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A Study of Vertical Discontinuity on RCC Frame Structures

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بسم الله الرحمن الرحيم

Declaration

This is to certify that the work presented in this thesis is an outcome of the experiment and research carried out by the authors under the supervision of Dr. Md. Jahidul Islam.

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Abstract

Irregularity is an inevitable occurrence in RCC frame structures now a days. The effect of this irregularity can be devastating. It can cause failure of structure in case of earthquake and heavy wind load. Sudden changes in stiffness and strength or mass in either vertical or horizontal planes of a building can result in distributions of lateral loads and deformations different from those that are anticipated for uniform structures. Large overturning forces that developed in the structure had to be carried by the columns that contributed to severe damage in the column. This work presents an investigation on vertical irregularity of RCC frame structures. The software based analysis that is called ETABS 9.7.0 is done for obtaining the behavior of structures in such irregular cases. Then some analytical analysis is done for obtaining the variation of stiffness and overturning.

A total number of 5 different cases have been analyzed, where all the properties of each case is compared to other cases. After comparing all the results, a possible solution and further recommendation is proposed.

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Chapter – 1

Introduction

1.1. General

Now a day, the population of the world is increasing at a very high rate. Due to this increase in population, the fundamental demands of people is increasing and developing day by day. One of the key fundamental demands is Accommodation. With the advancement of science and technology, the people all around the world have started building different kind of structures for various purposes. To build these structures, civil engineers play a very vital role. In one hand, the civil engineers have to design the structures economically; on the other hand, they also have to ensure the safety of the structure and comfort of the people.

To design any kind of structures, the civil engineers have to abide by some rules. They have to use different types of codes and design methods, use proper materials, ensure the safety of the structure and the economic feasibility of the structure.

In the recent practice of structural engineering, we can see that different kinds of structures are being built around the world; such as: Soft Story Structures, Irregular Structures, and Setback Structures etc. In our topic, we want to focus on structures where the space is narrow and want to provide vertical discontinuity to increase the parking facilities and resist the lateral loads.If we see all around us, we can find different kinds of buildings. In the recent times, the demand of apartments, office spaces and other facilities are increasing at an alarming rate, although the available land is not increasing in that ratio. Now a day, soft story buildings are much more preferred than irregular buildings. But if the space of the building is not sufficient, it is very difficult to design it as a soft story structure. For this reason, a vertical discontinuity in the plan occurs to provide sufficient parking facilities. But the design of this kind of irregular structure needs lots of precautions like resisting the lateral loads, making the structure stiff, reducing the undesirable torsional effects. An essential characteristic of any lateral load resisting system is that it must provide a continuous load path to the foundation. Inertial loads that develop due to acceleration of individual elements must be transferred from the individual reactive elements to floor diaphragms to vertical elements in lateral load system. So, we want to focus on these things in our work.

1.2. Purpose

Many multistory buildings around the world have open first story as an unavoidable feature. This is primarily being adopted to accommodate parking or reception lobbies in the first story. Whereas the total seismic base shear as experienced by a building during an earthquake is dependent on its natural period, the seismic force distribution is dependent on the distribution of stiffness and mass along the height.

The behavior of a building during earthquakes depends critically on its overall shape, size and geometry, in addition to how the earthquake forces are carried to the ground. The earthquake forces developed at different floor levels in a building need to be brought down along the height to the ground by the shortest path; any deviation or discontinuity in this load transfer path results in poor performance of the building. Buildings with vertical setbacks (like the hotel buildings with a few stories wider than the rest) cause a sudden jump in earthquake forces at the level of discontinuity. Buildings that have fewer columns or walls in a particular story or with unusually tall story tend to damage or collapse which is initiated in that story. Many buildings with an open ground story intended for parking collapsed or were severely damaged in Gujarat during the 2001 Bhuj earthquake. Buildings with columns that hang or float on beams at an intermediate story and do not go all the way to the foundation have discontinuities in the load transfer path.

1.3. Literature Review

The seismic response of vertically irregular building frames, which has been the subject of numerous research studies, started getting attention in the late 1970s. A large number of studies have focused on plan irregularity resulting in torsion in structural systems. Vertical irregularities are characterized by vertical discontinuities in the distribution of mass, stiffness and strength. Very few research studies have been carried out to evaluate the effects of discontinuities in each one of these quantities independently, and majority of the studies have focused on the elastic response. There have also been detailed studies on real irregular buildings that failed during earthquakes

Moehle and Alarcon (1986) carried out an experimental response study on two small scale models of reinforced concrete frame-wall structures subjected to strong base motions by using shake table. One of the test structures, designated as FFW, had two nine-story, three-bay frames and a nine-story, prismatic wall. The other structure, designated as FSW, was identical to FFW except that the wall extended only to the first floor level. Thus the test structures FFW and FSW represent the buildings having “regular” and “irregular” distributions of stiffness and strength in vertical plane respectively. They compared the measured response with that computed by the inelastic dynamic response time-history analysis, inelastic static analysis, elastic modal spectral analysis, and elastic static analysis. Several inelastic response time history analyses were conducted for each test structure. For each analysis, different modeling assumptions were tried in an effort to establish a “best-fit” model. They compared maximum top-floor displacements obtained by the experiments and by different inelastic dynamic and elastic analysis methods. They concluded that the main advantage of dynamic methods is that those are capable of estimating the maximum displacement response, whereas the static methods cannot be used for this purpose. Further, they inferred that the inelastic static and dynamic methods are superior to the elastic methods in interpreting the structural discontinuities.

Ruiz and Diederich (1989) studied the influence of the lateral strength discontinuity on ductility demand at the first story under the action of the acceleration record with largest peak ground acceleration, as obtained on soft soil in Mexico City during the Mexico earthquake of September 19, 1985. A parametric study was carried out for 5- and 12-story buildings with weak first story, and with brittle infill wall in upper stories in some cases and ductile in others. The fundamental periods of these buildings were 0.67 and 1.4 s respectively. They noted that the behavior of weak first story buildings greatly depends on the ratio of the dominant periods of excitation and response, the resistances of upper and first stories, and on the seismic coefficient used for design. The ratio of dominant periods of response and excitation was found to be closely related to the formation of plastic hinges, yielding or failure of infill walls, and to the times of their occurrences.

Esteva (1992) studied the nonlinear seismic response of soft-first-story buildings subjected to narrowband accelerograms. The variables covered were: number of stories, fundamental period, form of the variation of story stiffness along height, ratio of post-yield to initial stiffness, in addition to the variable of primary interest, i.e., factor *r* expressing the ratio of the average value of the safety factor for lateral shear at the upper stories to that at the bottom story. He used shear-beam systems representative of buildings characterized by different number of stories and natural periods. The study included cases of stories with hysteretic bilinear behavior, both including and neglecting P-delta effects. The excitation was in some cases an accelerogram recorded on soft soil in Mexico City during the Mexico earthquake of September 19, 1985, and in some cases an ensemble of artificial accelerograms with similar statistical characteristics. He observed that the nature and magnitude of the influence of the ratio *r* on the maximum ductility demands at the first story depend on the low-strain fundamental period of the system.

For very short periods those ductility demands may be reduced by about 30% when *r* grows from 1.0 to 3.0. For intermediate periods, ductility demands are little sensitive to ***r*,** but for longer periods those may reach the increments of 50 to 100% while *r* varies within the mentioned interval. He also observed that the influence of *r* on the response of the first story is strongly enhanced if P-delta effects are taken into Valmudsson and Nau (1997) focused on evaluating building code requirements for vertically irregular frames. The earthquake response of 5-, 10-, and 20-story framed structures with uniform mass, stiffness, and strength distributions was evaluated. The structures were modeled as two-dimensional shear buildings. The response calculated from the time-history analysis was compared with that predicted by the ELF procedure as embodied in UBC (1994). Based on this comparison, they evaluated the requirements under which a structure can be considered regular and the ELF provisions are applicable. They concluded that when the mass of one floor increases by 50%, the increase in ductility demand is not greater than 20%. Reducing the stiffness of the first story by 30%, while keeping the strength constant, increases the first story drift by 20-40%, depending on the design ductility (μ). Reducing the strength of the first story by 20% increases the ductility demand by 100-200%, depending on design ductility. Reducing the first story strength and stiffness proportionally by 30% increases the ductility demand by 80-200%, depending on the design ductility as shown in. Thus strength criterion results in large increases in response quantities and is not consistent with the mass and stiffness requirements.

Al-Ali and Krawinkler (1998) carried out evaluation of the effects of vertical irregularities by considering height-wise variations of seismic demands. They used a 10-story building model designed according to the strong-beam-weak-column (column hinge model) philosophy and an ensemble of 15 strong ground motions, recorded on rock or firm soil during Western U.S. earthquakes after 1983, for the parametric study. The effects of vertical irregularities in the distributions of mass, stiffness and strength were considered separately and in combinations, and the seismic response of irregular structures was assessed by means of the elastic and inelastic dynamic analyses. They found that:

(a) The effect of mass irregularity is the smallest,

(b) The effect of strength irregularity is larger than the effect of stiffness irregularity

(c) The effect of combined-stiffness-and-strength irregularity is the largest.

Das and Nau (2003) investigated the definition of irregular structure for different vertical irregularities: stiffness, strength, mass, and that due to the presence of non-structural masonry infill as prescribed in building codes. Linear and nonlinear dynamic time-history (TH) analyses were performed on an ensemble of 78 buildings of 5, 10, and 20 stories and with different story stiffness, strength, and mass ratios. All buildings had three bays in the direction of the ground motion. The lateral force-resisting systems considered were special moment resisting frames (SMRF) designed based on the forces obtained from the ELF(Equivalent Lateral Force) procedure according to the strong-column-weak-beam (SCWB) criteria of ACI 318-99 (ACI, 1999) and UBC (1997). They observed that most structures considered in their study performed well when subjected to the design earthquake ground motion. Hence they concluded that the restrictions on the applicability of the ELF procedure given in building codes are unnecessarily conservative for certain types of vertical irregularities considered. Response of a regular structure is shown by the continuous line. Letter “A” in the legend refers to SMRF with a taller (softer and weaker) first story. The numbers following this letter represent the ‘number of stories’ in the structure, the ‘height of the first story’ (in feet), and the ‘bay size’ (in feet). A201525 thus represents a 20-story SMRF with a 15-feet-tall first story and 25 feet of bay size. Further, letters “t”, “m”, and “b” denote the location of the heavier mass: “t” for top, “m” for mid-height, and “b” for bottom. The numbers before these letters denote the ‘number of stories’ in the structure and the numbers following denote the ‘mass ratios’. For example, 20t5 refers to a 20-story structure with a mass ratio of 5.0 on the top floor, whereas 20t25 refers to the same location of the heavier mass but with a mass ratio of 2.5 (a mass ratio of 2.5 is denoted by “25”). ELFR refers to the equivalent lateral force procedure that considers the ‘actual’ first mode shape, ‘actual’ fundamental period, and the corresponding ‘effective mass’. The response of an irregular structure designed by the ELF procedure is close to that of the regular structure. The presence of irregularity alters the inelastic response of the building, and there are marked increases in the inelastic story drift in the vicinity of the irregularity. However, in no case did the drift exceed the code-specified limit of 2%. The structure damage indices (a measure of the overall structural damage suffered by the building subjected to scaled ground motion) for all buildings were found to be less than 0.40, i.e., the threshold of repairable damage. The damage indices are insensitive to both the mass ratios and the location of the heavier mass. For all categories of the buildings studied, despite large increases on curvature ductility demands in the plastic regions in the vicinity of the irregularities, the demands did not exceed the computed curvature ductility capacities for which the members were designed. In general, it may be seen that the presence of irregularities has relatively little influence on the responses computed via ELF.

Chintanapakdee and Chopra (2004) studied the effects of stiffness and strength irregularities on story drift demand and floor displacement responses. They considered 48 frames, all 12-stories high and designed according to the strong-column-weak-beam (beam hinge model) philosophy. Three types of irregularities in the height-wise distributions of frame properties were considered: stiffness irregularity (KM), strength irregularity (SM), and combined-stiffness-and-strength irregularity (KS). They studied the influence of vertical irregularities in the stiffness and strength distributions, separately and in combination, on the seismic demands of strong-column-weak-beam frames.

During the January 26, 2001 earthquake, numerous mid- to high-rise residential buildings collapsed in the city of Ahmedabad leading to several hundred causalities and significant financial loss. The city of Ahmedabad lies about 300 km (400 km by road) east of the epicenter of the January 26 event and falls in the seismic Zone III (IS: 1893-1976) of India. The lateral design forces for this region are about 4 to 6% of total weight of the building, depending on the foundation type and soil conditions. Given that the horizontal accelerations recorded in Ahmedabad during the earthquake event are about 10% of gravity, the buildings may be expected to deform slightly into the inelastic range. However, the extent of damage observed was significantly more than expected in such a moderate seismic region. Following is a brief summary of the reasons that contributed to this unexpected damage in residential construction.

Figure 1.1: Collapse of structure due to earthquake

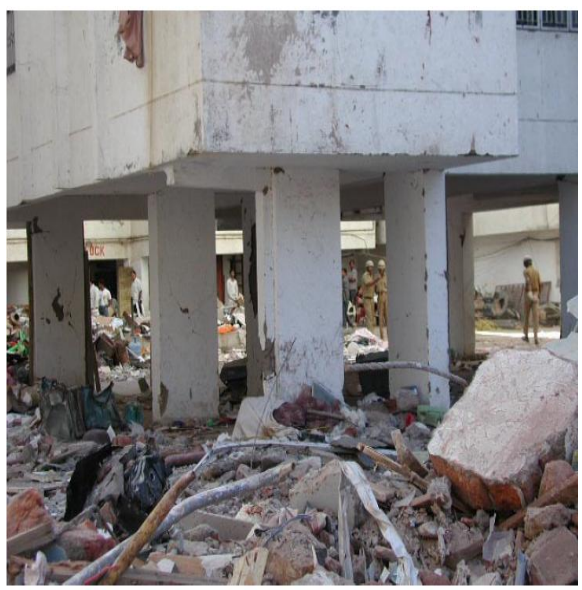


Figure1.2: Collapse of structure at Ahemdabad due to earthquake

The typical residential construction in Ahmedabad consists of reinforced concrete moment resisting frame system. The frame at the ground floor is open while frames at the upper floors are filled with un-reinforced brick panels. This type of lateral load resisting system leads to what is commonly known as a “soft-story” system. Most buildings also have overhanging covered balconies at higher floors; the overhangs were observed to be about 5 feet. The columns at the ground floor may not align with the columns at the upper floors giving rise to vertical discontinuities in the lateral load resisting system. The above-described lateral load resisting system occurs because of two factors. First, the open ground floor is needed to provide car parking; the buildings are usually built on very small land lots with little room for open parking. Second, the Floor Surface Index (FSI) used by the local municipal corporation for residential construction permits the land developers to cover more area at upper floor than the ground floor. The FSI only counts the area of within the column footprints at the ground floor. Therefore the developers are tempted to design the lateral load resisting system with only two to three columns in a frame on the ground floor with a beam overhangs on both sides. The upper floors may or may not continue these columns. But at least two floating columns are added, one on each end of the cantilever beam, starting from the first floor and running the entire height of the building. The most residential buildings appear to be designed primarily for gravity load; there are some indications that the lateral loads may not have been properly considered in design of these buildings. There is insufficient confining steel to provide required ductility in the lateral load resisting system, and column reinforcement is spliced just above the beam level, with often insufficient development length.

1.4. Various Definitions

1.4.1. Vertical Discontinuity in building structures

Aesthetic and architectural considerations often call for irregular structures with discontinuity in mass, stiffness, strength, geometry or structural form. Past earthquakes have shown that buildings with irregular configuration or asymmetrical distribution of structural properties trigger an increase in seismic demand, causing greater damage. Therefore, seismic codes provide elaborate empirical rules for the classification of buildings into regular, and various irregular categories as a function of asymmetries, to evaluate seismic demand. The codes have become increasingly cumbersome, with a plethora of experiential rules to account for irregularities from a multitude of structural asymmetries observed in the real world. There is a need to define and measure structural irregularity in a rational manner to assess its relative significance in different structures, and to develop seismic codes on a sound theoretical foundation. The major issue is the identification of a measurement scale for irregularity levels produced by asymmetric structural properties. This scale can then be used to specify quantifiable limits that delineate regular and different irregular building categories.

1.4.2. Soft story

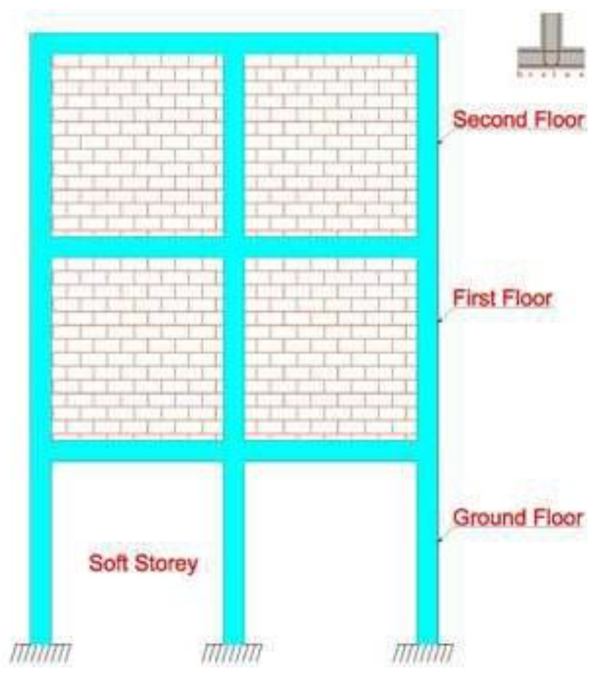


Figure 1.3:Soft story structure

The lowest story in a building which qualifies as a story as defined herein, except that a floor level in a building having only one floor level shall be classified as a soft story, provided such floor level is not more than 1.25m below grade, as defined herein, for more than 50 percent of the total perimeter, nor more than 2.5m below grade at any point.

1.4.3. Irregular structure

The structures having significant physical discontinuity in configuration or in their lateral force resisting systems may be defined as “Irregular Structures”.

An irregular structure may have:

- Vertical Irregularity
- Plan Irregularity
- Both Vertical and Plan Irregularity

1.4.4. Bending moment and shear force

A bending moment is a measure of the average internal stress induced in a structural element when an external force or moment is applied to the element causing the element to bend.

The internal stresses in a cross-section of a structural element can be resolved into a resultant force and a resultant couple. For equilibrium, the moment created by external forces (and external moments) must be balanced by the couple induced by the internal stresses. The resultant internal couple is called the bending moment while the resultant internal force is called the shear force (if it is transverse to the plane of element) or the normal force (if it is along the plane of the element).

1.4.5. Stiffness

Stiffness is the rigidity of an object — the extent to which it resists deformation in response to an applied force. The complementary concept is flexibility or pliability: the more flexible an object is the less stiff it is.

1.5. Objectives and Scope of Study

The research study deals with the analysis of changing lateral displacement and drift ratio in RC frame building under different conditions.The main purpose of this study is to develop software based structural models and analysis system for such kind of structures.

- To analyze the effects of the displacement of the columns at the ground floor of the buildings.
- To find a solution to provide maximum parking facilities at the ground floor.
- To study the irregularity that forms due to the displacement of the columns at the ground floor.
- To make structure safe against overturning moments.

During literature review we have found that some buildings in BHUJ collapsed due to earthquake and the reasons behind that failure was the displacement of columns at the ground floor without proper analysis. We have performed our study as follows:

- We considered five different cases by placing columns at the ground floor providing different spacing.
- We performed different numerical and analytical analysis.
- Then evaluated the comparisons of the results and suggested the best solution by analyzing various girder designs, stiffness calculations and factor of safety against overturning.

1.6. Organization of Thesis

Our study has been organized as follows:

- Chapter 2 presents the five cases that we are going to study. It also includes some theories and formulations used for developing both numerical and analytical analysis.

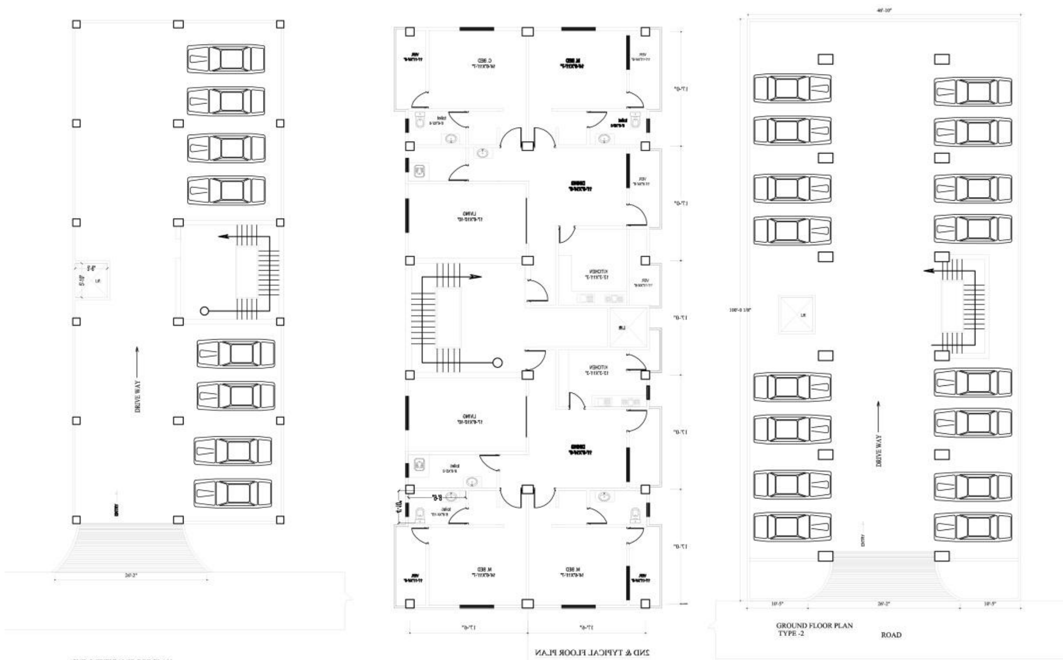
- Chapter 3 presents the deflected shape and displacements due to wind load and earthquake load. It also includes girder design, stiffness calculation and factor of safety against overturning.
- Chapter 4 presents the evaluation of the results and discussions on the best possible case.
- Chapter 5 concludes the present work and account of possible scope of extension to the present study has been appended to the concluding remarks.
- Some important publications and books have been listed in the reference which was used during our investigations.

Chapter – 2

Methodology

2.1. General

We have analyzed a case where the columns in the ground floor are to be displaced in order to increase parking facility. Here is the typical floor plan and the ground floor plan for our case of study. This is a plan for a nine story building with ground floor reserved for parking.



Regular Structure   Typical floor plan   Irregular Structure

Figure 2.1: Typical floor plan and ground floor plan of case

If we maintain the columns of typical floor plan at the ground floor and keep the structure regular, then there should be parking facility for only 8 cars. But if we displace the columns at the ground floor and make the structure irregular, then we can accommodate 16 cars in the parking.

To do so, we have used 5 alternatives and analyzed all of them by both analytical analysis and numerical analysis. For numerical analysis, we have used a software named “ETABS”, version 9.7.0, where the effects of earthquake load and wind load are given automatically by the structural software ETABS 9.7.0 according to the UBC-94 which is similar to BNBC code. In our analytical analysis, we have included girder design, stiffness calculation and factor of safety against overturning.

2.2. Case Study

We have analyzed five different alternatives, where we have used different spacing between the two columns in the ground floor. We have also tried by extending the corner columns to the ground floor. The five different cases are demonstrated below.

2.2.1. Case 1

In the first case, we have used girder at ground floor which separates the upper floor from contact with the ground. Here, the spacing between two ground floor columns is 17.5’.

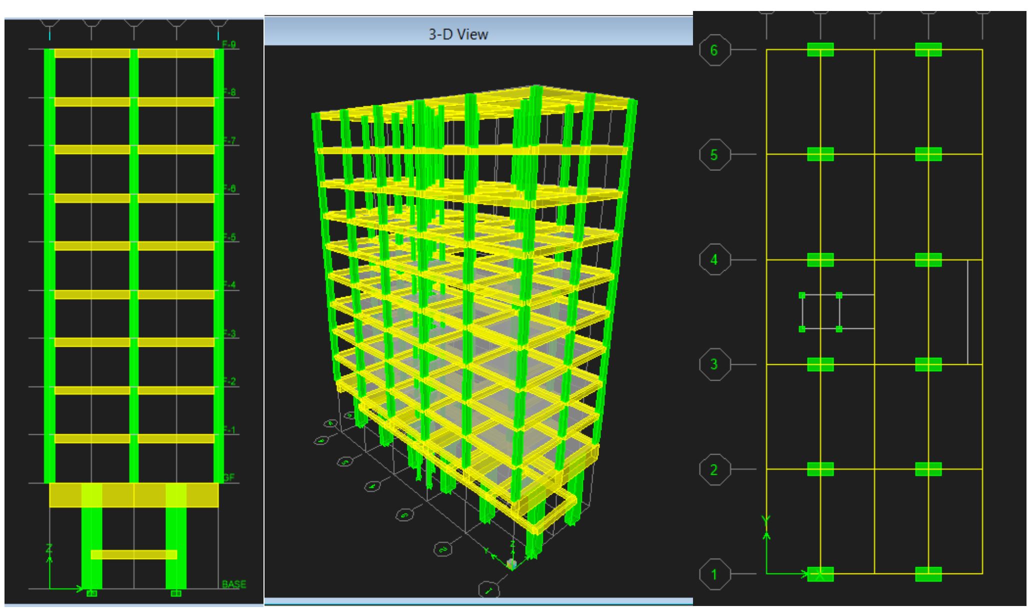


Figure 2.2: Elevation view, 3D view and ground floor plan view of case 1

2.2.2. Case 2

In the second case, we have used girder at ground floor which separates the upper floor from contact with the ground. Here, the spacing between two ground floor columns is 26’.

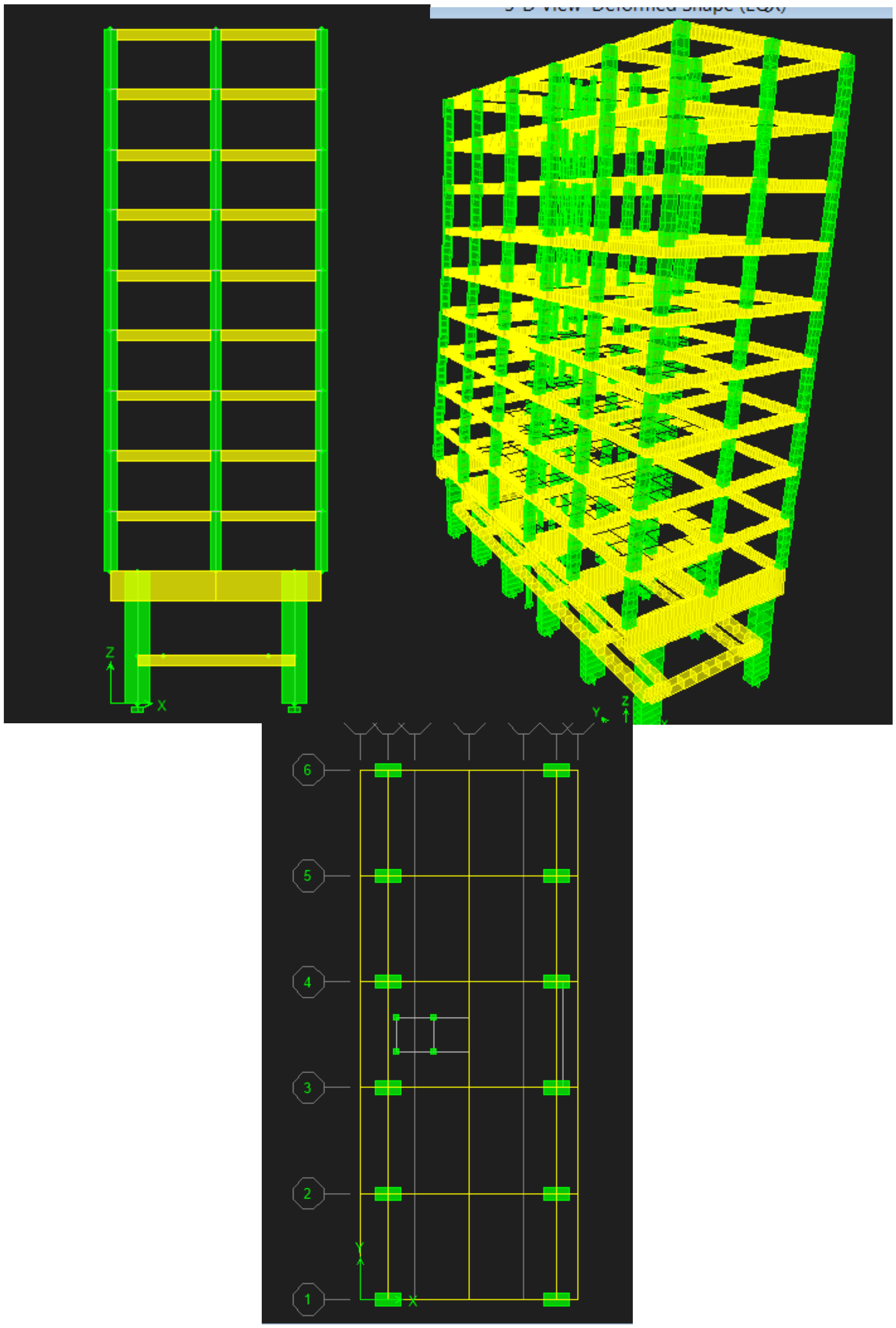


Figure 2.3: Elevation view, 3D view and ground floor plan viewof case 2

2.2.3. Case 3

In the third case, we have used girder at ground floor which separates the upper floor from contact with the ground. Here, the spacing between two ground floor columns is 35’.

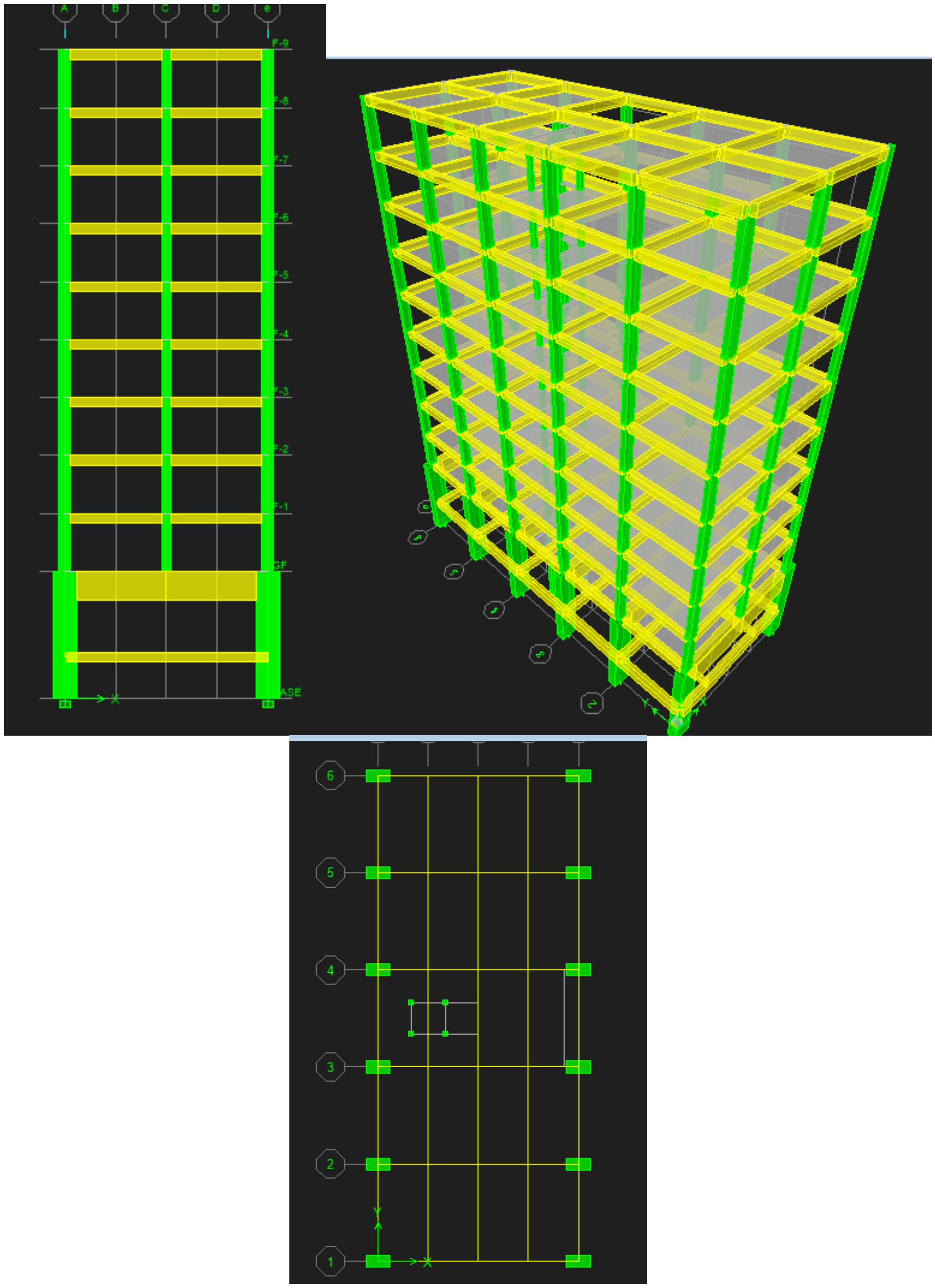


Figure 2.4: Elevation view, 3D viewand ground floor plan view of case 3

2.2.4. Case 4

In the fourth case, we have used girder in the ground floor and extending the corner column which creates direct contact in the ground. Here, spacing between two displaced columns is 17.5’.

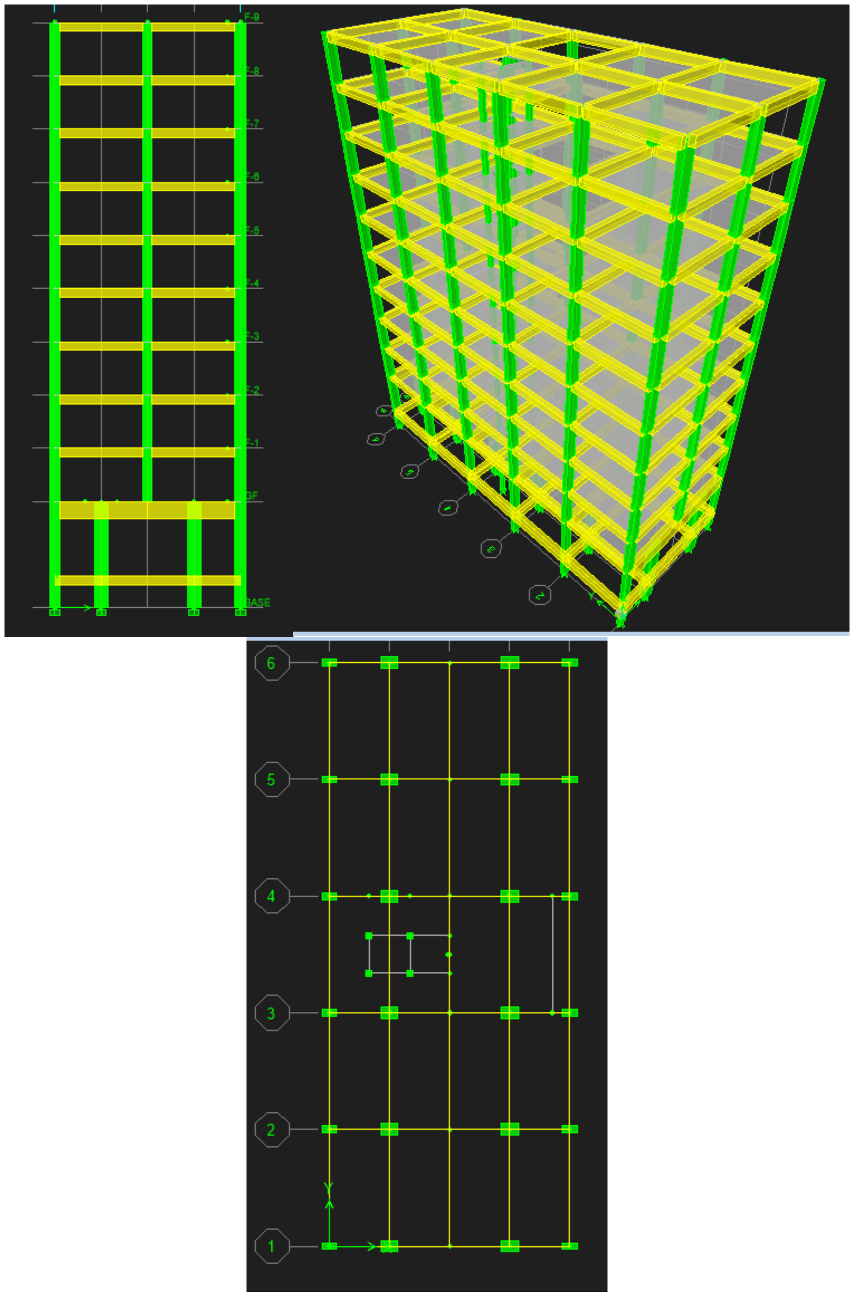


Figure 2.5: Elevation view, 3D viewand ground floor plan viewof case 4

2.2.5. Case 5

In the fourth case, we have used girder in the ground floor and extending the corner column which creates direct contact in the ground. Here, spacing between two displaced columns is 26’.

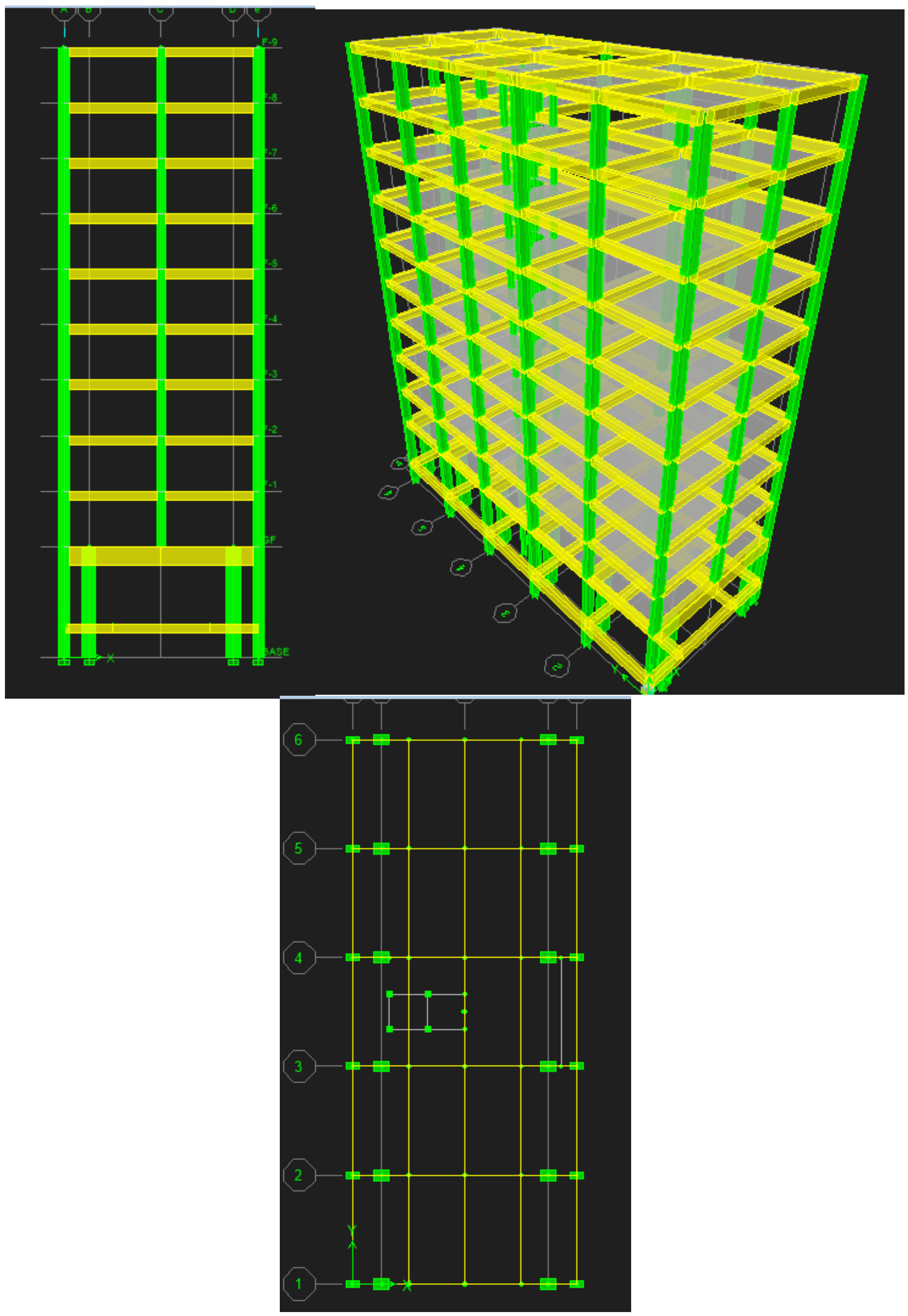


Figure 2.6: Elevation view, 3D viewand ground floor plan view of case 5

2.3. Numerical Analysis

We have performed the numerical analysis by using the software “ETABS” version 9.7.0.

2.3.1. Introduction

ETABS means extended 3D (Three-Dimensional) Analysis of Building Systems. It is used for special purpose analysis and design program developed specially for building system. ETABS version 9.7 features an intuitive and powerful graphical interfaced coupled with unmatched modeling , analysis and design procedure, all integrated data using a common database. ETABS can also give the solution of largest and most complex building. It also include a wide range of linear with nonlinear modeling.

Building construction is a very special class of structure. ETABS provide input, output and numerical solution of building type structure offering a significant saving of time and increase accuracy of the solution.

At present ETABSS added computationally complex analysis options such as dynamic nonlinear behavior and powerful CAD-like drawing tools. Its mission is to provide the performance with the most efficient and comprehensive software for the analysis and design of building.

- Most buildings are of straightforward geometry with horizontal beams and vertical columns. Although any building configuration is possible with ETABS, in most cases, a simple grid system defined by horizontal floors and vertical column lines can establish building geometry with minimal effort.
- Many of the floor levels in buildings are similar. This commonality can be used numerically to reduce computational effort.
- The input and output conventions used correspond to common building terminology. With ETABS, the models are defined logically floor-by-floor, column-by-column, bay-by-bay and wall-by-wall and not as a stream of non-descript

nodes and elements as in general purpose programs. Thus the structural definition is simple, concise and meaningful.

- In most buildings, the dimensions of the members are large in relation to the bay widths and story heights. Those dimensions have a significant effect on the stiffness of the frame. ETABS corrects for such effects in the formulation of the member stiffness, unlike most general-purpose programs that work on center-line-to-centerline dimensions.
- The results produced by the programs should be in a form directly usable by the engineer. General-purpose computer programs produce results in a general form that may need additional processing before they are usable in structural design.

2.3.2. Functions of ETABS

A widest amount of analysis and design facility with specific design tools are available for the structural work on building structure. Here some important types of system and analysis that can be done easily by ETABS are given

• Multi-story commercial, government and health care facilities

• Parking garages with circular and linear ramps

• Staggered truss buildings

• Buildings with steel, concrete, composite or joist floor framing

• Buildings based on multiple rectangular and/or cylindrical grid systems

• Flat and waffle slab concrete buildings

•Buildings subjected to any number of vertical and lateral load cases and combinations, include automated wind and seismic loads

• Multiple response spectrum load cases, with built-in input curves

• Automated transfer of vertical loads on floors to beams and walls

•P-Delta analysis with static or dynamic analysis

• Explicit panel-zone deformations

• Construction sequence loading analysis

•Multiple linear and nonlinear time history load cases in any direction

•Foundation/support settlement

• Large displacement analyses

•Nonlinear static pushover

• Buildings with base isolators and dampers

• Floor modeling with rigid or semi-rigid diaphragms

•Automated vertical live load reductions

2.3.3.The ETABS System

ETABS analyzes and designs your building structure using a model that you create using the graphical user interface. The key to successfully implementing ETABS is to understand the unique and powerful approach the program takes in modeling building systems. This chapter will provide an overview of some of the key components and their associated terminology.

2.3.4.Story Definition

The first and the most powerful features that ETABS offers is to assemble of story levels, allowing for the input of building data in a logical and convenient manner.

Users may define their models on a floor-by-floor, height-by-height basis, analogous to the way a designer works when laying out building drawings. Story levels help identify, locate and view specific areas and objects of your model; column and beam objects are easily located using their plan location and story level labels.

In ETABS terminology, a story level represents a horizontal plane cut through a building at a specified elevation, and all of the objects below this plane down to the next story level. Because ETABS inherently understands the geometry of building systems, a user can specify that an object being drawn in plan be replicated at all stories, or at all similar stories as identified by the user. This option works not only for repetitive floor framing, but also for columns and walls. Story labeling, the height of each story level, as well as the ability to mark a story as similar, are all under the control of the user.

2.3.5. Units

ETABS functionswith four basic units: force, length, temperature and time. Many different compatible sets of force, length and temperature units to choose from, such as “Kip, in, F” or “N, mm, C.” Time is always measured in seconds.

An important distinction is made between mass and weight. Mass is used for calculating dynamic inertia and for loads caused by ground acceleration only. Weight is a force that can be applied like any other force load. Be sure to use force units when specifying weight values, and mass units (force-sec^2 /length) when specifying mass values.

When you start a new model, you will be asked to specify a set of units. These become the “base units” for the model. Although you may provide input data and view output results in any set of units, those values are always converted to and from the base units of the model.

Angular measure always uses the following units:

• Geometry, such as axis orientation, is always measured in degrees.

• Rotational displacements are always measured in radians.

• Frequency is always measured in cycles/second (Hz).

2.3.6.Structural Objects

As stated previously, ETABS uses objects to represent physical structural members. When creating a model, the user starts by drawing the geometry of the object, and then assigning properties and loads to completely define the building structure.

The following object types are available, listed in order of geometrical dimension:

• **Point objects** of two types:

• **Joint objects** are automatically created at the corners or ends of all other types of objects, and they can be explicitly added anywhere in the model.

• **Grounded (one joint) link objects** are used to model special support behavior, such as isolators, dampers, gaps, multi-linear springs and more.

• **Line objects** of two types:

• **Frame objects** are used to model beams, columns, braces and trusses.

• **Connecting (two-joint) link objects** are used to model special member behavior, such as isolators, dampers, gaps, multi-linear springs, and more. Unlike frame objects, connecting link objects can have zero length.

• Area objects are used to model walls, slabs, decks, planks, and other thin-walled members. Area objects will be meshed automatically or manually into the elements needed for analysis if horizontal objects with the membrane definition are included in the model; otherwise, the user should specify the meshing option to be used.

As a general rule, the geometry of the object should correspond to that of the physical member. This simplifies the visualization of the model and helps with the design process. When you run an analysis, ETABS automatically converts your object-based model (except for certain Area objects; see previous bullet item) into an element-based model that is used for analysis. This element-based model is called the analysis model, and it consists of traditional finite elements and joints . After running the analysis, your object-based model still has the same number of objects in it as it did before the analysis was run.

Although the majority of the object meshing is performed automatically, you do have control over how the meshing is completed, such as the degree of refinement and how to handle the connections at intersecting objects. An option is also available to manually subdivide the model, which divides an object based on a physical member into multiple objects that correspond in size and number to the analysis elements.

2.3.7. Groups

A group is a named collection of objects. It may contain any number of objects of any number of types. Groups have many uses, including:

- Quick selection of objects for editing and assigning.
- Defining section cuts across the model.
- Grouping objects that are to share the same design.
- Selective output.

Define as many groups as needed. Using groups is a powerful way to manage larger models.

2.3.8. Properties

Properties are "assigned" to each object to define the structural behavior of that object in the model. Some properties, such as materials and section properties, are named entities that must be specified before assigning them to objects. For example, a model may have:

• A material property called CONCRETE.

• A rectangular frame section property called RECTANGLE, and a circular frame section called CIRCULAR, both using material property CONCRETE.

• A wall/slab section property called SLAB that also uses material property CONCRETE.

If you assign frame section property RECTANGLE to a line object, any changes to the definition of section RECTANGLE or material CONCRETE will automatically apply to that object. A named property has no effect on the model unless it is assigned to an object.

Other properties, such as frame releases or joint restraints, are assigned directly to objects. These properties can only be changed by making another assignment of that same property to the object; they are not named entities and they do not exist independently of the objects.

Static loads represent actions upon the structure, such as force, pressure, support displacement, thermal effects, and others. A spatial distribution of loads upon the structure is called a load case.

Define as many named static load cases as needed. Typically, separate load case definitions would be used for dead load, live load, static earth quake load, wind load, snow load, thermal load, and so on. Loads that need to vary independently, for design purposes or because of how they are applied to the building, should be defined as separate load cases.

After defining a static load case name, you must assign specific load values to the objects as part of the load case, or define an automated lateral load if the case is for quake or wind. The load values you assign to an object specify the type of load (e.g., force, displacement, temperature), its magnitude, and direction (if applicable). Different loads can be assigned to different objects as part of a single load case, along with the automated lateral load, if so desired. Each object can be subjected to multiple load cases.

2.3.9. Vertical Loads

Vertical loads may be applied to point, line and area objects. Vertical loads are typically input in the gravity, or -Z direction. Point objects can accept concentrated forces or moments. Frame objects may have any number of point loads (forces or moments) or distributed loads (uniform or trapezoidal) applied. Uniform loads can be applied to Area objects. Vertical load cases may also include element self-weight.

Some typical vertical load cases used for building structures might include:

- Dead load
- Superimposed dead load
- Live load
- Reduced live load
- Snow load

If the vertical loads applied are assigned to a reducible live load case, ETABS provides you with an option to reduce the live loads used in the design phase. Many different types of code-dependent load reduction formulations are available.

2.3.10. Wind and Seismic Lateral Loads

The lateral loads can be in the form of wind or seismic loads. The loads are automatically calculated from the dimensions and properties of the structure based on built-in options for a wide variety of building codes.

For rigid diaphragm systems, the wind loads are applied at the geometric centers of each rigid floor diaphragm. For modeling multi-tower systems, more than one rigid floor diaphragm may be applied at any one story.

The seismic loads are calculated from the story mass distribution over the structure using code-dependent coefficients and fundamental periods of vibration. For semi-rigid floor systems where there are numerous mass points, ETABS has a special load dependent Ritz-vector algorithm for fast automatic calculation of the predominant time periods. The seismic loads are applied at the locations where the inertia forces are generated and do not have to be at story levels only. Additionally, for semi-rigid floor systems, the inertia loads are spatially distributed across the horizontal extent of the floor in proportion to the mass distribution, thereby accurately capturing the shear forces generated across the floor diaphragms.

ETABS also has a very wide variety of Dynamic Analysis options, varying from basic Response spectrum analysis to large deformation nonlinear time history analysis. Code-dependent response spectrum curves are built into the system, and transitioning to a dynamic analysis is usually trivial after the basic model has been created.ETABS allows for the named combination of any previously defined load case or load combination. When a load combination is defined, it applies to the results for every object in the model.

The four types of combinations are as follows:

• ADD (Additive): Results from the included load cases or combos are added.

• ENVE (Envelope): Results from the included load cases or combos are enveloped to find the maximum and minimum values.

• ABS (Absolute): The absolute values of the results from the included load cases or combos are added.

• SRSS: The square root of the sum of the squares of the results from the included load cases or combos is computed.

2.3.11. Design Settings

ETABS offers the following integrated design postprocessors:

• Steel Frame Design

• Concrete Frame Design

• Composite Beam Design

• Steel Joist Design

• Shear Wall Design

The first four design procedures are applicable to line objects, and the program determines the appropriate design procedure for a line object when the analysis is run. The design procedure selected is based on the line object's orientation, section property, material type and connectivity.

Shear wall design is available for objects that have previously been identified as piers or spandrels by the user, and both piers and spandrels may consist of both area and line objects.

For each of the design postprocessors, several settings can be adjusted to affect the design of the model:

• The specific design code to be used for each type of object, e.g., AISC-LRFD93 for steel frames, EUROCODE 2-1992 for concrete frames, and BS8110 97 for shear walls.

• Preference settings of how these codes should be applied to your model.

• Load combinations for which the design should be checked.

• Groups of objects that should share the same design.

• For each object, optional "overwrite" values that supercede the default coefficients and parameters used in the design code formulas selected by the program.

For steel frame, composite beam, and steel joist design, ETABS can automatically select an optimum section from a list you define. You can also manually change the section during the design process. As a result, each line object can have two different section properties associated with it:

• An "analysis section" used in the previous analysis

• A "design section" resulting from the current design

The design section becomes the analysis section for the next analysis and the iterative analysis and design cycle should be continued until the two sections become the same.

Design results for the design section, when available, as well as all of the settings described herein, can be considered to be part of the model.

2.3.12. Output and Display Options

The ETABS model and the results of the analysis and design can be viewed and saved in many different ways, including:

• Two- and three-dimensional views of the model

• Input/output data values in plain text, spreadsheet, or database format

• Function plots of analysis results

• Design reports

• Export to other drafting and design programs

You may save named definitions of display views, sets of tables, and function plots as part of your model. Combined with the use of groups, this can significantly speed up the process of getting results while you are developing your model.

2.3.13. ETABS Analysis Techniques

This chapter provides an overview of some of the analysis techniques available within ETABS. The types of analyses described are linear static analysis, modal analysis, response spectrum analysis, time history analysis, P-Delta analysis and nonlinear analysis.

In a given analysis run, you may request an initial P-Delta analysis, a modal analysis, and multiple cases of linear static, response spectrum, and time history analyses. Multiple nonlinear static analysis cases may also be defined; these are performed separately from the other analysis cases.

2.3.14 Linear Static Analysis

A linear static analysis is automatically performed for each static load case that is defined. The results of different static load cases can be combined with each other and with other linear analysis cases, such as response spectrum analyses.

Geometric and material nonlinearity are not considered in linear static analysis, except that the effect of the initial P-Delta analysis is included in every static load case. For example, if you define an initial P-Delta analysis for gravity load, deflections and moments will be increased for lateral static load cases. Linear static load cases can still be combined when an initial P-Delta analysis has been performed, because the initial P-Delta load is the same for all static load and response spectrum cases.

2.3.15. Modal Analysis

Modal analysis calculates vibration modes for the structure based on the stiffnesses of the elements and the masses present. Those modes can be used to investigate the behavior of a structure, and are required as a basis for subsequent response spectrum and time history analyses.

Two types of modal analysis are available: eigenvector analysis and Ritz-vector analysis. Only one type can be used in a single analysis run.

2.3.16. Mass Source

To calculate modes of vibration, a model must contain mass. Mass may be determined and assigned in ETABS using any of the following approaches:

• ETABS determines the building mass on the basis of object self masses (defined in the properties assignment) and any additional masses that you specify. This is the default approach.

• ETABS determines the mass from a load combination that you specify.

• ETABS determines the mass on the basis of self masses, any additional masses you assign, and any load combination that you specify, which is a combination of the first two approaches.

Typically, masses are defined in all six degrees of freedom. However, ETABS has an option that allows only assigned translational mass in the global X and Y axes directions and assigned rotational mass moments of inertia about the global Z axis to be considered in the analysis. This option is useful when vertical dynamics are not to be considered in a model. In addition, an option exists for all lateral masses that do not occur at a story level to be lumped together at the story level above and the story level below the mass location. That approach is used primarily to eliminate the unintended dynamic out-of-plane behavior of walls spanning between story levels.

2.4. Analytical Analysis

In the analytical analysis, we have performed some calculations manually to check whether all the different cases satisfy our purpose.

2.4.1. Introduction

In our study we have used ETABS software to analyze the cases. As ETABS always does over design so we did analytical analysis to make the structures economical. In our analysis we have included

- Girder Design
- Stiffness Calculation
- Factor of Safety against Over Turning

2.4.2. Girder Design

To design the girder, we have used doubly reinforced beam design. The formula (Eqn 2.3.2.1) used for calculating the area of steel is:

$A_s = M_u / \phi f_y$ (d - –)………………………………….. (Eqn 2.1)

Here,

A_s = Area of steel

M_u = Ultimate moment of girder

f_y = Yield strength of steel

d = critical depth of girder

ϕ = Strength reduction factor

$a = A_s f_y / 0.85 f'_c b$………………….. (Eqn 2.2)

where f'_c = Concrete strength and b = width of girder

2.4.3. Stiffness Calculation

Analysis of frame buildings subjected to lateral loads such as those generated by earthquake motion and high wind , requires knowledge of lateral stiffness for calculation of lateral displacements in static analysis. For calculation of story stiffness we have used MUTO'S expression.

For typical story:

Column stiffness: …………… (Eqn 2.3)

For first story:

Column stiffness: …. (Eqn 2.4)

For ground floor:

Column stiffness: ……………. (Eqn 2.5)

Here,

E_c = Elasticity of concrete

I_c = Moment of inertia of column

H = Height

K_c = Column Stiffness

K_g = Girder Stiffness

2.4.4. Factor of safety against over turning

Over turning moment: The moment that is created by an applied force which causes a structure to turn over is called overturning moment.

Resisting moment: A moment produced by internal tensile and compressive forces that balances the external bending moment on a beam.

According to BNBC 2006, the factor of safety of any structure against overturning must be greater than 3. To calculate factor of safety against overturning, we need

Wind velocity: Wind velocity varies depending on the geographic location of the area. In our study, we have used the wind velocity of Dhaka city which is 120mph.

Average pressure: .0030 x (wind velocity)^2

Surface area: It is the area of our plan

V_{wind}= average pressure x surface area

Moment for overturning = V_{wind} x (Height of the building/2)

Minimum weight of building, which is generally considered as 0.2 kip/sq.ft./floor

Resisting moment = Weight of building x center of gravity of building

Factor of safety against overturning = ————————

Chapter – 3

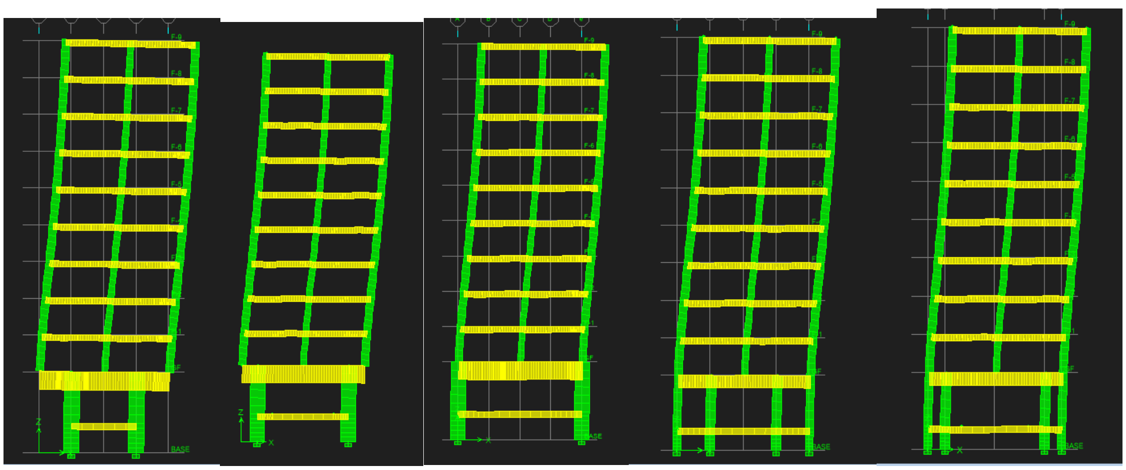
Analysis

3.1. Numerical Analysis

For numerical analysis, we have used "ETABS" version 9.7.0, to get the response of the alternatives in case of earthquake load and wind load.

As the structure is very stiff in Y-direction, it does not show any significant displacement along the Y-axis. The displacements due to earthquake load and wind load in each case are demonstrated below:

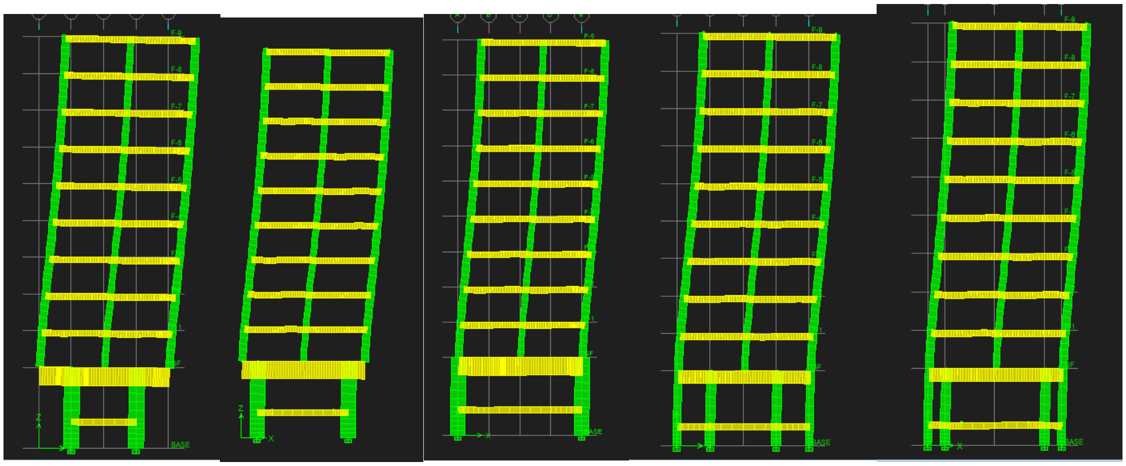
3.1.1. Displacement due to earthquake load in X-direction



Case 1 Case 2 Case 3 Case 4 Case 5

Figure 3.1: Displacement due to earthquake load in X-direction

3.1.2. Displacement due to wind load in X-direction



Case 1 Case 2 Case 3 Case 4 Case 5

Figure 3.2: Displacement due to wind load in X-direction

3.2. Analytical Analysis

3.2.1. Girder Design

In our study, from the numerical analysis, we have got the design moments of girder for all the five alternatives. They are stated below:

Table 3.1: Comparison of positive and negative moments of alternatives

	Case 1	Case 2	Case 3	Case 4	Case 5
Negative Moment (k-ft)	3948	2216	1836	1044	1685
Positive Moment (k-ft)	805	1314	2117	1216	1705

Using ETABS, we have found the following properties of each case.

Table 3.2: Design of girder

	Case 1	Case 2	Case 3	Case 4	Case 5
Concrete strength (f'_c)	3.5 ksi	3.5 ksi	3.5 ksi	3.5 ksi	3.5 ksi
Yield Strength (f_y)	60 ksi	60 ksi	60 ksi	60 ksi	60 ksi
Beam Depth (h)	45 in	36 in	34 in	30 in	34 in
M design (-ve)	3948 k-ft	2216 k-ft	1836 k-ft	1044 k-ft	1685 k-ft
Beam width (b)	25 in	25 in	25 in	20 in	20 in
Beam depth (d)	39 in	30 in	28 in	24 in	28 in
Pb	0.0249	0.0249	0.0249	0.0249	0.0249
ρ_{max}	0.0187	0.0187	0.0187	0.0187	0.0187
$A_s - A_s'$	18.24 in^2	14.03 in^2	13.09 in^2	8.98 in^2	10.48 in^2
A	14.715 in	11.31 in	10.56 in	9.06 in	10.56 in
M (max)	2597.2 k-ft < 3948 k-ft	1536.8 k-ft <2216 k-ft	1338.7 k-ft < 1836 k-ft	786.85 k-ft <1044 k-ft	1071 k-ft <1685 k-ft
M_1 = M design (-ve) – M (max)	1350.8 k-ft	679.18 k-ft	497.26 k-ft	257.15 k-ft	614.01 k-ft
	Case 1	Case 2	Case 3	Case 4	Case 5
A_s'	7.95 in^2	5.25 in^2	4.13 in^2	2.51 in^2	5.10 in^2
A_s	26.19 in^2	19.28 in^2	17.23 in^2	11.49 in^2	15.58 in^2
a (assumed)	28.7 in	19.7 in	16.9 in	13.5 in	22.5 in
A_s	35.59 in^2	24.44 in^2	20.87 in^2	13.45 in^2	22.36 in^2
A	28.71 in	19.72 in	16.84 in	13.56 in	22.54 in
M design (+ve)	805 k-ft	1314 k-ft	2117 k-ft	1216 k-ft	1705 k-ft
a (assumed)	3.5 in	9.5 in	12.3 in	12.3 in	12.3 in
A_s	4.80 in^2	11.56 in^2	21.53 in^2	15.14 in^2	17.34 in^2
A	3.87 in	9.33 in	17.37 in	15.27 in	17.49 in

After completing the design, we get the following girder sizes for each alternative:

Table 3.3: Girder size for each case

	Case 1	Case 2	Case 3	Case 4	Case 5
Girder Size (in x in)	25 x 45	25 x 36	25 x 34	20 x 30	20 x 34

3.2.2. Stiffness calculation

To calculate the story stiffness, we have used Muto's expressions (Eqn 2.3, 2.4, 2.5). The detailed calculated data are given in the following tables:

Table 3.4: Details of column

For column					
Case	Width (b in.)	Depth (h in.)	Length (L in.)	Moment of inertia (I_c in^4)	Individual column stiffness K_c
Case 1	12	25	99.96	15625	156.31
	15	20	99.96	10000	100.04
	20	40	135	106666.7	790.12
Case 2	12	25	99.96	15625	156.31
	15	20	99.96	10000	100.04
	20	40	135	106666.7	790.12
Case 3	12	25	99.96	15625	156.31
	15	20	99.96	10000	100.04
	20	40	135	106666.7	790.12
Case 4	12	25	99.96	15625	156.31
	15	20	99.96	10000	100.04
	20	25	135	26041.67	192.90
Case 5	12	25	99.96	15625	156.31
	15	20	99.96	10000	100.04
	20	25	135	26041.67	192.90

Table 3.5: Details of girder

For girder					
Case	Width (b in.)	Depth (h in.)	Length (L in.)	Moment of inertia (I_g in^4)	Individual girder stiffness K_g
Case 1	12	20	8000	192	41.67
	25	45	189843.8	180	1054.69
Case 2	12	20	8000	192	41.67
	25	36	97200	180	540
Case 3	12	20	8000	192	41.67
	25	34	81883.33	180	454.91
Case 4	12	20	8000	192	41.67
	20	30	45000	180	250
Case 5	12	20	8000	192	41.67
	20	34	65506.67	180	363.93

Table 3.6: Details of typical floor stiffness

Typical floor stiffness			
Case	K_c for corner column	K_c for middle column	Total story stiffness
Case 1	118.53	105.98	2058.19
Case 2	118.53	105.98	2058.19
Case 3	118.53	105.98	2058.19
Case 4	118.53	105.98	2058.19
Case 5	118.53	105.98	2058.19

Table 3.7: Details of first floor stiffness

First floor stiffness			
Case	K_c for corner column	K_c for middle column	Total story stiffness
Case 1	358.64	264.05	5888.03
Case 2	271.42	213.53	4538.22
Case 3	249.29	199.59	4189.02
Case 4	179.14	151.95	3061.45
Case 5	221.59	181.43	3747.61

Table 3.8: Details of ground floor stiffness

Ground floor stiffness			
Case	Total stiffness for displaced column	Total stiffness for extended column	Total story stiffness
Case 1	683.07	-	8196.85
Case 2	561	-	6731.98
Case 3	537.47	-	6449.63
Case 4	330.39	234.56	3389.7
Case 5	373.69	265.24	3833.65

3.2.3. Factor of safety against over turning

In our study, we have identified the factor of safety against over turning in each of the cases. The findings are provided below:

Table 3.9: Factor of safety calculation

	Case 1	Case 2	Case 3	Case 4	Case 5
Wind velocity (mph)	120	120	120	120	120
Average Pressure (psf)	43	43	43	43	43
Surface Area (ft^2)	8075	8075	8075	8075	8075
Wind Shear (kip)	347.23	347.23	347.23	347.23	347.23
Moment for overturning (k-ft)	16493	16493	16493	16493	16493
Minimum weight of building (kip/sq. ft./floor)	0.2	0.2	0.2	0.2	0.2
Area (ft^2)	2975	2975	2975	2975	2975
Total weight of building (kip)	5355	5355	5355	5355	5355
Resisting Moment (k-ft)	46856.3	69615	93712.5	93712.5	93712.5
Factor of safety	2.84	4.22	5.68	5.68	5.68

Chapter – 4

Result and Discussion

4.1. Introduction

After completing both the numerical analysis and analytical analysis, we have compared the results according to the following criteria:

- Variation of moments
- Variation of shear
- Variation of transfer girder section
- Variation of displacements
- Variation for story stiffness
- Variation for overturning

4.2. Variation of moments

After completing the numerical analysis, we get different positive and negative moments for each criterion. These variations of moments will help us to compare among the criterions.

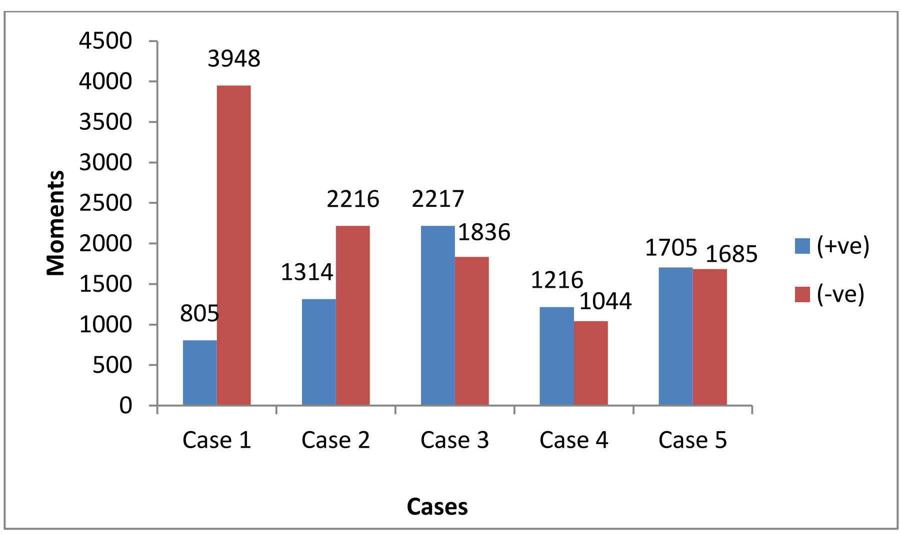


Figure 4.1: Variation of moments

After the comparison, we can see that the variation of positive moments and negative moments in case 1 is huge compared to others. That required huge girder size. On the other hand, in case 4, the value of moments and variation of moments is relatively low compared to others.in case 2 variation is less than case 1. In case 3 variation less than case 2.in case 5 variation is less but value of moment is high that require big girder size than case 4.

4.3. Variation of shear

From ETABS analysis we have got difference in shear for each five cases. These variations will also help us in case of choosing the best option.

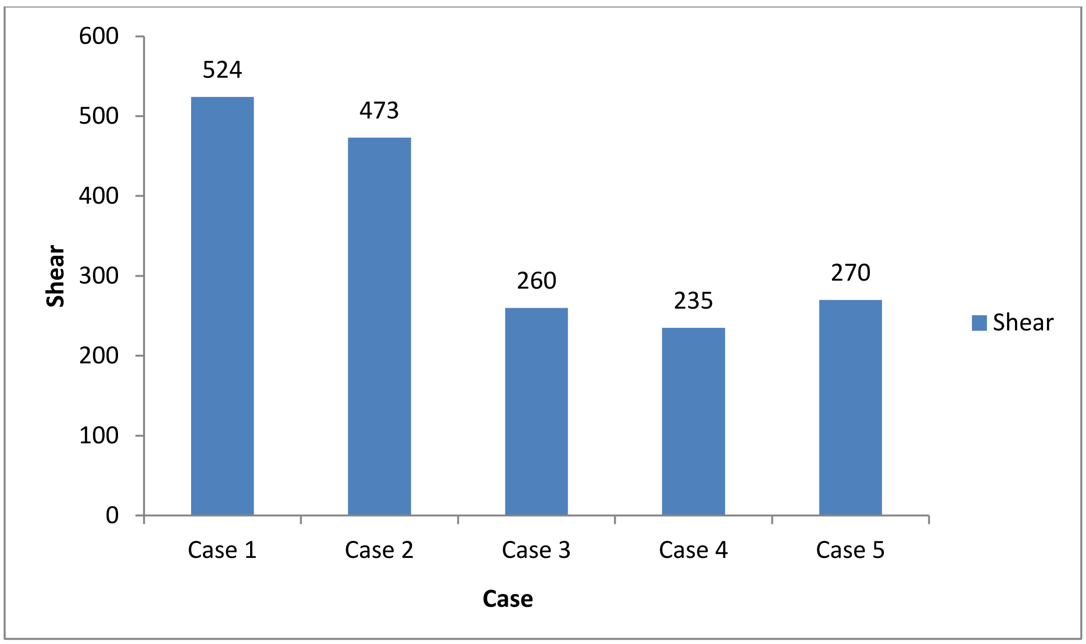


Figure 4.2: Variation of shear.

From the graphical representation we can see that in case 1 shear is highest and in case 4, shear is lowest.

4.4. Variation in girder section

As we have got variations in moments and shear, so there must be some variation on girder sizes. If moment and shear is high then girder size will be high. After calculating the girder sizes, steel area required for both positive and negative moments, we have got the following data:

Table 4.1: Variation on girder section

Case	Size	A_s (-ve) in^2	A_s (+ve) in^2
Case 1	25” x 45”	26.19	4.80
Case 2	25” x 36”	19.28	11.56
Case 3	25” x 34”	17.22	17.36
Case 4	20” x 30”	11.5	15.14
Case 5	20”x 34”	15.56	17.34

Case 1 has a higher value of shear, resulting with highest girder section. On the other hand, case 4 has the smallest section required for the girder.

4.5. Variations of displacements

From ETABS, we get displacement for both earthquake load and wind load. The detailed calculations are stated below:

Table 4.2: Displacement in X-direction (mm) due to earthquake load

Cases	1st story	2nd story	3rd story	4th story	5th story	6th story	7th story	8th story	9th story
Case 1	3.71	7.4	11.23	14.92	18.32	21.36	23.99	26.15	27.83
Case 2	3.66	8.27	13.11	17.74	21.99	25.75	28.93	31.46	33.31
Case 3	2.83	5.98	9.29	12.45	15.33	17.85	19.95	21.58	22.73
Case 4	3.47	6.69	10.06	13.23	16.21	18.79	20.97	22.69	23.93
Case 5	3.89	7.26	10.77	14.11	17.15	19.83	22.07	23.84	25.10

Table 4.3: Displacement in X-direction (mm) due to wind load

	1st story	2nd story	3rd story	4th story	5th story	6th story	7th story	8th story	9th story
Case 1	9.88	19.62	29.36	38.25	46.04	52.61	57.91	61.95	64.92
Case 2	6.94	15.66	24.42	32.33	39.14	44.72	49.04	52.09	54.07
Case 3	7.85	16.27	24.70	32.29	38.77	44.03	48.03	50.76	52.42
Case 4	9.80	18.54	27.25	35.11	41.88	47.42	51.69	54.71	56.64
Case 5	10.77	19.78	28.13	36.79	43.70	49.36	53.71	56.78	58.76

The graphical representation of the above provided data are given below:

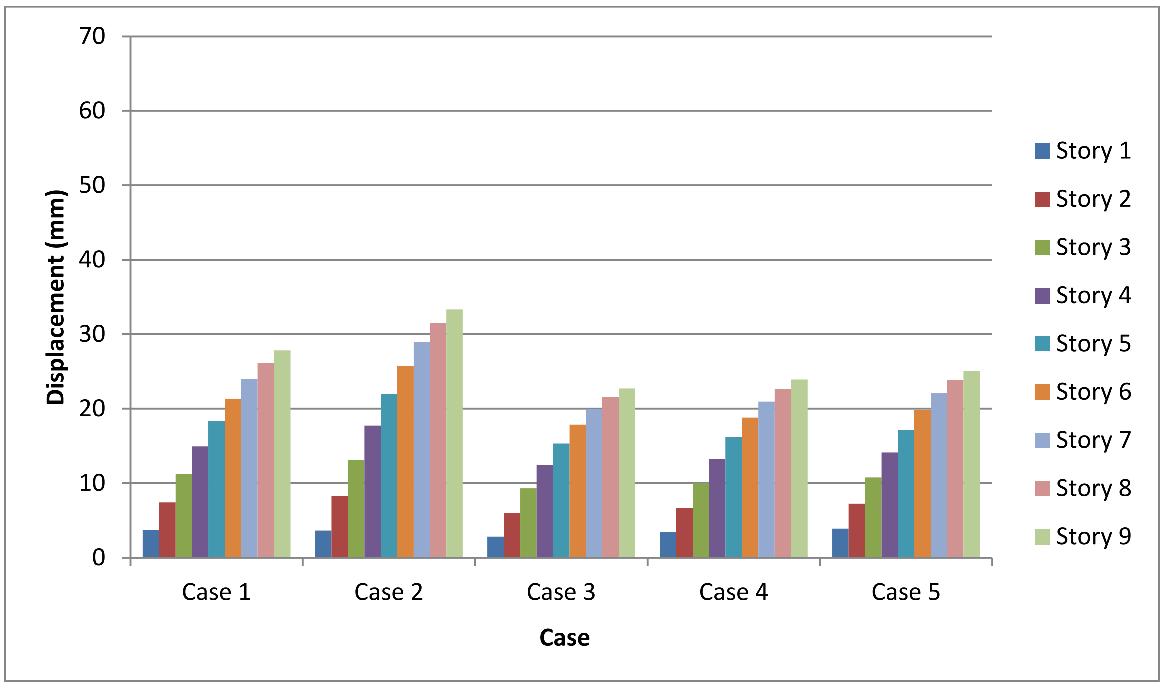


Figure 4.3: Comparison of displacements due to earthquake load in X-direction

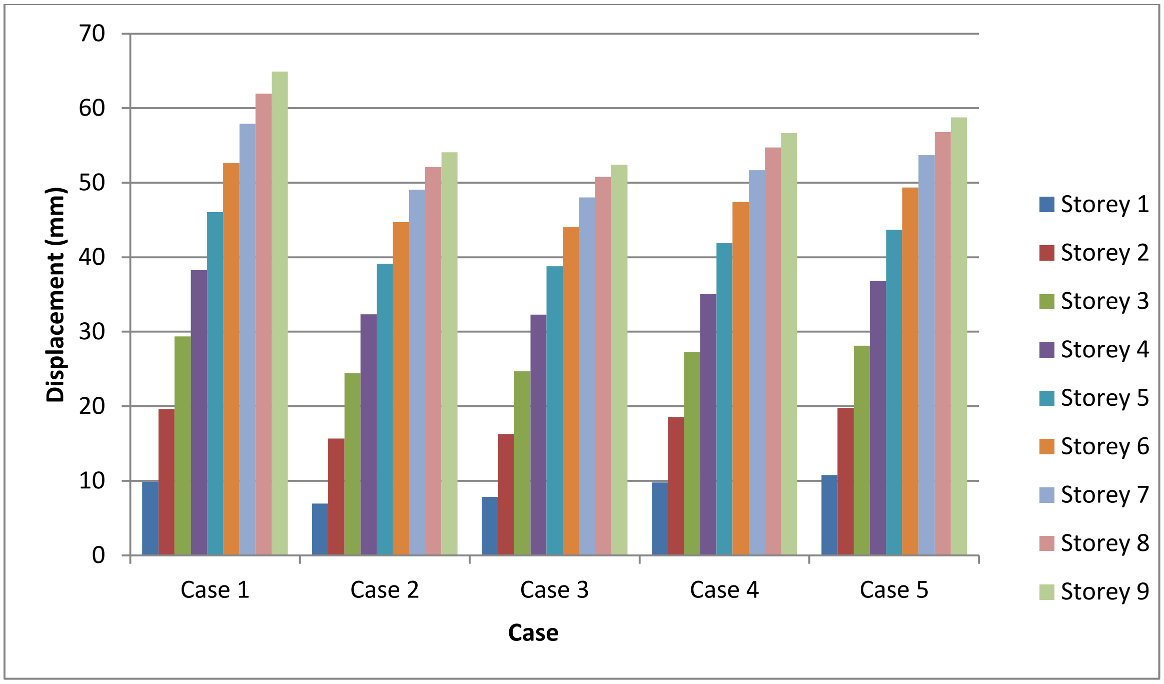


Figure 4.4: Comparison of displacements due to wind load in X-direction

From the displacement graph, we cannot actually see the proper variation of the five cases. But from the graph, we can see that the displacement due to wind load of case 1 is higher than the other cases. Similarly, the displacement due to earthquake load of case 2 is higher than the other cases.

4.6. Variation of story stiffness

We have calculated stiffness for all the five cases according to Muto's expression. The analytical data is given below:

Table 4.4: Variations of story stiffness

	Case 1		Case 2		Case 3		Case 4		Case 5	
H	Kc	Ratio	Kc	Ratio	Kc	Ratio	Kc	Ratio	Kc	Ratio
95	0	0	0	0	0	0	0	0	0	0
95~85	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
85~75	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
75~65	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
65~55	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
55~45	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
45~35	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
35~25	2058.18	1	2058.188	1	2058.18	1	2058.18	1	2058.18	1
25~15	5888.03	2.86	4538.223	2.20	4189.02	2.04	3061.44	1.48	3747.61	1.82
15~0	8196.85	3.98	6731.99	3.27	6449.63	3.13	3389.7	1.64	3833.65	1.86

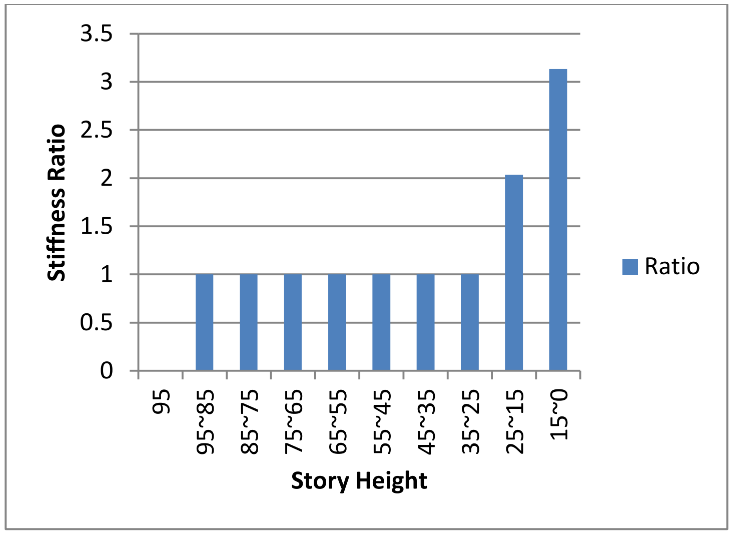


Figure 4.5: Graphical representation of stiffness ratio for case 1

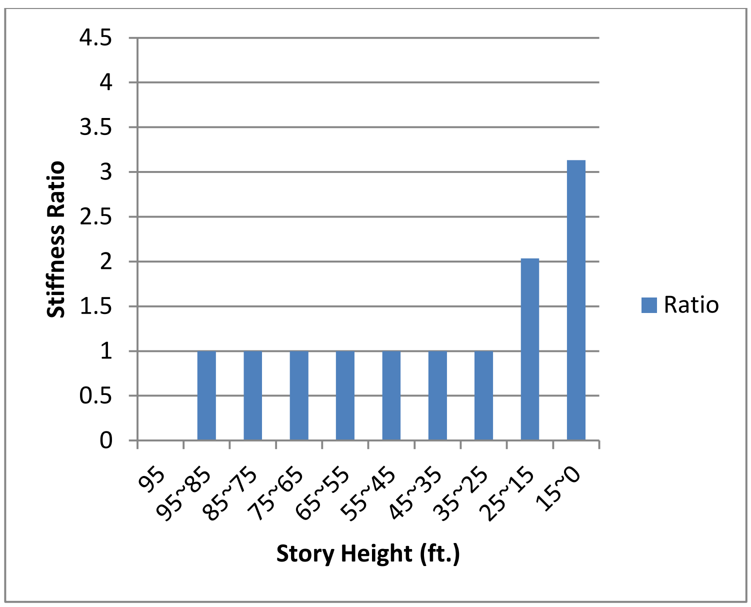


Figure 4.6: Graphical representation of stiffness ratio for case 2

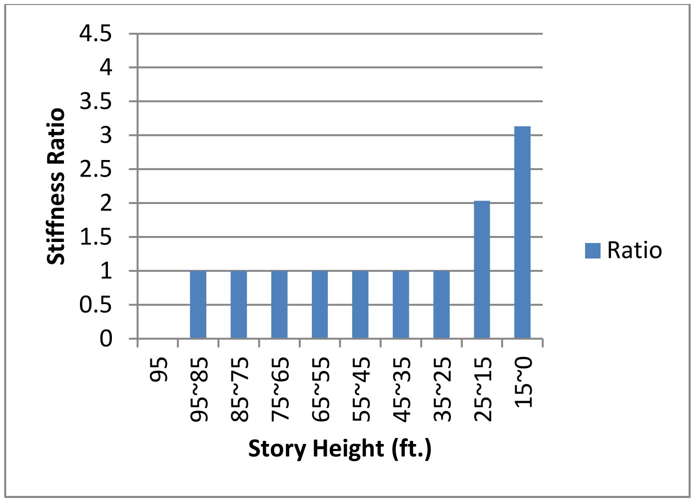


Figure 4.7: Graphical representation of stiffness ratio for case 3

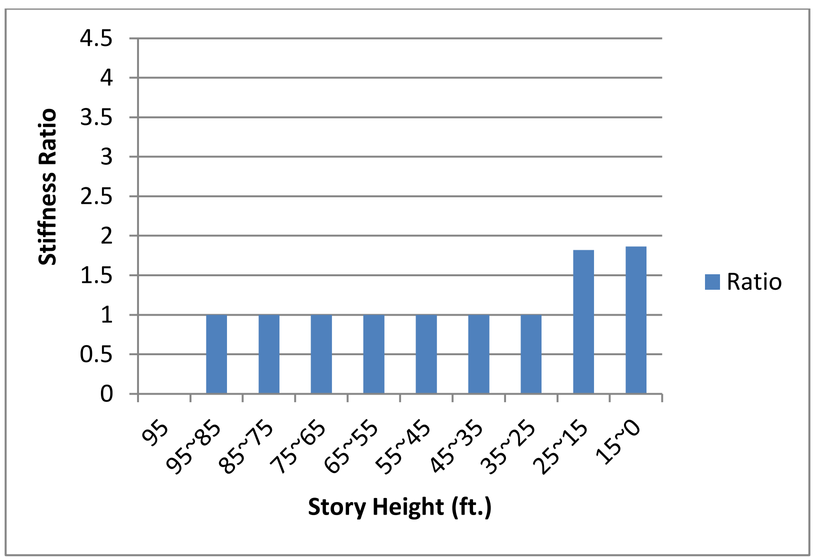


Figure 4.8: Graphical representation of stiffness ratio for case 4

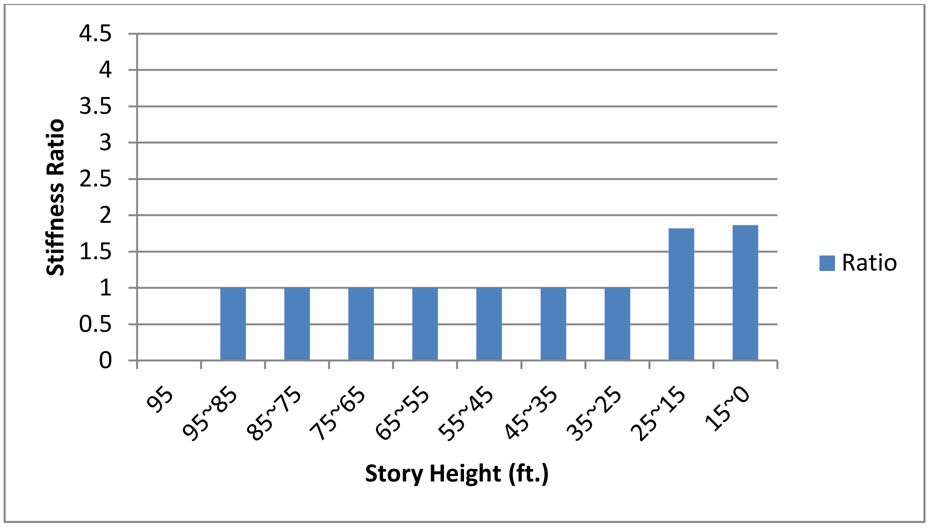


Figure 4.9: Graphical representation of stiffness ratio for case 5

As we can see, the stiffness ratio varies only for 0 – 25'. The stiffness ratio for the remaining stories remains constant. Here is the graphical comparison of variation of stiffness ratio:

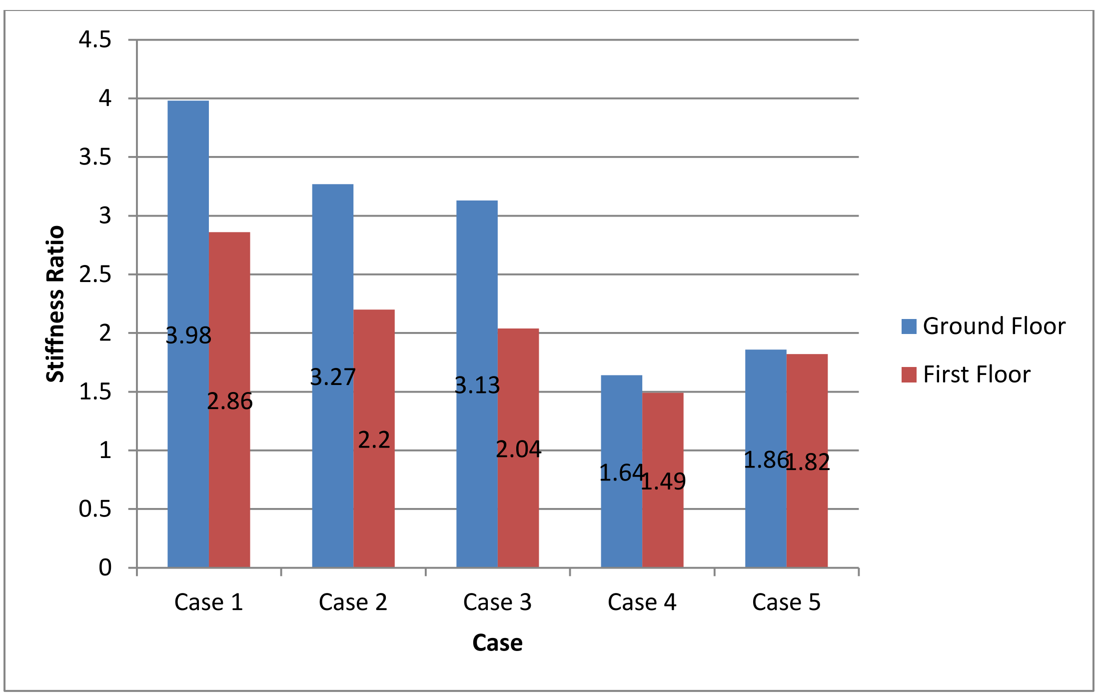


Figure 4.10: Comparison of stiffness ratio

From the comparison of stiffness ratio, we can see that the stiffness ratio is higher in case 1 than other cases. For case 4, the stiffness ratio is lower than the other cases.

4.7. Variations of factor of safety against over turning

We have calculated the factor of safety against overturning for each cases and the minimum factor of safety for any structure against overturning should be 3. If it is greater than 3, then the structure is safe against overturning. If it is not, then the structure is not safe against overturning.

Table 4.5: Factor of safety against overturning

Case	Factor of Safety
Case 1	2.84
Case 2	4.22
Case 3	5.68
Case 4	5.68
Case 5	5.68

From the comparison we can see that factor of safety in case 1 is less than 3. But in other cases, the factor of safety is greater than 3. In case 2, 3, 4 and 5, the structure is ok regarding factor of safety against overturning. But in case 1, the structure is not ok as its factor of safety against overturning is less than 3.

4.8. Discussion

Table 4.6: Comparison between different cases

Criterion	Case 1	Case2	Case3	Case4	Case5
Girder size	High compare to others	Higher than others except case1	Higher than case 3and4 but lower than case1and2	Low compare to other cases	Lower than others but higher than case 4
Ground floor column size	high	high	high	Lower than case 1,2,3	Lower than case 1,2,3
Moment variation	high	High except case1	Higher than case 3and4 but lower than case1and2	low	low
Shear	severe	high	moderate	Very low	low
Overturning	Less than 3	Greater than 3	Greater than 3	Greater than 3	Greater than 3
Story stiffness	high	High but lower than case1	Higer than 4and5 but lower than 1and2	low	Low but higher than case4
displacements	high	high	lower	lower	high

From the comparison we can see that factor of safety in case 1 is less than 3. But in other cases, the factor of safety is greater than 3. In case 2, 3, 4 and 5, the structure is ok regarding factor of safety against overturning. But in case 1, the structure is not ok as its factor of safety against overturning is less than 3.

Chapter – 5

Conclusions and recommendations

5.1. General

In our study if we can see that from the literature review this type of structure is built but the analysis was not accurate. That why during earthquake load and heavy wind load structures got damaged.at ground floor the load distribution path is not accurate as regular structure. Though the displacements graph is same but some more analysis should be needed and that is done is in our study.we analysis the girder size, factor of safety against overturning, stiffness ratio.

5.2 Summary of Conclusion

During literature review ,we have found that some buildings in BHUJ was failed due to lack of analysis of the displacement of the columns at the ground floor to provide maximum parking facilities.This incidence have caught our attention to work on this topic.Thus,we have performed various numerical and analytical analysis for five different cases and we have studied various girder designs,stiffness calculations and factor of safety against overturning.Our main goal is to attain maximum parking spaces by ensuring safety of structure against wind loads,earthquake loads and various overturning moments.

5.3. Recommendations

- Further analysis can be done by using the softwares called ANSYS,ABAQUS etc.
- In our analysis we have followed the linear method but it can also be done by dynamic analysis.

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