Static & Response Spectrum Analysis of High Rise Multi Planned Residential Building using ETABS

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Abstract— In these moderns days the buildings are made to fulfil our basic aspects and better serviceability. It is not an issue to construct a building any how its, important to construct an efficient building which will serve for many years without showing any failure. The project titled "planning and designing of multistory residential building", aims in finding better technique for creating geometry, defining the cross sections for column and beam etc., creating specification and supports (to define a support whether it is fixed or pinned), then the loads are calculated manually and applied to the model in ETABS. After that the model is analysed by 'run analyses'. This project are going to deal about dead load calculation like external wall load, internal wall load, parapet wall are calculated a per IS 875-part 1 live load has been taken from as per IS 875 part 2 to the building (G+10) and earth quake load has been taken from IS 1893-2002. Wind load has been taken from IS 875 part 3. The high rise building with 10 stores is analysed for its displacement, strength and stability using ETABS software. For the analysis of the building for seismic loading with two different zones (Zone-II & Zone-V) is considered with all soil types. The analysis of the building is done by using equivalent static method and dynamic method. The results from the analysis obtained from both the methods are presented in tabular form and the results are compared using graphical form. The results of the analysis on the displacement shear force, and bending moment are compared. The results are presented in tabular and graphical form. The results on the drift and displacement are checked with serviceability conditions and are compared and presented in tabular form. The zone wise results are also compared.

Key words: ETABS, Spectrum Analysis

I. INTRODUCTION

Earthquake is the result of a sudden release of energy in the earth's crust that creates seismic waves. The seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time.

A list of natural and man-made earthquake sources

Natural Source	Man-made Source
Tectonic Earthquakes	– Controlled Sources (Explosives)
Volcanic Earthquakes	– Reservoir Induced Earthquakes
Rock Falls/Collapse of Cavity	– Mining Induced Earthquakes
Microseism	– Cultural noise (Industry, Traffic, etc)

Buildings are subjected to ground motion. PGA (Peak Ground Acceleration), PGV (Peak Ground Velocity), PGD (Peak Ground Displacement), Frequency Content, and Duration play predominant rule in studying the behaviour of buildings under seismic loads.

The new RESIDENTIAL block is located at the province of Towlichowki, Hyderabad. The total built up area of the RESIDENTIAL building is 611.2 square meters and has five floors (Ground floor +10). The building is located in seismic prone zone (zone factor II & V). Since RESIDENTIALs are very important buildings and need to remain standing after the earthquake, the design of such buildings needs to be done as per earthquake design considerations.

A. General

Structural analysis is the backbone of civil engineering. During recent years, there has been a growing emphasis on using computer aided software and tools to analyze the structures. There has also been advancement in finite element analysis of structures using Finite Element Analysis methods or matrix analysis. These developments are most welcome, as they relieve the engineer of the often lengthy calculations and procedures required to be followed while large or complicated structures are analyzed using classical methods. But not all the time such detailed analysis are necessary to be performed i.e. sometimes, just approximate analysis could suffice our requirements as in case of preparing the rough estimates and participating in the bidding process for a tender. Now-a-days, high rise buildings and multi-bay-multi-story buildings are very common in metropolitan cities. The analysis of frames of multistoried buildings proves to be rather cumbersome as the frames have a large number of joints which are free to move. Even if the commonly used Moment distribution method is applied to all the joints, the work involved shall be tremendous. However, with certain assumptions, applying the substitute analysis methods like substitute frame method, portal method, cantilever method or factor method, the structures can be analyzed approximately.

B. Geometry of the Building

The plan of the Residential building is irregular. It has a story height of H = 3.0m where all stories are of the same height. The RESIDENTIAL building consist of Ten stories, it is eleven stories including ground floor. The RESIDENTIAL building length is m and width is m so the area is m2. The building consist of square columns with cross-section (0.5 x 0.5)m, rectangular beams with cross- section (0.6 x 0.3)m and slab thickness of 150mm. The size of column is constant for all stories. In each storey, the size of the beam is constant. The elastic rigidity of outer beams and columns are half that of interior ones and elastic rigidity of corner columns is one fourth that of interior ones.

C. Objective & Scope

Since RESIDENTIAL is the most important place during a disaster to give humanitarian aid and medical treatment, it is important to make sure that the RESIDENTIAL building can withstand the earthquake. The objective of this study is to make comparisons of analysis and design of a (G+5) story

RESIDENTIAL building. Several cases of seismic loads will be applied to the building.

In Afghanistan there is no particular seismic analysis code for buildings, therefore Indian Standard Code (IS 1893-2002) will be used for this study. The building will be designed according to the Earthquake resistant considerations. The present study deals with an Equivalent Static Analysis of 6 story RCC RESIDENTIAL building using Structural Analysis and Design (ETABS.) software.

D. *Methodology*

- Review the existing literature and Indian design code provision for analysis and design of the earthquake resistant building.
- The different types of structures are selected.
- The selected structures are modelled.
- Performing linear analysis for selected building for both gravity load, and earthquake loads and then a comparative study of both is obtained from the analysis.
- Also design the building manually for design and analysis results obtained and compare with the area of steel of the models obtained.
- Using structural analysis and design Software ETABS and comparing results.
- Observation of results and discussions.

E. *ETABS*

ETABS is engineering software that caters to multi-story & High Rise buildings to analysis and design. Modeling tools and templates, code-based load prescriptions, analysis methods and solution techniques, all coordinate with the grid-like geometry unique to this class of structure. Basic or advanced systems under static or dynamic conditions may be evaluated using ETABS. For a sophisticated assessment of seismic performance, modal and direct-integration time-history analyses may couple with P-Delta and Large Displacement effects. Interoperability with a series of design and documentation platforms makes ETABS a coordinated and productive tool for designs which range from simple 2D frames to elaborate modern high-rises.

ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analyses

1) *ETABS can Deal with Easily*

- Multi-story industrial, authorities and health care centers
- Parking garages with circular and linear ramps
- buildings with curved beams, walls and floor edges
- homes with steel, concrete, composite or joist floor framing
- tasks with multiple towers
- complex shear partitions and cores with arbitrary openings
- buildings based on a couple of square and/or cylindrical grid systems
- Flat and waffle slab concrete homes
- buildings subjected to any variety of vertical and lateral load
- cases and combos, consisting of automatic wind and seismic hundreds
- multiple reaction spectrum load cases, with integrated enter curves
- computerized switch of vertical masses on floors to beams and partitions
- capability test of beam-to-column and beam-to-beam metal connections
- P-Delta evaluation with static or dynamic evaluation
- express panel-quarter deformations
- creation collection loading analysis
- a couple of linear and nonlinear time history load instances in any route

II. HISTORY OF STRUCTURAL ANALYSIS

A structure refers to a system of two or more connected parts used to support a load. It may be considered as an assembly of two or more basic components connected to each other so that they carry the design loads safely without causing any serviceability failure. Once a preliminary design of a structure is fixed, the structure then must be analysed to make sure that it has its required strength and rigidity. The loadings are supposed to be taken from respective design codes and local specifications, if any. The forces in the members and the displacements of the joints are found using the theory of structural analysis. The whole structural system and its loading conditions might be of complex nature, so to make the analysis simpler, certain simplifying assumptions related to the material quality, member geometry, nature of applied loads, their distribution, the type of connections at the joints and the support conditions are used. This shall help making the process of structural analysis simpler to quite an extent.

A. *Methods of Structural Analysis*

When the number of unknown reactions or the number of internal forces exceeds the number of equilibrium equations available for the purpose of analysis, the structure is called a statically indeterminate structure. Many structures are statically indeterminate. This indeterminacy may be as a result of added supports or extra members, or by the general form of the structure. While analysing any indeterminate structure, it is essential to satisfy equilibrium, compatibility, and force-displacement conditions for the structure. The fundamental methods to analyse the statically indeterminate structures are discussed below.

1) *Force Method*

The force method developed first by James Clerk Maxwell and further developed later by Otto Mohr and Heinrich Muller-Breslau was one of the first methods available for analysis of statically indeterminate structures. This method is also called compatibility method or the method of consistent displacements. In this method, the compatibility and force displacement requirements for the given structure are first defined in order to determine the redundant forces. Once these forces are determined, the remaining reactive forces on the given structure are found out by satisfying the equilibrium requirements.

2) *Displacement Method*

In the displacement method, first load-displacement relations for the members of the structure are written and then the equilibrium requirements for the same are satisfied. The unknowns in the equations are displacements. Unknown displacements are written in terms of the loads or forces by using the load-displacement relations and then these equations are solved to determine the displacements. As the displacements are determined, the loads are found out from the compatibility and load- displacement equations. Some classical techniques used to apply the displacement method are discussed.

3) Slope Deflection Method

This method was first devised by Heinrich Manderla and Otto Mohr to study the secondary stresses in trusses and was further developed by G. A. Maney in order to extend its application to analyze indeterminate beams and framed structures. The basic assumption of this method is to consider the deformations caused only by bending moments. It is assumed that the effects of shear force or axial force deformations are negligible in indeterminate beams or frames. The fundamental slope-deflection equation expresses the moment at the end of a member as the superposition of the end moments caused due to the external loads on the member, with the ends being assumed as restrained, and the end moments caused by the displacements and actual end rotations. Slope-deflection equations are applied to each of the members of the structure. Using appropriate equations of equilibrium for the joints along with the slope-deflection equations of each member, a set of simultaneous equations with unknowns as the displacements are obtained. Once the values of these displacements are found, the end moments are found using the slope-deflection equations.

4) Moment Distribution Method

This method of analyzing beams and multi-storey frames using moment distribution was introduced by Prof. Hardy Cross in 1930, and is also sometimes referred to as Hardy Cross method. It is an iterative method. Initially all the joints are temporarily restrained against rotation and fixed end moments for all the members are written down. Each joint is then released one by one in succession and the unbalanced moment is distributed to the ends of the members in the ratio of their distribution factors. These distributed moments are then carried over to the far ends of the joints. Again the joint is temporarily restrained before moving on to the next joint. Same set of operations are performed at each joint till all the joints are completed and the results obtained are up to desired accuracy. The method does not involve solving a number of simultaneous equations, which may get quite complicated while dealing with large structures, and is therefore preferred over the slope-deflection method.

5) Kani's Method

This method was first developed by Prof. Gasper Kani of Germany in the year 1947. This is an indirect extension of slope deflection method. This is an efficient method due to simplicity of moment distribution. The method offers an iterative scheme for applying slope deflection method of structural analysis. Whereas the moment distribution method reduces the number of linear simultaneous equations and such equations needed are equal to the number of translator displacements, the number of equations needed is zero in case of the Kani's method

III. Literature Review

A. General

Valmundsson and Nau (1997) studied the earthquake response of 5-, 10-, and 20-story framed structures with non-uniform mass, stiffness, and strength distributions. The object of the study was to investigate the conditions for which a structure can be considered regular.

Das (2000) found that the structures designed by ELF method performed reasonably well. He concluded that capacity based criteria must be suitably applied in the vicinity of the irregularity.

Sadjadi et al. (2007) presented an analytical approach for seismic assessment of RC frames using nonlinear time history analysis and push-over analysis. The results from analytical models were validated against available experimental results. He observed that ductile and less ductile frames behaved very well under the earthquake considered.

Adiyanto (2008) analyzed a 3-storey RESIDENTIAL building using ETABS. Seismic loads were applied to the building. The dead loads and live loads were taken from BS6399:1997 and seismic loads intensity is based on equivalent static force procedure in UBC1994. Result showed that the building can withstand any intensity of earthquake. It means that the buildings were suitable to be built in any area located near the epicenter of the earthquake.

Kim and Elnashai (2009) observed that buildings for which seismic design was done using contemporary codes survived the earthquake loads. However the vertical motion significantly reduced the shear capacity in vertical members.

Griffith and Pinto (2009) investigated a 4-storey, 3-bay reinforced concrete frame test structure with unreinforced brick masonry (URM) infill walls for its weaknesses with regard to seismic loading. The concrete frame was shown to be essentially a "weak-column strong-beam frame" which was likely to exhibit poor post yield hysteretic behavior.

Di Sarno and Elnashai (2011) assessed the seismic performance of a 9-storey steel perimeter MRF (moment resisting frame) using the three types of bracings: special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). Local (member rotations) and global (inter-storey and roof drifts) deformations were employed to compare the inelastic response of the retrofitted frames. MBFs were found to be the most cost-effective bracing system with the least storey drifts.

Kumar et al (2011) experimentally investigated the behavior of retrofitted FRP (fiber reinforced polymer) wrapped exterior beam-column joint of a G+4 building in Salem. The test specimen was taken to be one fifth model of beam column joint from the prototype specimen and was evaluated in terms of load displacement relation, ductility, stiffness, and load ratio and cracking pattern. On comparing the test results with the analytical modeling of the joint on ANSYS and ETABS, it was found that such external confinement of concrete increased the load carrying capacity of the control specimen by 60% and energy absorption capacity by 30-60%.

Agrawal (2012) did seismic evaluation of an old institute building. Agrawal evaluated the member for seismic loads by determining the Demand Capacity Ratio (DCR) for beams and columns. Being an old building, the reinforcement details of the building were not available. Hence Design-1 was done applying only DEAD and LIVE loads according to IS 456:2000 to estimate the reinforcement present in the building and assuming that this much reinforcement was present. In Design-2, seismic loads were applied and for this demand obtained from design-2 and capacity from design -1, the DCR was calculated.

Poonam et al. (2012) concluded that any storey must not be softer than the storeys above or below. Irregularity in mass distribution contributed to the increased response of the buildings.

Rajeeva and Tesfamariam (2012) studied the Fragility based seismic vulnerability of structures considering soft -storey and quality of construction. Their study was demonstrated on three, five, and nine storey RC building frames designed prior to 1970s. Probabilistic seismic demand model was developed using non-linear finite element analysis considering the interactions between soft -storey and quality of construction. Their study showed the sensitivity of the model parameter to the interaction of soft -storey and quality of construction.

B. Summary of Literature Review

The present study aims to design a RESIDENTIAL building in Afghanistan using Equivalent Static analysis method. The base shear is calculated on the basis of mass and fundamental period of vibration and mode shape. The base shear is distributed along the height of structure in terms of forces according to IS 1893 (2002) Code formula. No similar code for design is available in Afghanistan. The Equivalent Static method is usually used for low, medium height buildings. This method is simple and accurate enough to use

IV. NUMERICAL MODELING

A. Geometrical Properties

1) 1Height of typical storey = 3 m
2) Height of groundstorey= 3.5m
3) Length of the building = 49.56m
4) Width of the building = 17.65 m
5) Height of the building = 45 m
6) Number of stores = 15
7) Wall thickness = 230 mm
8) Slab Thickness =115 mm
9) Grade of the concrete =M20
10) Grade of the steel = Fe415
11) Support = fixed
12) Column sizes1to10th storey =0.53m X 0.53 m 11 to 15th storey= 0.45m X 0.45 m
13) Beam sizes
    1 to 10th storey =0.23m X 0.4 m
    11 to 15th storey =0.23m X 0.45 m

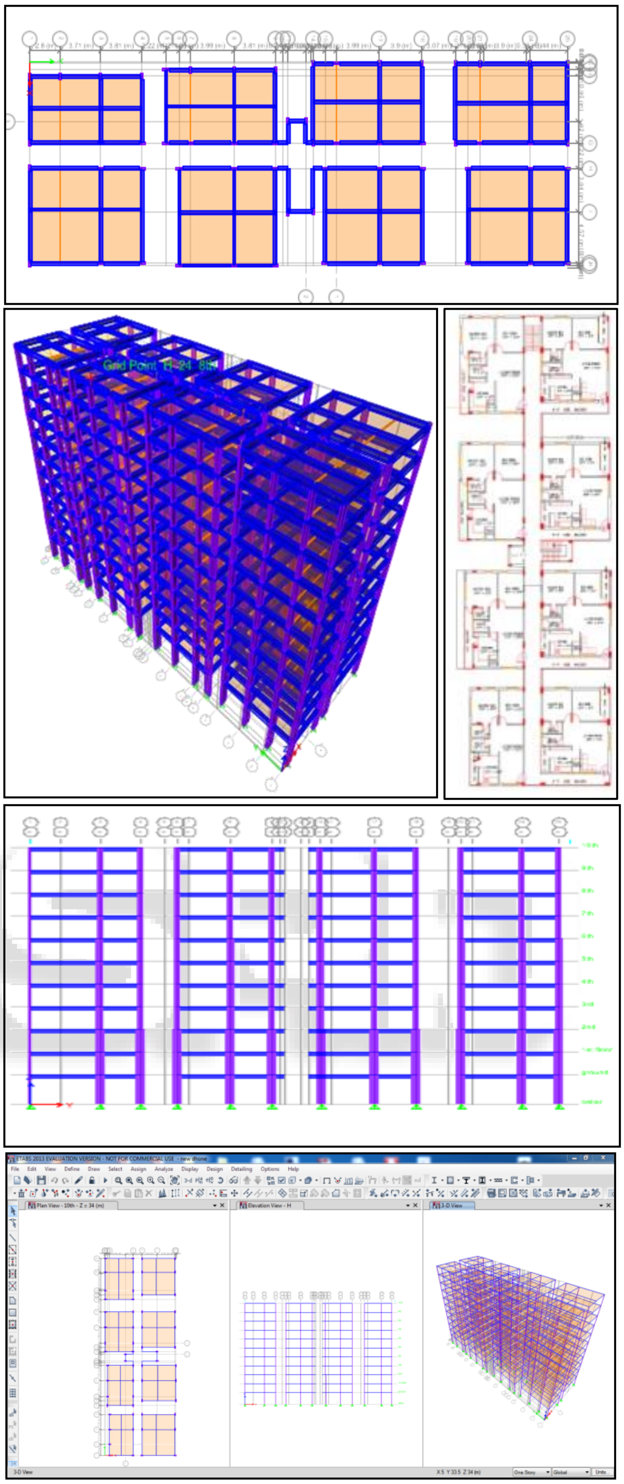


Fig. 1: Showing plan, 2D elevation and 3D elevation of proposed structure

B. Loads

1) Live Load

Live load from 1st floor to 15th floor =3.5 kN/m^2

2) Dead Load

Dead load is taken as prescribe by the IS: 875 -1987 (Part-I) [3] Code of Practice Design Loads (other than earthquake) for Buildings and structure.

- Unit weight of R.C.C. = 25 kN/m^3
- Unitweight of brick masonry=19 kN/m^3

- Floor finish= 1.5 kN/m^2
- Wall load= 13.8 kN/m on all floors

3) Wind Load

The basic wind speed (V_b) for any site shall be obtained from IS 875(Part 3 -1987) [4] it is 44 m/sec and shall be modified to include the following effects to get design wind velocity at any height (V_z) for the chosen the structure.

Risk level Terrain roughness, height and size of structure, and Local topography It can be mathematically expressed as follows:

- Vz = Vb K1.K2.K3 Eq. (4.1l) [5] Where,
- Vz= design wind speed at any height z in m/s
- K1 =probability factor (risk coefficient) (Refer 5.3.1 of is 875(Part 3 -1987))
- K2 = terrain, height and structure size factor (Refer 5.3.2 of IS 875(Part 3 – 1987))
- K3 = topography factor (Refer 5.3.3 of IS 875(Part 3 - 1987))

a) Wind Exposure Parameters

- Wind direction angle = 0 Degree
- Windward coff. Cp = 0.8
- Leeward coff Cp = 0.5

b) Wind Coefficients

1) Wind speed = 39 m/s
2) Terrain category = 4
3) Structure class = A
4) Risk coefficient (k1) = 1.05
5) Topography (k3) = 1

4) Seismic Loading

In the present work the building is located in Hyderabad which comes under –zone-II, using the IS 1893 (Part-I) – 2002(1) the following are the various values for the building considered.

a) Zone Factor (Z)

It is a factor to obtain the design spectrum depending on (lie perceived maximum seismic risk characterized by Maximum considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate of effective peak ground acceleration.

1) Zone factor = 0.36 (Zone-V) (from IS 1893 (Part-I)-2002, Table- 2).
2) Zone factor = 0.10 (Zone-II) (from IS 1893 (Part-I)-2002, Table- 2).

b) Response Reduction Factor I

It is the factor by which the actual base-shear force that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall by reduced to obtain the design lateral force.

Response reduction factor = 5.0 for Zone-V (from IS 1893 (Part-1)-2002, Table-7. I

Response reduction factor = 3.0 for Zone-II (from IS 1893 (Part-1)-2002, Table-7. I

c) Importance Factor (I)

It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterized by hazardous consequences of post-earthquake functional need, historical value, or economic importance.

- Importance factor (1) = 1.5 (from IS 1893-2002 (Part-I), Table-6).

d) Soil Type

Soil site factor (1 for hard soil, 2 for medium soil, and 3 for soft soil) depending on type of soil average response acceleration coefficient Sa/g is calculated corresponding to 5% damping Refer Clause 6.4.5 of IS 1893-2002. In the present work three type of soils are used.

- Soil type considered is hard soil, factor 1.
- Soil type considered is medium soil, factor 2.
- Soil type considered is loose soil, factor 3.

e) Damping

The effect of internal friction, imperfect elasticity of material, slipping, sliding etc. in reducing the amplitude of vibration and is expressed as a percentage critical damping. Damping – 5%

C. Material Properties

1) Modulus of Elasticity

The modulus of elasticity is primarily influenced by the elastic properties of the aggregate and to a lesser extent by the condition of curing and age of the concrete, the mix proportions and the type of cement. The modulus of elasticity is normally related to the compressive strength of concrete.

The modulus of elasticity of reinforced concrete members can be assumed as

$$E = 5000\sqrt{fck} \text{ Eq, (17) [3]}$$

Where,

- E=Short term static modulus of elasticity in kN/m^2
- f_{ck} = Characteristic cube compressive strength of concrete in N/mm^2

The modulus of elasticity of reinforced concrete members for M20 grade concrete is E= 22360679 kN/m^2

2) Poisson's Ratio

Poisson's ratio is the ratio between lateral strains to the longitudinal strain. It is generally denoted by the letters for normal concrete the value of Poisson's ratio lies in the range of 0.15 to 0.20 when actually determined from strain measurement. For the present work Poisson's ratio is assumed as 0.2 for reinforced concrete.

V. STATIC & DYNAMIC ANALYSIS

A. Design Aspect

Earthquakes can occur on both land and sea, at any place on the surface of the earth where there is a major fault. When earthquake occurs on land it affects the man made structure surrounding its origin leading to human lose. When a major earthquake occurs underneath the ocean or sea, it not only affects the structures near it, but also produces large tidal waves known as Tsunami, thus affecting the places far away from its origin. All the structures are designed for the combined effects of gravity loads and seismic loads to verify that sufficient vertical and lateral strength and stiffness are achieved to satisfy the structural concert and acceptable deformation levels prescribed in the governing building code. Because of the innate factor of safety used in the design specifications, most structures tend to be adequately protected against vertical shaking. Vertical acceleration should also be considered in structures with large spans, those in which stability for design, or for overall stability analysis of structures. When planning a building against natural hazards like earthquakes or cyclones, the following three limit states considerations are taken into account:

1) Serviceability Limit State

The structure undergoes little or no structural damage in this case. Important buildings such as RESIDENTIALs, atomic power stations, places of assembly etc., which affects a community, should be designed for elastic behavior under expected earthquake forces. These types of structures should be serviceable even after the occurrence of earthquake or cyclones.

2) Damage Controlled Limit State

In this case, if an earthquake or cyclone occurs, there can be some damage to the structure but it can be repaired even after the occurrence of the disaster. Most of the permanent buildings should come under this category, so, the structure should be designed for limited ductility response only.

3) Survival Limit State

In this case, the structure is allowed to be damaged in the event of earthquake or cyclone disasters. But, the supports should stand and support the permanent loads coming on to it so that there should be no caving in of the structure and no loss of life. By avoiding the brittle failure of the members and also ensuring that supporting elements like columns do not collapse, we can achieve the above specified behavior. Earthquake energy should also be designed to be dissipated by 'full' ductile behavior response of the structure.

We can design famed structures for the first two limit states stated above, by elastic or restricted ductile response of the structure using conventional methods of design and incorporating ductile detailing. Designing for full elastic response is costly. Limited ductile response is cheaper and full ductile response is cheapest. The full ductile detailing is achieved by the theory of plastic hinge formation and also by careful ductile detailing. The current design practice is to construct the structures for the first two limit states as the other is under development stage only.

B. Design Approach in IS 1893 (2002)

The title of IS 1893-2002 is "Criteria for earthquake resist design of structures" and part 1 of this code deals with General Provisions and buildings [1]. According to this code we consider the following magnitudes of earthquakes:

1) Design Basic Earthquake (DBE)

It is the earthquake which occurs reasonably at least once during the designed life of the structure.

2) Maximum Considered Earthquake (MCE)

This is the most severe earthquake that can occur in that region as considered by the code. It is divided by factor 2 to get design basic earthquake.

The value of Z, the seismic zone factor given in the code relates the realistic values of effective peak ground acceleration considering MCE and the service life of the structure. The following principles are the basis for the design approach recommended by IS 1893-2002.

1) The structure should have the strength to withstand minor earthquakes less than DBE without any damage.
2) The structure should able to resist earthquakes equal to DBE without significant damage though some nonstructural damage may occur.
3) The structure should able to withstand an earthquake equal to MCE without collapse so that there is no loss of life As, the actual forces will be much larger than the design forces specified by the code, the ductility arising from the inelastic material behavior and detailing along with the reserve strength are relied upon to account for the difference in the actual and the design lateral loads. Conceptual representation of earthquake resistant design philosophy is depicted in the following figure;

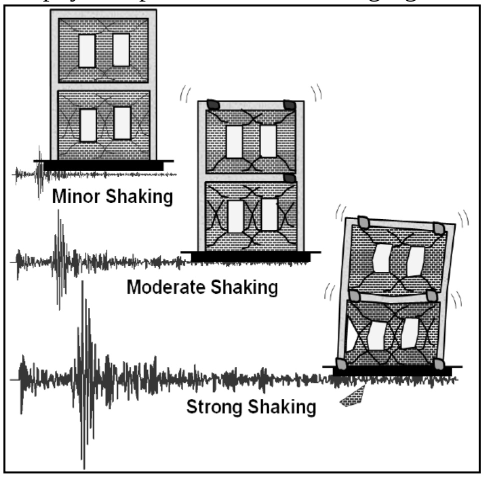


Fig. 4.1: Schematic Diagram Depicting Earthquake Resistant Design Philosophy for Different Levels Shaking

C. Factors Affecting Earthquake Design of Structure

There are many factors of the building that affect the behavior of the building when subjected to an earthquake. The following factors are considered of major importance.

1) Natural frequency of the building
2) Damping factor of the structure
3) Type of foundation of the structure
4) Importance of the building
5) Ductility of the structure

Frames that are specially designed for ductility are known as special moment resisting frames; where as those detailed with fewer considerations are known as ordinary moment resisting frames. For satisfactory performance, if a building is designed as SMRF frame, it needs to be designed for only lesser forces than if it is designed as on OMRF frame.

The analysis can be carried out on the basis of the external action, the behavior of the structure or structural materials, and the type of structural model selected. Based on the height of the structure and zone to which it belongs, type of analysis is selected. In all the methods of analyzing multi-storey buildings recommended in the code, the structure is treated as discrete system having concentrated masses at floor levels, which include half that of columns and walls above and below the floor. In addition, suitable amount of live load at this floor is also lumped with it.

Quite a few methods are available for the earthquake analysis of buildings; two of them are presented here:

a) Equivalent Static Lateral Force Method (pseudo static method).
b) Dynamic analysis.
1) Response spectrum method.
2) Time history method.

D. Equivalent Static Method

$$A_h = \frac{Z\ I\ S_A}{2\ R\ g}$$

The equivalent static method of finding lateral forces is also known as the static method or the seismic coefficient method. This method is the simplest one and it requires less computational attempt and is based on formulae given in the code of practice. In all the methods of analyzing a multi storey buildings recommended in the code, the structure is treated as discrete system having concentrated masses at floor levels which comprise the weight of columns and walls in any storey should be equally distributed to the floors above and below the storey. In addition, the suitable amount of imposed load at this floor is also lumped with it. It is also assumed that the structure flexible and will deflect with respect to the position of foundation; the lumped mass system reduces to the solution of a system of second order differential equations. These equations are formed by distribution of mass and stiffness in a structure, together with its damping characteristics of the ground motion.

E. Design Seismic Base Shear

The design seismic base shear or total design lateral force (V_B) along any principal direction shall be determined by the following expression:

$$V_B = A_h \text{ X } W$$

Where,
A_h = Design horizontal acceleration spectrum value using the fundamental natural period 'T' in the considered direction of vibration
W = seismic weight of the building
The A_h shall be determined by the following expression:
Provided that for any building with T less than 0.1s, the value of A_h shall not be taken less than Z/2 whatever be the value of I/R.
Where,
Z = Zone factor is determined from the following table

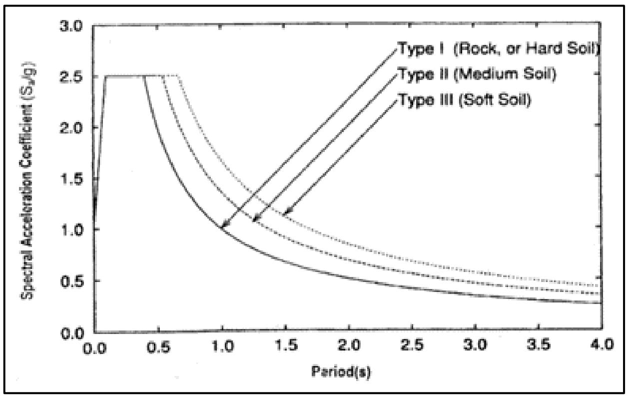
Seismic Zone	II	III	IV	V
Seismic intensity	Low	Moderate	severe	Very severe
Z	0.10	0.16	0.24	0.36

Zone factor given in the above table is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE).

I = I represents the importance factor and it depends upon the functional use of the structures. It is characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value or economic importance. 1.5 is considered for the important structures like RESIDENTIALs, schools, monumental buildings etc. and the rest of the buildings it is taken as 1.

R = It is Response reduction factor which depends on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations of the structure. This ration should not be greater than one. The values for R are given in Table 7 of IS: 1893. The value for R varies between 3 and 5 with respect to ductile reinforcement detailing.

S_a/g = Average response acceleration coefficient as per clause 6.4.5 of IS 1893:2002 as given by below figure and it is based on the damping and the natural periods of the structures.



NOTE: The value of Ah will not be taken less than ½ where ever the value of l/R.

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ 1.00/T & 0.40 \le T \le 4.00 \end{cases}$$

For medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.00 \end{cases}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.00 \end{cases}$$

F. Time Period

The approximate fundamental natural period of vibration T_a in seconds, of a moment resisting frame building without brick infill panels may be estimated by the following empirical formula

$$T_a = 0.075h^{0.75} \text{For RC frame building}$$
$$T_a = 0.085h^{0.75} \text{For steel frame building}$$

The approximate fundamental natural period of vibration in seconds of all other, buildings including moment resisting frame buildings with brick infill panels may be estimated by the following expression.

$$T = \frac{0.09\ H}{\sqrt{d}}$$

Where,
H = Height of building in meters (This excludes the basement storey, where basement walls are connected with the ground floor deck or fitted between the columns. But, it includes the basement storey, when they are not connected).
D = Base dimensions of the building at the plinth level, in m, along the considered direction of the lateral force.

G. Seismic Weight

The seismic weight of a structure is the sum of seismic weight of all the floors in the structure. The seismic weight of every floor is the sum of its full dead load and appropriate amount of imposed load, the latter being that element of the imposed loads that may sensibly be expected to be attached to the structure at the time of earthquake movement. It includes the weight of permanent and movable partitions, permanent equipment, a part of the live load, etc. While computing the seismic weight of walls and columns in any storey shall be equally distributed to the floors above and below the storey.

H. Distribution of Design Force

The computed base shear is now distributed along the height of the building. The shear force, at any level depends on the mass at that level and tends to deform the shape of the structure. Earth quake forces deflect the structure into number of shapes known as the natural mode shapes and the number of natural mode shapes depends up on the degree of freedom of the system. Generally a structure has continuous system with infinite degree of freedom. The magnitude of the lateral force at a particular floor depends on the mass of the node, the distribution of stiffness over the height of the structure and the nodal displacement in the given mode. The actual distribution of base share over the height of the building is obtained as the superposition of all the mode of vibration of the multi degree of freedom system.

$$Q_i = V_B \frac{W_i\, h_i^2}{\sum_{j=1}^{n} W_j\, h_j^2}$$

The design base shear (V_B) computed by using the above expression shall be distributed along the height of the building as per the following expression:

Where,

Q_i = Design lateral force at floor i.

W_i= Seismic weight of floor i.

H_i= Height of floor i measured from base, and

n = Number of stores in the building i.e., the number of levels at which the masses are located.

The distribution suggested in the code gives parabolic distribution of seismic forces such that seismic shears are higher near top storey for the same base shear. The assumptions involved in the static procedure reflected in the expression are

a) fundamental mode of the building makes the most significant contribution to base shear, and
b) The total building mass is considered as against the modal mass that would be used in a dynamic procedure.

The mass and stiffness are evenly distributed in the building.

I. Dynamic Analysis

Dynamic analysis shall be carried out to obtain the design seismic force, and its distribution in different levels along the height of the building, and in the various lateral loads resisting element, for the following buildings:

1) Regular Buildings

Those greater than 40m in height in zones IV and V, those greater than 90m in height in zone II and III.

2) Irregular Buildings

All framed buildings higher than 12m in zones IV and V, and those greater than 40m in height in zones II and III.

The analysis of model by dynamic analysis of buildings with unusual configuration should be such that it sufficiently models the types of irregularities present in the building configuration. Buildings with plan irregularities, as defined in Table 4 of IS code: 1893-2002 cannot be modeled for dynamic analysis.

Dynamic analysis may be performed either by the TIME HISTORY METHOD or by the RESPONSE SPECTRUM METHOD,

J. Time History Method

The usage of this method shall be on an appropriate ground motion and shall be performed using accepted principles of dynamics. In this method, the mathematical model of the building is subjected to accelerations from earthquake records that represent the expected earthquake at the base of the structure.

K. Response Spectrum Method

The word spectrum in engineering conveys the idea that the response of buildings having a broad range of periods and is summarized in a single graph. This method shall be performed using the design spectrum specified in code or by a site-specific design spectrum for a structure prepared at a project site. The values of damping for building may be taken as 2 and 5 percent of the critical, for the purposes of dynamic analysis of steel and reinforce concrete buildings, respectively. For most buildings, inelastic response can be expected to occur during a major earthquake, implying that an inelastic analysis is more proper for design. However, in spite of the availability of nonlinear inelastic programs, they are not used in typical design practice because:

1) Their proper use requires knowledge of their inner workings and theories. Design criteria, and
2) Result produced are difficult to interpret and apply to traditional design criteria, and
3) The necessary computations are expensive.

Therefore, analysis in practice typically use linear elastic procedures based on the response spectrum method. The response spectrum analysis is the preferred method because it is easier to use.

L. Response Spectrum Analysis

This method is also known as modal method or mode superposition method. It is based on the idea that the response of a building is the superposition of the responses of individual modes of vibration, each mode responding with its own particular deformed shape, its own frequency, and with its own modal damping.

According to IS-1893(Part-l):2002, high rise and irregular buildings must be analyzed by response spectrum method using design spectra shown in Figure 4.1. There are significant computational advantages using response spectra method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves only the calculation of the maximum values of the displacements and member forces in each mode using smooth spectra that are the average of several earthquake motions. Sufficient modes to capture such that at least 90% of the participating mass of the building (in each of two orthogonal principle horizontal directions) have to be considered for the analysis. The analysis is performed to determine the base shear for each mode using given building characteristics and ground motion spectra. And then the storey forces, accelerations, and displacements are calculated for each mode, and are combined statistically using the SRSS combination.

However, in this method, the design base shear (V_B) shall be compared with a base shear (V_b) calculated using a fundamental period T. If V_B is less than V_b response quantities are (for example member forces, displacements, storey forces, storey shears and base reactions) multiplied by V_B/V_b Response spectrum method of analysis shall be performed using design spectrum. In case design spectrum is specifically prepared for a structure at a particular project site, the same may be used for design at the discretion of the project authorities. Figure 4.1 shows the proposed 5% spectra for rocky and soils sites.

M. Modal Combination

Modal Response quantities (member forces, displacements, storey forces, storey shears and base reactions) for each mode of response may be combined by the complete quadratic combination (CQC) technique or by taking the square root of the sum of the squares (SRSS) of each mode of the moral values or absolute sum (ABS) method.

1) CQC Method

The peak response quantities shall be combined as per the complete quadratic combination (CQC) method.

$$\lambda = \sqrt{\sum_{i=1}^{r}\sum_{j=1}^{r} \lambda_i \, \rho_{ij} \, \lambda_j}$$

Where

r= Number of modes being considered

ρ_{ij}= Cross – modal coefficient

λ_i =Response quantity in mode I (including sign)

λ_j=Response quantity in mode j (including sign)

$$\rho_{ij} = \frac{8\zeta^2(1+\beta)\beta^{1.5}}{(1+\beta)^2 + 4\zeta^2\beta(1+\beta)^2}$$

β = Modal damping ratio (in fraction) as specified in 7.8.2.1

ζ = Frequency ratio $=\frac{\omega_j}{\omega_i}$

2) SRSS Method

If the building does not have closely spaced modes than the peak response quantity due to all modes considered shall be obtained as

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2}$$

Where

λ_k = Absolute value of quantity in mode k, and

r = Number of modes being considered.

3) ABS Method

If the building has a few closely spaced modes, then the peak response quantity due to all modes considered shall be obtained as:

$$\lambda^* = \sqrt{\sum_{c}^{r} \lambda_c}$$

Where, the summation is for the closely-spaced modes only. This peak response quantity due to the closely spaced modes (λ^*) is then combined with those of the remaining well-separated modes by the method described above.

N. Modal Analysis

Building with regular, or nominally irregular, plan configuration may be modeled as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In the modal analysis, the variability in masses and stiffness is accounted for in the computation of lateral force coefficients. The following expressions are used for the computation of various quantities:

$$M_k = \frac{[\sum_{i=1}^{n} W_i \phi_{ik}]^2}{g \sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

Modal mass:The modal mass (Mk) of mode 'k' is given by

Where

g =Acceleration due to gravity, and

ϕ_{ik}=Mode shape coefficient at floor (i) in mode 'k' and

W_i= Seismic weight of floor (i)

Modal participation factor: The modal participation factor (P_k) of mode k is given by

$$p_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

Design lateral force at each floor in each mode: The peak lateral force ϕ_i at floor I in mode k^{th} is given by

Where,

Ak= Design $\phi_{ik} = A_k \phi_{ik} p_k W_i$ horizontal acceleration spectrum value as per Eq. (3.2). Using the natural period of vibration (Tk)of mode 'k'.

Storey shear forces in each mode:

The peak shear force (V_{ik}) acting in storey 'i' in mode 'k' is given by

$$V_{ik} = \sum_{j=i+1}^{n} \phi_{ik}$$

Storey Shear forces due to all modes considered: The peak storey shear force (Vi) in storey i due to all modes considered is obtained by combining those due to each mode in accordance with 3.2.1.2.

Lateral force at each storey due to all modes considered: The design lateral forces F roof and Fi at roof and floor i.

$$F_{roof} = V_{roof}$$
$$F_i = V_i - V_{i+1}$$

O. Equivalent Lateral Force vs Response Spectrum Analysis Procedure

Both, the equivalent lateral force procedure and the response spectrum analysis procedure, are based on the same basic assumptions and are applicable to buildings, which exhibit dynamic response behavior in reasonable conformity with the implications of the assumptions made in the analysis. The main difference between the two procedures lies in the magnitude lies of the base shear and distribution of the lateral force. Whereas in the modal method the force calculations are based on compound periods and mode shapes of several modes of vibration, in the equivalent lateral force method they are based on an estimate of the fundamental period and simple formulae for distribution of forces which are appropriate for buildings with regular distribution of mass and stiffness over height. It would be adequate to use the equivalent lateral force procedure for buildings with the following properties-seismic force-resisting system has the same configuration in all storey and in all floors, floor masses do not differ by more than, say, 30 percent in adjacent floor and cross-sectional areas and moments of inertia of structural members do not differ by more than about 30 percent in adjacent storey. For other buildings, the following sequence of steps may be employed to decide whether the modal analysis procedure ought to be used:

1) Compute lateral forces and storey shears using the equivalent lateral force procedure.
2) Approximate the dimensions of structural members.
3) Compute lateral displacements of the structure as designed in step 2 due to lateral forces in step 1.
4) Computer new sets of lateral forces and storey shears with the displacements computed in step3.
5) If at any storey the recomputed storey shear (step 4) differs from the corresponding original value (step 1) by more than 30 percent, the structure should be analyzed by the modal analysis procedure.

If the difference is less than this value the modal analysis procedure in unnecessary, and the structure should be designed using the storey shears obtained in step 4, they represent an improvement over the results of step 1.

This method for determining modal analysis is efficient and effective. It requires far less computational effort than the use of the modal analysis procedure.

The seismicity of the area and the potential hazard due to failure of the building should also be considered in deciding whether the equivalent lateral force procedure is adequate. For example, even irregular buildings that may require modal analysis according to the criterion described, may be analyzed by the equivalent lateral force procedure if they are not located in higher seismic zones and do not house the critical facilities necessary for post-disaster recovery or a large number of people.

VI. RESULTS

A. *Linear Static Analysis*

DISPLACEMENT			
LOAD COMBINATION :1.2(DL+LL+EQX)			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	10.20	13.70	16.70
9	9.80	13.20	16.00
8	9.10	12.20	14.90
7	8.20	11.00	13.40
6	7.10	9.50	11.60
5	6.10	8.20	10.00
4	5.00	6.70	8.20
3	3.90	5.20	6.40
2	2.70	3.70	4.50
1	1.70	2.30	2.80
GROUND	0.80	1.00	1.20
BASE	0.00	0.00	0.00

Table & Graph 6.1: Showing Storey Displacement for 1.2(DL+LL+EQX) for Zone-II

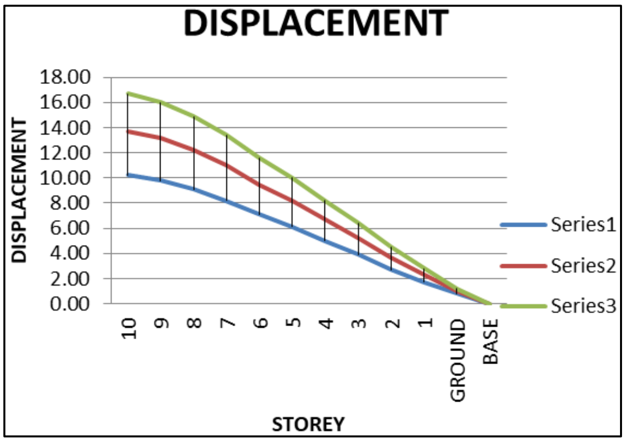


Table & Graph 6.2: Showing Storey Displacement for 1.2(DL+LL+EQY) for Zone-II

STOREY	SOIL-I	SOIL-II	SOIL-III
10	10.2	13.7	16.7
9	9.8	13.2	16
8	9.1	12.2	14.9
7	8.2	11	13.4
6	7.1	9.5	11.6
5	6.1	8.2	10
4	5	6.7	8.2
3	3.9	5.2	6.4
2	2.7	3.7	4.5
1	1.7	2.3	2.8
GROUND	0.8	1	1.2
BASE	0	0	0

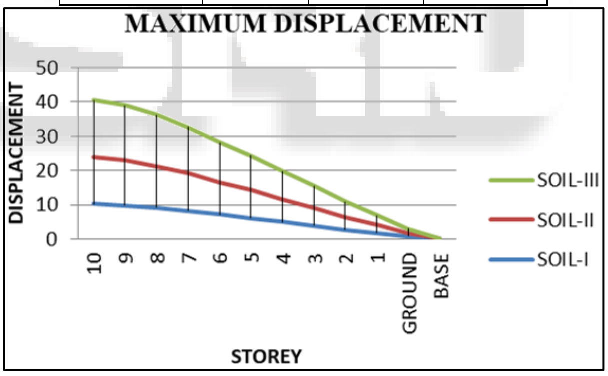


Table & Graph 6.3: Showing Storey Displacement for 1.2(DL+LL+EQX) for Zone-V

DISPLACEMENT			
LOAD COMBINATION :1.2(DL+LL+EQX)			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	35.30	47.80	58.60
9	33.90	45.80	56.20
8	31.50	42.60	52.20
7	28.30	38.30	47.00
6	24.60	33.30	40.80
5	21.10	28.50	35.00
4	17.30	23.50	28.80
3	13.40	18.20	22.30
2	9.50	12.80	15.70
1	5.90	8.00	9.70
GROUND	2.60	3.50	4.30
BASE	0.00	0.00	0.00

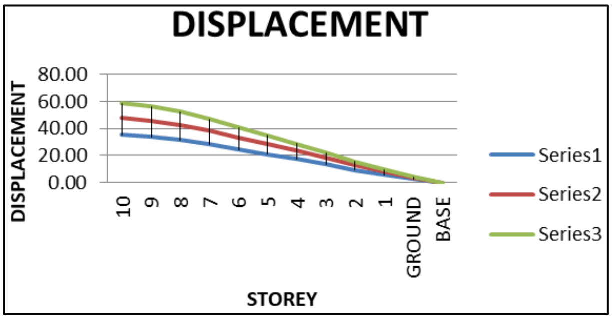


Table & Graph 6.4: Showing Storey Displacement for 1.2(DL+LL+EQY) for Zone-V

DISPLACEMENT			
LOAD COMBINATION :1.2(DL+LL+EQY)			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	9.70	19.00	19.00
9	10.70	19.40	19.40
8	11.00	18.90	18.90
7	10.70	17.70	17.70
6	9.80	15.80	15.80
5	8.80	13.90	13.90
4	7.60	11.70	11.70
3	6.10	9.30	9.30
2	4.50	6.70	6.70
1	3.00	4.40	4.40
GROUND	1.50	2.20	2.20
BASE	0.00	0.00	0.00

Table & Graph 6.5: Max Displacement Comparison for 1.2(DL+LL+EQX) for Soil-I

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX DISPLACEMENT	
SOIL-1	
ZONE-II	ZONE-V
10.20	35.30

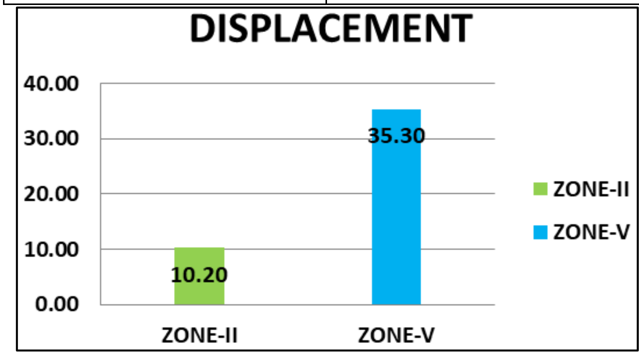


Table & Graph 6.6: Max Displacement Comparison for 1.2(DL+LL+EQX) for Soil-II

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX DISPLACEMENT	
SOIL-2	
ZONE-II	ZONE-V
13.70	47.80

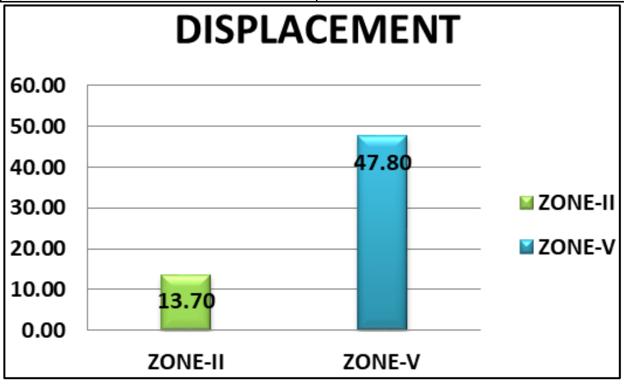


Table & Graph 6.7: Max Displacement Comparison for 1.2(DL+LL+EQX) for Soil-III

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX DISPLACEMENT	
SOIL-3	
ZONE-II	ZONE-V
16.70	58.60

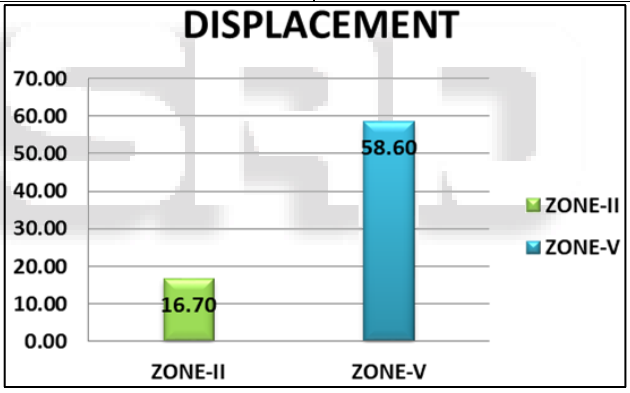


Table & Graph 6.8: Max Displacement Comparison for 1.2(DL+LL+EQY) for Soil-I

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX DISPLACEMENT	
SOIL-1	
ZONE-II	ZONE-V
8.80	11.00

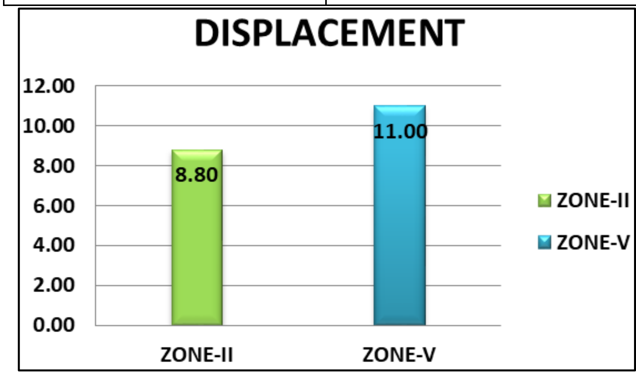


Table & Graph 6.9: Max Displacement Comparison for 1.2(DL+LL+EQY) for Soil-II

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX DISPLACEMENT	
SOIL-2	
ZONE-II	ZONE-V
6.20	19.40

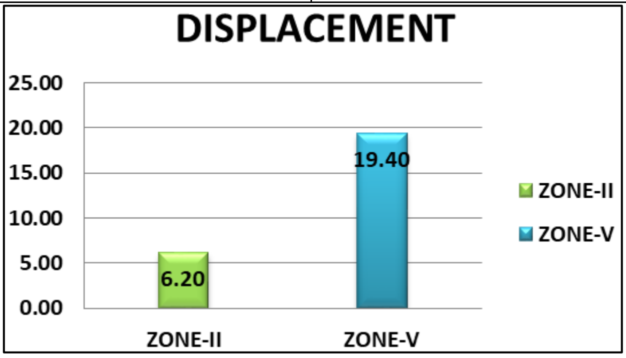


Table & Graph 6.10: Max Displacement Comparison for 1.2(DL+LL+EQY) for Soil-III

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX DISPLACEMENT	
SOIL-3	
ZONE-II	ZONE-V
4.00	19.40

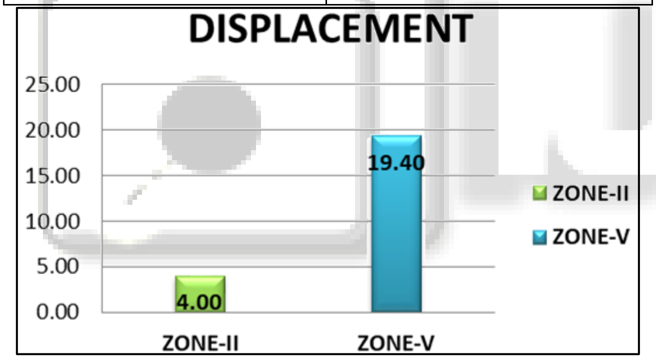


Table & Graph 6.11: Showing Storey Shear force for 1.2(DL+LL+EQX) for Zone-II

SHEARFORCE			
LOAD COMBINATION :1.2(DL+LL+EQX)			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	107.90	105.87	106.22
9	107.90	108.64	109.28
8	67.33	68.91	70.27
7	79.18	81.38	83.29
6	69.04	71.88	74.32
5	85.94	88.56	90.82
4	76.73	79.80	82.45
3	80.55	83.92	86.73
2	71.68	75.12	78.08
1	87.51	91.07	94.14
GROUND	87.50	91.16	94.31
BASE	46.12	49.98	54.17

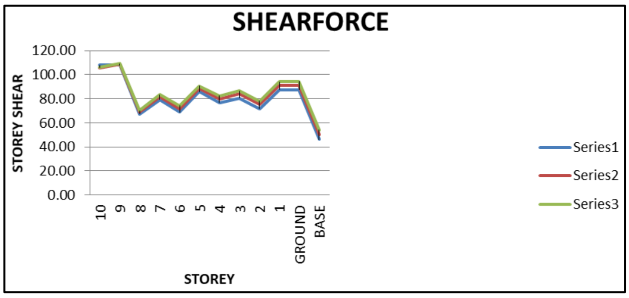


Table & Graph 6.12: Showing Storey Shear force for 1.2(DL+LL+EQY) for Zone-II

SHEARFORCE			
LOAD COMBINATION :1.2(DL+LL+EQY)			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	105.71	105.66	105.62
9	105.71	105.66	105.62
8	62.81	62.76	62.72
7	72.88	72.82	72.77
6	61.02	60.96	60.91
5	78.39	78.29	78.20
4	67.95	67.86	67.79
3	71.31	71.22	71.14
2	61.90	61.82	61.75
1	55.32	55.51	55.18
GROUND	45.62	45.32	45.69
BASE	106.20	106.30	106.20

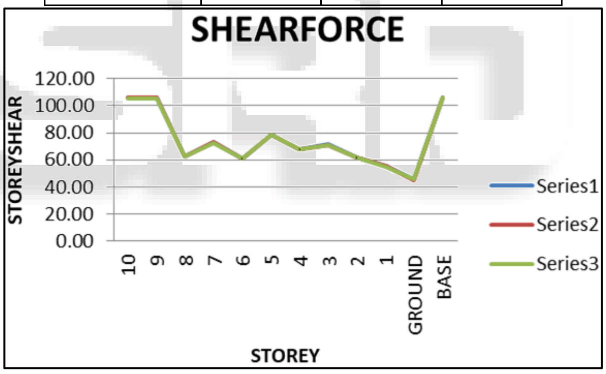


Table & Graph 6.13: Showing Storey Shearforce for 1.2(DL+LL+EQX) for Zone-V

SHEARFORCE			
LOAD COMBINATION :1.2(DL+LL+EQY)			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	105.37	105.20	105.20
9	105.37	105.20	105.20
8	62.46	62.29	62.29
7	72.44	72.22	72.22
6	60.61	60.41	60.41
5	77.68	77.33	77.33
4	67.33	67.03	67.03
3	70.65	70.32	70.32
2	61.33	61.05	61.05
1	46.74	46.43	46.43
GROUND	36.47	36.15	36.15
BASE	106.00	106.10	106.20

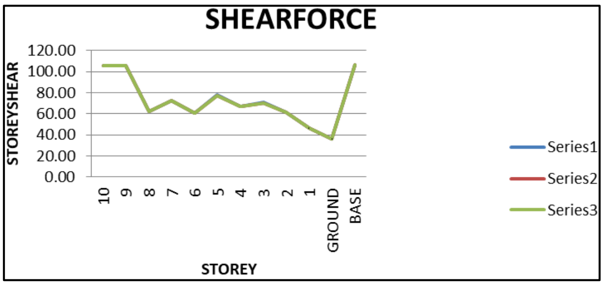


Table & Graph 6.14: Showing Storey Shear force for 1.2(DL+LL+EQY) for Zone-V

SHEARFORCE			
LOAD COMBINATION :1.2(DL+LL+EQY)			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	105.37	105.20	105.20
9	105.37	105.20	105.20
8	62.46	62.29	62.29
7	72.44	72.22	72.22
6	60.61	60.41	60.41
5	77.68	77.33	77.33
4	67.33	67.03	67.03
3	70.65	70.32	70.32
2	61.33	61.05	61.05
1	76.74	76.43	76.43
GROUND	76.47	76.15	76.15
BASE	31.34	31.25	31.25

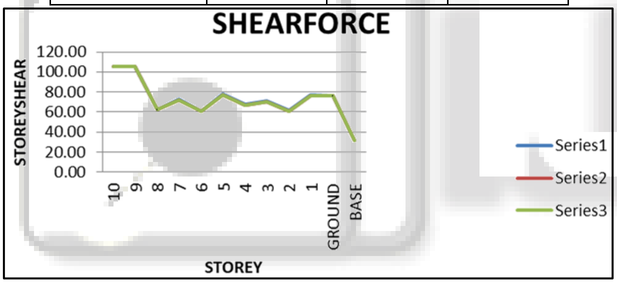


Table & Graph 6.15: Max Shearforce Comparison for 1.2(DL+LL+EQX) for Soil-I

EQUIVALENT STATIC METHODLOAD COMBINATION:1.2(DL+LL+EQX) MAX SHEAR	
SOIL-1	
ZONE-II	ZONE-V
107.90	113.27

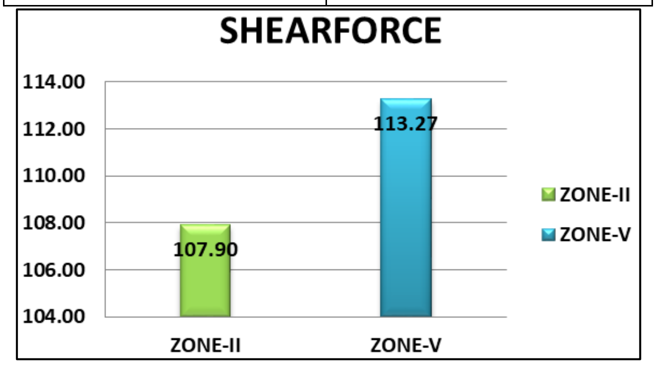


Table & Graph 6.16: Max Shear force Comparison for 1.2(DL+LL+EQX) for Soil-II

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX SHEAR	
SOIL-2	
ZONE-II	ZONE-V
108.64	115.92

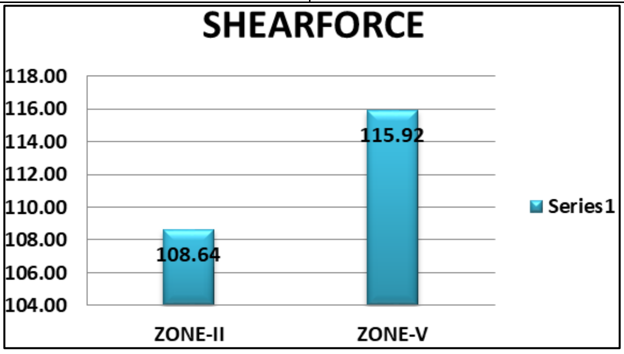


Table & Graph 6.17: Max Shear force Comparison for 1.2(DL+LL+EQX) for Soil-III

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX SHEAR	
SOIL-3	
ZONE-II	ZONE-V
109.28	138.54

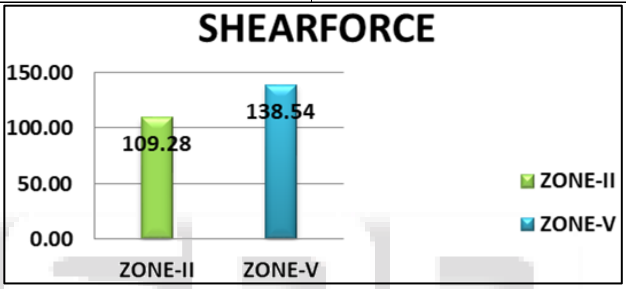


Table & Graph 6.18: Max Shear force Comparison for 1.2(DL+LL+EQY) for Soil-I

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX SHEAR	
SOIL-1	
ZONE-II	ZONE-V
105.71	105.37

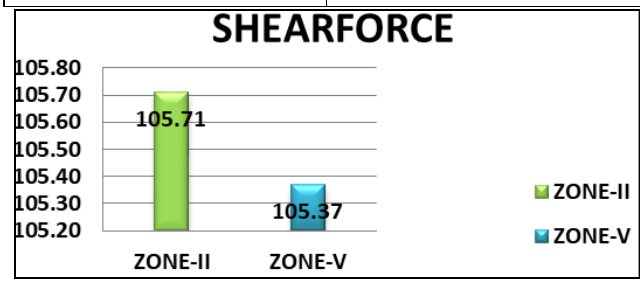


Table & Graph 6.19: Max Shear force Comparison for 1.2(DL+LL+EQY) for Soil-II

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX SHEAR	
SOIL-2	
ZONE-II	ZONE-V
105.66	105.20

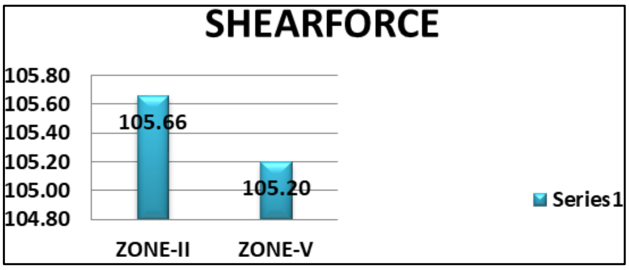


Table & Graph 6.20: Max Shearforce Comparison for 1.2(DL+LL+EQY) for Soil-III

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX SHEAR	
SOIL-3	
ZONE-II	ZONE-V
105.62	105.20

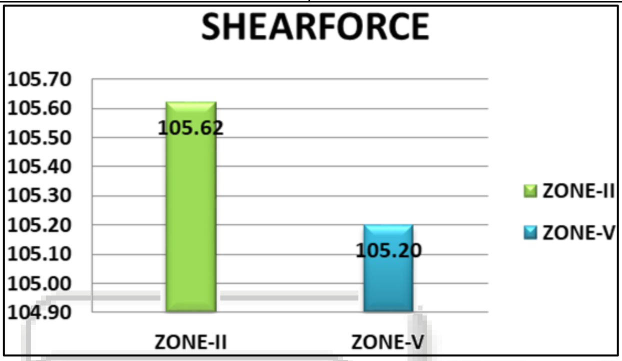


Table & Graph 6.21: Showing Storey Bending moment for 1.2(DL+LL+EQX) for Zone-II

BENDING MOMENT			
LOAD COMBINATION :1.2(DL+LL+EQX)			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	164.13	164.32	164.48
9	132.04	132.44	132.79
8	138.41	139.04	139.59
7	139.43	140.27	140.99
6	130.42	131.37	132.18
5	125.48	126.48	127.34
4	126.29	127.36	128.29
3	126.57	127.70	128.67
2	122.73	123.86	124.84
1	119.80	120.86	121.82
GROUND	111.23	111.22	111.25
BASE	0.00	0.00	0.00

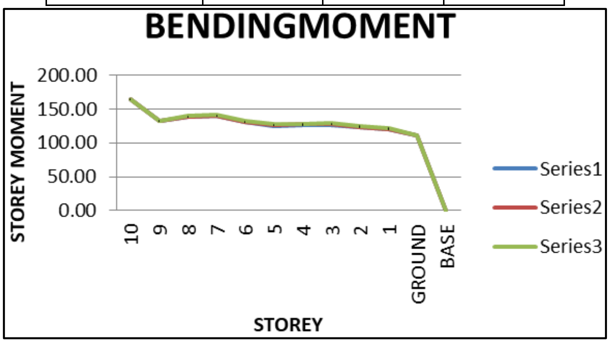


Table & Graph 6.22: Showing Storey Bending moment for 1.2(DL+LL+EQY) For Zone-II

BENDING MOMENT

LOAD COMBINATION :1.2(DL+LL+EQY)			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	163.35	163.26	163.18
9	130.62	130.51	130.41
8	136.29	136.16	136.05
7	136.72	136.58	136.46
6	127.39	127.24	127.12
5	122.28	122.12	121.99
4	122.87	122.71	122.58
3	123.02	122.38	122.75
2	119.20	119.07	118.95
1	116.13	116.31	116.21
GROUND	119.30	119.20	119.11
BASE	0.00	0.00	0.00

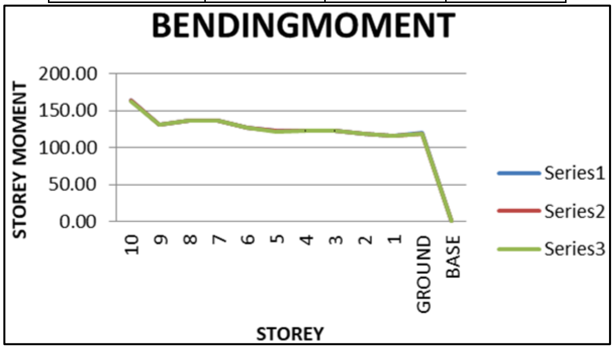


Table & Graph 6.23: Showing Storey Bending moment for 1.2(DL+LL+EQX)
For Zone-V

BENDING MOMENT			
LOAD COMBINATION :1.2(DL+LL+EQX)			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	164.49	166.17	166.75
9	134.94	136.38	137.62
8	143.00	145.29	147.26
7	145.45	148.47	151.06
6	137.23	140.62	143.54
5	132.69	136.29	139.38
4	134.07	137.98	141.80
3	134.73	138.80	142.31
2	130.93	135.02	138.54
1	127.68	131.61	135.00
GROUND	128.98	132.36	135.28
BASE	0.00	0.00	0.00

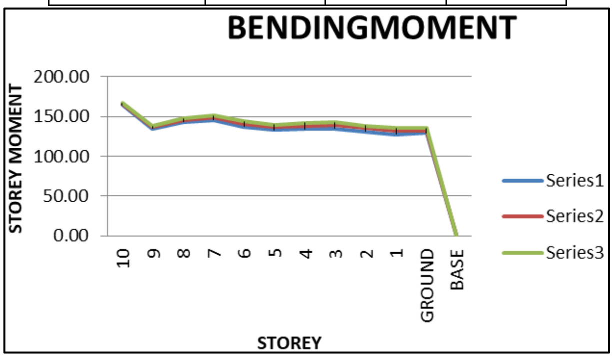


Table & Graph 6.24: Showing Storey Bending moment for 1.2(DL+LL+EQY)

For Zone-V

BENDING MOMENT			
LOAD COMBINATION :1.2(DL+LL+EQY)			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	162.69	162.36	162.36
9	129.81	129.41	129.41
8	135.36	134.90	134.90
7	135.71	135.21	135.21
6	126.32	125.78	125.78
5	121.17	120.62	120.62
4	121.37	121.22	121.22
3	121.96	121.43	121.43
2	118.23	117.75	117.75
1	115.57	115.14	115.14
GROUND	118.59	118.24	118.24
BASE	0.00	0.00	0.00

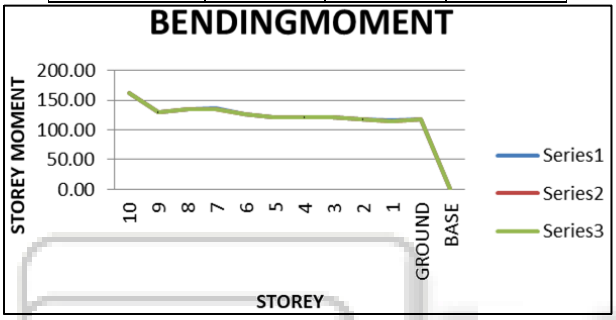


Table & Graph 6.25: Max Bending moment Comparison for 1.2(DL+LL+EQX) for Soil-V

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX BENDING MOMENT	
SOIL-1	
ZONE-II	ZONE-V
164.13	165.49

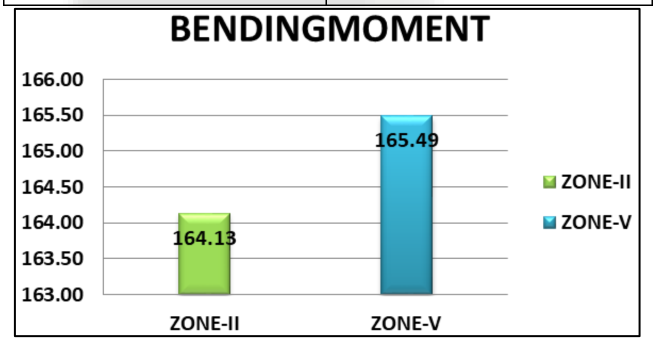


Table & Graph 6.26: Max Bending moment Comparison for 1.2(DL+LL+EQX) for Soil-II

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX BENDING MOMENT	
SOIL-2	
ZONE-II	ZONE-V
164.32	166.17

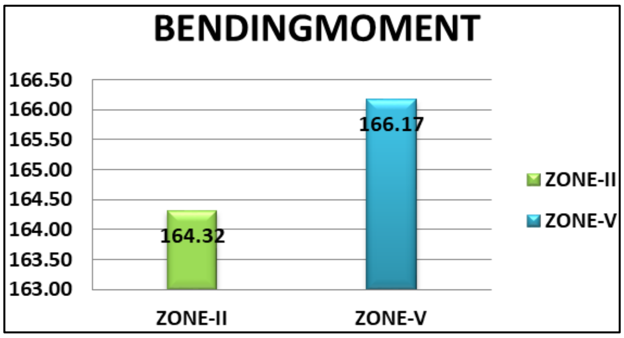


Table & Graph 6.27: Max Bending moment Comparison for 1.2(DL+LL+EQX) for Soil-III

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQX) MAX BENDING MOMENT	
SOIL-3	
ZONE-II	ZONE-V
164.48	166.75

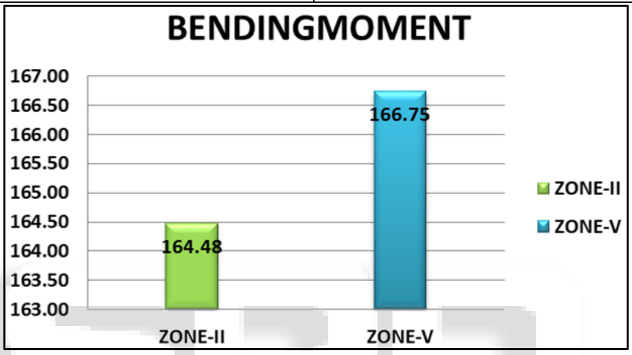


Table & Graph 6.28: Max Bending moment Comparison for 1.2(DL+LL+EQY) for Soil-I

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX BENDING MOMENT	
SOIL-1	
ZONE-II	ZONE-V
163.35	162.69

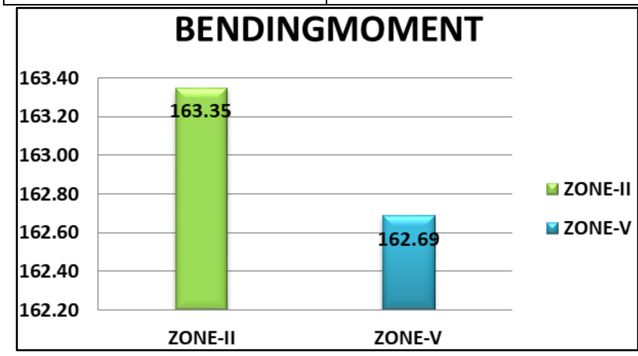


Table & Graph 6.29: Max Bending moment Comparison for 1.2(DL+LL+EQY) for Soil-II

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX BENDING MOMENT	
SOIL-2	
ZONE-II	ZONE-V
163.18	162.36

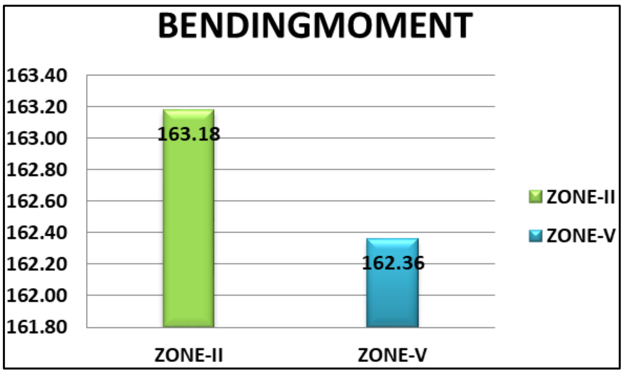
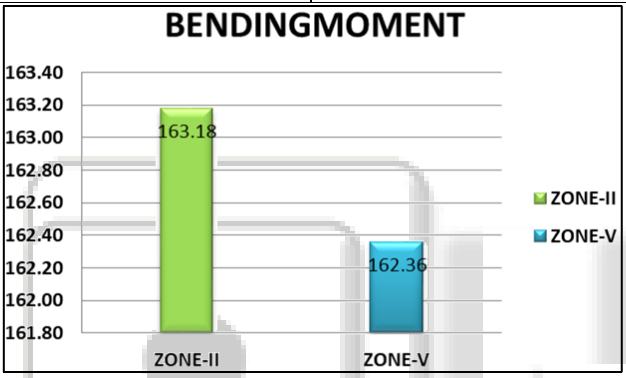


Table & Graph 6.30: Max Bending moment Comparison for 1.2(DL+LL+EQY) for Soil-III

EQUIVALENT STATIC METHOD LOAD COMBINATION:1.2(DL+LL+EQY) MAX BENDING MOMENT	
SOIL-3	
ZONE-II	ZONE-V
163.18	162.36



B. Response Spectrum Analysis

Table & Graph 6.31: Showing Storey Displacement for Response spectrum for Zone-II

DISPLACEMENT			
LOAD COMBINATION :RESPONSE SPECTRUM			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	51.20	69.70	85.60
9	49.50	67.30	82.60
8	46.60	63.40	77.90
7	42.70	58.00	71.30
6	37.80	51.30	63.10
5	32.90	44.70	54.90
4	27.50	37.40	45.90
3	21.70	29.40	36.20
2	15.50	21.10	25.90
1	9.70	13.20	16.20
GROUND	4.30	5.90	7.20
BASE	0.00	0.00	0.00

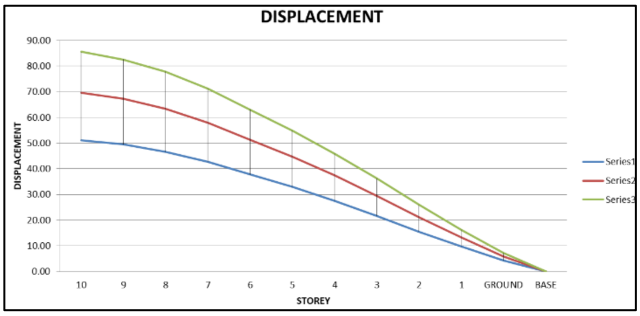


Table & Graph 6.32: Showing Storey Displacement for Response spectrum for Zone-V

DISPLACEMENT			
LOAD COMBINATION :RESPONSE SPECTRUM			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	184.50	250.90	308.10
9	178.10	242.20	297.40
8	167.80	228.30	280.30
7	153.60	208.90	256.60
6	135.90	184.80	227.00
5	118.40	161.10	197.80
4	99.00	134.70	165.40
3	77.90	106.00	130.20
2	55.80	75.80	93.10
1	34.90	47.80	58.30
GROUND	15.50	21.10	25.90
BASE	0.00	0.00	0.00

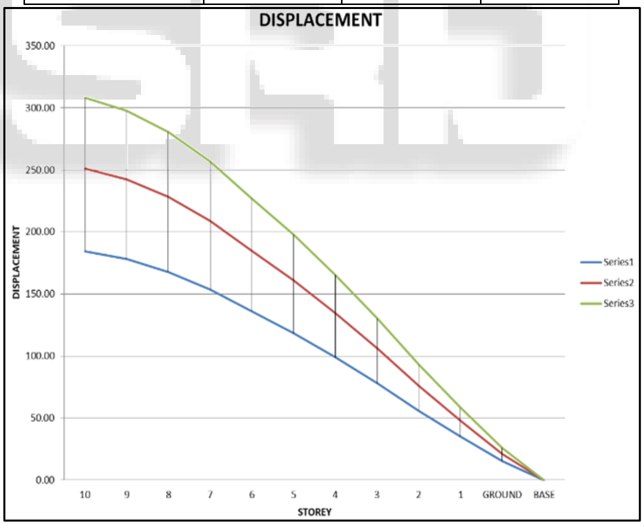


Table & Graph 6.33: Max Displacement Comparison for Response Spectrum for Soil-I

RESPONSE SPECTRUM MAX DISPLACEMENT	
SOIL-1	
ZONE-II	ZONE-V
51.20	184.50

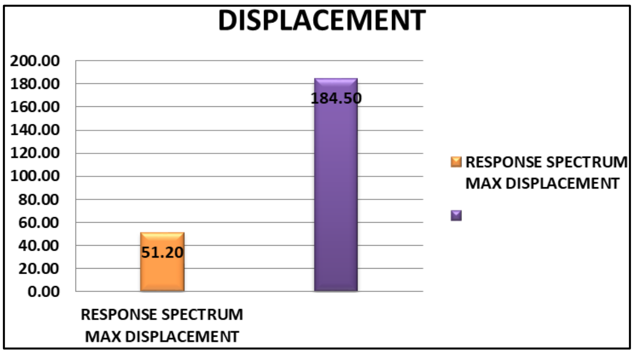


Table & Graph 6.34: Max Displacement Comparison for Response spectrum for Soil-II

RESPONSE SPECTRUM MAX DISPLACEMENT	
SOIL-2	
ZONE-II	ZONE-V
69.70	250.90

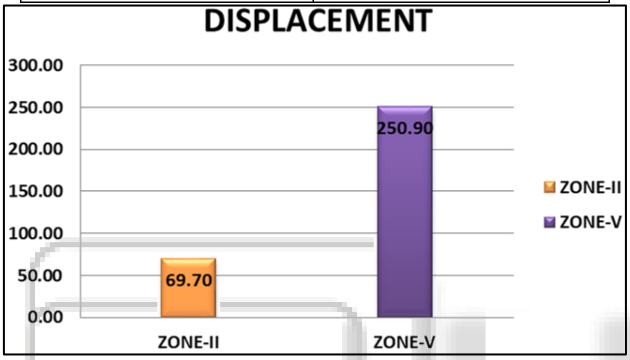


Table & Graph 6.35: Max Displacement Comparison for Response spectrum for Soil-III

RESPONSE SPECTRUM MAX DISPLACEMENT	
SOIL-3	
ZONE-II	ZONE-V
85.60	308.10

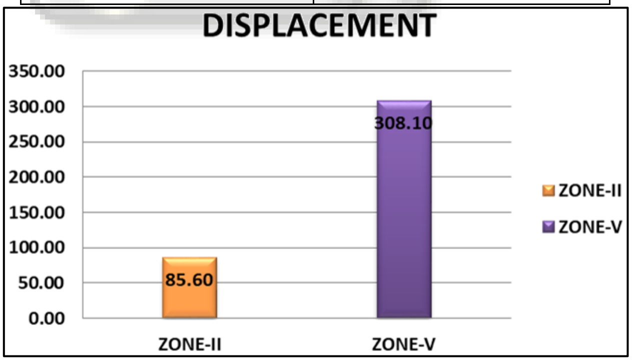


Table & Graph 6.36: Showing Storey Shear force for Response spectrum for Zone-II

SHEARFORCE			
LOAD COMBINATION :RESPONSE SPECTRUM			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	7.39	10.06	12.35
9	7.39	10.06	12.35
8	17.99	24.47	30.05
7	27.11	36.87	45.28
6	37.30	50.73	62.29
5	36.06	49.04	60.22
4	44.78	60.90	74.78
3	49.99	67.98	83.48
2	54.58	74.21	91.16
1	58.57	79.66	97.82
GROUND	61.58	83.75	102.84
BASE	83.71	113.84	139.79

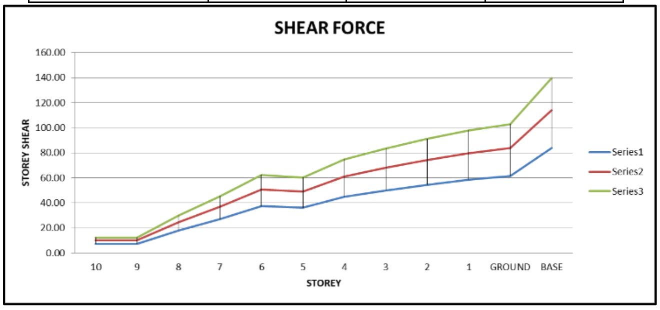


Table & Graph 6.37: Showing Storey Shearforce for Response spectrum for Zone-V

SHEARFORCE			
LOAD COMBINATION :RESPONSE SPECTRUM			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	26.50	20.06	44.48
9	26.63	36.22	44.48
8	64.79	88.12	108.21
7	97.61	132.76	163.02
6	134.29	182.64	224.27
5	129.83	176.57	216.82
4	161.22	219.26	269.24
3	179.96	244.75	300.54
2	196.52	267.27	328.19
1	210.88	286.80	352.18
GROUND	221.69	301.50	370.22
BASE	301.35	409.84	503.27

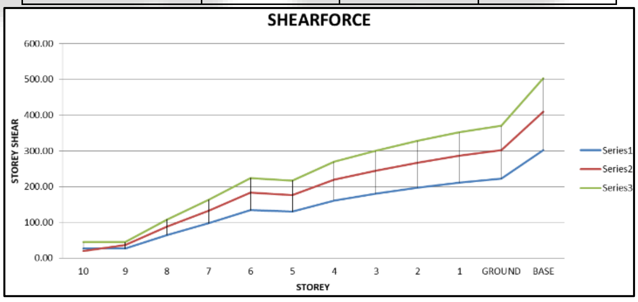


Table & Graph 6.38: Max Shear force Comparison for Response spectrum for Soil-I

RESPONSE SPECTRUM MAX SHEAR	
SOIL-1	
ZONE-II	ZONE-V
83.71	301.35

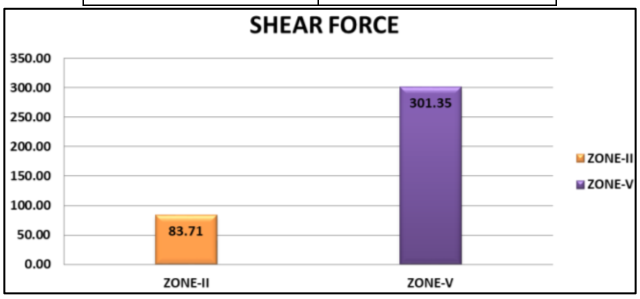


Table & Graph 6.39: Max Shear force Comparison for Response spectrum for Soil-II

RESPONSE SPECTRUM MAX DISPLACEMENT	
SOIL-2	
ZONE-II	ZONE-V
69.70	250.90

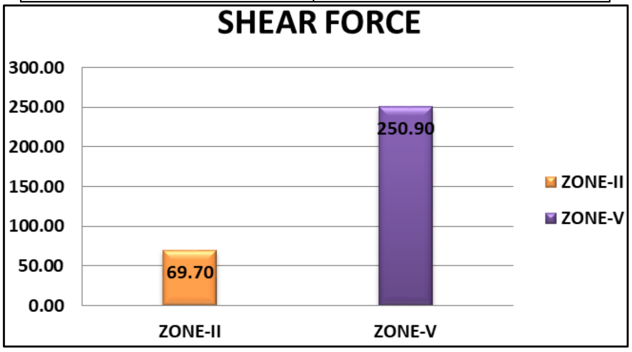


Table & Graph 6.40: Max Shear force Comparison for Response spectrum for Soil-III

RESPONSE SPECTRUM MAX SHEAR FORCE	
SOIL-3	
ZONE-II	ZONE-V
85.60	308.10

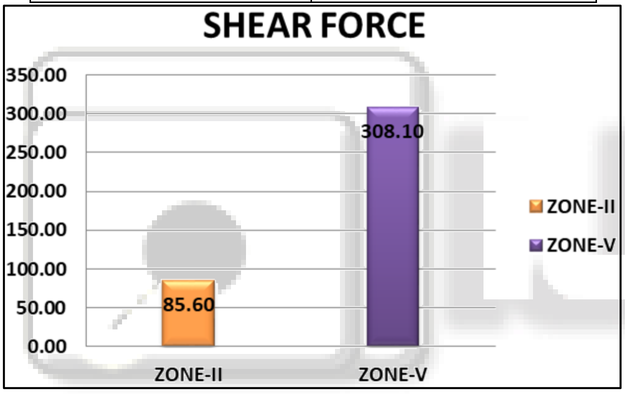
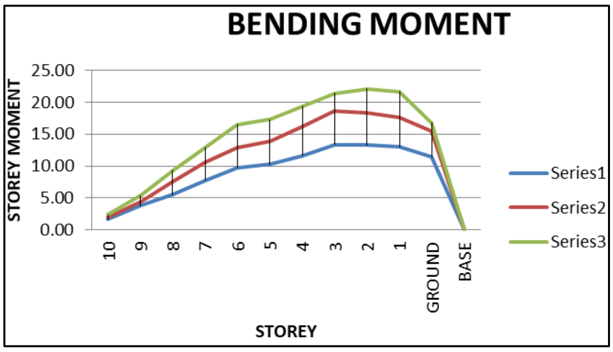


Table & Graph 6.41: Showing Storey Bending moment for Response spectrum For Zone-II

BENDING MOMENT			
LOAD COMBINATION :RESPONSE SPECTRUM			
SNO	ZONE-II		
	SOIL-1	SOIL-2	SOIL-3
10	1.65	1.90	2.36
9	3.76	4.47	5.37
8	5.51	7.49	9.30
7	7.74	10.53	12.93
6	9.76	12.88	16.52
5	10.25	13.94	17.30
4	11.60	16.13	19.37
3	13.35	18.62	21.39
2	13.30	18.41	22.11
1	12.98	17.65	21.67
GROUND	11.37	15.46	16.78
BASE	0.00	0.00	0.00



TABLE&GRAPH 6.42: Showing Storey Bending moment for Response spectrum For Zone-V

BENDING MOMENT			
LOAD COMBINATION :RESPONSE SPECTRUM			
SNO	ZONE-V		
	SOIL-1	SOIL-2	SOIL-3
10	5.25	6.94	9.23
9	11.57	15.74	19.97
8	20.32	26.99	33.15
7	27.88	37.92	46.57
6	33.35	45.35	55.69
5	37.78	54.78	61.64
4	42.23	61.97	69.74
3	45.75	67.71	78.66
2	47.66	65.48	79.66
1	47.25	74.47	78.03
GROUND	48.02	55.68	68.37
BASE	0.00	0.00	0.00

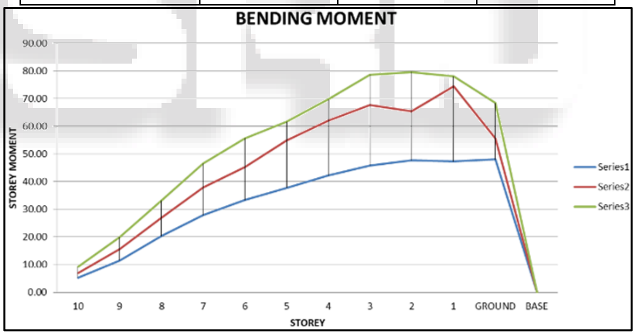


Table & Graph 6.43: Max Bending moment Comparison for Response spectrum For Soil-I

RESPONSE SPECTRUM MAX BENDINGMOMENT	
SOIL-1	
ZONE-II	ZONE-V
13.35	48.02

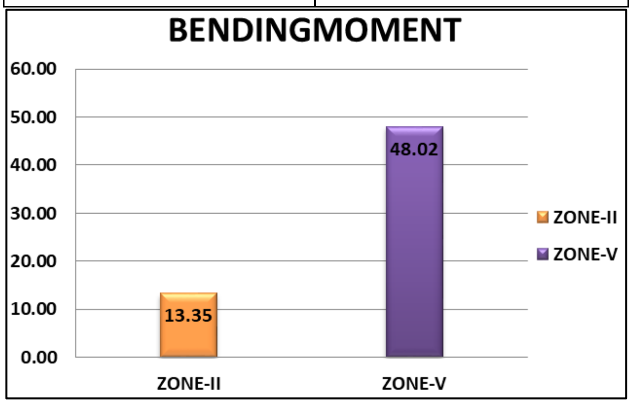


Table & Graph 6.44: Max Bending moment Comparison for Response spectrum For Soil-II

RESPONSE SPECTRUM MAX BENDINGMOMENT	
SOIL-2	
ZONE-II	ZONE-V
18.62	74.47

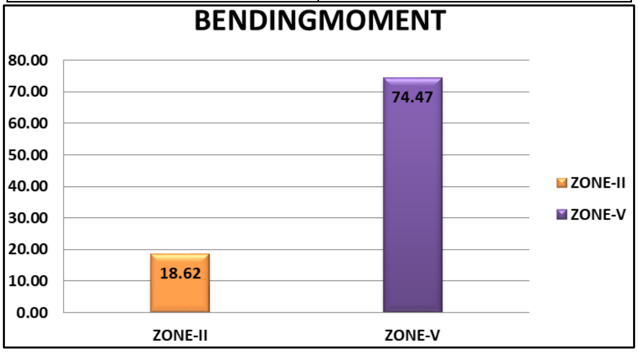
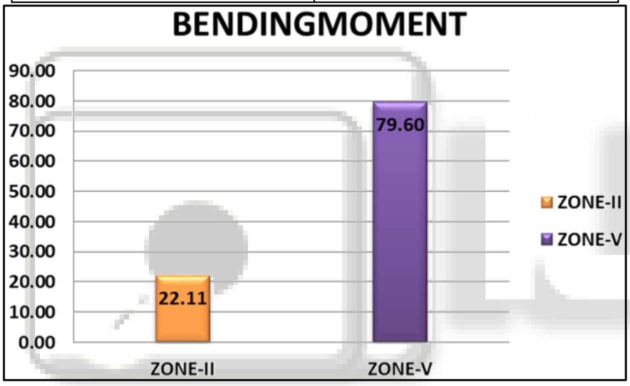


Table & Graph 6.45: Max Bending moment Comparison for Response spectrum For Soil-III

ESPONSE SPECTRUM MAX BENDINGMOMENT	
SOIL-3	
ZONE-II	ZONE-V
22.11	79.60



VII. CONCLUSIONS

In the present study, G+10 RESIDENTIAL building has been drawn in Auto CAD software and designed (Beams, Columns, Footings and Seismic load analysis by using Equivalent Static method & Response spectrum Method) using ETABS software. The dead load, live load and earthquake loads are calculated using IS: 456-2000 and IS 1893: 2002. Concrete grade M25 and HYSD bars Fe415 as per IS: 1786-1985 are used.

Originally, the building was designed without earthquake loads as per IS456:2000. Then building is designed considering the earthquake loads as per IS1893: 2002. The detailing has been done as per both approaches. Indian Standard codes have been used in the analysis and design.

Dynamic story shear is less than static story shear for all cases. From all cases, it is concluded that lateral force obtained from response spectrum method is higher than those obtained by equivalent static lateral force method for story one up to five and the rest higher stories have less values. The maximum story displacement, overturning moment obtained from response spectrum method is lesser than those obtained by equivalent static lateral force method.

For the case (Zone-V & OMRF), the static displacement of the story exceeds the limitation of the maximum story drift ratio of 0.004. The followings are the stories which exceed the maximum drift limitation: in the XDirection:- Story 2-7 and in the Y-Direction:- Story 3-5 Hence zone-V is extremely exposed to earthquake rather than increasing the dimension of columns, it's preferable to use SMRF frame type for the same dimension of columns.

Story stiffness is varying in X- and Y-directions for both methods. Because lateral stiffness of a story is not a stationary property, but an apparent one that depends on lateral load distribution. The base shear, lateral force, story shear, maximum story displacement and overturning moment are increased in both directions (i.e., X & Y) as the seismic zone goes from II to V for the same frame type building in both methods. The use of SMRF frame type in zones V, reduced the values of base shear, lateral force, story shear, Overturning Moment and maximum story displacement which obtained by using OMRF frame type by 40.0% for both methods. If a building is designed as an SMRF frame, it needs to be designed only for lesser forces than if it is designed as an OMRF frame. Equivalent static lateral force method gives higher values of forces and moments which makes building uneconomical hence consideration of response spectrum method is also needed.

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