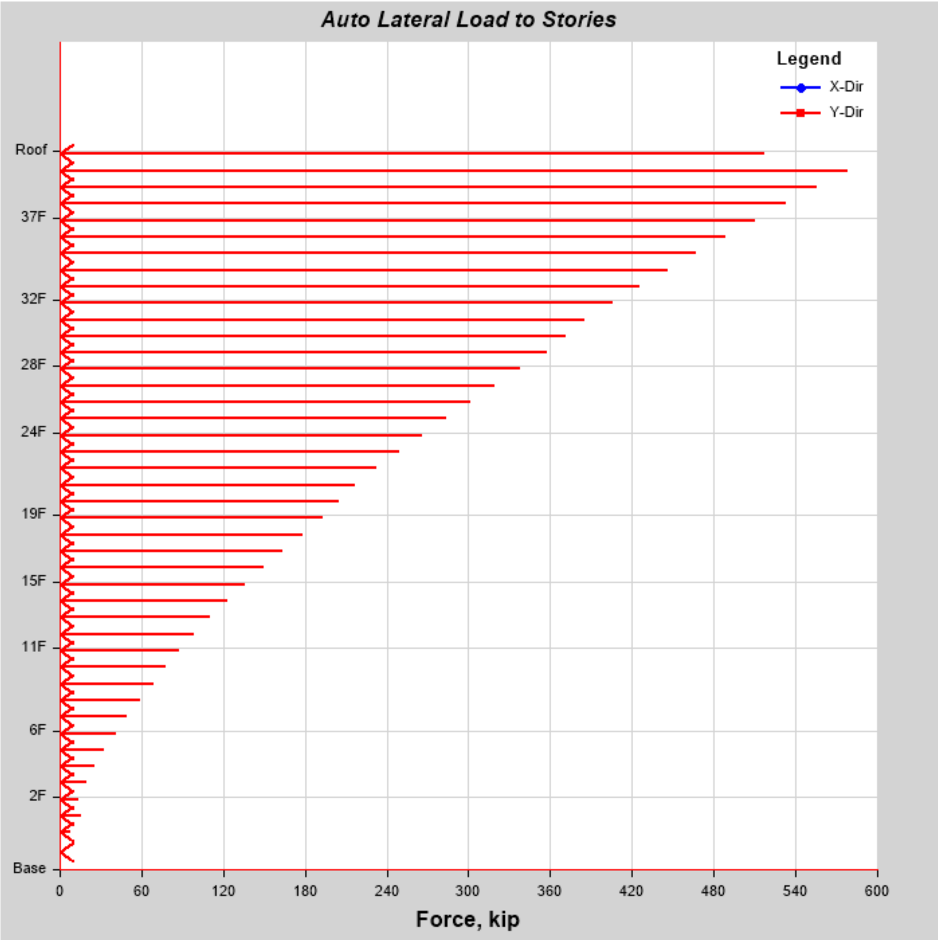


(a) Dynamic Loading

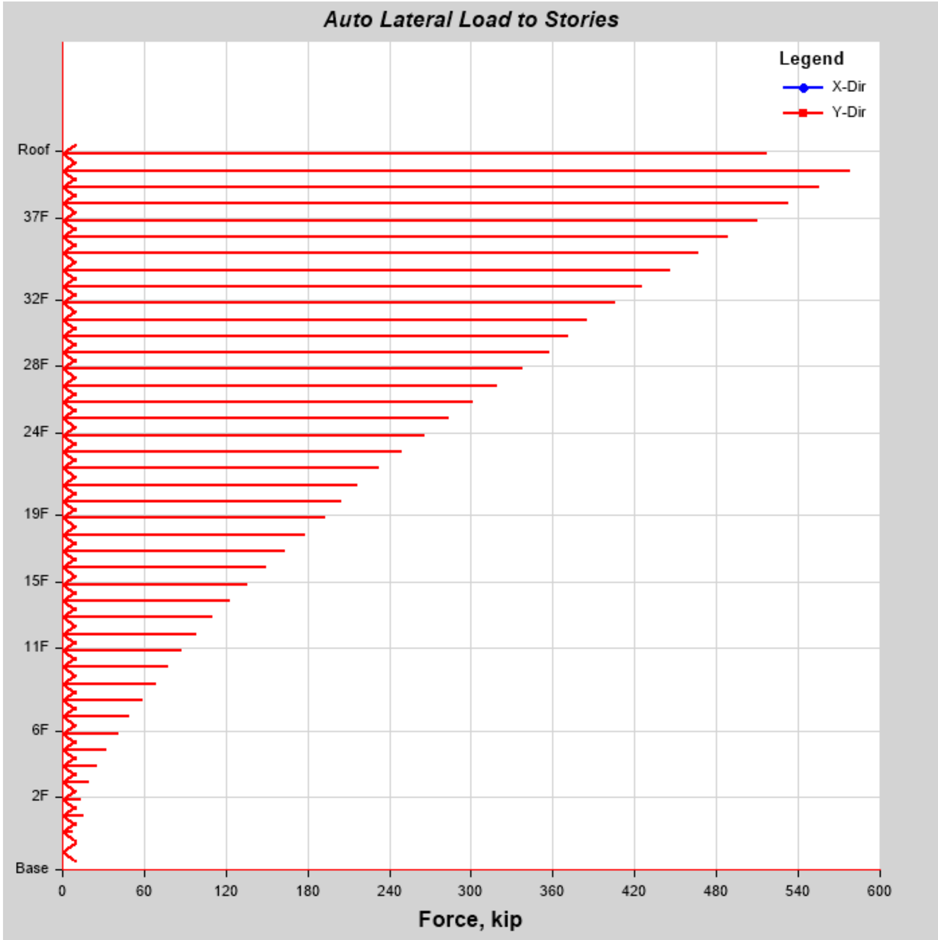


(b) Static Loading

Figure 5.2: Resisting loads by each story due to Earthquake in X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.3: Resisting loads by each story due to Earthquake in Y-direction

Table 5.1: Resisting lateral load to stories due to Earthquake in X-direction

Storey	Elevation (ft.)	Dynamic Loading Resisting Lateral Loads (kips)	Static Loading Resisting Lateral Loads (kips)
Roof	434.00	516.83	516.83
40F	424.00	578.581	578.581
39F	414.00	555.575	555.575
38F	404.00	532.955	532.955
37F	394.00	510.724	510.724
36F	384.00	488.884	488.884
35F	374.00	467.438	467.438
34F	364.00	446.39	446.39
33F	354.00	425.744	425.744
32F	344.00	405.501	405.501
31F	334.00	385.666	385.666
30F	324.00	370.958	370.958
29F	314.00	357.262	357.262
28F	304.00	338.136	338.136
27F	294.00	319.445	319.445
26F	284.00	301.195	301.195
25F	274.00	283.388	283.388
24F	264.00	266.031	266.031
23F	254.00	249.129	249.129
22F	244.00	232.685	232.685
21F	234.00	216.707	216.707
20F	224.00	204.697	204.697
19F	214.00	193.401	193.401
18F	204.00	178.375	178.375
17F	194.00	163.767	163.767
16F	184.00	149.677	149.677
15F	174.00	136.112	136.112
14F	164.00	123.083	123.083
13F	154.00	110.598	110.598
12F	144.00	98.669	98.669
11F	134.00	87.305	87.305
10F	124.00	77.821	77.821
9F	114.00	68.739	68.739
8F	104.00	58.806	58.806
7F	94.00	49.52	49.52
6F	84.00	40.901	40.901
5F	74.00	32.973	32.973
4F	64.00	25.761	25.761
3F	54.00	19.299	19.299
2F	44.00	13.625	13.625
1F	34.00	15.828	15.828
GF	24.00	8.111	8.111
BF	12.00	2.612	2.612
Base	0.00	0	0

Table 5.2: Resisting lateral load to stories due to Earthquake in Y-direction

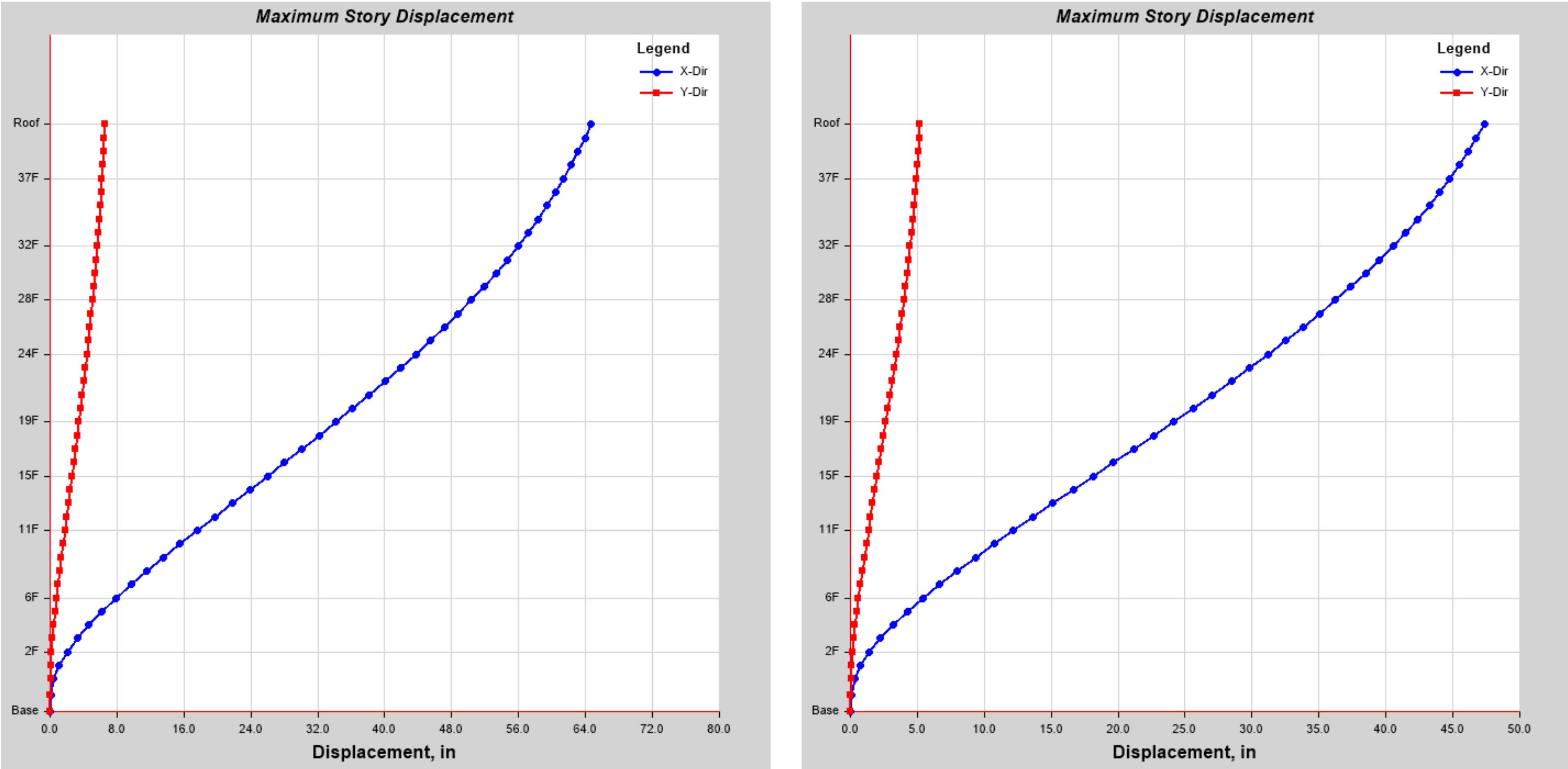
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Resisting Lateral Loads (kips)	Resisting Lateral Loads (kips)
Roof	434.00	516.83	516.83
40F	424.00	578.581	578.581
39F	414.00	555.575	555.575
38F	404.00	532.955	532.955
37F	394.00	510.724	510.724
36F	384.00	488.884	488.884
35F	374.00	467.438	467.438
34F	364.00	446.39	446.39
33F	354.00	425.744	425.744
32F	344.00	405.501	405.501
31F	334.00	385.666	385.666
30F	324.00	370.958	370.958
29F	314.00	357.262	357.262
28F	304.00	338.136	338.136
27F	294.00	319.445	319.445
26F	284.00	301.195	301.195
25F	274.00	283.388	283.388
24F	264.00	266.031	266.031
23F	254.00	249.129	249.129
22F	244.00	232.685	232.685
21F	234.00	216.707	216.707
20F	224.00	204.697	204.697
19F	214.00	193.401	193.401
18F	204.00	178.375	178.375
17F	194.00	163.767	163.767
16F	184.00	149.677	149.677
15F	174.00	136.112	136.112
14F	164.00	123.083	123.083
13F	154.00	110.598	110.598
12F	144.00	98.669	98.669
11F	134.00	87.305	87.305
10F	124.00	77.821	77.821
9F	114.00	68.739	68.739
8F	104.00	58.806	58.806
7F	94.00	49.52	49.52
6F	84.00	40.901	40.901
5F	74.00	32.973	32.973
4F	64.00	25.761	25.761
3F	54.00	19.299	19.299
2F	44.00	13.625	13.625
1F	34.00	15.828	15.828
GF	24.00	8.111	8.111
BF	12.00	2.612	2.612
Base	0.00	0	0

- **Maximum Story Displacement**

Figures 5.4 and 5.5 illustrated the maximum story displacement, where the horizontal axis represent displacement in inch and vertical axis represent the stories; blue curve state the story displacement implying in X-direction and red curve in Y-direction. Also comparison between maximum story displacement under static and dynamic loadings in X and Y direction are clearly shown in Tables 5.3 and 5.4 respectively.

From figures, it is clearly seen that curve starts from base and sharply goes on top in both X & Y direction. The displacement curve under static loading of the structure fluctuates slightly from the displacement curve under dynamic loading of the structure in both X & Y direction. It is also see that the story displacement due to earthquake in X-direction, a minor story displacement is found in Y-direction; and same as due to earthquake in Y-direction, a minor story displacement is found in X-direction. When earthquake on a direction, at the same time there is a minor effect on other direction because of cyclic loading behavior of earthquake.

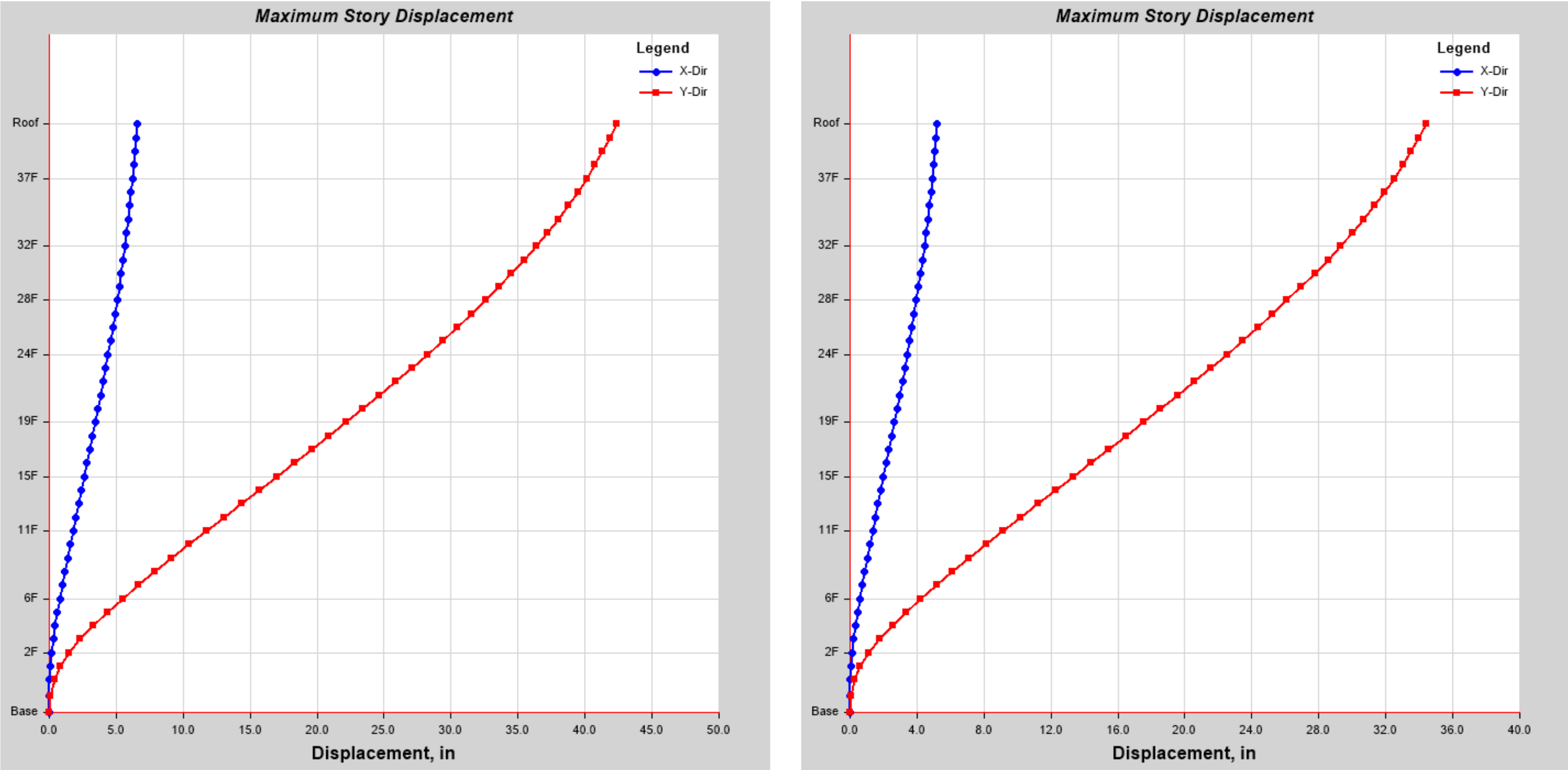
It is found the maximum displacement of the structure is larger under dynamic loading corresponding to the static loading in both X and Y direction. The maximum displacement of the structure is found under dynamic loading at roof top is 64.68 inch in X-direction, while the maximum displacement under static loading at roof top is 47.34 in X-direction.



(a) Dynamic Loading

(b) Static Loading

Figure 5.4: Maximum Story Displacement due to Earthquake loads in global X-direction



(a) Dynamic Loading (b) Static Loading

Figure 5.5: Maximum Story Displacement due to Earthquake loads in global Y-direction

Table 5.3: Maximum story displacement due to Earthquake in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Drift Displacement (inch)	Storey Drift Displacement (inch)
Roof	434.00	64.67511	47.337214
40F	424.00	63.914196	46.741658
39F	414.00	63.118783	46.118761
38F	404.00	62.28253	45.463774
37F	394.00	61.393791	44.767885
36F	384.00	60.444853	44.025483
35F	374.00	59.430037	43.232635
34F	364.00	58.345557	42.386954
33F	354.00	57.189232	41.48735
32F	344.00	55.960076	40.533701
31F	334.00	54.659372	39.52765
30F	324.00	53.282661	38.466484
29F	314.00	51.849977	37.366336
28F	304.00	50.348879	36.217915
27F	294.00	48.782721	35.02455
26F	284.00	47.152564	33.78764
25F	274.00	45.460386	32.509282
24F	264.00	43.708647	31.1919
23F	254.00	41.9003	29.838262
22F	244.00	40.038683	28.451381
21F	234.00	38.128371	27.035109
20F	224.00	36.167814	25.589012
19F	214.00	34.179918	24.130061
18F	204.00	32.15824	22.652848
17F	194.00	30.108432	21.162044
16F	184.00	28.035494	19.661465
15F	174.00	25.945325	18.15548
14F	164.00	23.84474	16.649031
13F	154.00	21.74162	15.147715
12F	144.00	19.645045	13.657886
11F	134.00	17.565542	12.186807
10F	124.00	15.515232	10.742954
9F	114.00	13.512471	9.338408
8F	104.00	11.558707	7.972997
7F	94.00	9.672445	6.659356
6F	84.00	7.872612	5.410094
5F	74.00	6.181167	4.239786
4F	64.00	4.62376	3.165442
3F	54.00	3.230438	2.207001
2F	44.00	2.036468	1.387914
1F	34.00	1.085859	0.737581
GF	24.00	0.437703	0.295707
Basement Floor	12.00	0.109426	0.073593
Base	0.00	0	0

Table 5.4: Maximum story displacement due to Earthquake in Y-direction

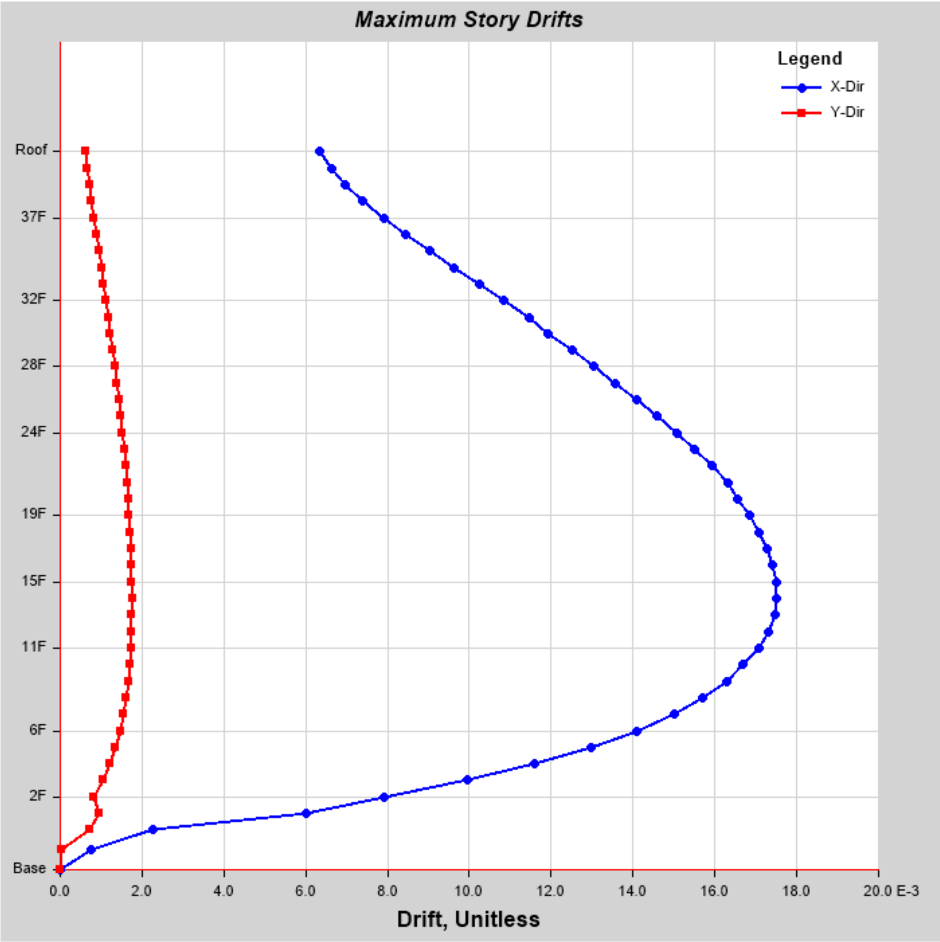
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Drift Displacement (inch)	Storey Drift Displacement (inch)
Roof	434.00	42.400167	34.403634
40F	424.00	41.87976	33.965412
39F	414.00	41.330224	33.501753
38F	404.00	40.748681	33.010262
37F	394.00	40.128048	32.485121
36F	384.00	39.464347	31.923204
35F	374.00	38.755222	31.322799
34F	364.00	37.999696	30.683389
33F	354.00	37.197872	30.005387
32F	344.00	36.350615	29.289862
31F	334.00	35.460152	28.539028
30F	324.00	34.523148	27.750462
29F	314.00	33.563661	26.94476
28F	304.00	32.56529	26.108197
27F	294.00	31.531217	25.243829
26F	284.00	30.462393	24.352746
25F	274.00	29.360271	23.436454
24F	264.00	28.226539	22.49664
23F	254.00	27.063092	21.535155
22F	244.00	25.871934	20.553929
21F	234.00	24.655319	19.55509
20F	224.00	23.411022	18.537298
19F	214.00	22.157836	17.515915
18F	204.00	20.886939	16.483262
17F	194.00	19.601031	15.441904
16F	184.00	18.302315	14.393808
15F	174.00	16.993407	13.341205
14F	164.00	15.677338	12.286621
13F	154.00	14.357649	11.232939
12F	144.00	13.038471	10.183478
11F	134.00	11.724777	9.142187
10F	124.00	10.422244	8.113649
9F	114.00	9.142618	7.10667
8F	104.00	7.88412	6.119175
7F	94.00	6.656618	5.158832
6F	84.00	5.470912	4.233843
5F	74.00	4.340299	3.354244
4F	64.00	3.281388	2.532546
3F	54.00	2.315304	1.784667
2F	44.00	1.469658	1.131446
1F	34.00	0.783759	0.602666
GF	24.00	0.377643	0.290054
Basement Floor	12.00	0.079798	0.061107
Base	0.00	0	0

➢ **Maximum Story Drift**

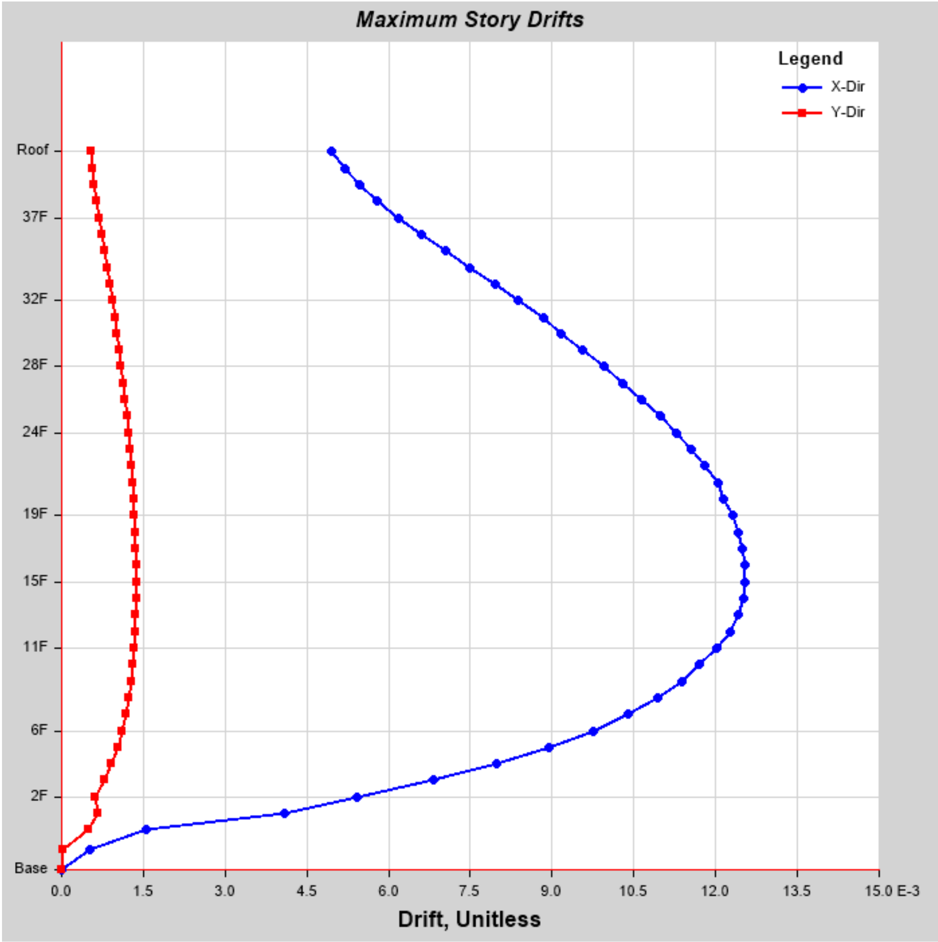
Figures 5.6 and 5.7 shows the story drift, where horizontal axis represent drift and vertical axis represent the stories; blue curve state the story drift implying in X-direction and red curve in Y-direction. Also comparison between maximum story drift under static and dynamic loadings in X and Y direction are clearly shown in Tables 5.5 and 5.6 respectively.

From figures, it is clearly seen that the story drift form a parabolic shape with zero drift at bottom, increases toward close to one-third height of the building and finally decreases at roof. Under dynamic loading curve start from base with zero value and sharply rises to 12^{th} floor and then gradually decreases to roof in both X & Y direction. It means that the incremental ratio of story displacement is large up to 12^{th} floor and then smaller compared to those floors. On the other hand, under static loading curve start from base with zero value and sharply rises to 14^{th} floor and then gradually decreases to roof top in both X & Y direction. It means that the incremental ratio of story displacement is larger up to 14^{th} floor and then smaller small compared to those floors. At the same time, it is also found that the story drift due to earthquake in both X & Y direction, a minor story drift is found in other direction; because of cyclic loading behavior of earthquake.

It is found the maximum drift ratio under dynamic loading 0.0175 at 14^{th} floor in X-direction, and under static loading 0.0125 at 15^{th} floor in X-direction, where the allowable drift ratio is 0.020. Hence, the story drift is larger under the dynamic loading of the structure.

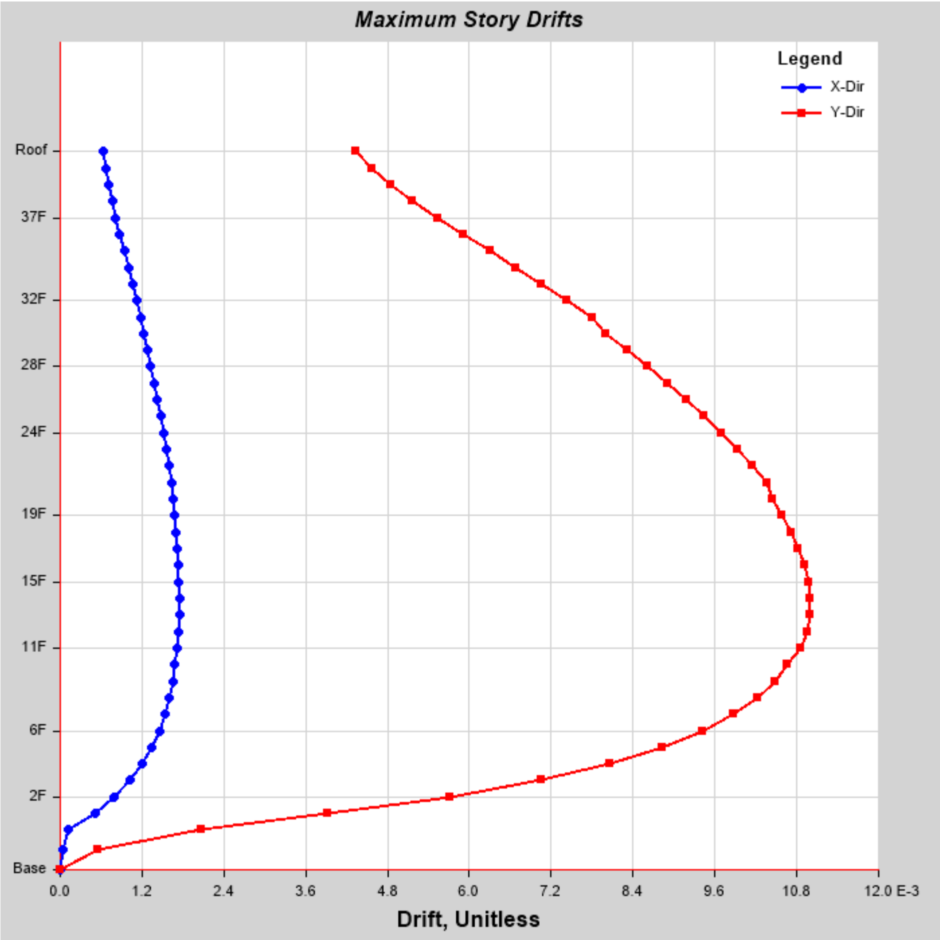


(a) Dynamic Loading

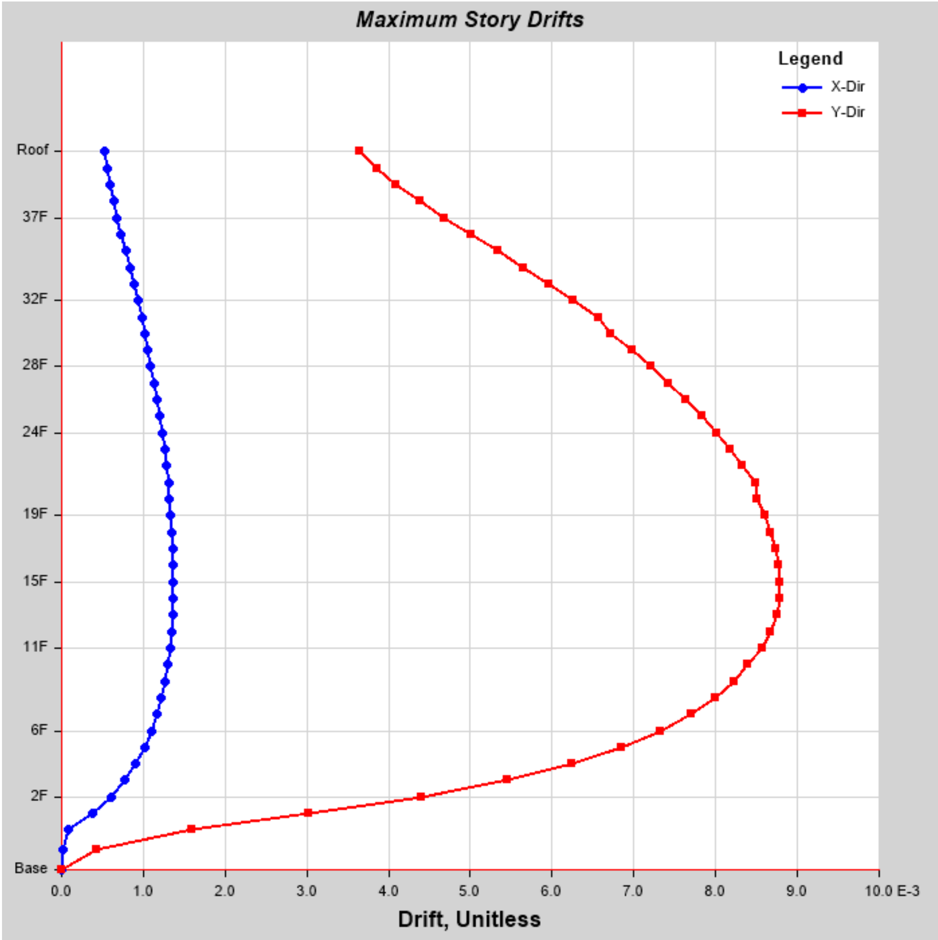


(b) Static Loading

Figure 5.6: Maximum Story Drift due to Earthquake loads in global X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.7: Maximum Story Drift due to Earthquake loads in global Y-direction

Table 5.5: Maximum story drift due to Earthquake in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Drift Ratio (Unitless)	Storey Drift Ratio (Unitless)
Roof	434.00	0.006339	0.004962
40F	424.00	0.006626	0.005189
39F	414.00	0.006966	0.005456
38F	404.00	0.007403	0.005797
37F	394.00	0.007905	0.006184
36F	384.00	0.008454	0.006604
35F	374.00	0.009034	0.007045
34F	364.00	0.009632	0.007494
33F	354.00	0.010239	0.007944
32F	344.00	0.010835	0.00838
31F	334.00	0.011468	0.008839
30F	324.00	0.011934	0.009164
29F	314.00	0.012504	0.009566
28F	304.00	0.013046	0.009941
27F	294.00	0.013579	0.010303
26F	284.00	0.014096	0.010649
25F	274.00	0.014592	0.010974
24F	264.00	0.015064	0.011276
23F	254.00	0.015508	0.011553
22F	244.00	0.015913	0.011797
21F	234.00	0.016332	0.012046
20F	224.00	0.01656	0.012153
19F	214.00	0.016841	0.012305
18F	204.00	0.017075	0.012418
17F	194.00	0.017268	0.0125
16F	184.00	0.017412	0.012545
15F	174.00	0.017498	0.012549
14F	164.00	0.017519	0.012506
13F	154.00	0.017465	0.01241
12F	144.00	0.017323	0.012254
11F	134.00	0.017079	0.012027
10F	124.00	0.016683	0.0117
9F	114.00	0.016275	0.011374
8F	104.00	0.015713	0.010942
7F	94.00	0.014993	0.010406
6F	84.00	0.01409	0.009748
5F	74.00	0.012973	0.008949
4F	64.00	0.011606	0.007984
3F	54.00	0.009946	0.006823
2F	44.00	0.007919	0.005417
1F	34.00	0.005994	0.00408
GF	24.00	0.00228	0.001542
Basement Floor	12.00	0.00076	0.000511
Base	0.00	0	0

Table 5.6: Maximum story drift due to Earthquake in Y-direction.

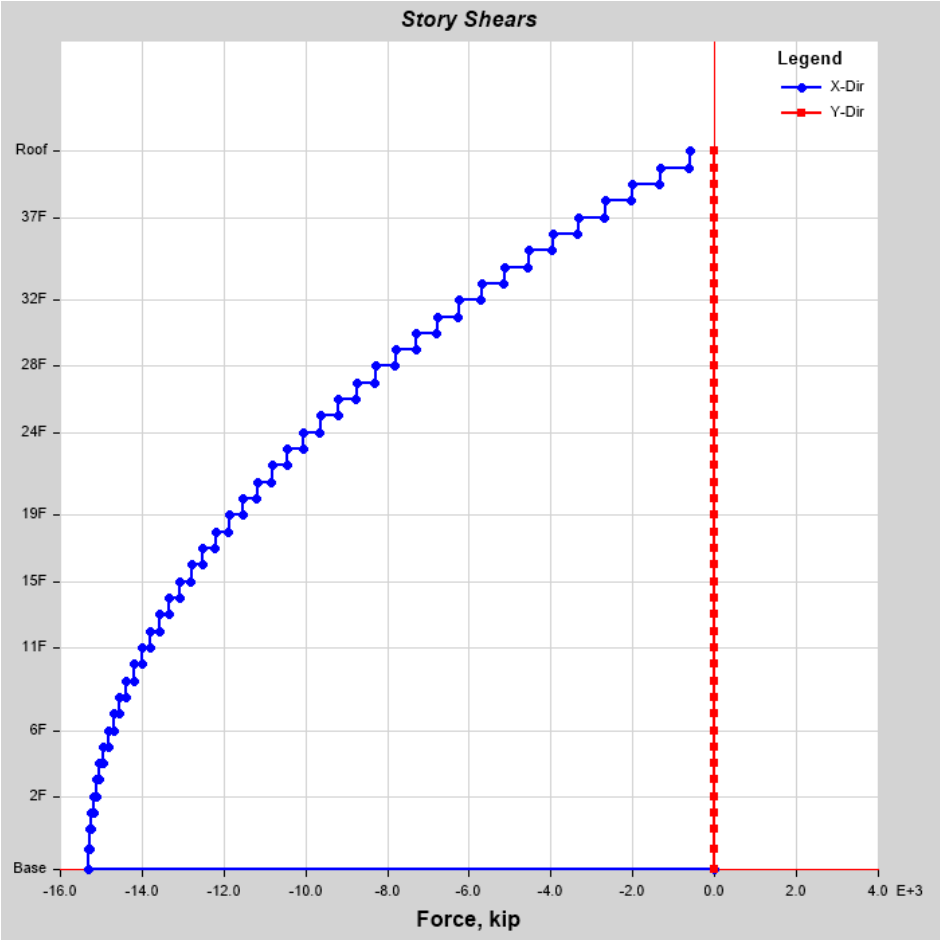
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Drift Ratio (Unitless)	Storey Drift Ratio (Unitless)
Roof	434.00	0.000634	0.003652
40F	424.00	0.000669	0.003864
39F	414.00	0.000709	0.004096
38F	404.00	0.000759	0.004376
37F	394.00	0.000815	0.004683
36F	384.00	0.000876	0.005003
35F	374.00	0.000938	0.005328
34F	364.00	0.001	0.00565
33F	354.00	0.001062	0.005963
32F	344.00	0.001122	0.006257
31F	334.00	0.001186	0.006571
30F	324.00	0.001224	0.006714
29F	314.00	0.001279	0.006971
28F	304.00	0.00133	0.007203
27F	294.00	0.001379	0.007426
26F	284.00	0.001427	0.007636
25F	274.00	0.001473	0.007832
24F	264.00	0.001516	0.008012
23F	254.00	0.001557	0.008177
22F	244.00	0.001594	0.008324
21F	234.00	0.001634	0.008482
20F	224.00	0.001651	0.008512
19F	214.00	0.001677	0.008605
18F	204.00	0.001699	0.008678
17F	194.00	0.001718	0.008734
16F	184.00	0.001733	0.008772
15F	174.00	0.001743	0.008788
14F	164.00	0.001748	0.008781
13F	154.00	0.001746	0.008746
12F	144.00	0.001737	0.008677
11F	134.00	0.001719	0.008571
10F	124.00	0.001684	0.008391
9F	114.00	0.00165	0.008229
8F	104.00	0.001601	0.008003
7F	94.00	0.001536	0.007708
6F	84.00	0.001451	0.00733
5F	74.00	0.001342	0.006847
4F	64.00	0.001203	0.006232
3F	54.00	0.001026	0.005444
2F	44.00	0.000798	0.004407
1F	34.00	0.000504	0.003015
GF	24.00	0.000118	0.00159
Basement Floor	12.00	0.000033	0.000424
Base	0.00	0	0

➢ Story Shear

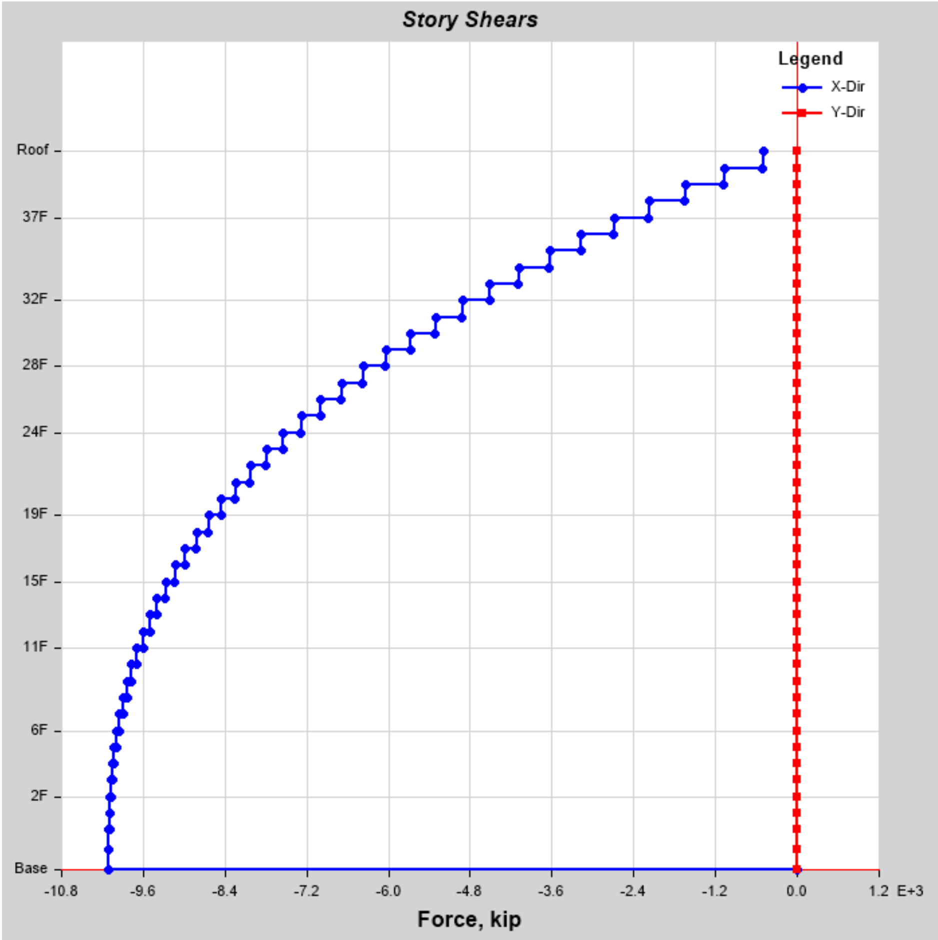
Figures 5.8 and 5.9 illustrated below provide the information about story shear, where horizontal axis represent story shear in kips and vertical axis represent the stories; blue curve state the response due to story shear implying in X-direction and red curves in Y-direction. Also comparison between responses about the story shear under static and dynamic loadings in X and Y direction are clearly shown in Tables 5.7 and 5.8 respectively.

From figures, it is clearly seen that the response curves are different under dynamic and static loadings in both X and Y direction. It is observed that the maximum shear at base and gradually decrease at up to top. Shear force is decreasing with respect to increase of height. The negative value of shear force indicates the resisting shear force that resist the positive shear in the corresponding direction. It is also shown that structure under dynamic loading have to be resist greater story shear force compared to the static loading.

We found the maximum story shear to resist the structure under dynamic loading 15300.36 kips, while under static loading structure have to be resist only 10108.947 kips.

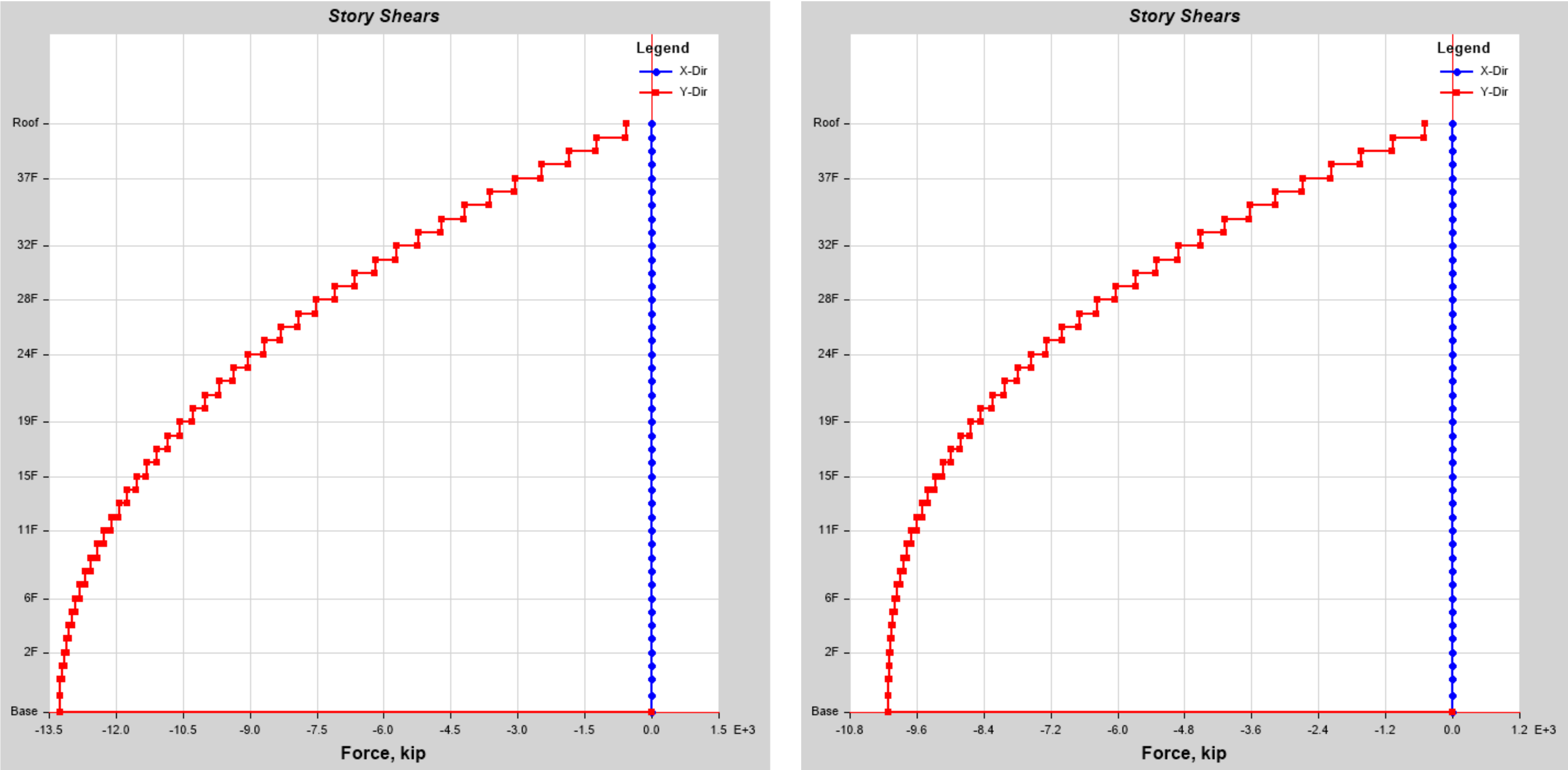


(a) Dynamic Loading



(b) Static Loading

Figure 5.8: Story Shear due to Earthquake loads in global X-direction



(a) Dynamic Loading

(b) Static Loading

Figure 5.9: Story Shear due to Earthquake loads in global Y-direction

Table 5.7: Maximum Story Shear due to Earthquake in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Shear resisted by each Storey (kips)	Shear resisted by each Storey (kips)
Roof	434.00	-638.778	-516.832
40F	424.00	-1347.386	-1095.415
39F	414.00	-2034.471	-1650.992
38F	404.00	-2700.406	-2183.949
37F	394.00	-3345.54	-2694.675
36F	384.00	-3970.206	-3183.56
35F	374.00	-4574.719	-3651
34F	364.00	-5159.386	-4097.392
33F	354.00	-5724.5	-4523.137
32F	344.00	-6270.346	-4928.64
31F	334.00	-6797.206	-5314.307
30F	324.00	-7311.883	-5685.267
29F	314.00	-7815.739	-6042.53
28F	304.00	-8300.911	-6380.667
27F	294.00	-8767.667	-6700.114
26F	284.00	-9216.265	-7001.31
25F	274.00	-9646.954	-7284.7
24F	264.00	-10059.972	-7550.733
23F	254.00	-10455.551	-7799.862
22F	244.00	-10833.909	-8032.549
21F	234.00	-11195.259	-8249.257
20F	224.00	-11545.77	-8453.955
19F	214.00	-11886.439	-8647.357
18F	204.00	-12210.219	-8825.733
17F	194.00	-12517.121	-8989.501
16F	184.00	-12807.3	-9139.178
15F	174.00	-13080.891	-9275.291
14F	164.00	-13338.001	-9398.375
13F	154.00	-13578.715	-9508.974
12F	144.00	-13803.091	-9607.643
11F	134.00	-14011.159	-9694.949
10F	124.00	-14206.186	-9772.77
9F	114.00	-14388.048	-9841.51
8F	104.00	-14552.908	-9900.316
7F	94.00	-14700.609	-9949.836
6F	84.00	-14830.941	-9990.738
5F	74.00	-14943.644	-10023.711
4F	64.00	-15038.42	-10049.472
3F	54.00	-15114.962	-10068.771
2F	44.00	-15173.031	-10082.396
1F	34.00	-15244.721	-10098.224
GF	24.00	-15282.389	-10106.335
Basement Floor	12.00	-15300.36	-10108.947
Base	0.00	0	0

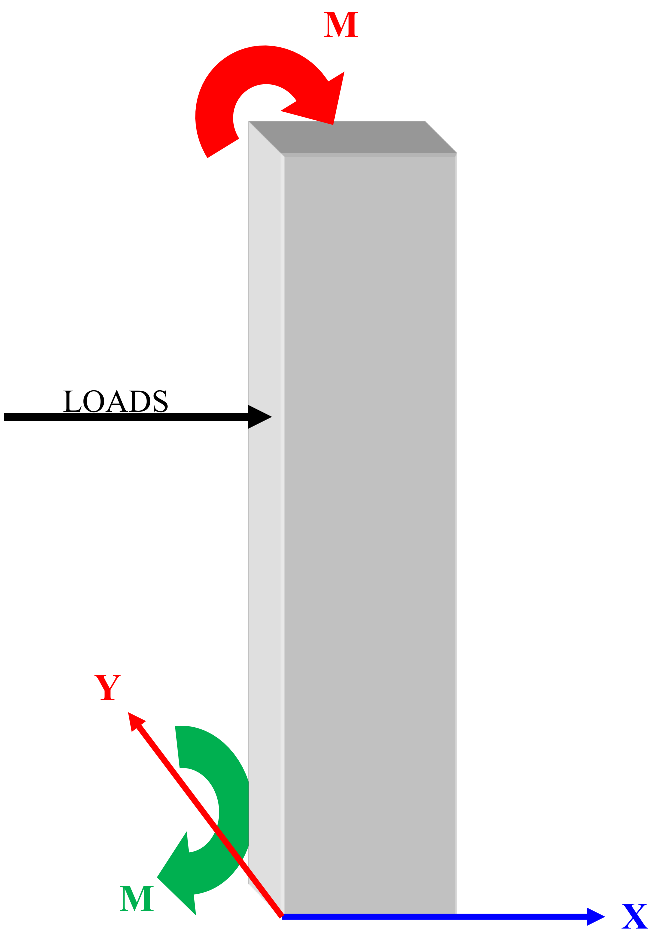
Table 5.8: Maximum Story Shear due to Earthquake in Y-direction

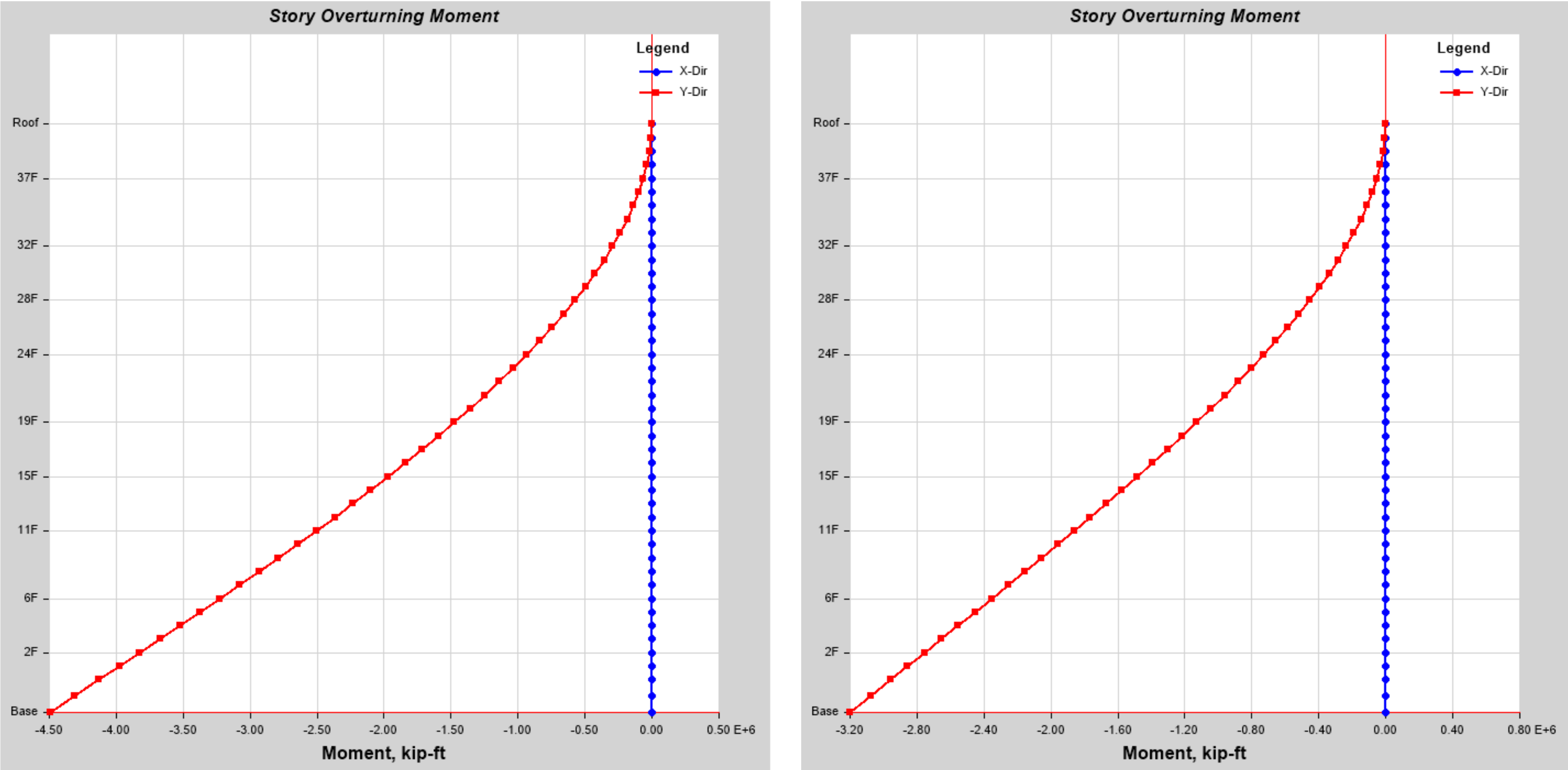
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Shear resisted by each Storey (kips)	Shear resisted by each Storey (kips)
Roof	434.00	-590.272	-516.811
40F	424.00	-1247.105	-1095.372
39F	414.00	-1881.76	-1650.927
38F	404.00	-2494.601	-2183.861
37F	394.00	-3085.978	-2694.565
36F	384.00	-3656.233	-3183.428
35F	374.00	-4205.706	-3650.847
34F	364.00	-4734.728	-4097.219
33F	354.00	-5243.632	-4522.944
32F	344.00	-5732.75	-4928.426
31F	334.00	-6202.418	-5314.074
30F	324.00	-6658.758	-5685.014
29F	314.00	-7103.02	-6042.259
28F	304.00	-7528.339	-6380.379
27F	294.00	-7935.068	-6699.809
26F	284.00	-8323.554	-7000.988
25F	274.00	-8694.148	-7284.363
24F	264.00	-9047.197	-7550.381
23F	254.00	-9383.046	-7799.496
22F	244.00	-9702.042	-8032.169
21F	234.00	-10004.527	-8248.863
20F	224.00	-10295.805	-8453.548
19F	214.00	-10576.814	-8646.938
18F	204.00	-10841.887	-8825.303
17F	194.00	-11091.239	-8989.061
16F	184.00	-11325.21	-9138.729
15F	174.00	-11544.128	-9274.833
14F	164.00	-11748.311	-9397.908
13F	154.00	-11938.063	-9508.499
12F	144.00	-12113.671	-9607.162
11F	134.00	-12275.402	-9694.462
10F	124.00	-12426.02	-9772.278
9F	114.00	-12565.645	-9841.012
8F	104.00	-12691.576	-9899.814
7F	94.00	-12803.937	-9949.331
6F	84.00	-12902.793	-9990.229
5F	74.00	-12988.14	-10023.2
4F	64.00	-13059.903	-10048.96
3F	54.00	-13117.93	-10068.258
2F	44.00	-13162.046	-10081.882
1F	34.00	-13216.642	-10097.709
GF	24.00	-13245.585	-10105.82
Basement Floor	12.00	-13259.305	-10108.432
Base	0.00	0	0

➢ Story Overturning Moment

Figures 5.10 and 5.11 illustrated the story overturning moment, where the horizontal axis represent overturning moment in kip-ft. and vertical axis represent the stories; blue curve state the response due to overturning moment implying in X-direction and red curve in Y-direction. Also comparison between the responses about story overturning moment under static and dynamic loadings in both X and Y direction are clearly shown in Tables 5.9 and 5.10 respectively.

From figures, it is clearly seen that the curve start from base with its peak value and sharply goes down to roof top in both X & Y direction and under dynamic loading structure can withstand greater story overturning compared to static loading. It is observed that due to earthquake force in X-direction, the whole structure will resist its overturn with respect to Y-axis and creates a resisting overturning moment M_R with respect to Y-axis. Similar case can be explained for loads in Y-direction. The value is negative in X-direction means the resisting overturning moment and it is at negative quadrant, whereas positive value in Y-direction means the resisting overturning moment at positive quadrant.

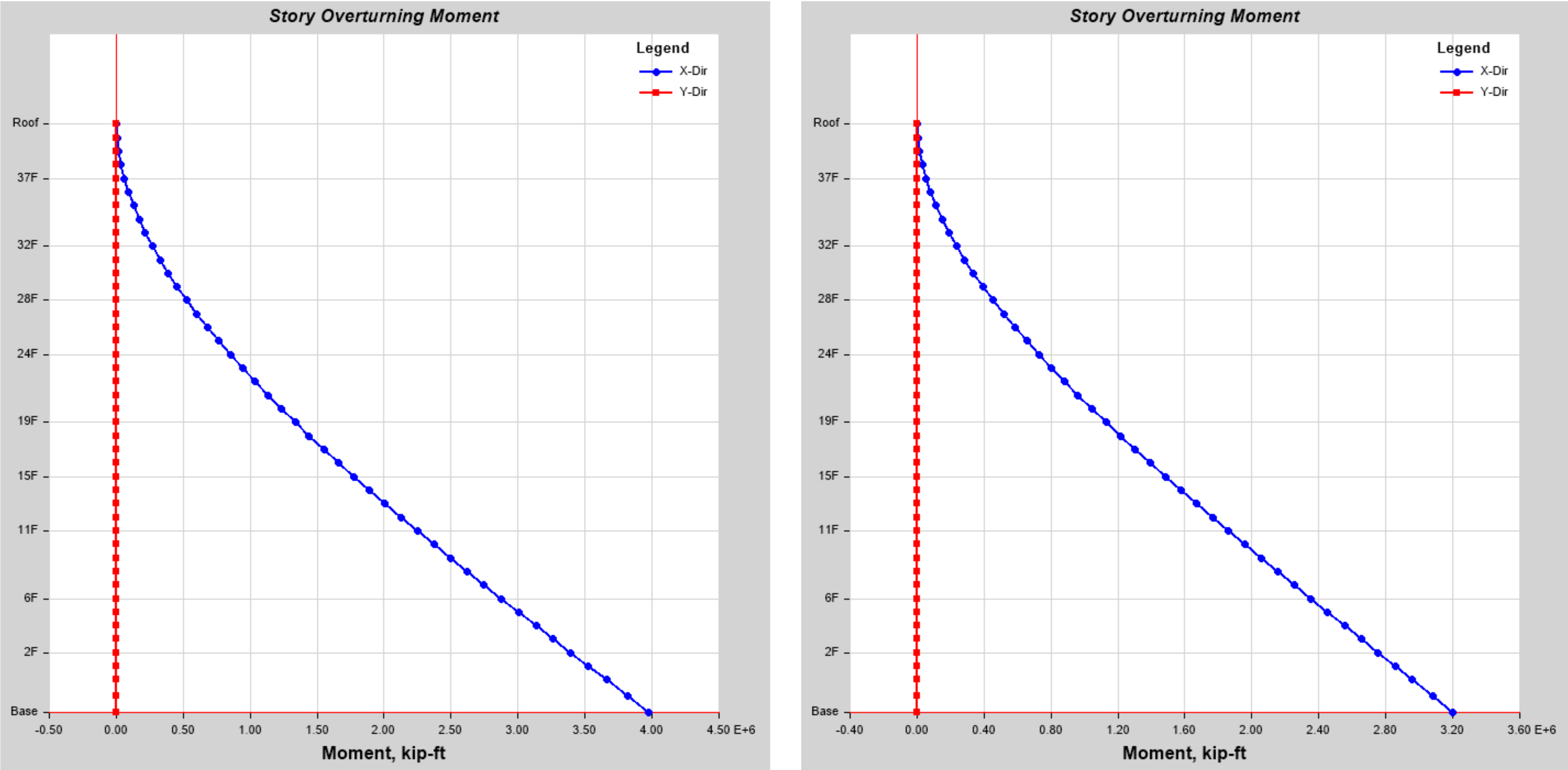




(a) Dynamic Loading

(b) Static Loading

Figure 5.10: Story Overturning Moments due to Earthquake loads in global X-direction



(a) Dynamic Loading (b) Static Loading

Figure 5.11: Story Overturning Moments due to Earthquake loads in global Y-direction

Table 5.9: Story Overturning Moments due to Earthquake in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Overturning Moments (kip-ft)	Storey Overturning Moments (kip-ft)
Roof	434.00	-92.33	0.00
40F	424.00	-6336.39	-5049.96
39F	414.00	-19670.93	-15890.35
38F	404.00	-39880.63	-32291.04
37F	394.00	-66753.94	-54025.75
36F	384.00	-100082.77	-80872.08
35F	374.00	-139662.38	-112611.56
34F	364.00	-185291.19	-149029.66
33F	354.00	-236770.67	-189915.81
32F	344.00	-293905.25	-235063.48
31F	334.00	-356502.19	-284270.15
30F	324.00	-424371.58	-337337.39
29F	314.00	-497391.55	-394118.05
28F	304.00	-575453.83	-454475.08
27F	294.00	-658371.53	-518217.14
26F	284.00	-745960.44	-585157.24
25F	274.00	-838038.95	-655112.78
24F	264.00	-934427.91	-727905.63
23F	254.00	-1034951.00	-803362.12
22F	244.00	-1139433.00	-881313.14
21F	234.00	-1247702.00	-961594.16
20F	224.00	-1359587.00	-1044045.00
19F	214.00	-1474982.00	-1128546.00
18F	204.00	-1593786.00	-1214984.00
17F	194.00	-1715832.00	-1303209.00
16F	184.00	-1840950.00	-1393074.00
15F	174.00	-1968973.00	-1484438.00
14F	164.00	-2099735.00	-1577166.00
13F	154.00	-2233071.00	-1671127.00
12F	144.00	-2368818.00	-1766197.00
11F	134.00	-2506812.00	-1862255.00
10F	124.00	-2646890.00	-1959188.00
9F	114.00	-2788921.00	-2056902.00
8F	104.00	-2932775.00	-2155305.00
7F	94.00	-3078280.00	-2254298.00
6F	84.00	-3225266.00	-2353787.00
5F	74.00	-3373558.00	-2453687.00
4F	64.00	-3522981.00	-2553918.00
3F	54.00	-3673355.00	-2654409.00
2F	44.00	-3824498.00	-2755093.00
1F	34.00	-3976226.00	-2855914.00
GF	24.00	-4128674.00	-2956895.00
Basement Floor	12.00	-4312052.00	-3078166.00
Base	0.00	-4495533.00	-3199472.00

Table 5.10: Story Overturning Moments due to Earthquake in Y-direction

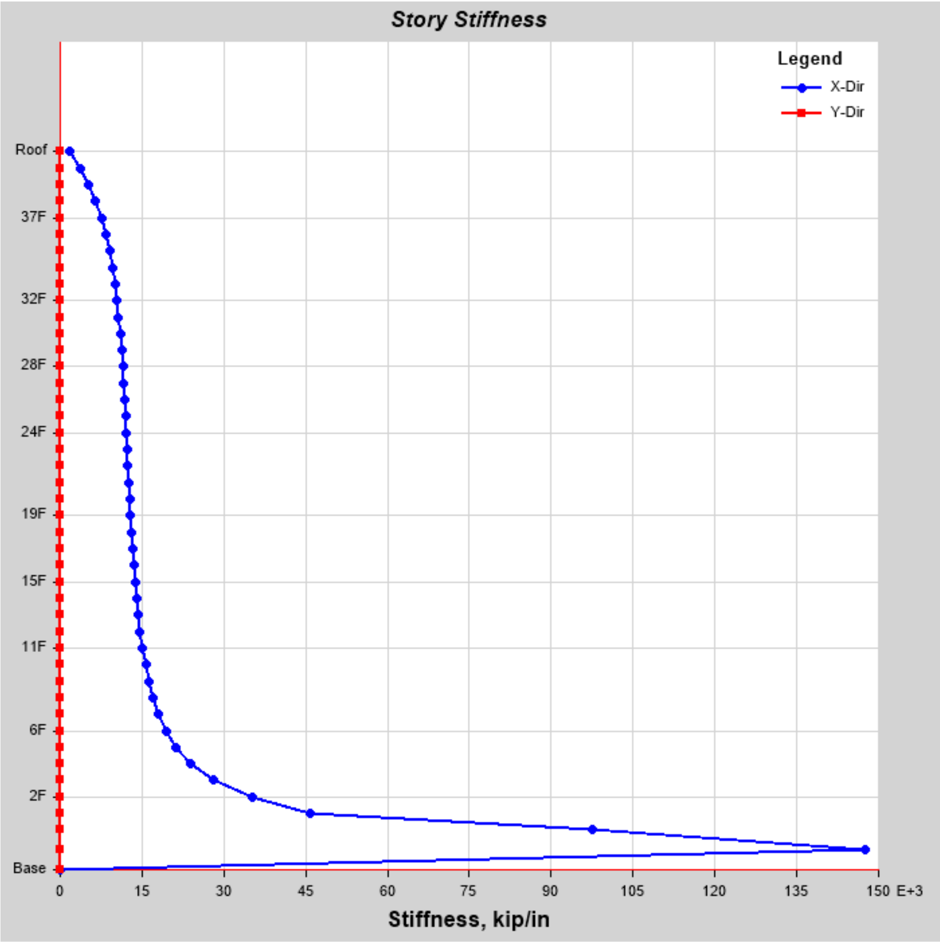
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Overturning Moments (kip-ft)	Storey Overturning Moments (kip-ft)
Roof	434.00	55.68	0.00
40F	424.00	5824.73	5049.76
39F	414.00	18166.54	15889.72
38F	404.00	36859.26	32289.75
37F	394.00	61684.68	54023.58
36F	384.00	92428.10	80868.81
35F	374.00	128878.24	112606.97
34F	364.00	170827.19	149023.54
33F	354.00	218070.39	189907.96
32F	344.00	270406.59	235053.69
31F	334.00	327637.88	284258.23
30F	324.00	389569.68	337323.14
29F	314.00	456068.64	394101.27
28F	304.00	527013.92	454455.60
27F	294.00	602216.02	518194.77
26F	284.00	681488.96	585131.81
25F	274.00	764650.27	655084.14
24F	264.00	851520.95	727873.62
23F	254.00	941925.48	803326.59
22F	244.00	1035691.80	881273.94
21F	234.00	1132651.31	961551.16
20F	224.00	1232638.85	1043998.38
19F	214.00	1335542.27	1128495.41
18F	204.00	1441258.81	1214929.22
17F	194.00	1549629.06	1303149.46
16F	184.00	1660495.73	1393009.95
15F	174.00	1773704.96	1484369.72
14F	164.00	1889106.16	1577093.03
13F	154.00	2006551.93	1671049.48
12F	144.00	2125897.89	1766114.14
11F	134.00	2247002.56	1862167.62
10F	124.00	2369727.13	1959096.18
9F	114.00	2493960.41	2056804.89
8F	104.00	2619592.46	2155202.81
7F	94.00	2746486.30	2254190.52
6F	84.00	2874506.21	2353675.04
5F	74.00	3003517.14	2453570.08
4F	64.00	3133384.02	2553796.23
3F	54.00	3263971.02	2654281.26
2F	44.00	3395140.82	2754960.41
1F	34.00	3526754.34	2855776.81
GF	24.00	3658916.28	2956752.35
Basement Floor	12.00	3817847.73	3078016.96
Base	0.00	3976962.67	3199316.56

➢ **Story Stiffness**

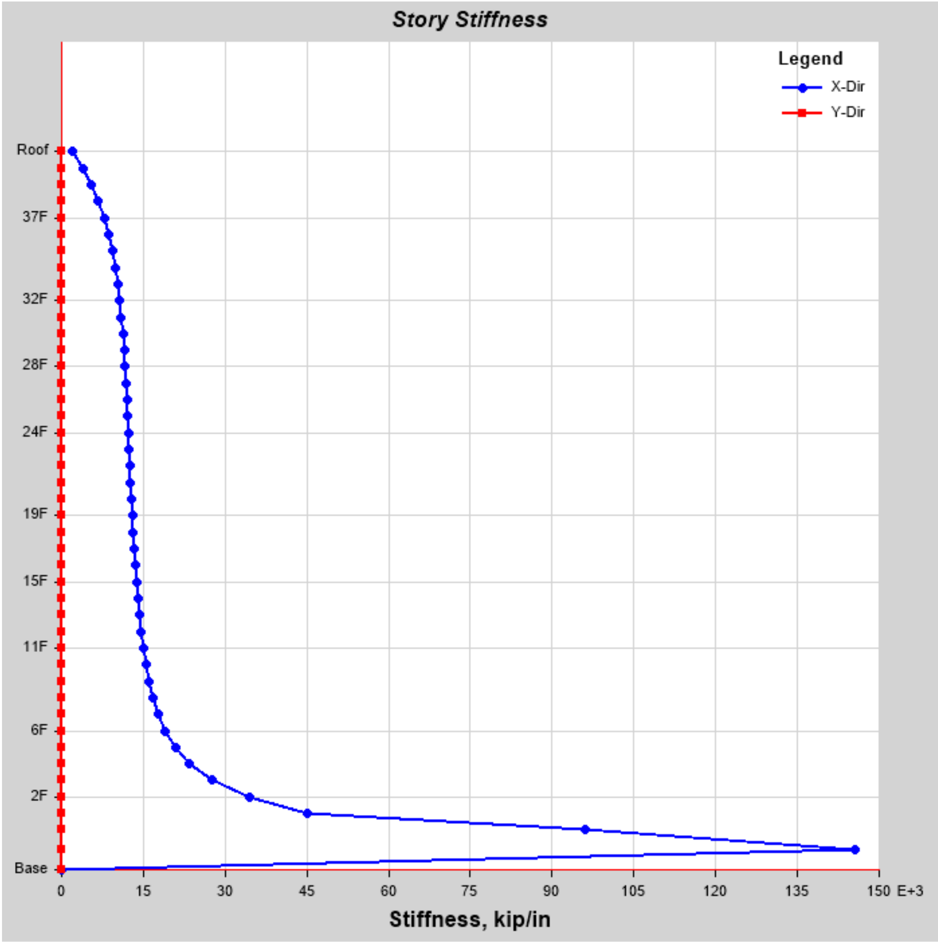
Figures 5.12 and 5.13 illustrated the response of story stiffness, where the horizontal axis represent story stiffness in kip/inch and vertical axis represent the stories; blue curve state the story stiffness implying in X-direction and red curve in Y-direction. Also comparison between the responses about story stiffness under static and dynamic loadings in both X and Y direction are clearly shown in Tables 5.11 and 5.12 respectively.

From figures, it is clearly seen that the story stiffness under both static & dynamic loading in both X and Y direction is almost close but not same. It is also see that the curve rapidly high at basement floor; after that the value decreases with large rate up to 2nd floor and then the decreases at a lower rate up to top. Its' indicate that up to 2nd floor of the building is more stiffer than others; because there is a concrete wall to the periphery of the building and also ramp connect in this level which makes their stronger and after ground floor the building floor plan is changed.

It is found that the maximum story stiffness under dynamic loading of the structure 177237.62 kip/in inch, while under static loading 176130.26 kip/inch.

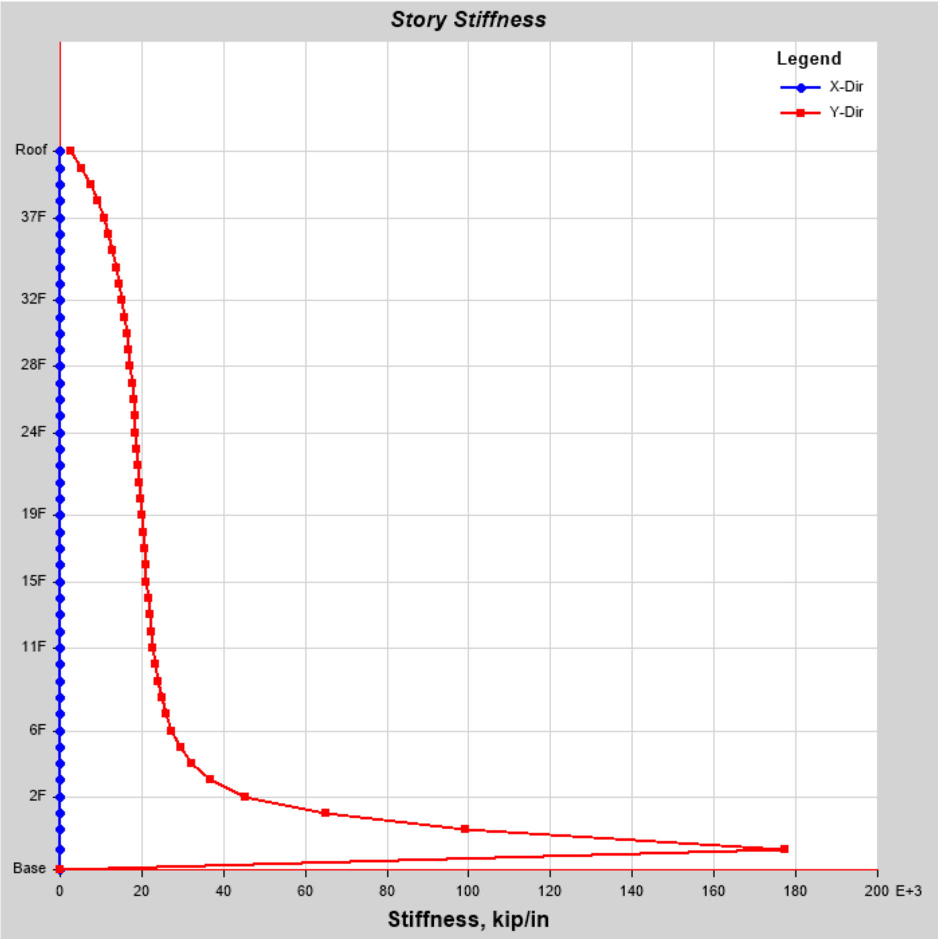


(a) Dynamic Loading

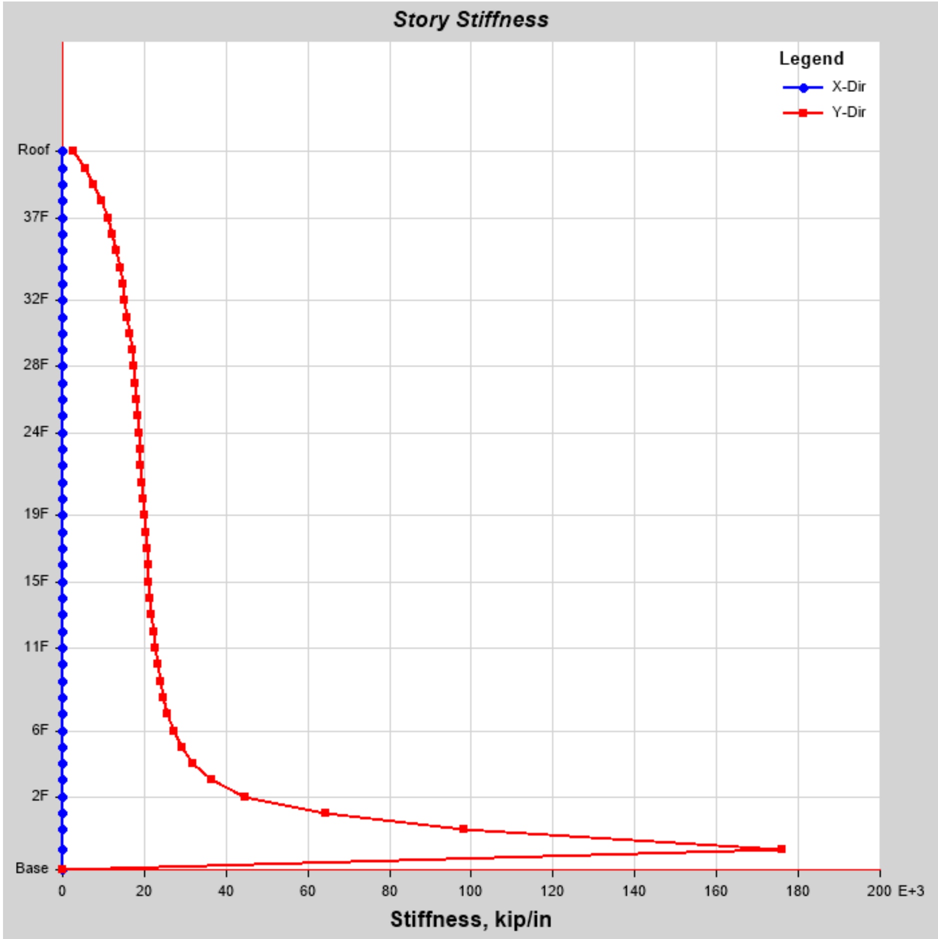


(b) Static Loading

Figure 5.12: Story Stiffness due to Earthquake loads in global X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.13: Story Stiffness due to Earthquake loads in global Y-direction

Table 5.11: Story Stiffness due to Earthquake in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Stiffness (kip/inch)	Storey Stiffness (kip/inch)
Roof	434.00	1813.57	1886.22
40F	424.00	3654.01	3818.01
39F	414.00	5218.70	5444.25
38F	404.00	6504.10	6767.12
37F	394.00	7541.93	7824.87
36F	384.00	8372.78	8663.55
35F	374.00	9037.80	9328.46
34F	364.00	9573.24	9858.79
33F	354.00	10010.96	10288.44
32F	344.00	10361.20	10628.93
31F	334.00	10698.01	10956.32
30F	324.00	11012.59	11259.48
29F	314.00	11257.80	11490.48
28F	304.00	11468.57	11687.42
27F	294.00	11649.63	11854.65
26F	284.00	11809.18	12000.52
25F	274.00	11953.72	12131.57
24F	264.00	12088.66	12253.31
23F	254.00	12219.16	12370.91
22F	244.00	12337.10	12477.21
21F	234.00	12494.65	12623.21
20F	224.00	12685.31	12796.42
19F	214.00	12861.70	12950.78
18F	204.00	13046.26	13114.34
17F	194.00	13244.00	13291.11
16F	184.00	13461.82	13487.71
15F	174.00	13707.94	13693.97
14F	164.00	13976.61	13926.87
13F	154.00	14276.53	14204.16
12F	144.00	14635.87	14540.32
11F	134.00	15073.60	14956.37
10F	124.00	15651.51	15505.18
9F	114.00	16256.96	16071.01
8F	104.00	17040.75	16815.13
7F	94.00	18050.51	17782.27
6F	84.00	19388.33	19072.85
5F	74.00	21227.01	20857.17
4F	64.00	23882.25	23446.34
3F	54.00	27998.02	27475.09
2F	44.00	35211.83	34557.76
1F	34.00	45908.38	45060.79
GF	24.00	97654.36	95945.44
Basement Floor	12.00	147517.07	145553.46
Base	0.00	0.00	0.00

Table 5.12: Story Stiffness due to Earthquake in Y-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Stiffness (kip/inch)	Storey Stiffness (kip/inch)
Roof	434.00	2636.21	2731.67
40F	424.00	5251.32	5448.28
39F	414.00	7457.39	7716.42
38F	404.00	9255.76	9547.25
37F	394.00	10717.98	11022.27
36F	384.00	11911.17	12215.83
35F	374.00	12896.53	13194.07
34F	364.00	13722.86	14009.00
33F	354.00	14438.98	14711.63
32F	344.00	15009.16	15266.93
31F	334.00	15677.91	15924.09
30F	324.00	16336.89	16569.42
29F	314.00	16789.19	17003.10
28F	304.00	17212.30	17409.29
27F	294.00	17585.57	17765.96
26F	284.00	17922.92	18087.32
25F	274.00	18232.68	18381.83
24F	264.00	18521.23	18656.04
23F	254.00	18797.13	18918.64
22F	244.00	19014.86	19126.69
21F	234.00	19336.28	19430.52
20F	224.00	19709.16	19796.25
19F	214.00	19987.74	20051.74
18F	204.00	20273.34	20317.98
17F	194.00	20560.15	20586.41
16F	184.00	20857.12	20865.62
15F	174.00	21173.42	21164.66
14F	164.00	21520.09	21463.22
13F	154.00	21881.03	21800.02
12F	144.00	22290.13	22193.16
11F	134.00	22771.64	22662.78
10F	124.00	23451.43	23321.64
9F	114.00	24094.74	23931.15
8F	104.00	24924.20	24729.16
7F	94.00	25994.42	25767.44
6F	84.00	27419.85	27159.02
5F	74.00	29395.51	29097.32
4F	64.00	32285.13	31943.42
3F	54.00	36862.66	36465.84
2F	44.00	45240.99	44762.58
1F	34.00	65134.63	64421.67
GF	24.00	99185.14	98174.04
Basement Floor	12.00	177237.62	176130.26
Base	0.00	0.00	0.00

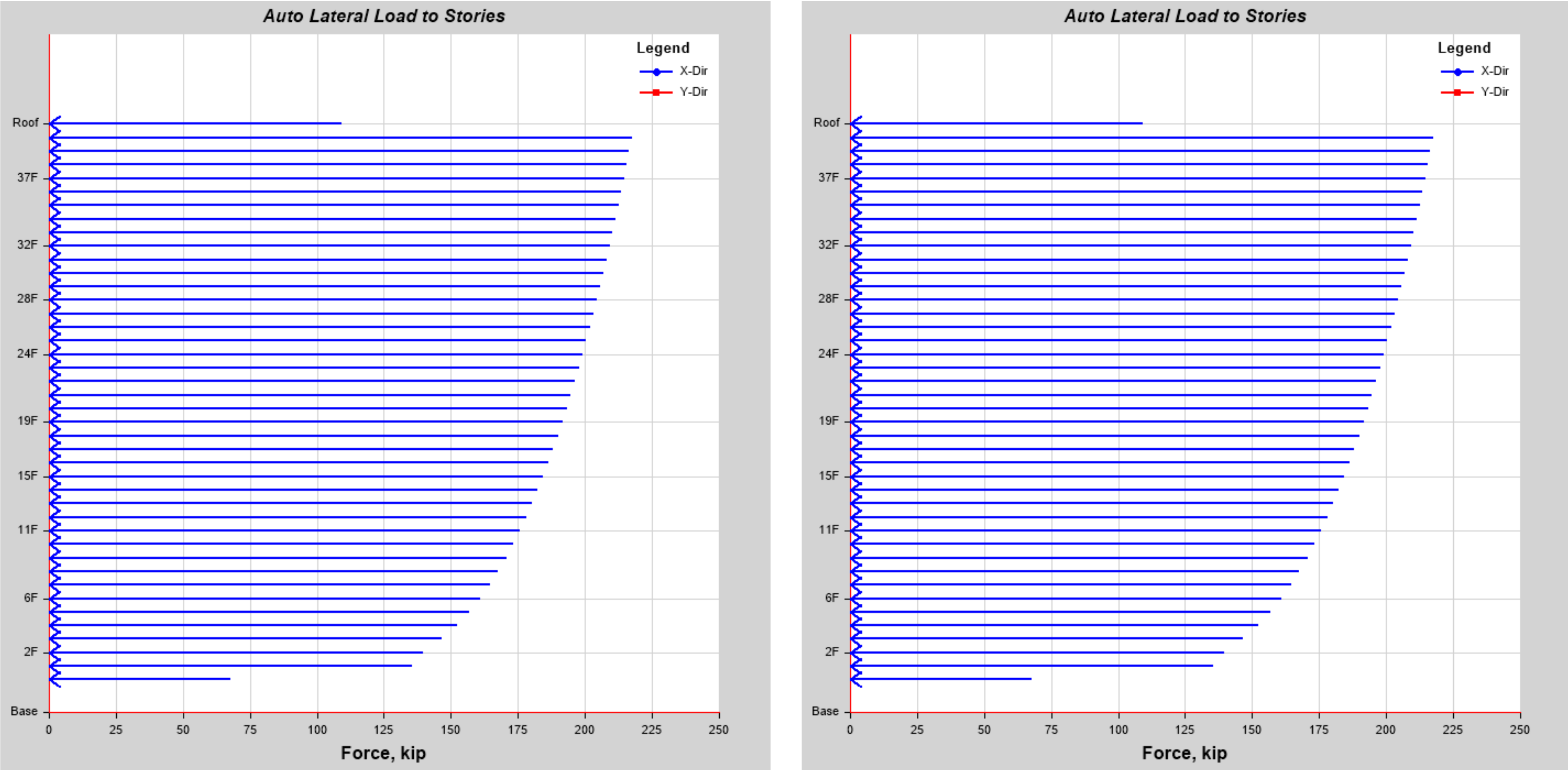
5.3.2 Response due to Wind

- **Auto Lateral Load to Stories**

Figures 5.14 and 5.15 illustrated the response for lateral loads to stories, where the horizontal axis represent lateral load in kips and vertical axis represent the stories; blue curve state the response due to lateral loads implying in X-direction and red curve in Y-direction. Also comparison between lateral loads resisting capacities under dynamic and static loadings in X and Y direction are clearly shown in Tables 5.13 and 5.14 respectively.

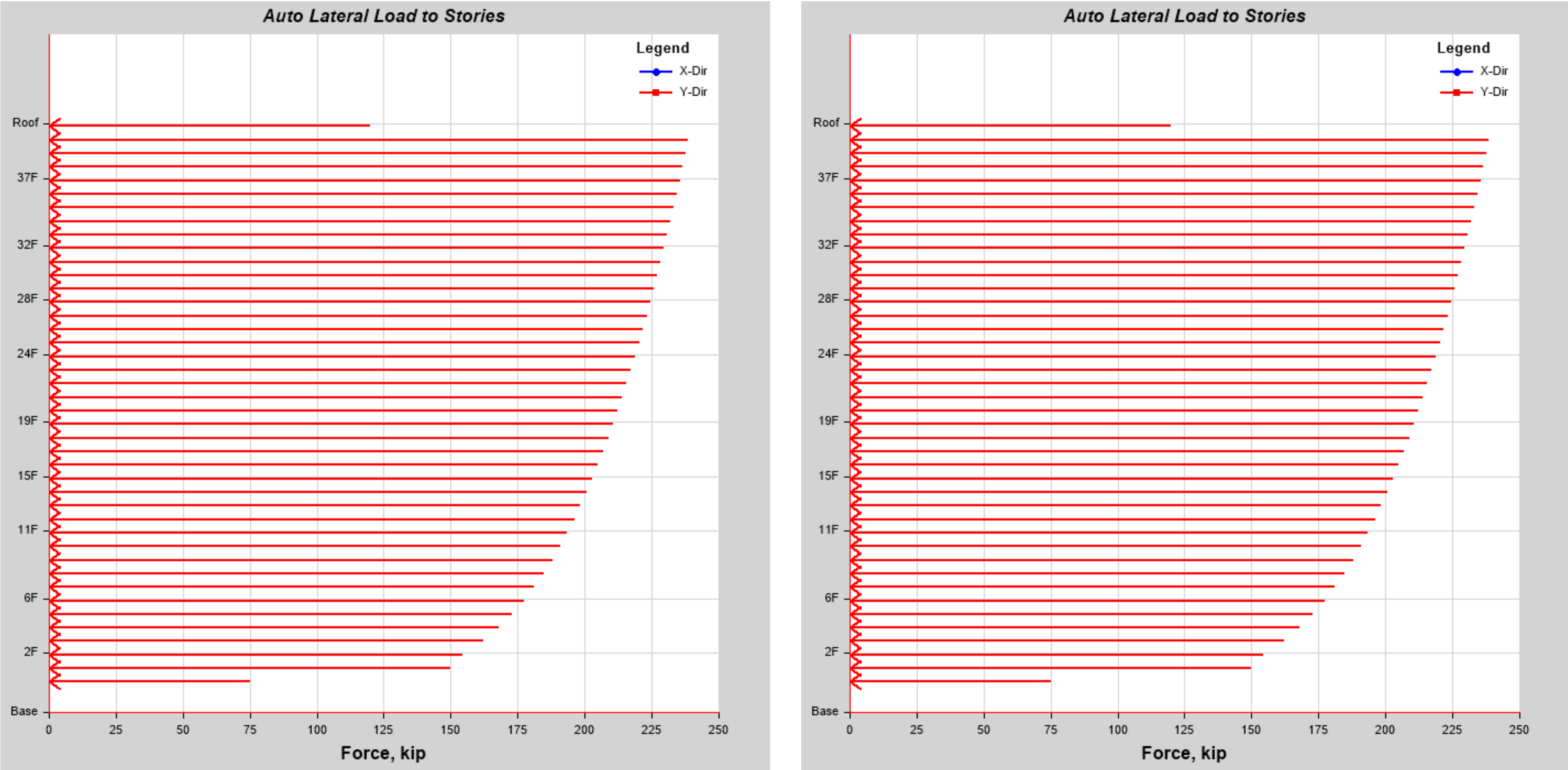
From figures, it is observe that the curves are same under both dynamic and static loadings while the value changes rapidly to ground floor to 1st floor, and then gradually increases to 40th floor, and after that the value is decreases suddenly at roof. The value at ground floor is so small because at this level wind speed is very small almost zero and tributary area is less compared to other story in both X & Y direction. The wind speed is high at roof top but the value is small because the less tributary area due to no parapet wall exist.

It is found the maximum lateral force due to wind at 40th floor under both static & dynamic loading 217.51 kips in X and 238.71 kips in Y direction.



(a) Dynamic Loading (b) Static Loading

Figure 5.14: Resisting loads by each story due to Wind in X-direction



(a) Dynamic Loading (b) Static Loading

Figure 5.15: Resisting loads by each story due to Wind in Y-direction

Table 5.13: Resisting loads by each story due to Wind in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Resisting Lateral Loads (kips)	Resisting Lateral Loads (kips)
Roof	434.00	109.077	109.077
40F	424.00	217.514	217.514
39F	414.00	216.539	216.539
38F	404.00	215.546	215.546
37F	394.00	214.535	214.535
36F	384.00	213.503	213.503
35F	374.00	212.451	212.451
34F	364.00	211.378	211.378
33F	354.00	210.281	210.281
32F	344.00	209.161	209.161
31F	334.00	208.015	208.015
30F	324.00	206.842	206.842
29F	314.00	205.641	205.641
28F	304.00	204.411	204.411
27F	294.00	203.148	203.148
26F	284.00	201.852	201.852
25F	274.00	200.519	200.519
24F	264.00	199.148	199.148
23F	254.00	197.735	197.735
22F	244.00	196.278	196.278
21F	234.00	194.772	194.772
20F	224.00	193.215	193.215
19F	214.00	191.6	191.6
18F	204.00	189.924	189.924
17F	194.00	188.18	188.18
16F	184.00	186.361	186.361
15F	174.00	184.458	184.458
14F	164.00	182.463	182.463
13F	154.00	180.363	180.363
12F	144.00	178.143	178.143
11F	134.00	175.788	175.788
10F	124.00	173.273	173.273
9F	114.00	170.571	170.571
8F	104.00	167.645	167.645
7F	94.00	164.443	164.443
6F	84.00	160.893	160.893
5F	74.00	156.887	156.887
4F	64.00	152.252	152.252
3F	54.00	146.677	146.677
2F	44.00	139.714	139.714
1F	34.00	135.41	135.41
GF	24.00	67.761	67.761
BF	12.00	0	0
Base	0.00	0	0

Table 5.14: Resisting loads by each story due to Wind in Y-direction

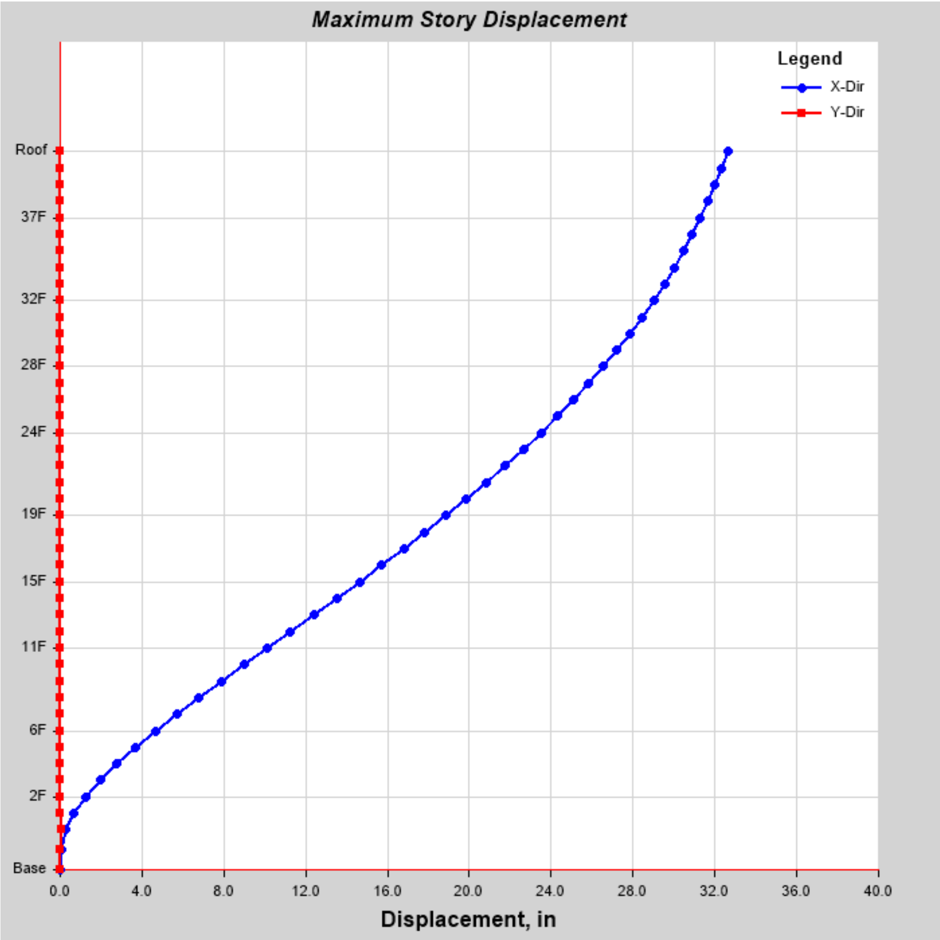
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Resisting Lateral Loads (kips)	Resisting Lateral Loads (kips)
Roof	434.00	119.7	119.7
40F	424.00	238.707	238.707
39F	414.00	237.651	237.651
38F	404.00	236.575	236.575
37F	394.00	235.479	235.479
36F	384.00	234.362	234.362
35F	374.00	233.222	233.222
34F	364.00	232.059	232.059
33F	354.00	230.871	230.871
32F	344.00	229.657	229.657
31F	334.00	228.416	228.416
30F	324.00	227.146	227.146
29F	314.00	225.845	225.845
28F	304.00	224.511	224.511
27F	294.00	223.144	223.144
26F	284.00	221.739	221.739
25F	274.00	220.296	220.296
24F	264.00	218.81	218.81
23F	254.00	217.28	217.28
22F	244.00	215.701	215.701
21F	234.00	214.07	214.07
20F	224.00	212.382	212.382
19F	214.00	210.634	210.634
18F	204.00	208.818	208.818
17F	194.00	206.928	206.928
16F	184.00	204.957	204.957
15F	174.00	202.896	202.896
14F	164.00	200.734	200.734
13F	154.00	198.459	198.459
12F	144.00	196.055	196.055
11F	134.00	193.503	193.503
10F	124.00	190.779	190.779
9F	114.00	187.852	187.852
8F	104.00	184.682	184.682
7F	94.00	181.213	181.213
6F	84.00	177.367	177.367
5F	74.00	173.028	173.028
4F	64.00	168.007	168.007
3F	54.00	161.966	161.966
2F	44.00	154.423	154.423
1F	34.00	149.76	149.76
GF	24.00	74.932	74.932
BF	12.00	0	0
Base	0.00	0	0

➢ Maximum Story Displacement

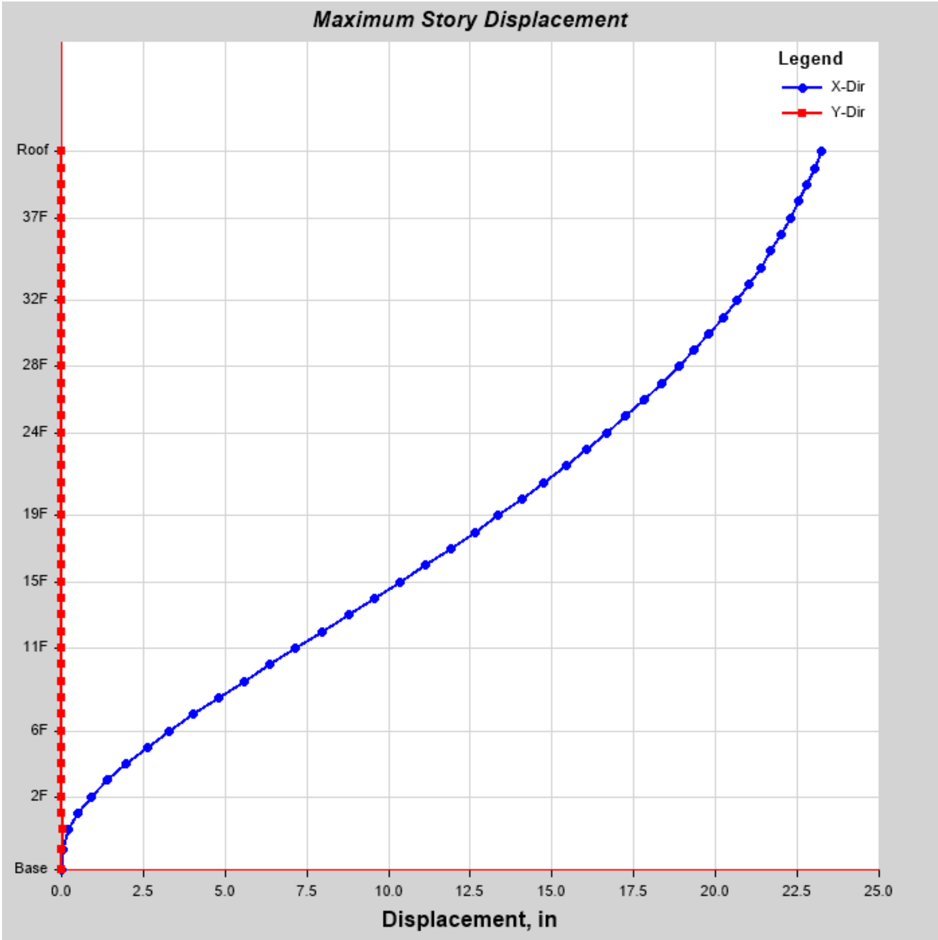
Figures 5.16 and 5.17 illustrated the maximum story displacement, where the horizontal axis represent displacement in inch and vertical axis represent the stories; blue curve state the story displacement implying in X-direction and red curve in Y-direction. Also comparison between maximum story displacement under dynamic and static loadings in X and Y direction are clearly shown in Tables 5.15 and 5.16 respectively.

From figures, it is clearly seen that the curve starts from base and sharply goes on top in both X & Y direction. The displacement curve under static loading of the structure fluctuates slightly from the displacement curve under dynamic loading of the structure in both X & Y direction.

It is found the maximum displacement of the structure larger under dynamic loading corresponding to the static loading in both X and Y direction. The maximum displacement of the structure is found under dynamic loading at roof top 32.62 inch in X-direction, while the maximum displacement under static loading at roof top 23.25 inch in X-direction.

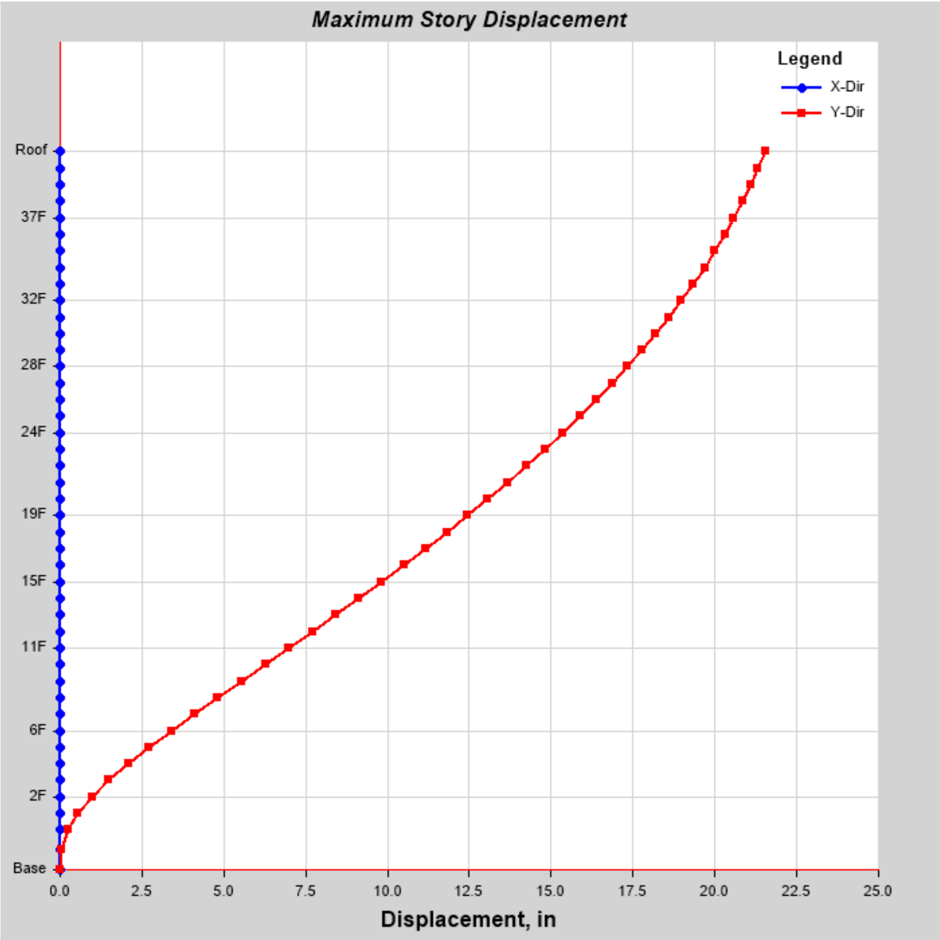


(a) Dynamic Loading

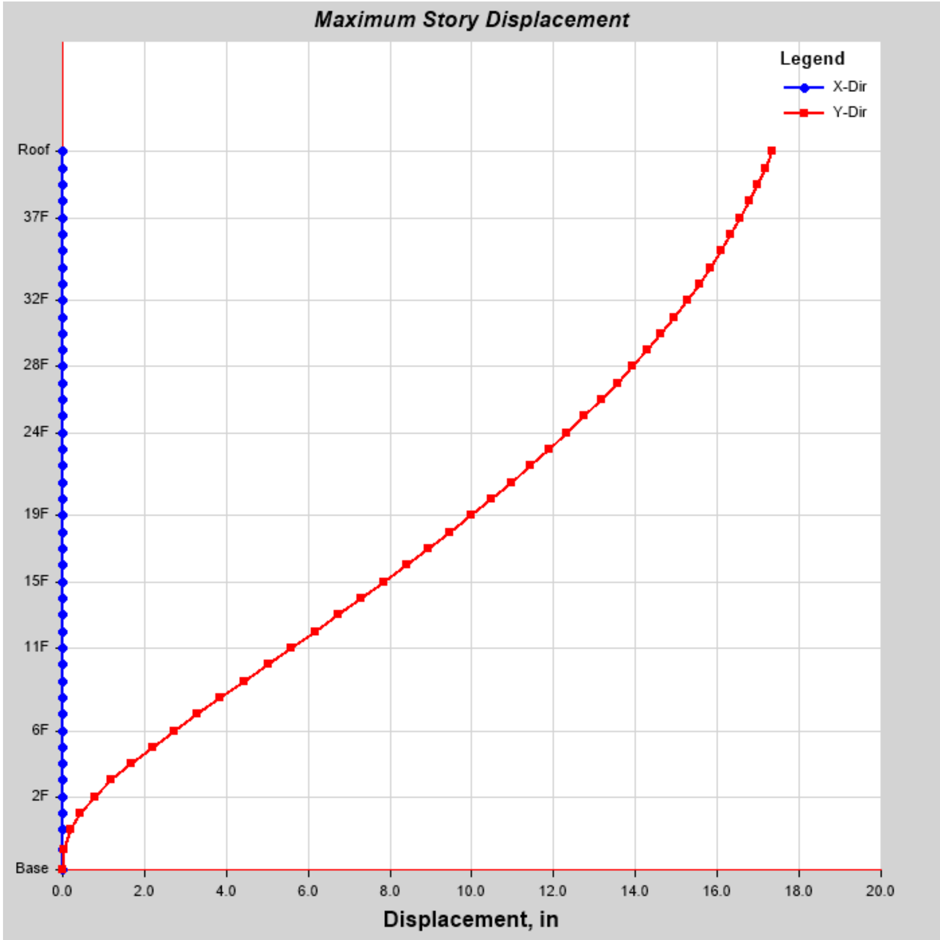


(b) Static Loading

Figure 5.16: Maximum Story Displacement due to Wind loads in global X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.17: Maximum Story Displacement due to Wind loads in global Y-direction

Table 5.15: Maximum story displacement due to Wind in X-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Drift Displacement (inch)	Storey Drift Displacement (inch)
Roof	434.00	32.618736	23.248798
40F	424.00	32.307911	23.025015
39F	414.00	31.983182	22.791061
38F	404.00	31.641311	22.544549
37F	394.00	31.276858	22.281539
36F	384.00	30.885826	21.999161
35F	374.00	30.464949	21.695101
34F	364.00	30.011668	21.367571
33F	354.00	29.524037	21.015242
32F	344.00	29.000573	20.637135
31F	334.00	28.440766	20.232977
30F	324.00	27.841569	19.800684
29F	314.00	27.210329	19.345667
28F	304.00	26.541462	18.863979
27F	294.00	25.835217	18.355914
26F	284.00	25.091121	17.821223
25F	274.00	24.309098	17.259942
24F	264.00	23.489331	16.672283
23F	254.00	22.632276	16.058646
22F	244.00	21.738613	15.419585
21F	234.00	20.809769	14.756174
20F	224.00	19.843788	14.067096
19F	214.00	18.851584	13.360143
18F	204.00	17.83118	12.633827
17F	194.00	16.784428	11.889508
16F	184.00	15.713472	11.1287
15F	174.00	14.621049	10.35332
14F	164.00	13.510522	9.565712
13F	154.00	12.385975	8.768717
12F	144.00	11.252317	7.965743
11F	134.00	10.11544	7.160882
10F	124.00	8.981924	6.35871
9F	114.00	7.863257	5.567249
8F	104.00	6.762467	4.788531
7F	94.00	5.69035	4.030116
6F	84.00	4.658574	3.300171
5F	74.00	3.680832	2.608308
4F	64.00	2.773198	1.965841
3F	54.00	1.954531	1.38609
2F	44.00	1.246915	0.884697
1F	34.00	0.677565	0.481005
GF	24.00	0.282425	0.200635
Basement Floor	12.00	0.071793	0.051041
Base	0.00	0	0

Table 5.16: Maximum story displacement due to Wind in Y-direction

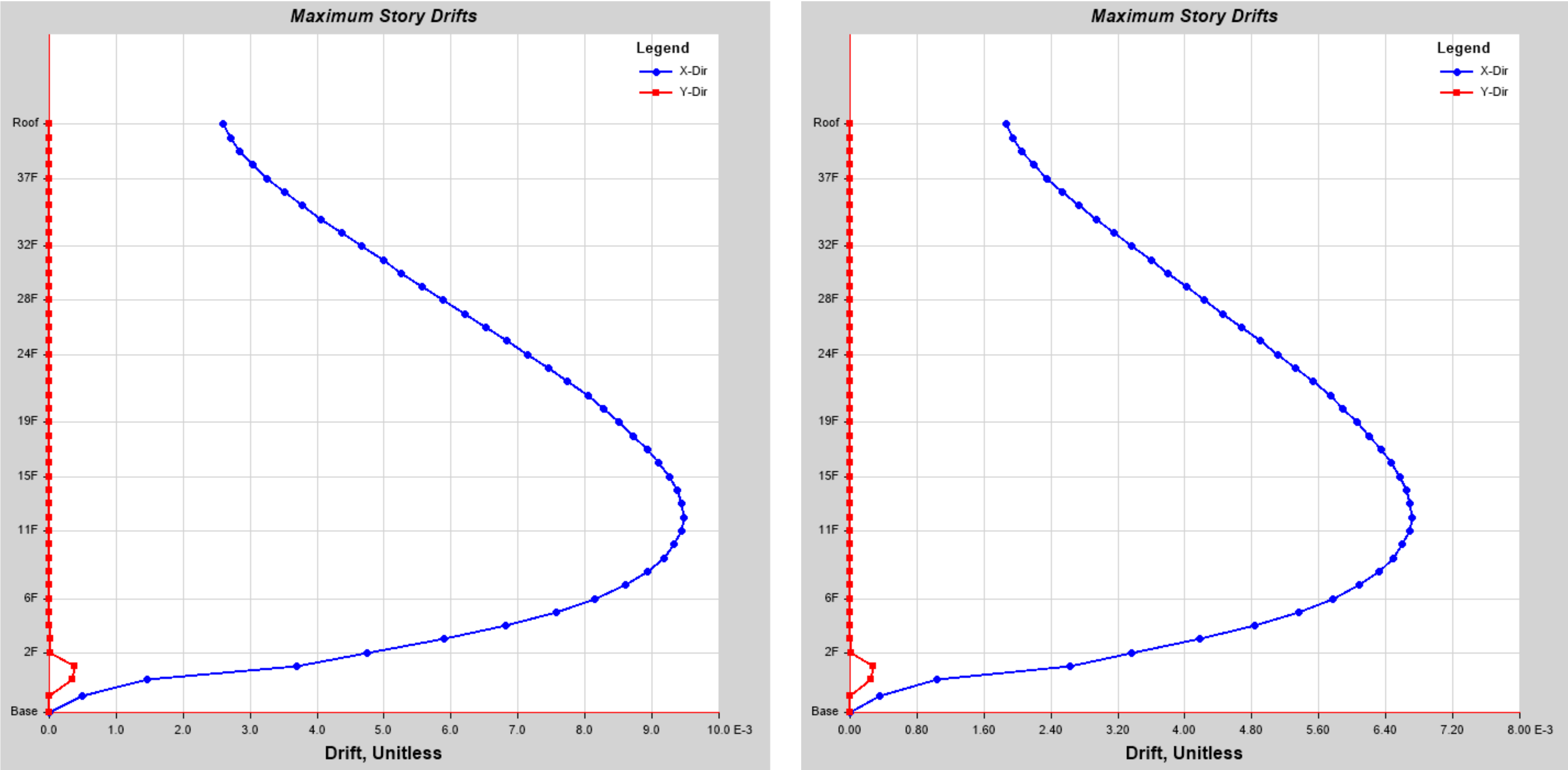
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Drift Displacement (inch)	Storey Drift Displacement (inch)
Roof	434.00	21.548731	17.344765
40F	424.00	21.325025	17.163962
39F	414.00	21.090783	16.974481
38F	404.00	20.84453	16.775062
37F	394.00	20.582789	16.562877
36F	384.00	20.303304	16.3361
35F	374.00	20.004442	16.093432
34F	364.00	19.685115	15.83402
33F	354.00	19.344665	15.557375
32F	344.00	18.982748	15.263263
31F	334.00	18.599601	14.951929
30F	324.00	18.192915	14.621557
29F	314.00	17.772798	14.280426
28F	304.00	17.331756	13.922485
27F	294.00	16.870341	13.54824
26F	284.00	16.388313	13.15755
25F	274.00	15.885659	12.750446
24F	264.00	15.362477	12.327053
23F	254.00	14.818968	11.887575
22F	244.00	14.255384	11.432257
21F	234.00	13.672079	10.961424
20F	224.00	13.066724	10.473267
19F	214.00	12.448673	9.975337
18F	204.00	11.814557	9.464863
17F	194.00	11.164702	8.942155
16F	184.00	10.49964	8.407652
15F	174.00	9.820223	7.862044
14F	164.00	9.127595	7.306241
13F	154.00	8.423234	6.741413
12F	144.00	7.708998	6.169029
11F	134.00	6.987249	5.590949
10F	124.00	6.260462	5.00913
9F	114.00	5.536196	4.429556
8F	104.00	4.81497	3.852566
7F	94.00	4.10218	3.282435
6F	84.00	3.404309	2.724294
5F	74.00	2.729642	2.184708
4F	64.00	2.088765	1.672087
3F	54.00	1.495377	1.197344
2F	44.00	0.96764	0.774987
1F	34.00	0.531603	0.425876
GF	24.00	0.262537	0.210344
Basement Floor	12.00	0.058372	0.046783
Base	0.00	0	0

➢ Maximum Storey Drift

Figures 5.18 and 5.19 shows the story drift, where horizontal axis represent drift and vertical axis represent the stories; blue curve state the story drift implying in X-direction and red curve in Y-direction. Also comparison between responses about the maximum story drifts under static and dynamic loadings in X and Y direction are clearly shown in Tables 5.17 and 5.18 respectively.

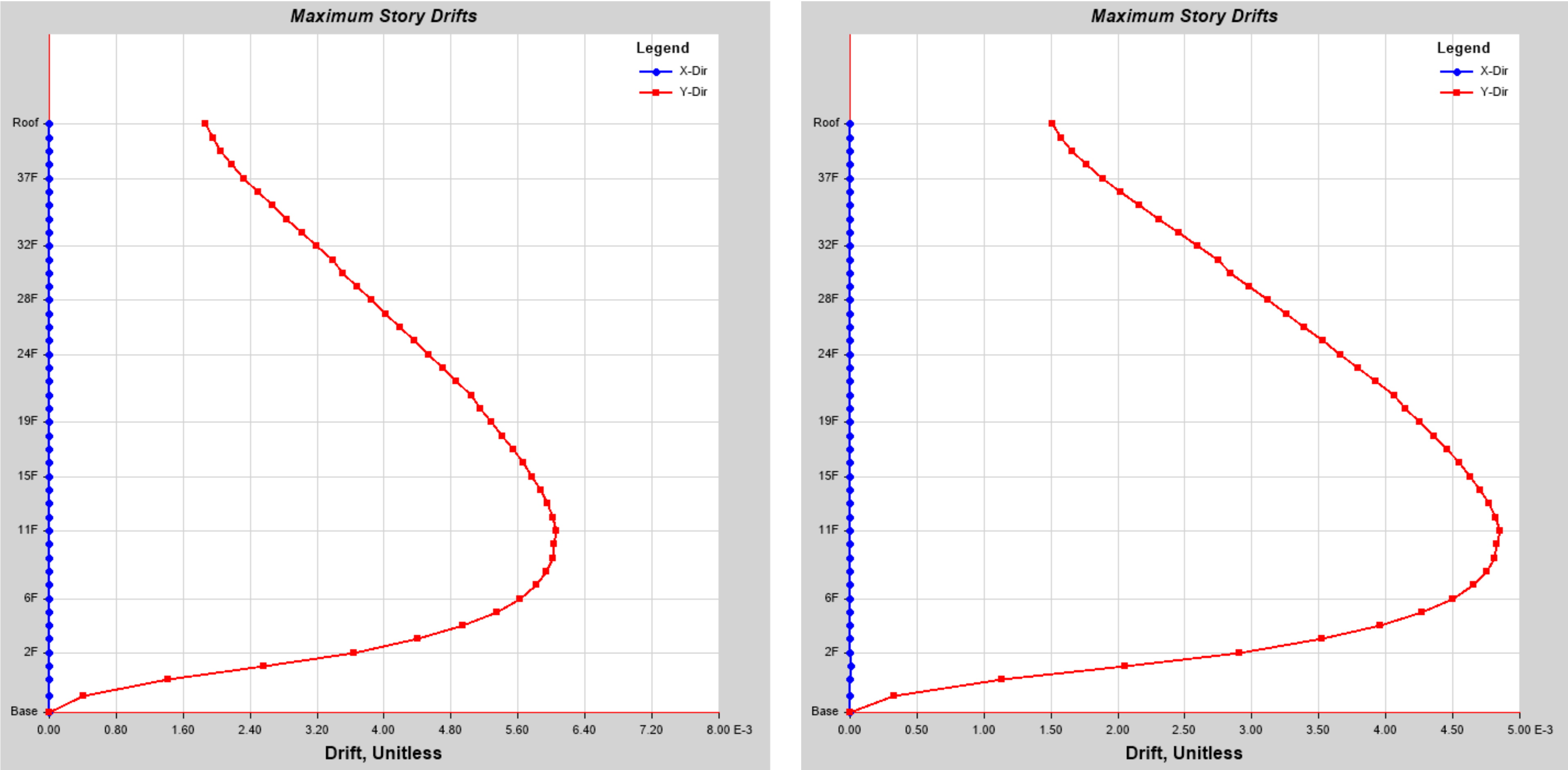
From figures, it is clearly that the story drift form a parabolic shape with zero drift at bottom, increases toward close to 1/3.5 of height of building and finally decreases at roof. Under both static & dynamic loading, curve start from base with zero value and sharply rises to 12^{th} floor and then gradually decreases to roof in both X & Y direction. It means that the incremental ratio of story displacement is large up to 12^{th} floor and then smaller compared to those floors.

It is found the maximum drift ratio under dynamic loading 0.0095 at 12^{th} floor in X-direction, and under static loading 0.0067 at 12^{th} floor in X-direction. Hence, the story drift is larger under the dynamic loading of the structure.



(a) Dynamic Loading (b) Static Loading

Figure 5.18: Maximum Story Drift due to Wind loads in global X-direction



(a) Dynamic Loading

(b) Static Loading

Figure 5.19: Maximum Story Drift due to Wind loads in global Y-direction

Table 5.17: Maximum story drift due to Wind in X-direction

Storey	Elevation (ft.)	Dynamic Loading Storey Drift Ratio (Unitless)	Static Loading Storey Drift Ratio (Unitless)
Roof	434.00	0.00259	0.001865
40F	424.00	0.002706	0.00195
39F	414.00	0.002849	0.002054
38F	404.00	0.003037	0.002192
37F	394.00	0.003259	0.002353
36F	384.00	0.003507	0.002534
35F	374.00	0.003777	0.002729
34F	364.00	0.004064	0.002936
33F	354.00	0.004362	0.003151
32F	344.00	0.004665	0.003368
31F	334.00	0.004993	0.003602
30F	324.00	0.00526	0.003792
29F	314.00	0.005574	0.004014
28F	304.00	0.005885	0.004234
27F	294.00	0.006201	0.004456
26F	284.00	0.006517	0.004677
25F	274.00	0.006831	0.004897
24F	264.00	0.007142	0.005114
23F	254.00	0.007447	0.005326
22F	244.00	0.00774	0.005528
21F	234.00	0.00805	0.005742
20F	224.00	0.008268	0.005891
19F	214.00	0.008503	0.006053
18F	204.00	0.008723	0.006203
17F	194.00	0.008925	0.00634
16F	184.00	0.009104	0.006461
15F	174.00	0.009254	0.006563
14F	164.00	0.009371	0.006642
13F	154.00	0.009447	0.006691
12F	144.00	0.009474	0.006707
11F	134.00	0.009446	0.006685
10F	124.00	0.009322	0.006596
9F	114.00	0.009173	0.006489
8F	104.00	0.008934	0.00632
7F	94.00	0.008598	0.006083
6F	84.00	0.008148	0.005766
5F	74.00	0.007564	0.005354
4F	64.00	0.006822	0.004831
3F	54.00	0.005897	0.004178
2F	44.00	0.004745	0.003364
1F	34.00	0.003694	0.002622
GF	24.00	0.001463	0.001039
Basement Floor	12.00	0.000499	0.000354
Base	0.00	0	0

Table 5.18: Maximum story drift due to Wind in Y-direction

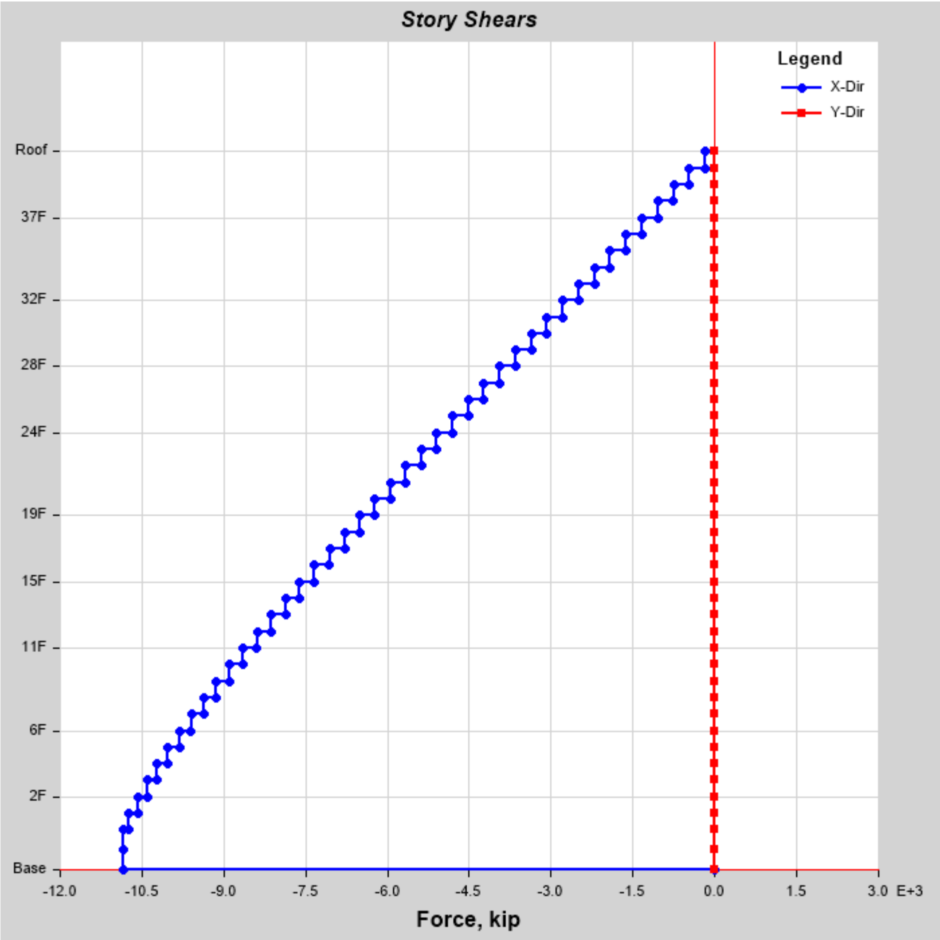
Storey	Elevation (ft.)	Dynamic Loading Storey Drift Ratio (Unitless)	Static Loading Storey Drift Ratio (Unitless)
Roof	434.00	0.001864	0.001507
40F	424.00	0.001952	0.001579
39F	414.00	0.002052	0.001662
38F	404.00	0.002181	0.001768
37F	394.00	0.002329	0.00189
36F	384.00	0.002491	0.002022
35F	374.00	0.002661	0.002162
34F	364.00	0.002837	0.002305
33F	354.00	0.003016	0.002451
32F	344.00	0.003193	0.002594
31F	334.00	0.003389	0.002753
30F	324.00	0.003501	0.002843
29F	314.00	0.003675	0.002983
28F	304.00	0.003845	0.003119
27F	294.00	0.004017	0.003256
26F	284.00	0.004189	0.003393
25F	274.00	0.00436	0.003528
24F	264.00	0.004529	0.003662
23F	254.00	0.004697	0.003794
22F	244.00	0.004861	0.003924
21F	234.00	0.005045	0.004068
20F	224.00	0.00515	0.004149
19F	214.00	0.005284	0.004254
18F	204.00	0.005415	0.004356
17F	194.00	0.005542	0.004454
16F	184.00	0.005662	0.004547
15F	174.00	0.005772	0.004632
14F	164.00	0.00587	0.004707
13F	154.00	0.005952	0.00477
12F	144.00	0.006015	0.004817
11F	134.00	0.006057	0.004848
10F	124.00	0.006036	0.00483
9F	114.00	0.00601	0.004808
8F	104.00	0.00594	0.004751
7F	94.00	0.005816	0.004651
6F	84.00	0.005622	0.004497
5F	74.00	0.005341	0.004272
4F	64.00	0.004945	0.003956
3F	54.00	0.004398	0.00352
2F	44.00	0.003634	0.002909
1F	34.00	0.002561	0.002051
GF	24.00	0.001418	0.001136
Basement Floor	12.00	0.000405	0.000325
Base	0.00	0	0

➢ Storey Shear

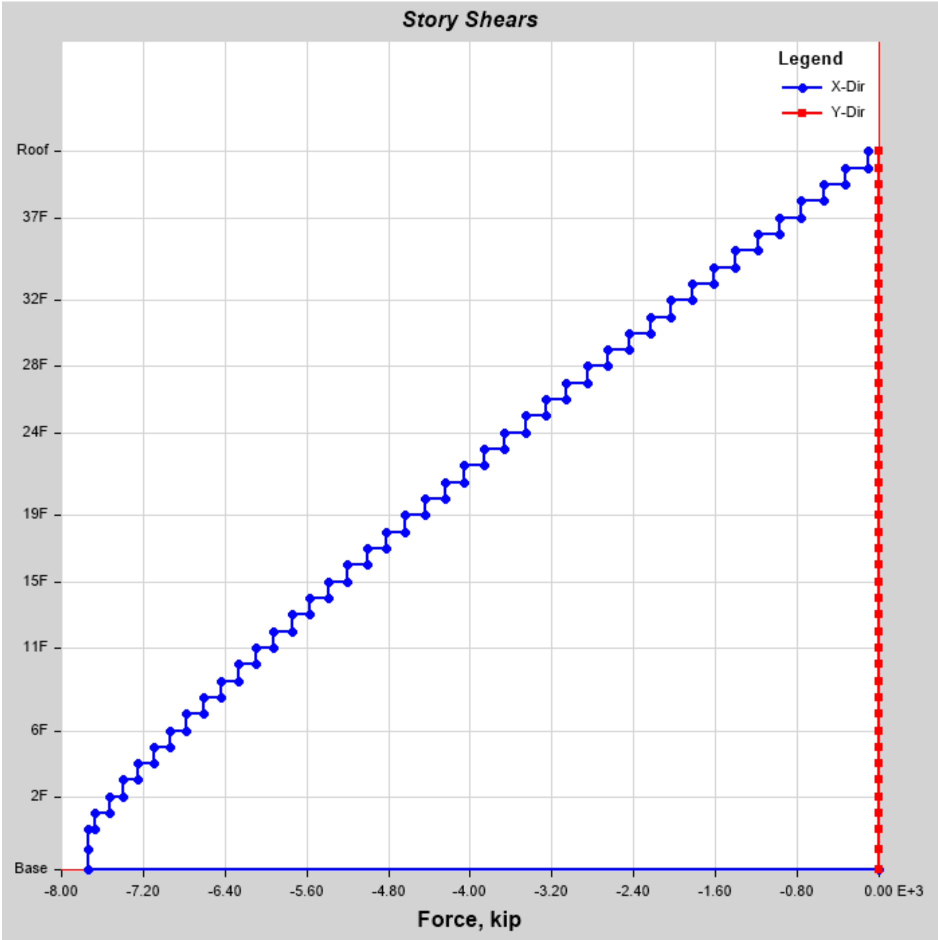
Figures 5.20 and 5.21 illustrated below provide the information about story shears, where horizontal axis represent story shear in kips and vertical axis represent the stories; blue curves state the response due to story shear implying in X-direction and red curves in Y-direction. Also comparison between responses about the story shear under static and dynamic loadings in X and Y direction are clearly shown in Tables 5.19 and 5.20 respectively.

From figures, it is clearly seen that the response curves are different under dynamic and static loading in both X and Y direction. It is observed that the maximum shear at base and gradually decrease at up to top. Shear force is decreasing with respect to increase of height. The negative value of shear force indicates the resisting shear force that resist the positive shear in the corresponding direction. It is also shown that structure under dynamic loading have to be resist greater story shear force compared to the static loading.

It is found the maximum story shear to resist the structure under dynamic loading 10851.31 kips in X-direction, while under static loading structure have to be resist only 8504.31 kips in Y-direction.

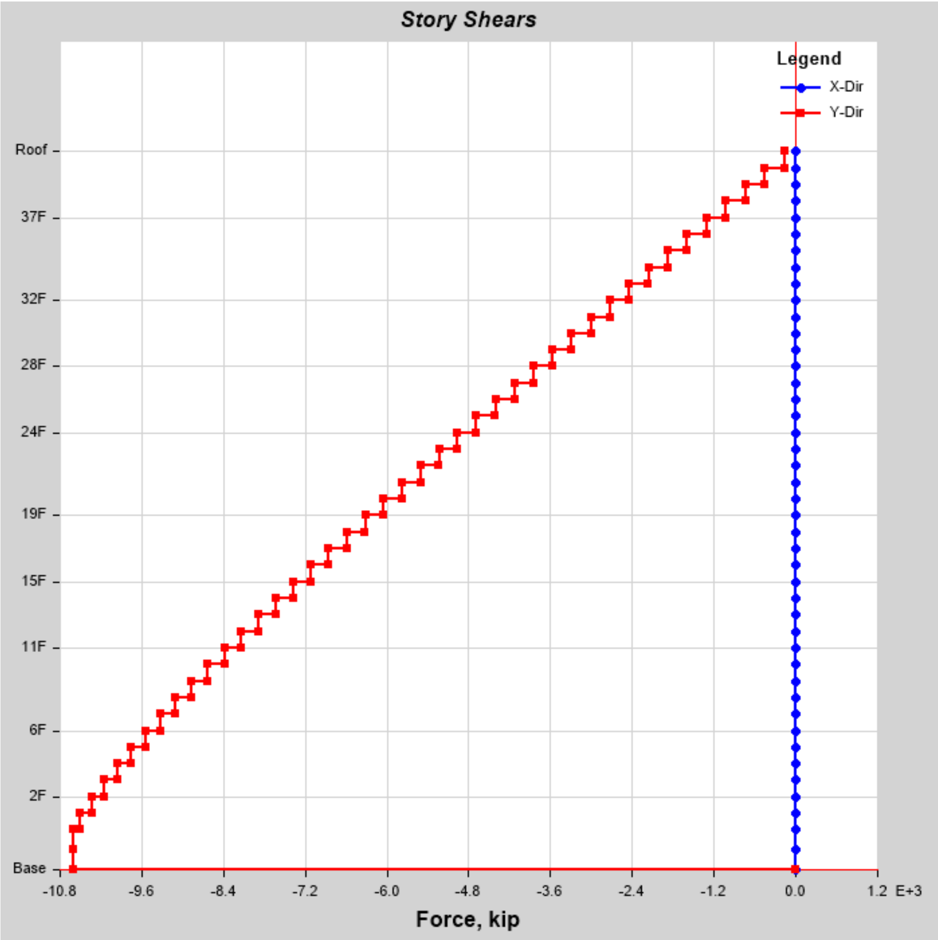


(a) Dynamic Loading

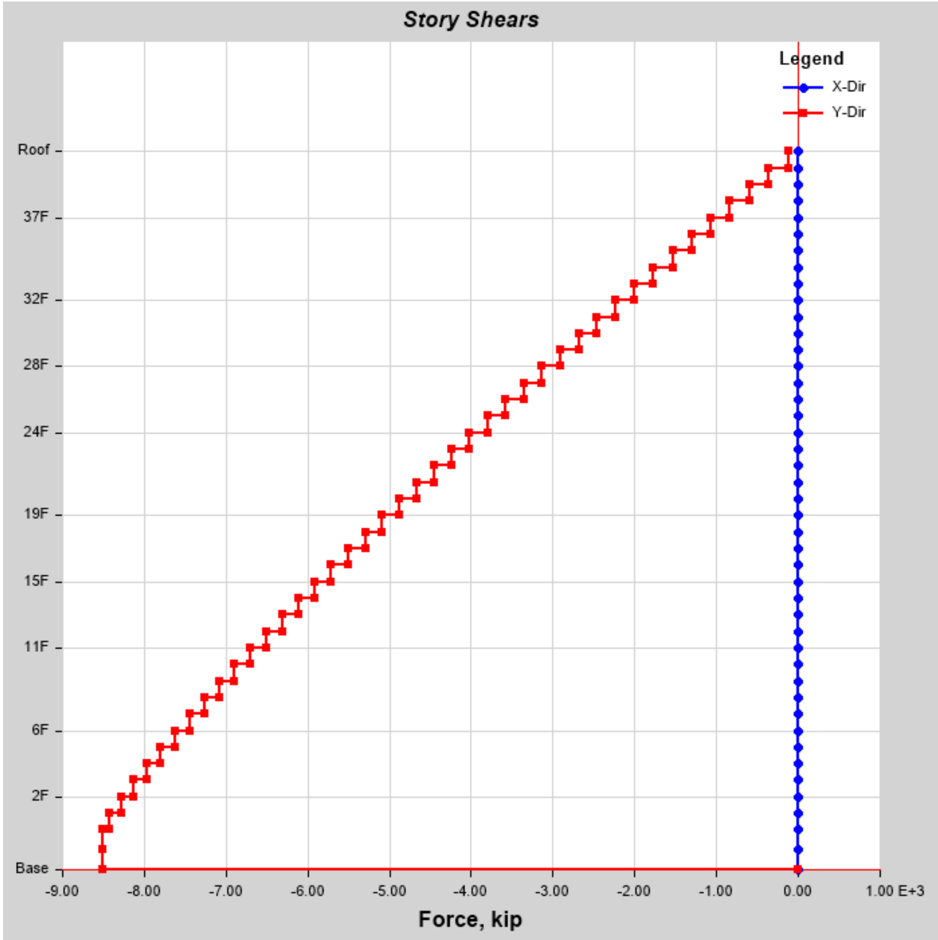


(b) Static Loading

Figure 5.20: Story Shear due to Wind loads in global X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.21: Story Shear due to Wind loads in global Y-direction

Table 5.19: Maximum Story Shear due to Wind in X-direction

Storey	Elevation (ft.)	Dynamic Loading Shear resisted by each Storey (kips)	Static Loading Shear resisted by each Storey (kips)
Roof	434.00	-176.91	-109.078
40F	424.00	-466.926	-326.593
39F	414.00	-756.973	-543.133
38F	404.00	-1047.041	-758.68
37F	394.00	-1337.107	-973.216
36F	384.00	-1627.14	-1186.721
35F	374.00	-1917.094	-1399.173
34F	364.00	-2206.918	-1610.551
33F	354.00	-2496.549	-1820.834
32F	344.00	-2785.917	-2029.995
31F	334.00	-3074.944	-2238.011
30F	324.00	-3364.59	-2444.854
29F	314.00	-3655.051	-2650.496
28F	304.00	-3944.916	-2854.908
27F	294.00	-4234.072	-3058.057
26F	284.00	-4522.391	-3259.909
25F	274.00	-4809.738	-3460.429
24F	264.00	-5095.959	-3659.577
23F	254.00	-5380.89	-3857.313
22F	244.00	-5664.348	-4053.591
21F	234.00	-5946.137	-4248.364
20F	224.00	-6227.536	-4441.58
19F	214.00	-6508.67	-4633.18
18F	204.00	-6787.468	-4823.105
17F	194.00	-7063.61	-5011.285
16F	184.00	-7336.788	-5197.646
15F	174.00	-7606.661	-5382.105
14F	164.00	-7872.853	-5564.568
13F	154.00	-8134.949	-5744.931
12F	144.00	-8392.487	-5923.075
11F	134.00	-8644.959	-6098.862
10F	124.00	-8893.049	-6272.136
9F	114.00	-9136.199	-6442.707
8F	104.00	-9372.241	-6610.352
7F	94.00	-9600.331	-6774.796
6F	84.00	-9819.5	-6935.689
5F	74.00	-10028.613	-7092.576
4F	64.00	-10226.311	-7244.829
3F	54.00	-10410.886	-7391.506
2F	44.00	-10580.195	-7531.22
1F	34.00	-10753.047	-7666.629
GF	24.00	-10840.794	-7734.391
Basement Floor	12.00	-10851.306	-7734.391
Base	0.00	0	0

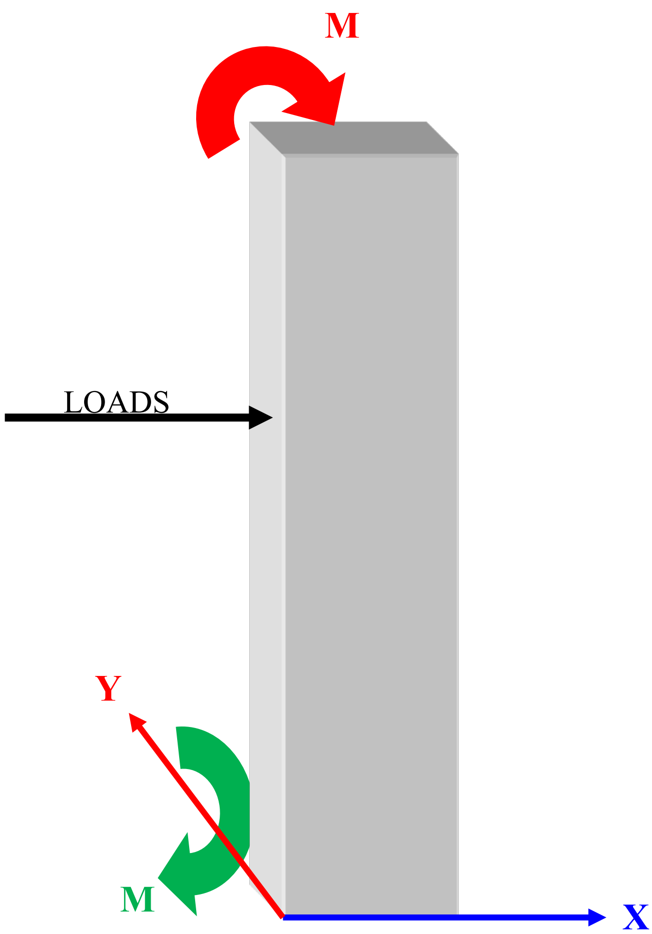
Table 5.20: Maximum Story Shear due to Wind in Y-direction

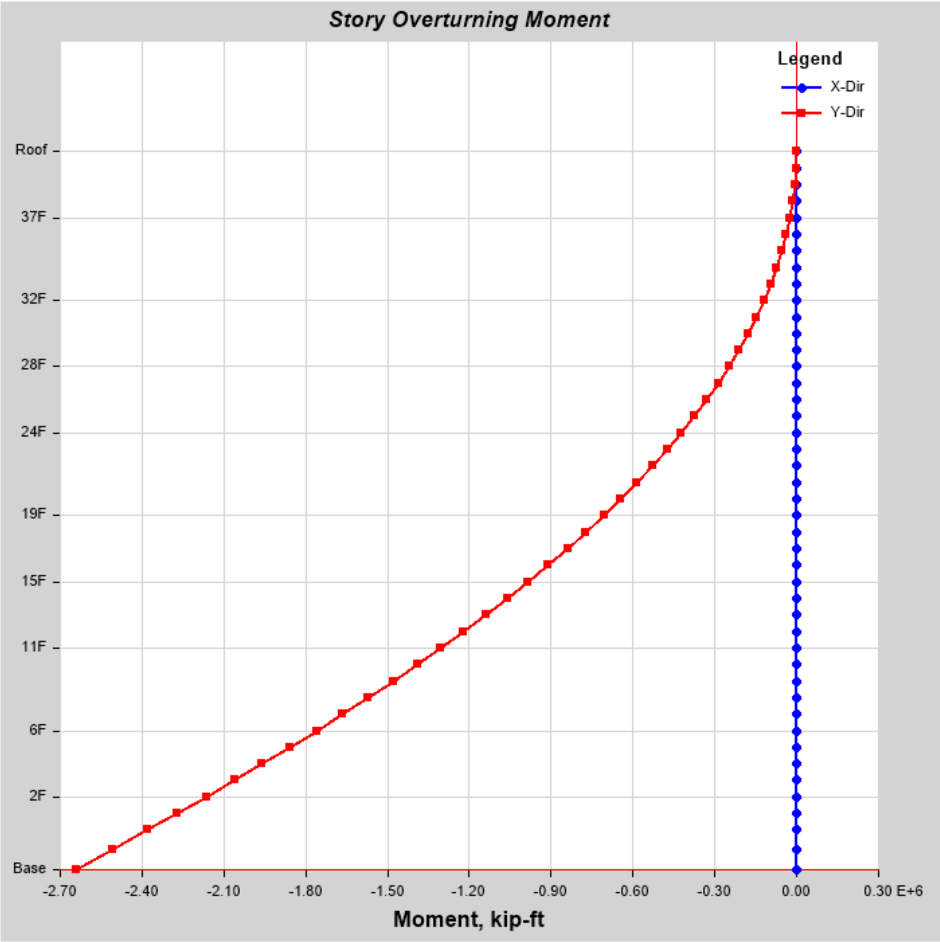
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Shear resisted by each Storey (kips)	Shear resisted by each Storey (kips)
Roof	434.00	-164.516	-119.689
40F	424.00	-451.078	-358.383
39F	414.00	-737.201	-596.021
38F	404.00	-1022.869	-832.584
37F	394.00	-1308.055	-1068.05
36F	384.00	-1592.726	-1302.4
35F	374.00	-1876.841	-1535.61
34F	364.00	-2160.356	-1767.658
33F	354.00	-2443.222	-1998.518
32F	344.00	-2725.384	-2228.163
31F	334.00	-3006.784	-2456.568
30F	324.00	-3288.044	-2683.702
29F	314.00	-3569.298	-2909.536
28F	304.00	-3849.62	-3134.038
27F	294.00	-4128.937	-3357.171
26F	284.00	-4407.164	-3578.901
25F	274.00	-4684.213	-3799.187
24F	264.00	-4959.986	-4017.988
23F	254.00	-5234.375	-4235.259
22F	244.00	-5507.265	-4450.951
21F	234.00	-5778.525	-4665.012
20F	224.00	-6048.998	-4877.387
19F	214.00	-6318.782	-5088.013
18F	204.00	-6586.515	-5296.823
17F	194.00	-6851.986	-5503.745
16F	184.00	-7114.989	-5708.696
15F	174.00	-7375.292	-5911.586
14F	164.00	-7632.635	-6112.315
13F	154.00	-7886.723	-6310.77
12F	144.00	-8137.222	-6506.82
11F	134.00	-8383.748	-6700.319
10F	124.00	-8626.734	-6891.094
9F	114.00	-8865.752	-7078.943
8F	104.00	-9099.204	-7263.622
7F	94.00	-9326.378	-7444.833
6F	84.00	-9546.414	-7622.198
5F	74.00	-9758.262	-7795.225
4F	64.00	-9960.595	-7963.23
3F	54.00	-10151.659	-8125.195
2F	44.00	-10329.158	-8279.618
1F	34.00	-10508.492	-8429.378
GF	24.00	-10599.481	-8504.31
Basement Floor	12.00	-10608.152	-8504.31
Base	0.00	0	0

➢ Story Overturning Moments

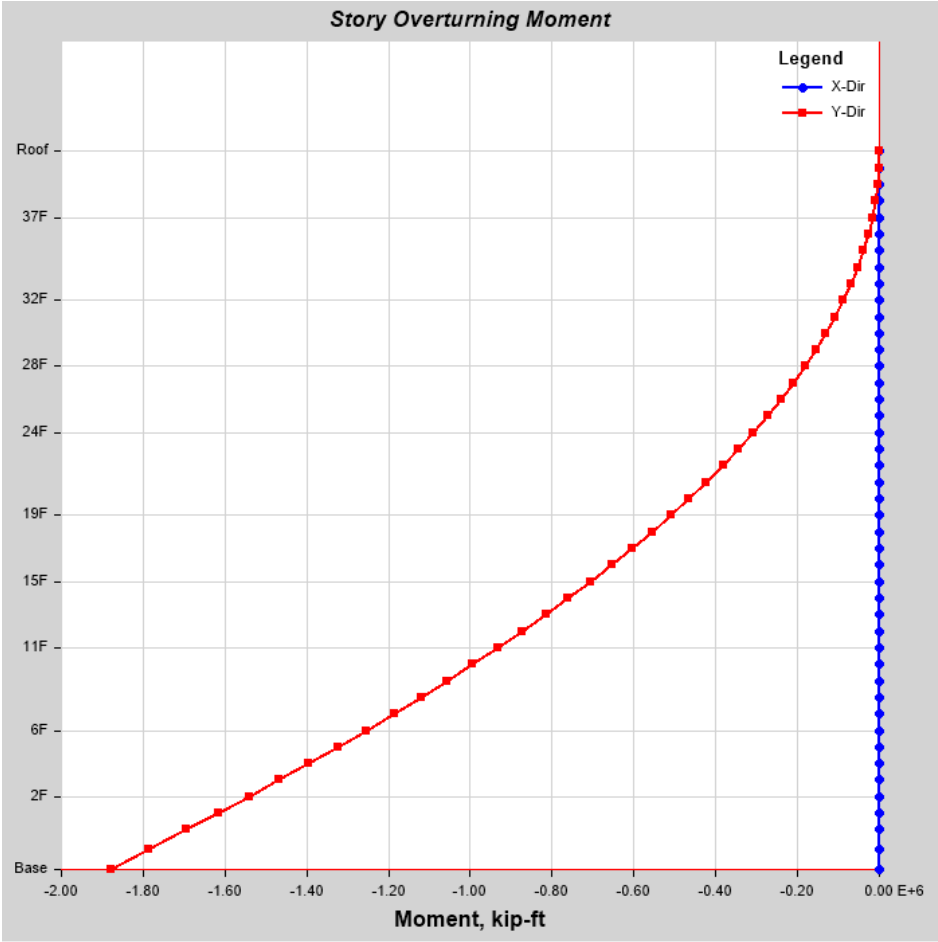
Figures 5.22 and 5.23 illustrated the story overturning moment, where horizontal axis represent overturning moment in kip-ft. and vertical axis represent the stories; blue curve state the response due to overturning moment implying in X-direction and red curve in Y-direction. Also comparison between responses about the story overturning moment under static and dynamic loading in both X and Y directions are clearly shown in Tables 5.21 and 5.22 respectively.

From figures, it is clearly seen that the curve start from base with its peak value and sharply goes down to roof top in both X & Y direction and under dynamic loading structure can withstand greater storey overturning compared to static loading. It is observed that due to wind forces in X-direction, the whole structure will resist its overturn with respect to Y-axis and creates a resisting overturning moment M_R with respect to Y-axis. Similar case can be explained for loads in Y-direction. The value is negative in X-direction means the resisting overturning moment is in negative quadrant, whereas positive value in Y-direction means the positive quadrant.



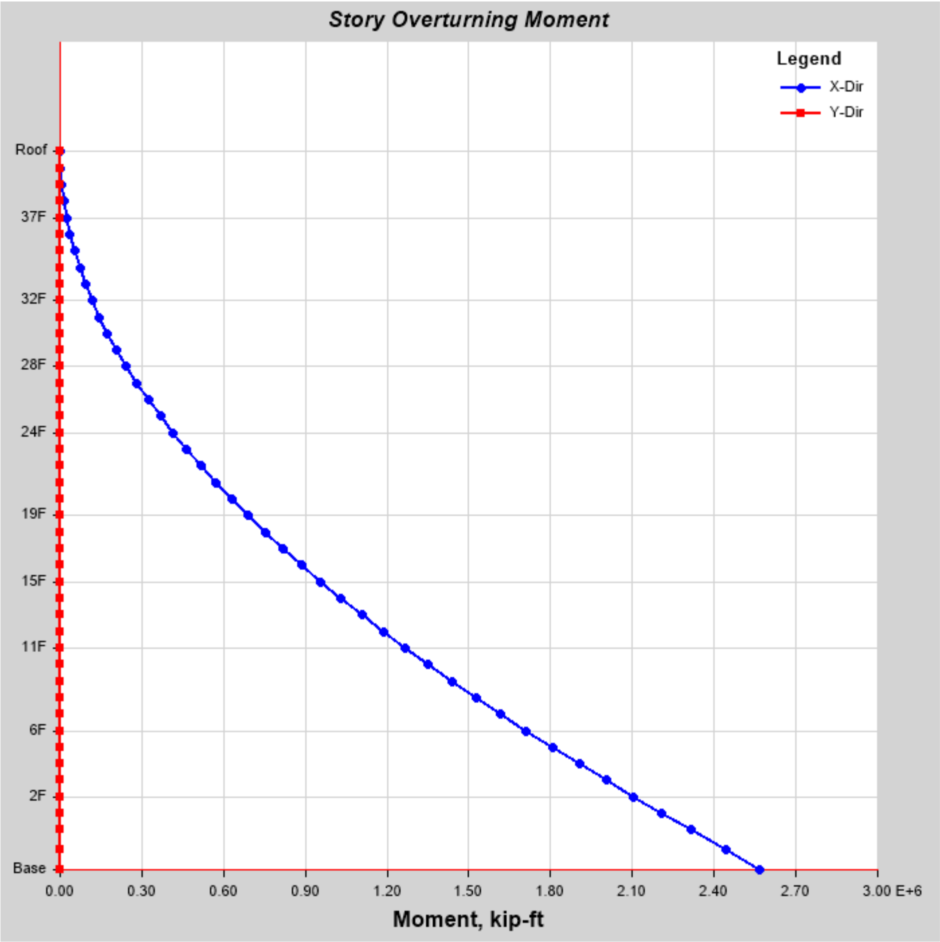


(a) Dynamic Loading

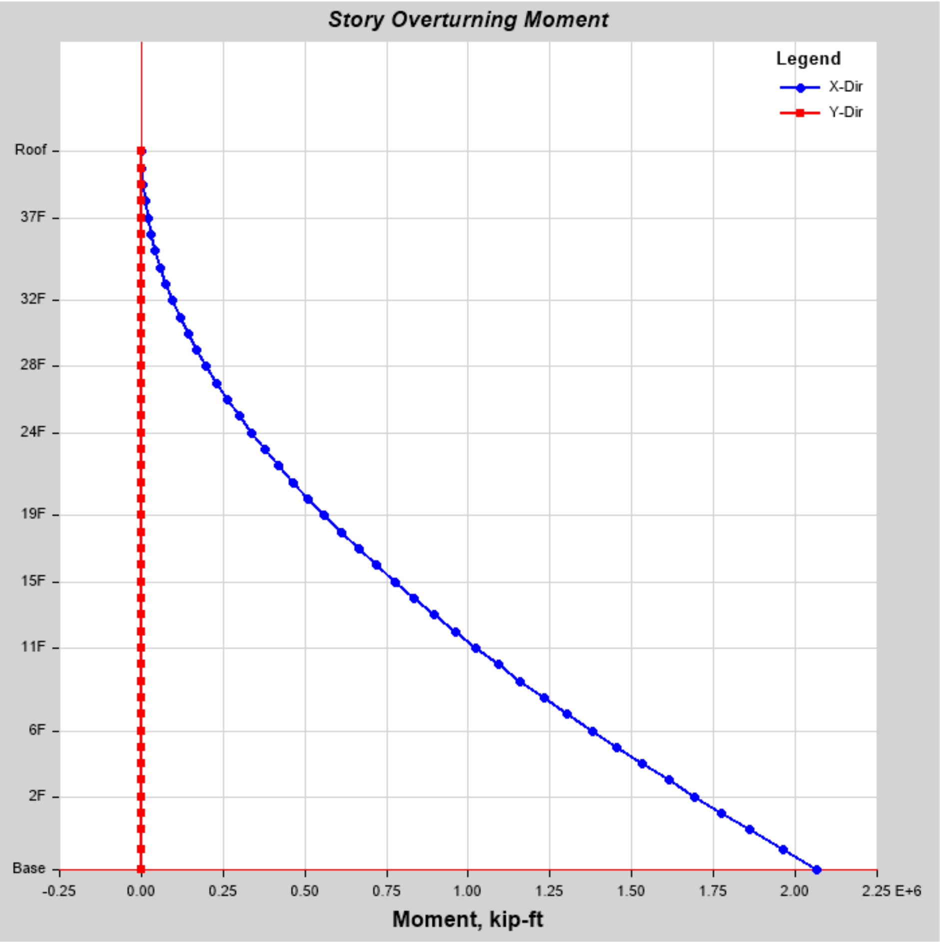


(b) Static Loading

Figure 5.22: Story Overturning Moments due to Wind loads in global X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.23: Story Overturning Moments due to Wind loads in global Y-direction

Table 5.21: Story Overturning Moments due to Wind in X-direction

Storey	Elevation (ft.)	Dynamic Loading Storey Overturning Moments (kip-ft)	Static Loading Storey Overturning Moments (kip-ft)
Roof	434.00	-51.25	0.00
40F	424.00	-1806.18	-1090.78
39F	414.00	-6461.08	-4356.71
38F	404.00	-14016.24	-9788.05
37F	394.00	-24471.89	-17374.85
36F	384.00	-37828.01	-27107.01
35F	374.00	-54084.25	-38974.22
34F	364.00	-73239.85	-52965.95
33F	354.00	-95293.51	-69071.46
32F	344.00	-120243.30	-87279.80
31F	334.00	-148086.62	-107579.75
30F	324.00	-178820.05	-129959.85
29F	314.00	-212449.79	-154408.39
28F	304.00	-248984.03	-180913.36
27F	294.00	-288416.80	-209462.43
26F	284.00	-330741.03	-240043.00
25F	274.00	-375948.38	-272642.09
24F	264.00	-424029.14	-307246.38
23F	254.00	-474972.07	-343842.15
22F	244.00	-528764.30	-382415.28
21F	234.00	-585391.14	-422951.20
20F	224.00	-644835.92	-465434.84
19F	214.00	-707094.76	-509850.64
18F	204.00	-772165.05	-556182.44
17F	194.00	-840023.49	-604413.49
16F	184.00	-910643.53	-654526.34
15F	174.00	-983995.58	-706502.81
14F	164.00	-1060047.00	-760323.86
13F	154.00	-1138760.00	-815969.54
12F	144.00	-1220095.00	-873418.85
11F	134.00	-1304005.00	-932649.59
10F	124.00	-1390441.00	-993638.22
9F	114.00	-1479358.00	-1056360.00
8F	104.00	-1570708.00	-1120787.00
7F	94.00	-1664419.00	-1186890.00
6F	84.00	-1760411.00	-1254638.00
5F	74.00	-1858596.00	-1323995.00
4F	64.00	-1958874.00	-1394921.00
3F	54.00	-2061130.00	-1467369.00
2F	44.00	-2165233.00	-1541284.00
1F	34.00	-2271030.00	-1616596.00
GF	24.00	-2378558.00	-1693263.00
Basement Floor	12.00	-2508638.00	-1786075.00
Base	0.00	-2638849.00	-1878888.00

Table 5.22: Story Overturning Moments due to Wind in Y-direction

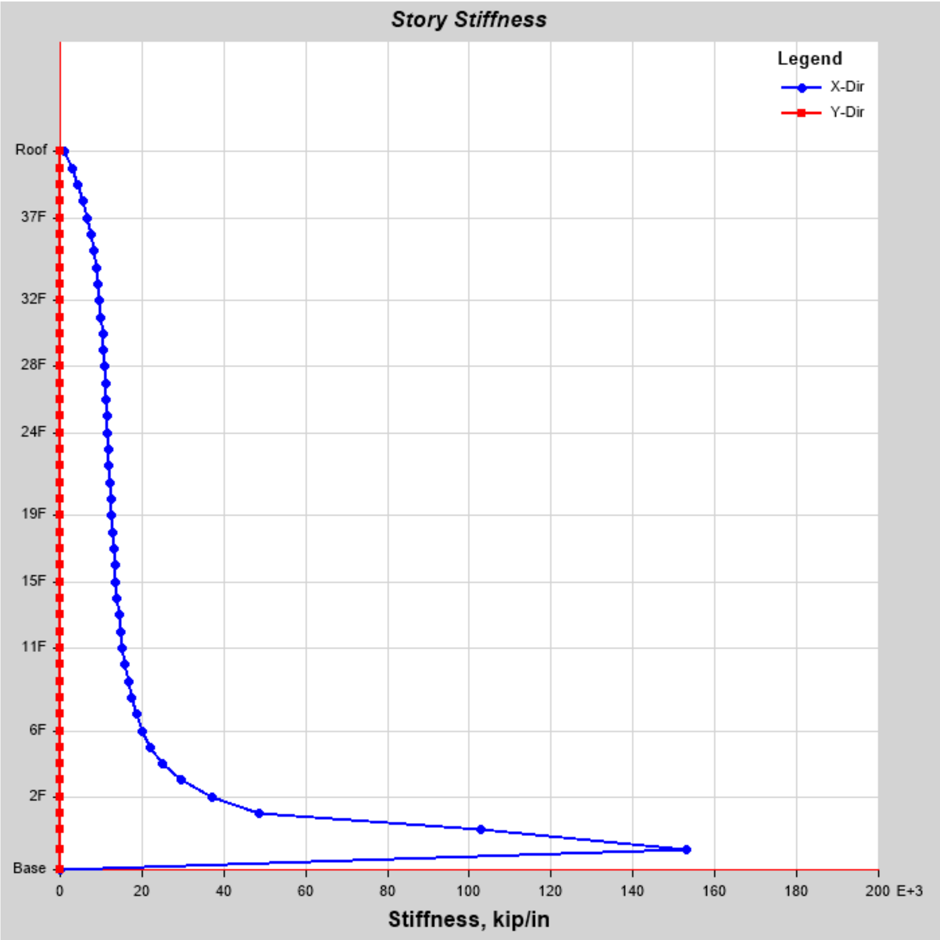
Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Overturning Moments (kip-ft)	Storey Overturning Moments (kip-ft)
Roof	434.00	33.92	0.00
40F	424.00	1669.73	1196.89
39F	414.00	6171.02	4780.72
38F	404.00	13533.43	10740.94
37F	394.00	23752.40	19066.77
36F	384.00	36823.11	29747.28
35F	374.00	52740.41	42771.28
34F	364.00	71498.75	58127.38
33F	354.00	93092.14	75803.96
32F	344.00	117514.07	95789.13
31F	334.00	144757.53	118070.76
30F	324.00	174814.90	142636.44
29F	314.00	207684.79	169473.46
28F	304.00	243367.13	198568.83
27F	294.00	281852.63	229909.20
26F	284.00	323131.23	263480.92
25F	274.00	367192.04	299269.92
24F	264.00	414023.30	337261.79
23F	254.00	463612.26	377441.67
22F	244.00	515945.09	419794.26
21F	234.00	571006.80	464303.77
20F	224.00	628781.13	510953.89
19F	214.00	689260.22	559727.76
18F	204.00	752437.18	610607.88
17F	194.00	818291.53	663576.12
16F	184.00	886800.67	718613.56
15F	174.00	957939.95	775700.52
14F	164.00	1031682.39	834816.39
13F	154.00	1107998.43	895939.54
12F	144.00	1186855.55	959047.24
11F	134.00	1268217.90	1024115.44
10F	124.00	1352045.80	1091118.63
9F	114.00	1438303.90	1160029.58
8F	104.00	1526952.57	1230819.01
7F	94.00	1617936.23	1303455.23
6F	84.00	1711192.17	1377903.56
5F	74.00	1806649.12	1454125.55
4F	64.00	1904225.29	1532077.79
3F	54.00	2003825.65	1611710.09
2F	44.00	2105337.66	1692962.05
1F	34.00	2208625.80	1775758.23
GF	24.00	2313708.47	1860052.01
Basement Floor	12.00	2440894.25	1962103.73
Base	0.00	2568188.50	2064155.45

➢ Story Stiffness

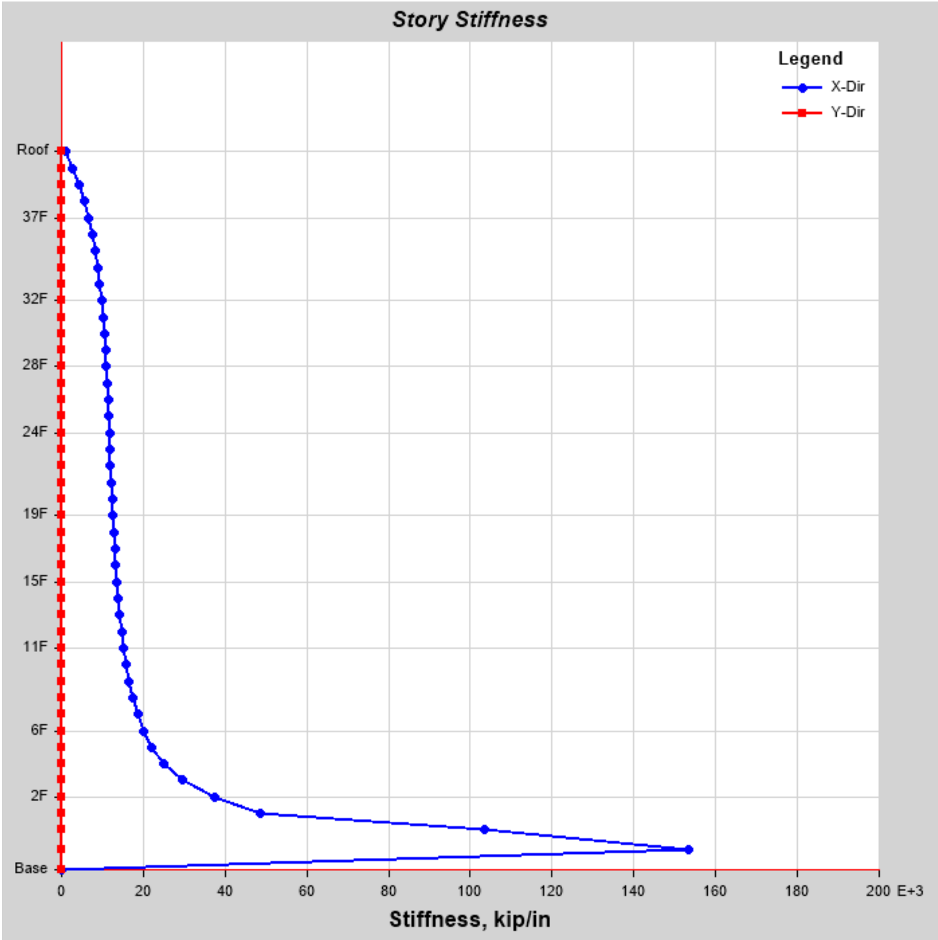
Figures 5.24 and 5.25 illustrated the response of story stiffness, where the horizontal axis represent story stiffness in kip/inch and vertical axis represent the stories; blue curve state the story stiffness implying in X-direction and red curve in Y-direction. Also comparison between the responses about story stiffness under static and dynamic loading in both X and Y direction are clearly shown in Tables 5.23 and 5.24 respectively.

From figures, it is clearly seen that the story stiffness under both static & dynamic loading in both X and Y direction is almost close but not same. It is also sees that the curve rapidly high at basement floor; after that the value decreases with large rate up to 2nd floor and then the decreases at a lower rate up to top. Its' indicate that up to 2nd floor of the building is more stiffer than others; because there is a concrete wall to the periphery of the building and also ramp connect in this level which makes their stronger and after ground floor the building floor plan is changed.

It is found that the maximum story stiffness under dynamic loading of the structure 181735.49 kip/in inch, while under static loading is 181782.35 kip/inch.

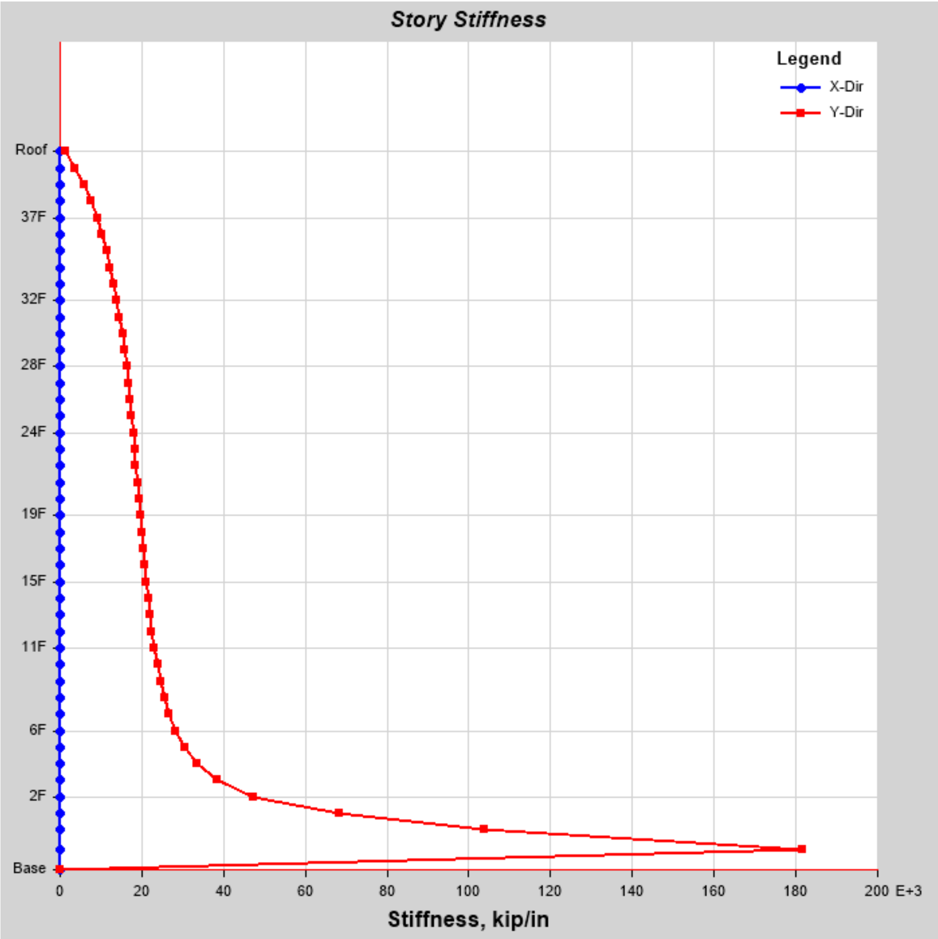


(a) Dynamic Loading

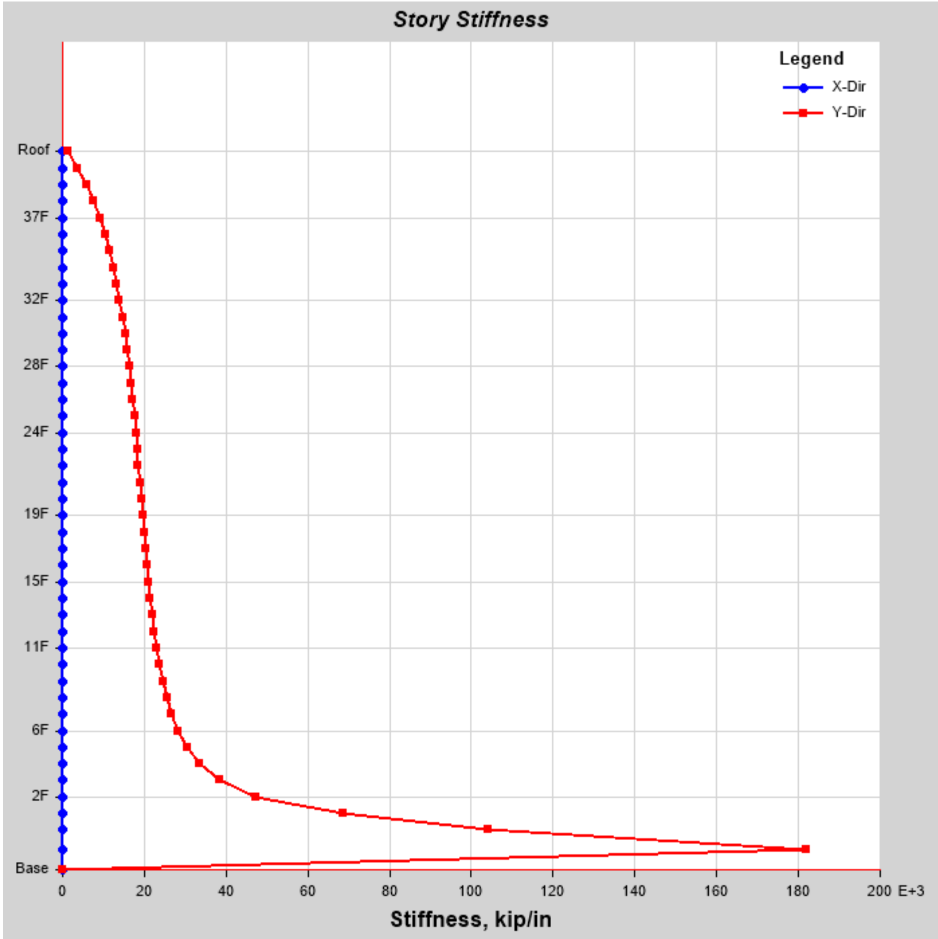


(b) Static Loading

Figure 5.24: Story Stiffness due to Wind loads in global X-direction



(a) Dynamic Loading



(b) Static Loading

Figure 5.25: Story Stiffness due to Wind loads in global Y-direction

Table 5.23: Story Stiffness due to Wind in X-direction

Storey	Elevation (ft.)	Dynamic Loading Storey Stiffness (kip/inch)	Static Loading Storey Stiffness (kip/inch)
Roof	434.00	1115.55	954.96
40F	424.00	2808.94	2725.73
39F	414.00	4294.22	4270.95
38F	404.00	5551.05	5571.37
37F	394.00	6594.01	6644.27
36F	384.00	7450.96	7520.70
35F	374.00	8154.17	8235.80
34F	364.00	8734.08	8822.31
33F	354.00	9218.86	9310.17
32F	344.00	9616.74	9708.83
31F	334.00	9997.60	10089.56
30F	324.00	10351.81	10439.50
29F	314.00	10634.12	10713.14
28F	304.00	10880.98	10951.68
27F	294.00	11098.14	11160.88
26F	284.00	11293.39	11348.61
25F	274.00	11473.04	11521.20
24F	264.00	11642.39	11683.99
23F	254.00	11806.52	11842.12
22F	244.00	11955.33	11985.74
21F	234.00	12143.83	12169.67
20F	224.00	12373.19	12391.16
19F	214.00	12592.18	12598.99
18F	204.00	12817.40	12814.18
17F	194.00	13056.44	13044.14
16F	184.00	13316.84	13296.33
15F	174.00	13607.16	13579.31
14F	164.00	13937.78	13903.54
13F	154.00	14321.85	14282.25
12F	144.00	14764.10	14718.25
11F	134.00	15253.35	15205.89
10F	124.00	15899.37	15849.51
9F	114.00	16599.34	16546.97
8F	104.00	17483.61	17432.00
7F	94.00	18609.35	18562.50
6F	84.00	20086.07	20049.30
5F	74.00	22098.35	22079.20
4F	64.00	24982.84	24992.87
3F	54.00	29425.25	29483.90
2F	44.00	37165.86	37311.73
1F	34.00	48516.87	48741.20
GF	24.00	102935.87	103404.56
Basement Floor	12.00	153125.05	153502.80
Base	0.00	0.00	0.00

Table 5.24: Story Stiffness due to Wind in Y-direction

Storey	Elevation (ft.)	Dynamic Loading	Static Loading
		Storey Stiffness (kip/inch)	Storey Stiffness (kip/inch)
Roof	434.00	1444.08	1299.40
40F	424.00	3760.85	3691.83
39F	414.00	5812.92	5800.27
38F	404.00	7566.48	7593.59
37F	394.00	9052.21	9105.74
36F	384.00	10309.41	10379.48
35F	374.00	11380.74	11460.40
34F	364.00	12303.52	12388.05
33F	354.00	13119.05	13205.46
32F	344.00	13790.54	13876.74
31F	334.00	14543.07	14629.39
30F	324.00	15281.57	15364.42
29F	314.00	15819.83	15894.20
28F	304.00	16321.50	16388.10
27F	294.00	16771.80	16831.06
26F	284.00	17183.24	17235.66
25F	274.00	17563.56	17609.70
24F	264.00	17918.41	17958.94
23F	254.00	18256.20	18291.83
22F	244.00	18521.50	18553.71
21F	234.00	18884.11	18914.12
20F	224.00	19324.43	19347.98
19F	214.00	19681.84	19694.90
18F	204.00	20032.15	20036.43
17F	194.00	20380.25	20376.74
16F	184.00	20736.22	20725.71
15F	174.00	21109.75	21092.97
14F	164.00	21511.96	21489.61
13F	154.00	21957.49	21930.34
12F	144.00	22451.71	22420.89
11F	134.00	23070.74	23032.33
10F	124.00	23822.01	23779.85
9F	114.00	24585.22	24537.50
8F	104.00	25531.21	25480.54
7F	94.00	26728.12	26677.25
6F	84.00	28299.62	28252.00
5F	74.00	30452.82	30413.22
4F	64.00	33571.93	33547.55
3F	54.00	38472.48	38475.43
2F	44.00	47377.39	47432.56
1F	34.00	68390.38	68492.18
GF	24.00	103832.08	103989.21
Basement Floor	12.00	181735.49	181782.35
Base	0.00	0.00	0.00

5.3.3 Response under Base Reaction

Table 5.25: Base Reaction, Forces

Load Cases	FX (kips)		FY (kips)		FZ (kips)	
	Dynamic	Static	Dynamic	Static	Dynamic	Static
Dead	0.0	0.0	0.0	0.0	339062.79	339062.79
Live	0.0	0.0	155.52	155.52	84426.24	84426.24
EQX	-10108.96	-10108.95	0.0	0.0	0.0	0.0
EQY	0.0	0.0	-10108.32	-10108.29	0.0	0.0
WX	-7734.40	-7734.39	0.0	0.0	0.0	0.0
WY	0.0	0.0	-8504.33	-8504.31	0.0	0.0
RS	9326.08	N/A	9395.16	N/A	0.0	N/A

Table 5.26: Base Reaction, Moments

Load Cases	MX (kip-ft.)		MY (kip-ft.)		MZ (kip-ft.)	
	Dynamic	Static	Dynamic	Static	Dynamic	Static
Dead	48802543.0	48802543.0	-52893795.0	-52893795	-0.0141	-0.0109
Live	12151847.0	12151847.0	-13170494.0	-13170494	24261.10	24261.10
EQX	-0.0104	0.0152	-3199477.0	-3199472	1594377.25	1316991.38
EQY	3199282.58	3199316.54	-0.0047	-0.0012	-1727143.0	-1426663.0
WX	1548100.11	0.004	-1429101	-1878888	-147145.55	1113752.31
WY	1548100.11	-0.004	-1429101.0	1878887.99	-147145.55	-1113752.0
RS	549419.50	N/A	522271.34	N/A	2131342.63	N/A

5.3.4 Comparison Table under Response of the Structure

Table 5.27: Comparison Table

Response Items	Storey Level	Dynamic Loading				Static Loading			
		EQX	EQY	WX	WY	EQX	EQY	WX	WY
Lateral Load to Stories (kips)	Roof	516.83	516.83	109.08	119.70	516.83	516.83	109.08	119.70
	GF	8.11	8.11	67.76	74.93	2.612	2.61	67.76	74.93
Maximum Storey Displacement (in)	Roof	64.68	42.40	32.62	21.55	47.34	34.40	23.25	17.34
	GF	0.44	0.3776	0.2824	0.2625	0.2958	0.2901	0.2006	0.2103
Maximum Storey Drift	Roof	0.0063	0.0043	0.0026	0.0015	0.0050	0.0037	0.0019	0.0015
	GF	0.0023	0.0021	0.0019	0.0014	0.0015	0.0016	0.0010	0.0011
Storey Shear (kips)	Roof	-638.78	-590.27	-176.91	-164.52	1886.22	2731.67	954.96	1299.40
	GF	-15282.39	-13245.59	-10840.79	-10599.48	95945.44	98174.04	103404.56	103989.21
Storey Overturning Moments (kip-ft.)	Roof	-92.33	55.68	-51.25	33.92	-0.0001	-0.0001	-2.274E-05	-0.0001
	GF	-4128674.0	3658916.28	-2378558.0	2313708.47	-2956895.0	2956752.35	-1693263.0	1860052.01
Storey Stiffness (kip/in)	Roof	1813.57	2636.21	1115.55	1444.08	1886.22	2731.67	954.96	1299.40
	GF	97654.36	99185.14	102935.87	103832.08	95945.44	98174.04	103404.56	103989.21

Chapter 6

CONCLUSIONS & RECOMMENDATIONS

CHAPTER SIX
CONCLUSIONS & RECOMMENDATIONS

6.1 CONCLUSIONS

From the analysis & findings as mentioned in the Chapter 5, it can be concluded that:

- Dynamic loading shows greater story displacement due to earthquake and wind compared to static loading. Under dynamic loading structure shows 36.05% (Average of X & Y directions) greater storey displacement due to earthquake and 38.91% (Average of X & Y directions) due to wind than the structure under static loading.
- Storey drift is 28.05% (Average of X & Y directions) greater due to earthquake and 36.36% (Average of X & Y directions) greater due to wind under the dynamic loading than the static loading.
- Under dynamic loading, structure has to resist higher story shear, higher lateral loads and higher story overturning moments due to earthquake & wind compared to static loading.

From above conclusions, we can realize that the response of structure based on Earthquake & Wind under static analysis result don't show precise effects of Earthquake & Wind compared to dynamic analysis. Because in dynamic analysis, we consider response spectrum cases, P-delta cases. If on high-rise structures, since the response is the important factor for its stability condition that case we should have correct responses and only dynamic analysis can confirm that results.

6.2 RECOMMENDATIONS

Based on the objectives, scopes and limitations of the study (stated in *Chapter 1*), few *recommendations* can be proposed for further studies:

- This study was conducted based on 41 storied building with edge supported floor system, further analyses considering more number of stories and also the other floor system to observe the response of structure according to Earthquake and Wind loads.
- The study was conducted on dual structural system, further analyses can be done considering the bracing system.

- Further study, to reduce the top story displacement under static and dynamic analysis, it is recommended to change the location of shear walls or increase the dimension or thickness of shear walls.

- It is recommended that materials properties can be changed to observe the variation in result. The compressive strength of concrete can be taken more than 5,000 psi and yield strength of reinforcing bar can be taken as 72,500 psi or 80,000 psi to observe the response of structure.

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APPENDIX

Table 4.A.1: Mass Irregularity in X-direction (both Static and Dynamic Loadings).

Story ID	Mass (lb-s^2/ft)	Irregular (Mi>1.5Mi+1) or, (Mi>1.5Mi-1)		Extreme Irregular (Mi>2Mi+1) or, (Mi>2Mi-1)	
Base	0.00	N/A	N/A	N/A	N/A
BF	610393.13	NO	N/A	NO	N/A
GF	580502.76	NO	NO	NO	NO
1F	691337.85	YES	NO	NO	NO
2F	376397.55	NO	NO	NO	NO
3F	376397.55	NO	NO	NO	NO
4F	376397.55	NO	NO	NO	NO
5F	376397.55	NO	NO	NO	NO
6F	376397.55	NO	NO	NO	NO
7F	376397.55	NO	NO	NO	NO
8F	376397.55	NO	NO	NO	NO
9F	376397.55	NO	NO	NO	NO
10F	369054.67	NO	NO	NO	NO
11F	362606.93	NO	NO	NO	NO
12F	362606.93	NO	NO	NO	NO
13F	362606.93	NO	NO	NO	NO
14F	362606.93	NO	NO	NO	NO
15F	362606.93	NO	NO	NO	NO
16F	362606.93	NO	NO	NO	NO
17F	362606.93	NO	NO	NO	NO
18F	362606.93	NO	NO	NO	NO
19F	362425.62	NO	NO	NO	NO
20F	354588.04	NO	NO	NO	NO
21F	348241.31	NO	NO	NO	NO
22F	348241.31	NO	NO	NO	NO
23F	348241.31	NO	NO	NO	NO
24F	348241.31	NO	NO	NO	NO
25F	348241.31	NO	NO	NO	NO
26F	348241.31	NO	NO	NO	NO
27F	348241.31	NO	NO	NO	NO
28F	348241.31	NO	NO	NO	NO
29F	348241.31	NO	NO	NO	NO
30F	342561.27	NO	NO	NO	NO
31F	337991.85	NO	NO	NO	NO
32F	337991.85	NO	NO	NO	NO
33F	337991.85	NO	NO	NO	NO
34F	337991.85	NO	NO	NO	NO
35F	337991.85	NO	NO	NO	NO
36F	337991.85	NO	NO	NO	NO
37F	337991.85	NO	NO	NO	NO
38F	337991.85	NO	NO	NO	NO
39F	337991.85	NO	NO	NO	NO
40F	337991.85	NO	NO	NO	NO
Roof	288590.09	YES	NO	YES	NO

Table 4.A.2: Mass Irregularity in Y-direction (both Static and Dynamic Loadings).

Story ID	Mass (lb-s^2/ft)	Irregular (Mi>1.5Mi+1) or, (Mi>1.5Mi-1)		Extreme Irregular (Mi>2Mi+1) or, (Mi>2Mi-1)	
Base	0.00	*N/A*	*N/A*	*N/A*	*N/A*
BF	610393.13	*NO*	*N/A*	*NO*	*N/A*
GF	580502.76	*NO*	*NO*	*NO*	*NO*
1F	691337.85	*YES*	*NO*	*NO*	*NO*
2F	376397.55	*NO*	*NO*	*NO*	*NO*
3F	376397.55	*NO*	*NO*	*NO*	*NO*
4F	376397.55	*NO*	*NO*	*NO*	*NO*
5F	376397.55	*NO*	*NO*	*NO*	*NO*
6F	376397.55	*NO*	*NO*	*NO*	*NO*
7F	376397.55	*NO*	*NO*	*NO*	*NO*
8F	376397.55	*NO*	*NO*	*NO*	*NO*
9F	376397.55	*NO*	*NO*	*NO*	*NO*
10F	369054.67	*NO*	*NO*	*NO*	*NO*
11F	362606.93	*NO*	*NO*	*NO*	*NO*
12F	362606.93	*NO*	*NO*	*NO*	*NO*
13F	362606.93	*NO*	*NO*	*NO*	*NO*
14F	362606.93	*NO*	*NO*	*NO*	*NO*
15F	362606.93	*NO*	*NO*	*NO*	*NO*
16F	362606.93	*NO*	*NO*	*NO*	*NO*
17F	362606.93	*NO*	*NO*	*NO*	*NO*
18F	362606.93	*NO*	*NO*	*NO*	*NO*
19F	362425.62	*NO*	*NO*	*NO*	*NO*
20F	354588.04	*NO*	*NO*	*NO*	*NO*
21F	348241.31	*NO*	*NO*	*NO*	*NO*
22F	348241.31	*NO*	*NO*	*NO*	*NO*
23F	348241.31	*NO*	*NO*	*NO*	*NO*
24F	348241.31	*NO*	*NO*	*NO*	*NO*
25F	348241.31	*NO*	*NO*	*NO*	*NO*
26F	348241.31	*NO*	*NO*	*NO*	*NO*
27F	348241.31	*NO*	*NO*	*NO*	*NO*
28F	348241.31	*NO*	*NO*	*NO*	*NO*
29F	348241.31	*NO*	*NO*	*NO*	*NO*
30F	342561.27	*NO*	*NO*	*NO*	*NO*
31F	337991.85	*NO*	*NO*	*NO*	*NO*
32F	337991.85	*NO*	*NO*	*NO*	*NO*
33F	337991.85	*NO*	*NO*	*NO*	*NO*
34F	337991.85	*NO*	*NO*	*NO*	*NO*
35F	337991.85	*NO*	*NO*	*NO*	*NO*
36F	337991.85	*NO*	*NO*	*NO*	*NO*
37F	337991.85	*NO*	*NO*	*NO*	*NO*
38F	337991.85	*NO*	*NO*	*NO*	*NO*
39F	337991.85	*NO*	*NO*	*NO*	*NO*
40F	337991.85	*NO*	*NO*	*NO*	*NO*
Roof	288590.09	*YES*	*NO*	*YES*	*NO*

Table 4.A.3: Stiffness Irregularity (Soft Storey) under Earthquake in X-direction (Dynamic Loading).

Story ID	Story Stiffness (kip/in)	Irregular Ki<0.7Ki+1 or, Ki<0.8/3(Ki+1+Ki+2+Ki+3)		Extreme Irregular Ki<0.6Ki+1 or, Ki<0.7/3(Ki+1+Ki+2+Ki+3)	
Base	0.00	*N/A*	*N/A*	*N/A*	*N/A*
BF	147517.07	*NO*	*NO*	*NO*	*NO*
GF	97654.36	*NO*	*NO*	*NO*	*NO*
1F	45908.38	*NO*	*NO*	*NO*	*NO*
2F	35211.83	*NO*	*NO*	*NO*	*NO*
3F	27998.02	*NO*	*NO*	*NO*	*NO*
4F	23882.25	*NO*	*NO*	*NO*	*NO*
5F	21227.01	*NO*	*NO*	*NO*	*NO*
6F	19388.33	*NO*	*NO*	*NO*	*NO*
7F	18050.51	*NO*	*NO*	*NO*	*NO*
8F	17040.75	*NO*	*NO*	*NO*	*NO*
9F	16256.96	*NO*	*NO*	*NO*	*NO*
10F	15651.51	*NO*	*NO*	*NO*	*NO*
11F	15073.60	*NO*	*NO*	*NO*	*NO*
12F	14635.87	*NO*	*NO*	*NO*	*NO*
13F	14276.53	*NO*	*NO*	*NO*	*NO*
14F	13976.61	*NO*	*NO*	*NO*	*NO*
15F	13707.94	*NO*	*NO*	*NO*	*NO*
16F	13461.82	*NO*	*NO*	*NO*	*NO*
17F	13244.00	*NO*	*NO*	*NO*	*NO*
18F	13046.26	*NO*	*NO*	*NO*	*NO*
19F	12861.70	*NO*	*NO*	*NO*	*NO*
20F	12685.31	*NO*	*NO*	*NO*	*NO*
21F	12494.65	*NO*	*NO*	*NO*	*NO*
22F	12337.10	*NO*	*NO*	*NO*	*NO*
23F	12219.16	*NO*	*NO*	*NO*	*NO*
24F	12088.66	*NO*	*NO*	*NO*	*NO*
25F	11953.72	*NO*	*NO*	*NO*	*NO*
26F	11809.18	*NO*	*NO*	*NO*	*NO*
27F	11649.63	*NO*	*NO*	*NO*	*NO*
28F	11468.57	*NO*	*NO*	*NO*	*NO*
29F	11257.80	*NO*	*NO*	*NO*	*NO*
30F	11012.59	*NO*	*NO*	*NO*	*NO*
31F	10698.01	*NO*	*NO*	*NO*	*NO*
32F	10361.20	*NO*	*NO*	*NO*	*NO*
33F	10010.96	*NO*	*NO*	*NO*	*NO*
34F	9573.24	*NO*	*NO*	*NO*	*NO*
35F	9037.80	*NO*	*NO*	*NO*	*NO*
36F	8372.78	*NO*	*NO*	*NO*	*NO*
37F	7541.93	*NO*	*NO*	*NO*	*NO*
38F	6504.10	*NO*	*NO*	*NO*	*NO*
39F	5218.70	*NO*	*NO*	*NO*	*NO*
40F	3654.01	*NO*	*NO*	*NO*	*NO*
Roof	1813.57	*NO*	*NO*	*NO*	*NO*

Table 4.A.4: Stiffness Irregularity (Soft Storey) under Earthquake in Y-direction (Dynamic Loading).

Story ID	Story Stiffness (kip/in)	Irregular Ki<0.7Ki+1 or, Ki<0.8/3(Ki+1+Ki+2+Ki+3)		Extreme Irregular Ki<0.6Ki+1 or, Ki<0.7/3(Ki+1+Ki+2+Ki+3)	
Base	0.00	*N/A*	*N/A*	*N/A*	*N/A*
BF	177237.62	*NO*	*NO*	*NO*	*NO*
GF	99185.14	*NO*	*NO*	*NO*	*NO*
1F	65134.63	*NO*	*NO*	*NO*	*NO*
2F	45240.99	*NO*	*NO*	*NO*	*NO*
3F	36862.66	*NO*	*NO*	*NO*	*NO*
4F	32285.13	*NO*	*NO*	*NO*	*NO*
5F	29395.51	*NO*	*NO*	*NO*	*NO*
6F	27419.85	*NO*	*NO*	*NO*	*NO*
7F	25994.42	*NO*	*NO*	*NO*	*NO*
8F	24924.20	*NO*	*NO*	*NO*	*NO*
9F	24094.74	*NO*	*NO*	*NO*	*NO*
10F	23451.43	*NO*	*NO*	*NO*	*NO*
11F	22771.64	*NO*	*NO*	*NO*	*NO*
12F	22290.13	*NO*	*NO*	*NO*	*NO*
13F	21881.03	*NO*	*NO*	*NO*	*NO*
14F	21520.09	*NO*	*NO*	*NO*	*NO*
15F	21173.42	*NO*	*NO*	*NO*	*NO*
16F	20857.12	*NO*	*NO*	*NO*	*NO*
17F	20560.15	*NO*	*NO*	*NO*	*NO*
18F	20273.34	*NO*	*NO*	*NO*	*NO*
19F	19987.74	*NO*	*NO*	*NO*	*NO*
20F	19709.16	*NO*	*NO*	*NO*	*NO*
21F	19336.28	*NO*	*NO*	*NO*	*NO*
22F	19014.86	*NO*	*NO*	*NO*	*NO*
23F	18797.13	*NO*	*NO*	*NO*	*NO*
24F	18521.23	*NO*	*NO*	*NO*	*NO*
25F	18232.68	*NO*	*NO*	*NO*	*NO*
26F	17922.92	*NO*	*NO*	*NO*	*NO*
27F	17585.57	*NO*	*NO*	*NO*	*NO*
28F	17212.30	*NO*	*NO*	*NO*	*NO*
29F	16789.19	*NO*	*NO*	*NO*	*NO*
30F	16336.89	*NO*	*NO*	*NO*	*NO*
31F	15677.91	*NO*	*NO*	*NO*	*NO*
32F	15009.16	*NO*	*NO*	*NO*	*NO*
33F	14438.98	*NO*	*NO*	*NO*	*NO*
34F	13722.86	*NO*	*NO*	*NO*	*NO*
35F	12896.53	*NO*	*NO*	*NO*	*NO*
36F	11911.17	*NO*	*NO*	*NO*	*NO*
37F	10717.98	*NO*	*NO*	*NO*	*NO*
38F	9255.76	*NO*	*NO*	*NO*	*NO*
39F	7457.39	*NO*	*NO*	*NO*	*NO*
40F	5251.32	*NO*	*NO*	*NO*	*NO*
Roof	2636.21	*NO*	*NO*	*NO*	*NO*

Table 4.A.5: Stiffness Irregularity (Soft Storey) under Earthquake in X-direction (Static Loading).

Story ID	Story Stiffness (kip/in)	Irregular Ki<0.7Ki+1 or, Ki<0.8/3(Ki+1+Ki+2+Ki+3)		Extreme Irregular Ki<0.6Ki+1 or, Ki<0.7/3(Ki+1+Ki+2+Ki+3)	
Base	0.00	*N/A*	*N/A*	*N/A*	*N/A*
BF	145413.2568	*NO*	*NO*	*NO*	*NO*
GF	91001.0122	*NO*	*NO*	*NO*	*NO*
1F	41247.5864	*NO*	*NO*	*NO*	*NO*
2F	31006.8858	*NO*	*NO*	*NO*	*NO*
3F	24585.3526	*NO*	*NO*	*NO*	*NO*
4F	20970.4636	*NO*	*NO*	*NO*	*NO*
5F	18660.1446	*NO*	*NO*	*NO*	*NO*
6F	17073.682	*NO*	*NO*	*NO*	*NO*
7F	15929.1441	*NO*	*NO*	*NO*	*NO*
8F	15073.1001	*NO*	*NO*	*NO*	*NO*
9F	14415.4453	*NO*	*NO*	*NO*	*NO*
10F	13915.9156	*NO*	*NO*	*NO*	*NO*
11F	13429.2793	*NO*	*NO*	*NO*	*NO*
12F	13062.0332	*NO*	*NO*	*NO*	*NO*
13F	12765.1835	*NO*	*NO*	*NO*	*NO*
14F	12520.1901	*NO*	*NO*	*NO*	*NO*
15F	12314.1066	*NO*	*NO*	*NO*	*NO*
16F	12137.1491	*NO*	*NO*	*NO*	*NO*
17F	11981.3695	*NO*	*NO*	*NO*	*NO*
18F	11840.2368	*NO*	*NO*	*NO*	*NO*
19F	11707.6651	*NO*	*NO*	*NO*	*NO*
20F	11589.0865	*NO*	*NO*	*NO*	*NO*
21F	11408.9916	*NO*	*NO*	*NO*	*NO*
22F	11343.2351	*NO*	*NO*	*NO*	*NO*
23F	11248.0628	*NO*	*NO*	*NO*	*NO*
24F	11156.2007	*NO*	*NO*	*NO*	*NO*
25F	11059.3644	*NO*	*NO*	*NO*	*NO*
26F	10953.5926	*NO*	*NO*	*NO*	*NO*
27F	10833.6387	*NO*	*NO*	*NO*	*NO*
28F	10693.5659	*NO*	*NO*	*NO*	*NO*
29F	10523.2023	*NO*	*NO*	*NO*	*NO*
30F	10335.4576	*NO*	*NO*	*NO*	*NO*
31F	10015.9767	*NO*	*NO*	*NO*	*NO*
32F	9797.9943	*NO*	*NO*	*NO*	*NO*
33F	9485.9557	*NO*	*NO*	*NO*	*NO*
34F	9109.3216	*NO*	*NO*	*NO*	*NO*
35F	8634.4615	*NO*	*NO*	*NO*	*NO*
36F	8030.6971	*NO*	*NO*	*NO*	*NO*
37F	7259.3453	*NO*	*NO*	*NO*	*NO*
38F	6276.7135	*NO*	*NO*	*NO*	*NO*
39F	5041.2938	*NO*	*NO*	*NO*	*NO*
40F	3517.1664	*NO*	*NO*	*NO*	*NO*
Roof	1735.6271	*NO*	*NO*	*NO*	*NO*

Table 4.A.6: Stiffness Irregularity (Soft Storey) under Earthquake in Y-direction (Static Loading).

Story ID	Story Stiffness (kip/in)	Irregular Ki<0.7Ki+1 or, Ki<0.8/3(Ki+1+Ki+2+Ki+3)		Extreme Irregular Ki<0.6Ki+1 or, Ki<0.7/3(Ki+1+Ki+2+Ki+3)	
Base	0.00	*N/A*	*N/A*	*N/A*	*N/A*
BF	176130.0789	*NO*	*NO*	*NO*	*NO*
GF	88277.4716	*NO*	*NO*	*NO*	*NO*
1F	55826.4976	*NO*	*NO*	*NO*	*NO*
2F	38132.5578	*NO*	*NO*	*NO*	*NO*
3F	30826.5205	*NO*	*NO*	*NO*	*NO*
4F	26873.2019	*NO*	*NO*	*NO*	*NO*
5F	24396.3198	*NO*	*NO*	*NO*	*NO*
6F	22715.4203	*NO*	*NO*	*NO*	*NO*
7F	21512.3286	*NO*	*NO*	*NO*	*NO*
8F	20617.2364	*NO*	*NO*	*NO*	*NO*
9F	19931.2731	*NO*	*NO*	*NO*	*NO*
10F	19409.0987	*NO*	*NO*	*NO*	*NO*
11F	18850.9506	*NO*	*NO*	*NO*	*NO*
12F	18452.407	*NO*	*NO*	*NO*	*NO*
13F	18120.7265	*NO*	*NO*	*NO*	*NO*
14F	17838.2335	*NO*	*NO*	*NO*	*NO*
15F	17589.5463	*NO*	*NO*	*NO*	*NO*
16F	17364.0652	*NO*	*NO*	*NO*	*NO*
17F	17153.1174	*NO*	*NO*	*NO*	*NO*
18F	16949.6089	*NO*	*NO*	*NO*	*NO*
19F	16747.0376	*NO*	*NO*	*NO*	*NO*
20F	16553.1405	*NO*	*NO*	*NO*	*NO*
21F	16209.3305	*NO*	*NO*	*NO*	*NO*
22F	16083.0074	*NO*	*NO*	*NO*	*NO*
23F	15897.4558	*NO*	*NO*	*NO*	*NO*
24F	15705.6569	*NO*	*NO*	*NO*	*NO*
25F	15501.7208	*NO*	*NO*	*NO*	*NO*
26F	15281.1259	*NO*	*NO*	*NO*	*NO*
27F	15037.4552	*NO*	*NO*	*NO*	*NO*
28F	14763.0986	*NO*	*NO*	*NO*	*NO*
29F	14445.4326	*NO*	*NO*	*NO*	*NO*
30F	14111.9597	*NO*	*NO*	*NO*	*NO*
31F	13477.817	*NO*	*NO*	*NO*	*NO*
32F	13127.8712	*NO*	*NO*	*NO*	*NO*
33F	12642.3021	*NO*	*NO*	*NO*	*NO*
34F	12086.1547	*NO*	*NO*	*NO*	*NO*
35F	11419.4303	*NO*	*NO*	*NO*	*NO*
36F	10604.2751	*NO*	*NO*	*NO*	*NO*
37F	9590.6042	*NO*	*NO*	*NO*	*NO*
38F	8317.2452	*NO*	*NO*	*NO*	*NO*
39F	6718.029	*NO*	*NO*	*NO*	*NO*
40F	4724.9077	*NO*	*NO*	*NO*	*NO*
Roof	2358.6734	*NO*	*NO*	*NO*	*NO*

Table 4.A.7: Discontinuity Capacity (Weak Storey) under Earthquake (Dynamic Loading).

X-Direction				*Y-Direction*			
Story ID	Story Shear (kip)	Irregular Stri<0.8Stri+1	Extreme Irregular Stri<0.65Stri+1	Story ID	Story Shear (kip)	Irregular Stri<0.8Stri+1	Extreme Irregular Stri<0.65Stri+1
Base	0.00	*N/A*	*N/A*	Base	0.00	*N/A*	*N/A*
BF	-15301.12	*NO*	*NO*	BF	-13259.344	*NO*	*NO*
GF	-15283.108	*NO*	*NO*	GF	-13245.624	*NO*	*NO*
1F	-15245.401	*NO*	*NO*	1F	-13216.68	*NO*	*NO*
2F	-15173.658	*NO*	*NO*	2F	-13162.084	*NO*	*NO*
3F	-15115.56	*NO*	*NO*	3F	-13117.968	*NO*	*NO*
4F	-15038.989	*NO*	*NO*	4F	-13059.94	*NO*	*NO*
5F	-14944.186	*NO*	*NO*	5F	-12988.177	*NO*	*NO*
6F	-14831.456	*NO*	*NO*	6F	-12902.828	*NO*	*NO*
7F	-14701.099	*NO*	*NO*	7F	-12803.972	*NO*	*NO*
8F	-14553.373	*NO*	*NO*	8F	-12691.611	*NO*	*NO*
9F	-14388.489	*NO*	*NO*	9F	-12565.679	*NO*	*NO*
10F	-14206.605	*NO*	*NO*	10F	-12426.052	*NO*	*NO*
11F	-14011.556	*NO*	*NO*	11F	-12275.434	*NO*	*NO*
12F	-13803.468	*NO*	*NO*	12F	-12113.701	*NO*	*NO*
13F	-13579.072	*NO*	*NO*	13F	-11938.093	*NO*	*NO*
14F	-13338.339	*NO*	*NO*	14F	-11748.34	*NO*	*NO*
15F	-13081.211	*NO*	*NO*	15F	-11544.156	*NO*	*NO*
16F	-12807.603	*NO*	*NO*	16F	-11325.237	*NO*	*NO*
17F	-12517.406	*NO*	*NO*	17F	-11091.265	*NO*	*NO*
18F	-12210.488	*NO*	*NO*	18F	-10841.912	*NO*	*NO*
19F	-11886.693	*NO*	*NO*	19F	-10576.838	*NO*	*NO*
20F	-11546.007	*NO*	*NO*	20F	-10295.827	*NO*	*NO*
21F	-11195.482	*NO*	*NO*	21F	-10004.548	*NO*	*NO*
22F	-10834.118	*NO*	*NO*	22F	-9702.062	*NO*	*NO*
23F	-10455.746	*NO*	*NO*	23F	-9383.065	*NO*	*NO*
24F	-10060.155	*NO*	*NO*	24F	-9047.215	*NO*	*NO*
25F	-9647.124	*NO*	*NO*	25F	-8694.165	*NO*	*NO*
26F	-9216.423	*NO*	*NO*	26F	-8323.57	*NO*	*NO*
27F	-8767.813	*NO*	*NO*	27F	-7935.082	*NO*	*NO*
28F	-8301.045	*NO*	*NO*	28F	-7528.353	*NO*	*NO*
29F	-7815.861	*NO*	*NO*	29F	-7103.032	*NO*	*NO*
30F	-7311.995	*NO*	*NO*	30F	-6658.769	*NO*	*NO*
31F	-6797.306	*NO*	*NO*	31F	-6202.428	*NO*	*NO*
32F	-6270.436	*NO*	*NO*	32F	-5732.759	*NO*	*NO*
33F	-5724.58	*NO*	*NO*	33F	-5243.64	*NO*	*NO*
34F	-5159.456	*NO*	*NO*	34F	-4734.735	*NO*	*NO*
35F	-4574.78	*NO*	*NO*	35F	-4205.712	*NO*	*NO*
36F	-3970.257	*NO*	*NO*	36F	-3656.239	*NO*	*NO*
37F	-3345.583	*NO*	*NO*	37F	-3085.982	*NO*	*NO*
38F	-2700.44	*NO*	*NO*	38F	-2494.604	*NO*	*NO*
39F	-2034.496	*NO*	*NO*	39F	-1881.762	*NO*	*NO*
40F	-1347.402	*NO*	*NO*	40F	-1247.106	*NO*	*NO*
Roof	-638.785	*NO*	*NO*	Roof	-590.273	*NO*	*NO*

Table 4.A.8: Discontinuity Capacity (Weak Storey) under Earthquake (Static Loading).

X-Direction				*Y-Direction*			
Story ID	Story Shear (kip)	Irregular Stri<0.8Stri+1	Extreme Irregular Stri<0.65Stri+1	Story ID	Story Shear (kip)	Irregular Stri<0.8Stri+1	Extreme Irregular Stri<0.65Stri+1
Base	0.00	*N/A*	*N/A*	Base	0.00	*N/A*	*N/A*
BF	-10108.947	*NO*	*NO*	BF	-10108.432	*NO*	*NO*
GF	-10106.335	*NO*	*NO*	GF	-10105.82	*NO*	*NO*
1F	-10098.224	*NO*	*NO*	1F	-10097.709	*NO*	*NO*
2F	-10082.396	*NO*	*NO*	2F	-10081.882	*NO*	*NO*
3F	-10068.771	*NO*	*NO*	3F	-10068.258	*NO*	*NO*
4F	-10049.472	*NO*	*NO*	4F	-10048.96	*NO*	*NO*
5F	-10023.711	*NO*	*NO*	5F	-10023.2	*NO*	*NO*
6F	-9990.738	*NO*	*NO*	6F	-9990.229	*NO*	*NO*
7F	-9949.836	*NO*	*NO*	7F	-9949.331	*NO*	*NO*
8F	-9900.316	*NO*	*NO*	8F	-9899.814	*NO*	*NO*
9F	-9841.51	*NO*	*NO*	9F	-9841.012	*NO*	*NO*
10F	-9772.77	*NO*	*NO*	10F	-9772.278	*NO*	*NO*
11F	-9694.949	*NO*	*NO*	11F	-9694.462	*NO*	*NO*
12F	-9607.643	*NO*	*NO*	12F	-9607.162	*NO*	*NO*
13F	-9508.974	*NO*	*NO*	13F	-9508.499	*NO*	*NO*
14F	-9398.375	*NO*	*NO*	14F	-9397.908	*NO*	*NO*
15F	-9275.291	*NO*	*NO*	15F	-9274.833	*NO*	*NO*
16F	-9139.178	*NO*	*NO*	16F	-9138.729	*NO*	*NO*
17F	-8989.501	*NO*	*NO*	17F	-8989.061	*NO*	*NO*
18F	-8825.733	*NO*	*NO*	18F	-8825.303	*NO*	*NO*
19F	-8647.357	*NO*	*NO*	19F	-8646.938	*NO*	*NO*
20F	-8453.955	*NO*	*NO*	20F	-8453.548	*NO*	*NO*
21F	-8249.257	*NO*	*NO*	21F	-8248.863	*NO*	*NO*
22F	-8032.549	*NO*	*NO*	22F	-8032.169	*NO*	*NO*
23F	-7799.862	*NO*	*NO*	23F	-7799.496	*NO*	*NO*
24F	-7550.733	*NO*	*NO*	24F	-7550.381	*NO*	*NO*
25F	-7284.7	*NO*	*NO*	25F	-7284.363	*NO*	*NO*
26F	-7001.31	*NO*	*NO*	26F	-7000.988	*NO*	*NO*
27F	-6700.114	*NO*	*NO*	27F	-6699.809	*NO*	*NO*
28F	-6380.667	*NO*	*NO*	28F	-6380.379	*NO*	*NO*
29F	-6042.53	*NO*	*NO*	29F	-6042.259	*NO*	*NO*
30F	-5685.267	*NO*	*NO*	30F	-5685.014	*NO*	*NO*
31F	-5314.307	*NO*	*NO*	31F	-5314.074	*NO*	*NO*
32F	-4928.64	*NO*	*NO*	32F	-4928.426	*NO*	*NO*
33F	-4523.137	*NO*	*NO*	33F	-4522.944	*NO*	*NO*
34F	-4097.392	*NO*	*NO*	34F	-4097.219	*NO*	*NO*
35F	-3651	*NO*	*NO*	35F	-3650.847	*NO*	*NO*
36F	-3183.56	*NO*	*NO*	36F	-3183.428	*NO*	*NO*
37F	-2694.675	*NO*	*NO*	37F	-2694.565	*NO*	*NO*
38F	-2183.949	*NO*	*NO*	38F	-2183.861	*NO*	*NO*
39F	-1650.992	*NO*	*NO*	39F	-1650.927	*NO*	*NO*
40F	-1095.415	*NO*	*NO*	40F	-1095.372	*NO*	*NO*
Roof	-516.832	*NO*	*NO*	Roof	-516.811	*NO*	*NO*

Table 4.A.9: Storey Drift under Earthquake (Dynamic Loading).

X-Direction					Y-Direction				
Story ID	Story height, hi (ft.)	Drift X, Δx (in)	Allowable Story Drift, Δa (in)	Drift Check	Story ID	Story height, hi (ft.)	Drift Y, ΔY (in)	Allowable Story Drift, Δa (in)	Drift Check
Base	0.0	0.00	0.00	*N/A*	Base	0.0	0.00	0.00	*N/A*
1BF	12.0	0.103814	2.88	*Safe*	1BF	12.0	0.074811	2.88	*Safe*
GF	12.0	0.164138	2.88	*Safe*	GF	12.0	0.148928	2.88	*Safe*
1F	10.0	0.359661	2.40	*Safe*	1F	10.0	0.235028	2.40	*Safe*
2F	10.0	0.475305	2.40	*Safe*	2F	10.0	0.342949	2.40	*Safe*
3F	10.0	0.596985	2.40	*Safe*	3F	10.0	0.422823	2.40	*Safe*
4F	10.0	0.696661	2.40	*Safe*	4F	10.0	0.483042	2.40	*Safe*
5F	10.0	0.778703	2.40	*Safe*	5F	10.0	0.529455	2.40	*Safe*
6F	10.0	0.845722	2.40	*Safe*	6F	10.0	0.565307	2.40	*Safe*
7F	10.0	0.899917	2.40	*Safe*	7F	10.0	0.592853	2.40	*Safe*
8F	10.0	0.943131	2.40	*Safe*	8F	10.0	0.613751	2.40	*Safe*
9F	10.0	0.976882	2.40	*Safe*	9F	10.0	0.629249	2.40	*Safe*
10F	10.0	1.001381	2.40	*Safe*	10F	10.0	0.639813	2.40	*Safe*
11F	10.0	1.025155	2.40	*Safe*	11F	10.0	0.651267	2.40	*Safe*
12F	10.0	1.039751	2.40	*Safe*	12F	10.0	0.656847	2.40	*Safe*
13F	10.0	1.048287	2.40	*Safe*	13F	10.0	0.659589	2.40	*Safe*
14F	10.0	1.051560	2.40	*Safe*	14F	10.0	0.659845	2.40	*Safe*
15F	10.0	1.050292	2.40	*Safe*	15F	10.0	0.658034	2.40	*Safe*
16F	10.0	1.045085	2.40	*Safe*	16F	10.0	0.654454	2.40	*Safe*
17F	10.0	1.036469	2.40	*Safe*	17F	10.0	0.649358	2.40	*Safe*
18F	10.0	1.024904	2.40	*Safe*	18F	10.0	0.642954	2.40	*Safe*
19F	10.0	1.010839	2.40	*Safe*	19F	10.0	0.635448	2.40	*Safe*
20F	10.0	0.993948	2.40	*Safe*	20F	10.0	0.626593	2.40	*Safe*
21F	10.0	0.980278	2.40	*Safe*	21F	10.0	0.622149	2.40	*Safe*
22F	10.0	0.955156	2.40	*Safe*	22F	10.0	0.608307	2.40	*Safe*
23F	10.0	0.930809	2.40	*Safe*	23F	10.0	0.595579	2.40	*Safe*
24F	10.0	0.904173	2.40	*Safe*	24F	10.0	0.581724	2.40	*Safe*
25F	10.0	0.875870	2.40	*Safe*	25F	10.0	0.566866	2.40	*Safe*
26F	10.0	0.846089	2.40	*Safe*	26F	10.0	0.551061	2.40	*Safe*
27F	10.0	0.815078	2.40	*Safe*	27F	10.0	0.534412	2.40	*Safe*
28F	10.0	0.783079	2.40	*Safe*	28F	10.0	0.517036	2.40	*Safe*
29F	10.0	0.750549	2.40	*Safe*	29F	10.0	0.499185	2.40	*Safe*
30F	10.0	0.716342	2.40	*Safe*	30F	10.0	0.479744	2.40	*Safe*
31F	10.0	0.688355	2.40	*Safe*	31F	10.0	0.468502	2.40	*Safe*
32F	10.0	0.650352	2.40	*Safe*	32F	10.0	0.445231	2.40	*Safe*
33F	10.0	0.614578	2.40	*Safe*	33F	10.0	0.423628	2.40	*Safe*
34F	10.0	0.578163	2.40	*Safe*	34F	10.0	0.400912	2.40	*Safe*
35F	10.0	0.542240	2.40	*Safe*	35F	10.0	0.377763	2.40	*Safe*
36F	10.0	0.507408	2.40	*Safe*	36F	10.0	0.354562	2.40	*Safe*
37F	10.0	0.474469	2.40	*Safe*	37F	10.0	0.331851	2.40	*Safe*
38F	10.0	0.444370	2.40	*Safe*	38F	10.0	0.310316	2.40	*Safe*
39F	10.0	0.418127	2.40	*Safe*	39F	10.0	0.290771	2.40	*Safe*
40F	10.0	0.397706	2.40	*Safe*	40F	10.0	0.274768	2.40	*Safe*
Roof	10.0	0.380457	2.40	*Safe*	Roof	10.0	0.260204	2.40	*Safe*

Table 4.A.10: Storey Drift under Earthquake (Static Loading).

X-Direction					*Y-Direction*				
Story ID	Story height, hi (ft.)	Drift X, Δx (in)	Allowable Story Drift, Δa (in)	Drift Check	Story ID	Story height, hi (ft.)	Drift Y, ΔY (in)	Allowable Story Drift, Δa (in)	Drift Check
Base	0.0	0.00	0.00	*N/A*	Base	0.0	0.00	0.00	*N/A*
1BF	12.0	0.069519	2.88	*Safe*	1BF	12.0	0.057392	2.88	*Safe*
GF	12.0	0.111057	2.88	*Safe*	GF	12.0	0.114478	2.88	*Safe*
1F	10.0	0.24482	2.40	*Safe*	1F	10.0	0.180877	2.40	*Safe*
2F	10.0	0.325166	2.40	*Safe*	2F	10.0	0.26439	2.40	*Safe*
3F	10.0	0.409543	2.40	*Safe*	3F	10.0	0.32661	2.40	*Safe*
4F	10.0	0.47922	2.40	*Safe*	4F	10.0	0.37394	2.40	*Safe*
5F	10.0	0.537172	2.40	*Safe*	5F	10.0	0.410849	2.40	*Safe*
6F	10.0	0.585154	2.40	*Safe*	6F	10.0	0.439799	2.40	*Safe*
7F	10.0	0.624631	2.40	*Safe*	7F	10.0	0.462494	2.40	*Safe*
8F	10.0	0.65682	2.40	*Safe*	8F	10.0	0.480172	2.40	*Safe*
9F	10.0	0.682706	2.40	*Safe*	9F	10.0	0.493747	2.40	*Safe*
10F	10.0	0.702273	2.40	*Safe*	10F	10.0	0.50349	2.40	*Safe*
11F	10.0	0.721926	2.40	*Safe*	11F	10.0	0.514269	2.40	*Safe*
12F	10.0	0.73554	2.40	*Safe*	12F	10.0	0.520645	2.40	*Safe*
13F	10.0	0.744915	2.40	*Safe*	13F	10.0	0.524731	2.40	*Safe*
14F	10.0	0.750658	2.40	*Safe*	14F	10.0	0.526841	2.40	*Safe*
15F	10.0	0.753225	2.40	*Safe*	15F	10.0	0.527292	2.40	*Safe*
16F	10.0	0.752992	2.40	*Safe*	16F	10.0	0.526301	2.40	*Safe*
17F	10.0	0.75029	2.40	*Safe*	17F	10.0	0.524048	2.40	*Safe*
18F	10.0	0.745402	2.40	*Safe*	18F	10.0	0.520679	2.40	*Safe*
19F	10.0	0.738606	2.40	*Safe*	19F	10.0	0.516326	2.40	*Safe*
20F	10.0	0.729476	2.40	*Safe*	20F	10.0	0.510691	2.40	*Safe*
21F	10.0	0.723049	2.40	*Safe*	21F	10.0	0.508896	2.40	*Safe*
22F	10.0	0.708136	2.40	*Safe*	22F	10.0	0.49942	2.40	*Safe*
23F	10.0	0.693441	2.40	*Safe*	23F	10.0	0.490613	2.40	*Safe*
24F	10.0	0.676819	2.40	*Safe*	24F	10.0	0.480743	2.40	*Safe*
25F	10.0	0.658691	2.40	*Safe*	25F	10.0	0.469907	2.40	*Safe*
26F	10.0	0.639179	2.40	*Safe*	26F	10.0	0.458146	2.40	*Safe*
27F	10.0	0.618455	2.40	*Safe*	27F	10.0	0.445541	2.40	*Safe*
28F	10.0	0.596683	2.40	*Safe*	28F	10.0	0.432184	2.40	*Safe*
29F	10.0	0.57421	2.40	*Safe*	29F	10.0	0.418282	2.40	*Safe*
30F	10.0	0.550074	2.40	*Safe*	30F	10.0	0.402851	2.40	*Safe*
31F	10.0	0.530583	2.40	*Safe*	31F	10.0	0.394283	2.40	*Safe*
32F	10.0	0.503025	2.40	*Safe*	32F	10.0	0.375417	2.40	*Safe*
33F	10.0	0.476825	2.40	*Safe*	33F	10.0	0.357763	2.40	*Safe*
34F	10.0	0.449802	2.40	*Safe*	34F	10.0	0.339001	2.40	*Safe*
35F	10.0	0.422841	2.40	*Safe*	35F	10.0	0.319705	2.40	*Safe*
36F	10.0	0.396424	2.40	*Safe*	36F	10.0	0.300202	2.40	*Safe*
37F	10.0	0.371201	2.40	*Safe*	37F	10.0	0.280959	2.40	*Safe*
38F	10.0	0.347945	2.40	*Safe*	38F	10.0	0.26257	2.40	*Safe*
39F	10.0	0.327494	2.40	*Safe*	39F	10.0	0.245746	2.40	*Safe*
40F	10.0	0.311448	2.40	*Safe*	40F	10.0	0.231829	2.40	*Safe*
Roof	10.0	0.297778	2.40	*Safe*	Roof	10.0	0.219111	2.40	*Safe*