A

MAJOR PROJECT REPORT

ON

EFFECT OF CONFIGURATION OF R.C.C. BUILDING SUBJECTED TO SEISMIC LOADING

SUBMITTED IN THE PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF DEGREE OF

MASTER OF ENGINEERING

(STRUCTURAL ENGINEERING)

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DELHI UNIVERSITY

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CERTIFICATE

It is certified that the work presented in this thesis entitled **"EFFECT OF CONFIGURATION OF R.C.C. BUILDING SUBJECTED TO SEISMIC LOADING"** by me, University Roll No. 9071 in partial fulfillment of the requirement for the award of the degree of Master of Engineering in Structural Engineering, Delhi College of Engineering (Now known as Delhi Technological University), Delhi, is an authentic record. The work is being carried out by under the guidance and supervision of Dr. A.K. Gupta in the academic year 2010-2011.

This is to hereby certify that this work has not been submitted by me, for the award of any other degree in any other institute.

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This is to certify that the above statement made by ABHISHEK KUMAR GUPTA bearing roll no. 9071 is correct to the best of my knowledge.

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It is a matter of great pleasure for me to present my major project report on **“EFFECT OF CONFIGURATION OF R.C.C. BUILDING SUBJECTED TO SEISMIC LOADING”**. First and foremost, I am profoundly grateful to my project guide **Dr. A. K. Gupta**, Professor & Head of Civil and Environment Engineering Department for their expert guidance and continuous encouragement during all stages of thesis. Their help in the form of valuable information and research thoughts at proper time has brought life in this thesis. I feel lucky to get an opportunity to work with him. I am thankful to the kindness and generosity shown by him towards me, as it helped me morally to complete the project.

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ABSTRACT

Multistoried buildings with open (soft) ground floor are inherently vulnerable to collapse due to earthquake load, their construction is still widespread in the developing nations due to social and functional need for provide car parking space at ground level. Engineering community warned against such buildings from time to time. In the present thesis, an analysis has been performed to study the lateral forces and base shear of a multistoried (G+3) building for five different cases. 1st case is when no infill wall is provided, 2nd case is when stiffness of each floor is same, 3rd case is when considering the stiffness of infill from top to bottom, 4th case is when the ground floor is used for parking and the above stories are provided with infill walls and the last 5th case is when Infill wall is provided at alternate floor levels. The structural action of masonry infill panels of has been taken into account by modeling them as diagonal struts. Building is subjected to earthquake load in accordance with equivalent static force method as well as response spectrum method as per IS 1893(Part 1) : 2002.To perform analysis by equivalent static force method as well as by response spectrum method computer programs are written in matlab. Equivalent static force method produces same magnitude of earthquake force regardless of the infill present in the model. However, when the same buildings are subjected to response spectrum method, significant increase in lateral forces as well as total base shear has been observed in presence of infill. It has been found that when infill wall is incorporated in lumped mass model, it shows different mode shapes indicating that dynamic behavior of buildings changes when infill is incorporated in the model. More than two fold increase in base shear has been observed when infill is present on upper floors with ground floor open when compared to the base shear given by equivalent static force method. Study of the sway characteristics also reveals significantly high demand for ductility for columns at ground floor level. There is substantial change in the lateral forces when any type of unsymmetrical like soft or weak storey, Infill wall etc. is incorporated in the building. Most severe case is observed when the Ground floor is devoid of infill walls.

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INTRODUCTION

1.1 GENRAL

Although there are so many studies about earthquakes but however it has not been possible to predict when and where earthquake will happen. It has been learned how to pinpoint the locations of earthquakes, how to accurately measure their sizes, and how to build flexible structures that can withstand the strong shaking produced by earthquakes and protect our loved ones.

Occurrence of recent earthquakes in India and in different parts of the world resulted losses, especially human lives. It has highlighted the structural inadequacy of buildings to carry seismic loads. There is an urgent need for assessment of existing buildings in terms of seismic resistance. In view of this various organizations in the earthquake threatened countries have come up with documents, which serve as guidelines for the assessment of the strength expected performance and safety of existing buildings so that they will carrying out the necessary rehabilitation, if required.

The Code of Practice on Earthquake Resistant Design of Buildings and Structures is in existence since 1962, it is being followed only by few government organizations, as a result non compliant buildings are being constructed in the country especially in private sector. Only recently, the codal provisions on Earthquake Resistant Design are made mandatory in few States and its implementation is yet to take full momentum. As a result, existing earthquake unsafe buildings are still glowing to an alarming proportion.

Like other earthquakes in the past, the recent earthquakes of KilIari 1993, Bhuj 2001, Kashmir 2005 and Haiti 2010 have exposed the seismic vulnerability of construction practices being followed in the country. It has clearly demonstrated that not only non-engineered rural houses are vulnerable to earthquakes but also engineered multistoried buildings in big cities are also mostly vulnerable due to faulty design and construction. Considering the large number of people, high fatality in RC buildings and volume of economic activities, the social risk involved in cities is also very high so the seismic retrofitting of the existing buildings has to be undertaken to make these unsafe houses safe to resist future earthquakes, thereby reducing the number of casualties significantly. The problem of seismic retrofitting of large stock of unsafe buildings is so big that any government action is just not feasible and therefore individual house owner/builder has to undertake the retrofitting measures. However, government can take up retrofitting of its own buildings and some public utility buildings which are of post earthquake importance.

The deficiencies in buildings and structures against earthquake may arise at (i) Planning stage with faulty configuration and irregularities, (ii) design stage due to inadequate strength and ductility, and (iii) construction stage due to faulty construction practices. Revision of design codes is a continuing process all over world and usually results in up-gradation of seismic hazard and increase in design forces. In India also several regions have been upgraded in terms of seismic ones thereby rendering buildings unsafe according to new code. All these factors make the retrofitting of existing structures necessary. The retrofitting may also be required if change in usage of a building takes place or there is a major alteration/extension of building. The level of retrofitting of a building depends on the seismic zone in which building is situated and the level of performance desired from the building. Important buildings are desired to have a higher performance level during future earthquakes. The seismic zone governs the design earthquake forces and the performance level governs the permissible damage or the permissible values of members' actions due to earthquake forces. Not only member forces and strength are important, the nonlinear deformations and ductile capacity of members are also important for seismic safety of building and need to be evaluated and examined. Much literature on retrofitting of building is already available including the Bureau of Indian Standards (BIS). The techniques have been presented for the type of construction prevailing in India. Emphasis has been on detailing the techniques with illustrations, so that these may be easily understood and applied by common engineers, architects and builders. A need has been felt to provide adequate information about seismic retrofit design of masonry and RC buildings which can be easily understood and implemented.

The Guidelines deal with important aspects of seismic hazard estimation, systematic inspection of existing buildings, tests for estimation of in-situ strength and extent of damage and, deterioration in masonry and RC buildings, mathematical modeling of frames, frame-tubes, shear walls and frames with infill, and various methods of analysis for earthquake forces for seismic evaluation which requires knowledge of structural behaviour, materials of construction, principles of seismic intervention and behaviour of modified structure, and various retrofitting materials. This includes performance levels of various types of buildings. The definition of these performance levels has been taken from Federal Emergency Management Agency (FEMA) and Applied Technology Council (ATC).

Two checklists have been given for systematic inspection of masonry and RC frames. These checklists are useful in preliminary evaluation and identification of major deficiencies in existing buildings. These Guidelines cover retrofitting of non-engineered, engineered and earthquake damaged buildings. These also cover non-engineered rural and semi-urban houses, these buildings are constructed in mud, stone or brick masonry, without any consideration to strength and ductility of the structure. 'The retrofitting techniques for such buildings are based on failure mode identification and behaviour of such buildings in past earthquakes. The techniques have been tested in laboratories and field, and known to provide adequate safety intended for such buildings.

Retrofitting of RC buildings is much more systematic and rational process than that of non-engineered load bearing wall buildings. The different techniques available for retrofitting of RC buildings have been described. The principles of retrofitting of RC buildings are:-

(i) Removal of irregularities and asymmetry,

(ii) Increasing the strength and stiffness of structure,

(iii) Enhancement of deformation capacity (or ductility), and

(iv) Earthquake demand reduction by Base-isolation (or Supplemental Energy Dissipation.)

Different techniques based on these principles have been illustrated. The emphasis on reinforcement detailing, bond of old and new concrete, and anchorage of new reinforcement is highlighted. Outline and principle of advanced techniques (e.g. Base-Isolation and Supplemental Damping) has also been provided. However, a detailed description and mathematical formulation of these advanced techniques are beyond the scope of these guidelines and references have been provided for further reference.

Evaluation and retrofitting of damaged structures is an urgent task after an earthquake, as safe shelter is under pressing demand after a damaging earthquake. This requires some quick evaluation and retrofitting techniques. The techniques for quick evaluation of need and viability of retrofitting, temporary emergency support of the damaged structures, and repair and retrofitting of structures are also covered. Retrofitting and strengthening of existing structures require use of special materials. Bonding of old and new concrete and shrinkage are the main governing factors in selection of material. A description of materials available for this purpose, including a range from ordinary cement-sand grout, concrete to polymers and epoxy, use of Fiber Reinforced Polymers/Plastics (FRP) in strengthening and retrofitting has also been described with the points of caution. Specialized machinery and preparations required for use of different retrofitting materials are also outlined.

1.2 CONFIGURATION OF A MULTISTOREY BUILDING

Configuration plays an important role in the seismic performance of structures subjected to earthquake actions. Post - earthquake reconnaissance has pointed towards the observation that buildings with irregular configurations are more vulnerable than their regular counterparts. There are several reasons for this observed poor structural performance of irregular structures. Concentrations of inelastic demand are likely to occur in zones of geometrical discontinuities and/or mass and stiffness irregularities. If the available ductility is limited, failure is initiated, thus possibly leading to collapse. Unexpected load paths and overstress of components can cause significant adverse effects. To prevent unfavorable failure modes, adequate ‘conceptual design’ is required at an early stage. In addition, thorough assessment of the structural configuration is vital to achieve adequate seismic performance.

Structural configuration has two fundamental aspects: the overall form and the type of lateral resisting system employed. The impact of structural configuration, in plan and elevation, on seismic performance depends upon:

(i) Size: As the absolute size of the structure increases, the range of cost - efficient configurations and systems is reduced. For example, while standardized simple and symmetrical shapes are generally used for high - rise buildings, more options are available for low - to medium–rise structures. The same is also true in bridge engineering where very long spans (> 600 – 800 m) impose the use of suspension cables. Size may also dictate the choice of specific materials of construction. For example, high - rise structures may require high - strength concrete (e.g. Laogan and Elnashai, 1999 ; Aoyama, 2001 , among others).

(ii) Proportion: Earthquake response of a structure depends on its relative proportions rather than absolute size. Low slenderness in plan and elevation is beneficial. Reduced elevation slenderness minimizes overturning effects. For buildings, the ratio of the height (H) to the smallest depth(B) should not exceed 4–5 (Dowrick, 1987). This figure is exceeded by far in modern tall buildings worldwide, which exhibit H / B of 10–15. Multi - storey structures may also employ narrow shapes. In this case, the slenderness ratio is critical. Large aspect ratios in plan render torsional effects more likely to occur. Asynchronous motions at the foundation of building structures may also be caused by high width - to - depth ratios.

(iii) Distribution and concentration: Vertical and plan distribution of stiffness and mass is important to achieve adequate seismic performance. In tall and slender buildings, lateral deformability reduces the earthquake - induced forces. Problems related to deflection control may arise, however, in earthquake and wind response of high - rise structures. Low - rise buildings should be flexible to reduce the shear forces due to ground motions. Tall buildings should be stiff to control the lateral deformations. Seismic motions are multi - dimensional, thus structures need to be able to resist the imposed loads and deformations in any direction. Adequate distributions of structural systems to resist loads (vertical and lateral) can prevent concentrations of inelastic demands. Structural elements can be arranged in orthogonal directions to ensure similar stiffness and resistance characteristics in both main directions, i.e. they should possess bidirectional resistance and stiffness.

(iv) Perimeter resistance*:* Torsional motion tends to stress lateral resisting systems non - uniformly. High earthquake - induced torsional moments can be withstood by lateral resisting components located along the perimeter of the structure as displayed in Figure 1 . Perimeter columns and walls create, for instance, structural configurations with high rigidity and strength (also referred to as ‘ torsional stiffness and resistance ’). The location in plan of systems for earthquake resistance significantly influences the dynamic response. The higher the radius of gyration of the plan layout of the structure, the higher the lever arm to resist overturning moments. In framed systems, the bending stiffness is significantly affected by the layout of columns in plan and elevation. Frames employing perimeter columns possess high bending stiffness and resistance; this is also true for frame - wall systems.

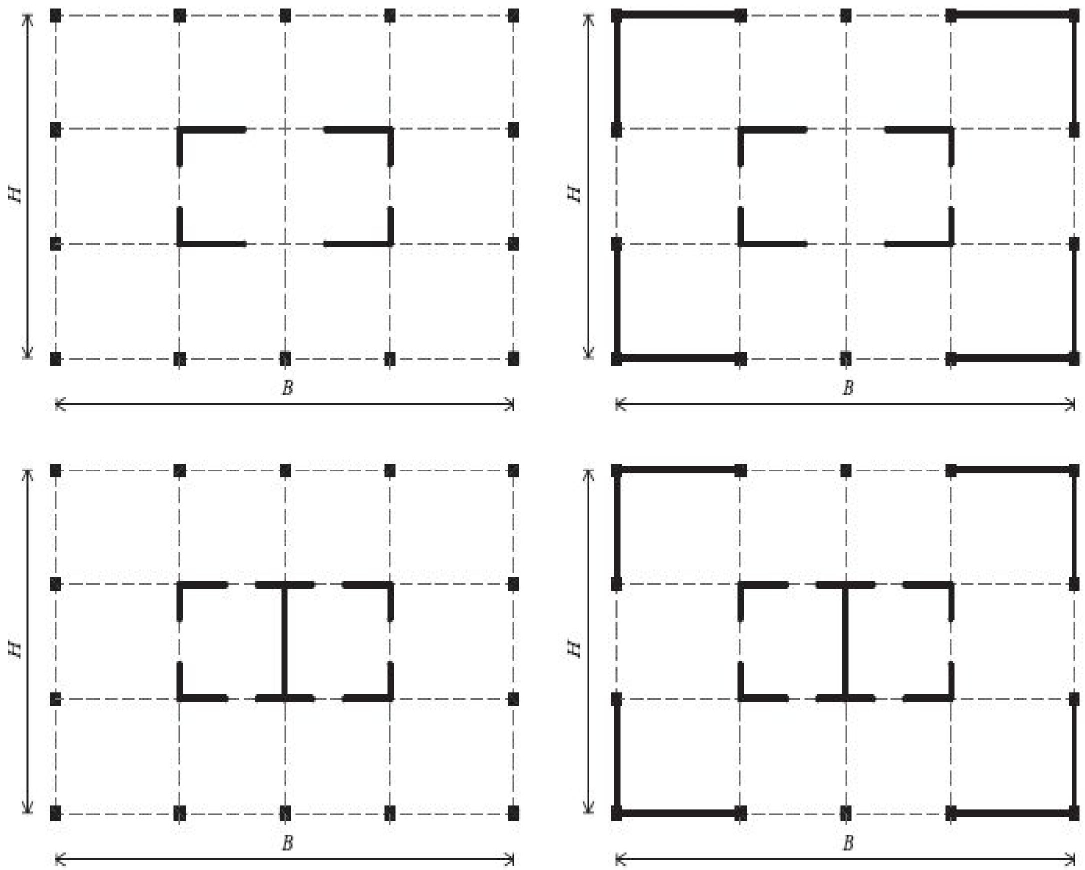

Figure 1: Configurations with different perimeter resistance: low (*left*) and high (*right*) torsional resistance (Amr S. Elnashai et al.,2008).

1.3 ORIGIN OF EARTHQUAKE

Earthquakes are one of the most devastating forces in nature. Earthquakes disasters have been known since ancient times. Earthquakes have been instrumental in changing the course of history. Some of the most significant disasters in the last hundred years have been caused by Earthquakes.

1.3.1 HISTORICAL BACKGROUND

Records of every major earthquakes in China during the last 3000 years. Records of major earthquakes in India up to last 2500 years. Records of major earthquakes over 2000 years in Middle-East. Legends about earthquakes in India and several other ancient civilizations.

1.3.2 EARTHQUAKES SOURCES

Most Earthquakes are concentrated along boundaries of earth's plates. Some Earthquakes also occur away from plate boundaries. Earthquakes in many parts are also associated with volcanic activities. In recent times, earthquakes may have been triggered by human structures and activities (dams, mining etc.)

1.3.3. PLATE TECTONICS

Motion of earth's plates explained using Plate Tectonics According to Plate Tectonics earth's land-mass were earlier joined together. The land-mass have broken up and have drifted apart. Relative motion is still continuing, relative motion at plate boundaries cause earthquakes. Considerable evidence now exist to support Plate Tectonics. Types of evidences are Geological and geomorphological – similar rock formations, Anthropological– similar vegetation and animal life, Geomagnetic–magnetic anomalies support drifting away of land and Mass from Atlantic ridge and other places.

1.3.4 ELASTIC REBOUND THEORY

Elastic rebound theory is used to explain occurrence of earthquakes. Earth's crust is under tremendous strain at the plate boundaries. Relative motion across a fault line will eventually lead to rupture. Fault rupture suddenly releases energy, causing an earthquake

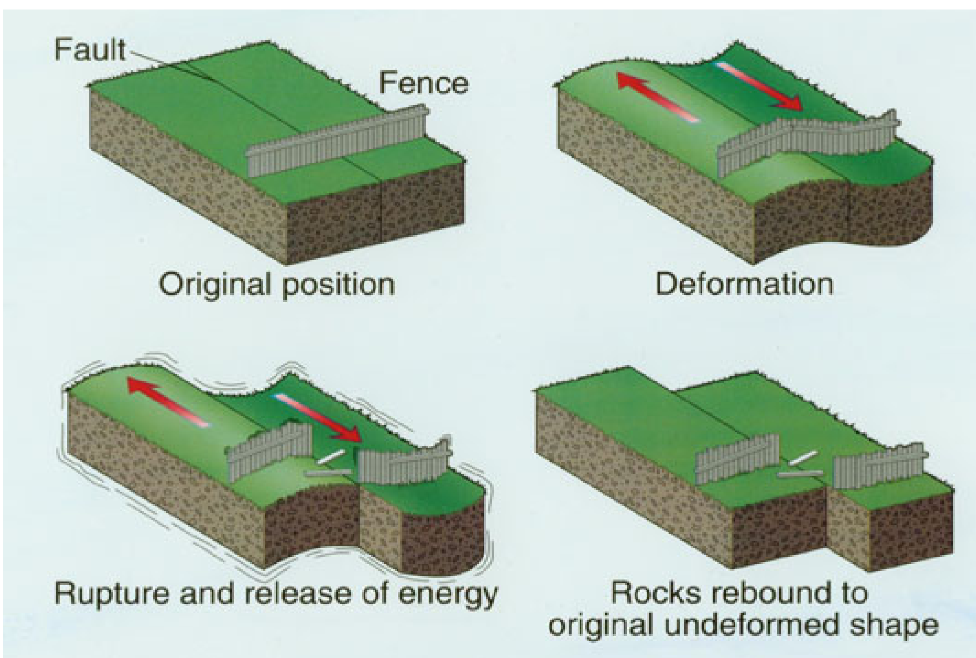

Figure 2: Showing the phenomena of elastic rebound theory (internet sources)

1.3.5 EARTHQUAKE WAVES

Elastic rebound produces waves from the point of rupture. The rupture may be localized at a point, along a slip line or a slip surface. Earthquake waves have clearly identifiable components. They are Primary wave (refractory), Secondary or shear wave (transverse), Raleigh wave (refractory) and Love wave (transverse).Figure of these waves are given in figure 3.

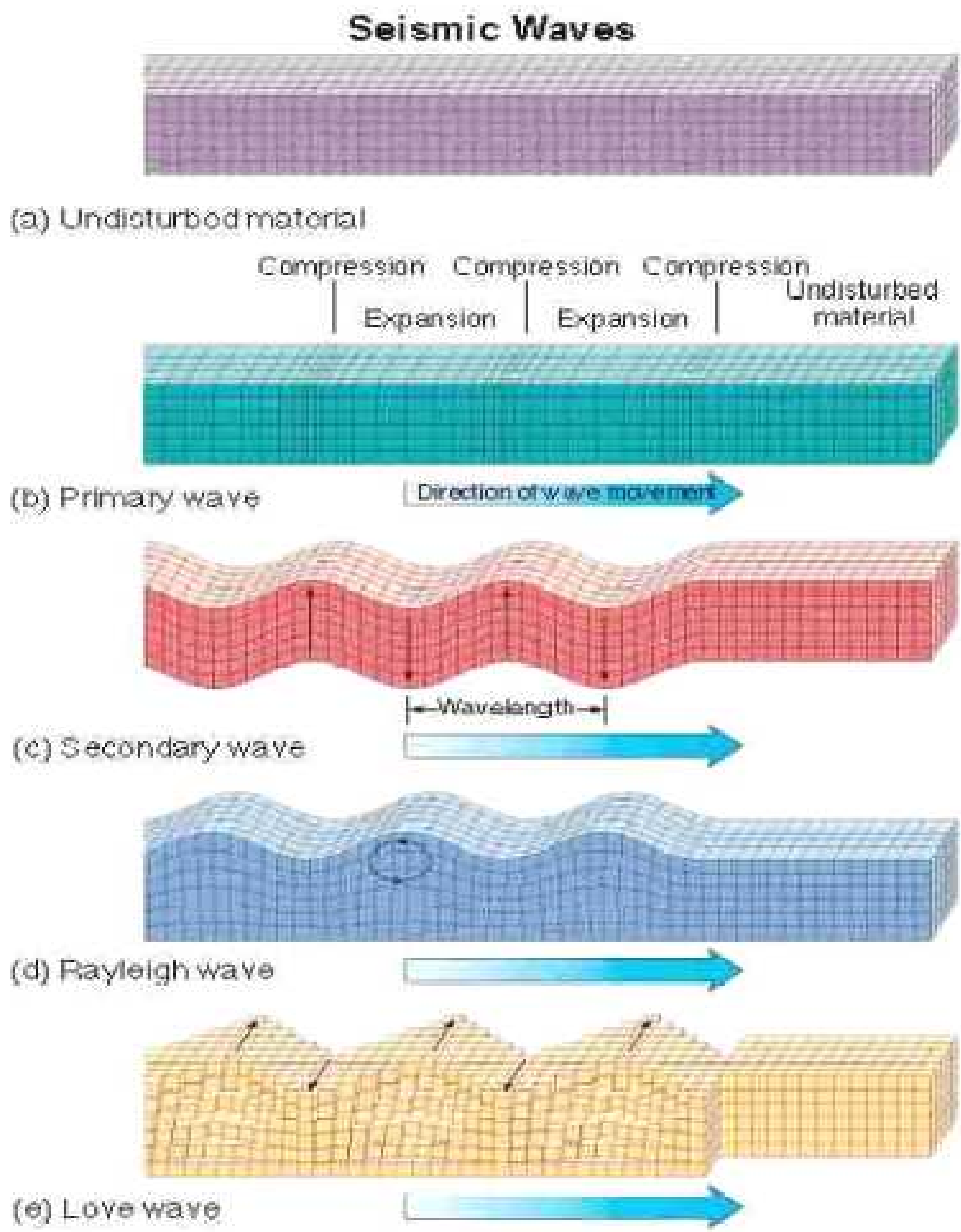

Figure 3: Differnent type of earthquake waves (internet sources)

1.3.6 EARTHQUAKE MAGNITUDE

Earthquake magnitude is most commonly defined in Richter magnitude. It is logarithm of the maximum displacement (in µm) recorded on a particular type of seismograph 100 km from the epicenter. Richter magnitude is open-ended and has no maximum value. Scientifically more useful measure is based on seismic moment and measures the total energy that is released. Both magnitudes give similar value for moderate earthquakes (M5.0–M7.5)

1.3.7 EARTHQUAKE INTENSITY

Earthquake intensity is a measure of its consequence. Most popular intensity scales are primarily based on structure damage.MMI (Define 12 intensities) based only on performance of buildings.MSK (Defines 12 intensities) base on building performance, geotechnical effects as human perception. Most countries use MSK intensity scale or its modifications to suit local conditions.

1.4 AIMS AND OBJECTIVES

In the present study optimum configuration of five different cases of a hypothetical building of different configurations for G+3 storey building have been analyzed as a lumped modal mass system using seismic coefficient method and response spectrum method. Different cases of configuration are as follows:

Case I **:** When no infill wall is provided.

Case II **:** When stiffness of each floor is same.

Case III **:** Considering the stiffness of infill from top to bottom.

Case IV **:** The ground floor is used for parking and the above stories are provided with infill walls.

Case V **:** Infill wall is provided at alternate floor levels.

CHAPTER 2

LITRATURE REVIEW

Important recent works carried out in this area are summarized below.

2.1 Ernesto F Cruz and Silvana Cominetti (2000)

Typically the evaluation of the seismic response of a building in Chile is based on the analysis of the structure excited by unidirectional earthquakes represented by design spectra. In this work the responses of buildings with elastic and with non-linear behavior, subjected to uni - and to bi-directional earthquakes have been studied. It can be concluded that the evaluation of the element design strengths based on an uni-directional analysis of the whole structure is generally adequate. Only in structures with very different transverse stiffness respect to the lateral stiffness a bi-directional analysis would be necessary. The maximum seismic axial forces in columns are underestimated by an uni-directional earthquake. The seismic axial forces are much more sensitive to the type of excitation in buildings with non-linear behavior than in buildings with elastic behavior. The maximum combined (seismic and gravitational force) elastic and inelastic axial forces are less sensitive to the type of excitation. The important inelastic torsional effect detected when the building is subjected to bi - directional earthquakes is related with an increase of the maximum local ductilities of rotation in the columns located in the flexible border, especially of rigid buildings. If the purpose of the analysis is to know the level of damage to which the columns of a structure are exposed, or the elastic or inelastic torsional behavior of building excited by seismic ground movements, this evaluation should be carried out considering the bidirectional earthquake. If the purpose is to define the design strengths, in most cases it would be adequate the use of uni directional earthquake acting on the whole structure in the two principal directions independently. In this work has been observed that structures with low redundancy level and small torsional and transverse stiffness, experience excessive non-linear deformations that represent their collapse. Normally the analysis is carried out considering elastic behavior, and there would be no way to predict a possible collapse during an earthquake.

2.2 Mario De Stefano and Barbara Pintucchi (2002)

A refined model of asymmetric building structure has been presented, which is capable to overcome limitations of widely used simplified models. From the results shown here in for torsionally-stiff systems, the following conclusions can be drawn:

1. Previous models of plan asymmetric building structures, which make no allowance for interaction phenomena, generally overestimate torsional response, as represented by floor rotations.

2. Inelastic interaction phenomena among axial force and bi-directional horizontal forces in vertical resisting elements result in a reduction of floor rotation ranging between 20% and 30% for systems having uncoupled lateral period T greater than 0.2 sec.

3. Axial force due to gravity loads, in the long period range, leads to a further reduction in torsional response, even if such effect is lower than that of interaction phenomena.

4. Variations r appear to be influenced to a larger degree by interaction phenomena than by Design ductility levels, except for short periods.

2.3 M. Piazza et al. (2008)

They concluded in-plane stiffness of the floors strongly affects the structural behavior of an existing masonry building subjected to seismic action. It defines the seismic distribution of forces on lateral walls and the request displacement for verifying the out–of–plane mechanism of the walls. The real size used for the specimens proved to be very important in order to determine the in-plane stiffness of the floor and to adequately simulate the real contribution of the secondary elements (planks and reinforcement elements).

Their tests showed also the efficiency and the contribution of the steel ring tie, mainly in terms of strength rather than of the initial stiffness. The possibility to have many connectors along the floor border guarantees a nearly uniform transmission of shear forces to lateral walls. Its strength contribution is essential in the tension zones of the deck. Finally, the ductility of steel curb ensures a constant strength contribution when cyclic loadings are applied.

2.4 Sharany Haque and Khan Mahmud Amanat (2008)

Earthquake vulnerability of buildings with open ground floors is well known around the world. However, under the present socio economic context of developing nations like Bangladesh, construction of such buildings is unavoidable. In such a situation, an investigation has been performed to study the behavior of such buildings subjected to earthquake load so that some guideline could be developed to minimize the risk involved in such type of buildings. It has been found that code provisions do not provide any guideline in this regard. Present study reveals that such types of buildings should not be treated as ordinary RC framed buildings. It has been found that calculation of earthquake forces by treating them as ordinary frames results in an underestimation of base shear. Calculation shows that, when RC framed buildings having brick masonry infill on upper floor with soft ground floor is subjected to earthquake loading, base shear can be more than twice to that predicted by equivalent earthquake force method with or without infill or even by response spectrum method when no infill in the analysis model. Since response spectrum method is seldom used in practice for the design of such buildings, it can be suggested that the base shear calculated by equivalent static method may at least be doubled for the safer design of the columns of soft ground floor.

2.5 M. Ashraf et al. (2008)

According to their study of the building having 25 stories and different positions of shear walls displaced on one side along the length leads to the following conclusions:

1. Beam moments at column points due to seismic loading are found to increase towards edge grids opposite to the displaced direction of shear walls at lower stories and on the contrary, the moments are found to have lesser values at the same grids of upper stories. It follows that the behavior becomes reversed for the edge grids from the position of shear walls location for lower stories and vice versa.

2. Torsion in beams increases with the enhancement in eccentricity of shear walls. Torsion in beams due to seismic loading has the maximum effect at top stories with the increase in eccentricity. Its maximum effect is closer to the edge grid of the building away from the displacing direction of shear walls and for members joining shear walls.

3. Column axial forces and moments due to seismic loading are found to increase with the enhancement in eccentricity towards the edge grid opposite to the displaced direction of shear walls. On the contrary, the behavior becomes reversed for the edge grid in the displacing direction of shear walls.

4. Torsion in columns also shows an increasing trend with the enhancement in eccentricity. It increases from base to maximum at storey level 2 to 3 and start decreasing towards upper stories.

5. Comparison of forces in shear walls shows that the eccentricity causes major effect on shear walls. It depends on its location in the building. For a given case, it causes maximum effect on pier members in the direction displaced of shear walls.

6. The displacement of building is uni-directional and uniform for all the grids in the case of zero eccentricity for seismic loading. With the increase in the eccentricity, the building shows non-uniform movement of right and left edges due to torsion.

7. Building receives more drifts with the increase in eccentricity.

8. The study indicates the significant effects on axial and shear forces along with bending and twisting moments of beams and columns at different levels of the building by shifting the shear wall location. Placing shear wall away from center of gravity resulted in increase in most of the members forces. It follows that shear walls should be placed in such a fashion that center of gravity of the building should be coinciding with the centroid of the building.

9. It is clear from the study that non-uniform placement of stiff elements cause the structure more harm than good by introducing torsion besides increase in beam and column moments due to their off-center locations.

2.6 Sharany Haque and Khan Mahmud Amanat (2009)

Their Present study reveals that open ground floors types of buildings should not be treated as ordinary RC framed buildings. Study of the sway characteristics of RC framed buildings with open ground floor reveals that the columns of open ground floor demands much higher allowance for drift. Drift demand of these columns are, in general, about 75% higher than that predicted by conventional equivalent static force method. Thus special detailing of reinforcement, based on designing the building as special moment resisting frame, may be adopted to meet that high ductility demand of the ground floor columns. However, they feel that more research in this area is need, It has been found that calculation of earthquake forces by treating the common RC framed buildings with open ground floor as ordinary frames results in an underestimation of design force and moment for ground floor columns. Calculation shows that, when RC framed buildings having brick masonry infill on upper floor with soft ground floor is subjected to earthquake loading, base shear can be more than twice to that predicted by equivalent earthquake force method with or without infill or even by response spectrum method when no infill in the analysis model. Since response spectrum method is seldom used in practice for the design of such buildings, it can be suggested that the design shear and moment calculated by equivalent static method may at least be doubled for the safer design of the columns of soft ground floor.

2.7 D. Güney and A. O. Kurusçu (2011)

The effect of the infill walls on the behavior of the RC frames is quite complicated in terms of boundary conditions material properties and geometry. Despite all of the uncertainties and difficulties involved, including the infill in the model, somehow becomes a compulsory parameter for an accurate estimation of the behavior and vulnerability prediction of RC buildings. As shown in the analysis of structural models, infill walls directly effects displacement response of the structure under the earthquake excitation. Generally, during the design stage, infill walls are not taken into account. However, analysis results show bare frame structures displacement response is much higher than structure with infill walls. Depending on location of infill walls can cause increase in lateral stiffness of structure, then this decrease displacement response of structure. So that it is necessary to take into account these infill walls during structural analysis case. In addition to this result, the configuration of infill walls also can amplify of attenuate displacement of the structure. The optimization of the location of infill walls is not only technical problem but also economical problem. That is why the ratio between total area of structural elements and infill walls is one of the most important parameter. In this study, the best result is taken for 0.68.If the structural model cases increases , it is possible to get better and more realistic results.

2.8 Andrea Lucchini et al.(2011)

Results of nonlinear dynamic analyses carried out on a two-way asymmetric single-story frame structure have been reported. The evolution of the maximum displacement demand in the different resisting elements of the system and of the corresponding global restoring forces has been investigated for earthquakes of increasing intensities characterized by different angles of incidence. The results obtained from the considered case study are found to be consistent with those obtained by the writers in previous investigations on one-way asymmetric-plan structures Lucchini et al. 2009. The main findings of the work can be briefly summarized as follows. First, with increasing response into the nonlinear range, the different global forces acting on the system that produce the maximum demand in the resisting elements tend to converge toward a single distribution. Second, this distribution is related to the resistance distribution only, not to the elastic properties of the system. In particular, it has been found that the nonlinear response is governed by specific points of that surface known in the literature as the BST.

CHAPTER 3

STRUCTURAL BEHAVIOUR DURING GROUND MOTION

3.1 GENRAL

When a structure is subjected to ground motions in an earthquake, it responds by vibrating. The random motion of the ground caused by an earthquake can be resolved in any three mutually perpendicular directions: the two horizontal directions (x and y) and the vertical direction (z). This motion causes the structure to vibrate or shake in all three directions; the predominant direction of shaking is horizontal. All the structures are primarily designed for gravity loads — force equal to mass times gravity in the vertical direction. Because of the inherent factor of safety used in the design specifications, most structures tend to be adequately protected against vertical shaking. Generally, however, the inertia forces generated by the horizontal components of ground motion require greater consideration in seismic design. Earthquake-generated vertical inertia force must be considered in the design unless checked and proved to be insignificant, In general, buildings are not particularly susceptible to vertical ground motion, but its effect should be borne in mind in the design of RCC columns, steel column connections, and prestressed beams. Vertical acceleration should also be considered in structures with large spans, those in which stability is a criterion for design, or for overall stability analysis of structures with large spans. Structures designed only for vertical shaking, in general, may not be able to safely sustain the effect of horizontal shaking. Hence, it is necessary to ensure that the structure is adequately resistant to horizontal earthquake shaking too.

As the ground on which a building rest is displaced, the base of the building moves suddenly with it, but the roof has a tendency to stay in its original position. The tendency to continue to remain in its original position is known as inertia. So the upper part of the structure will not respond instantaneously but will lag because of inertial resistance and flexibility of structure. Since the roofs and foundations are connected with the walls and columns, the roofs are dragged along with the walls/columns. The building is thrown backwards and the roof experiences a force called the inertia force (figure 4). The maximum inertia force acting on a simple structure during an earthquake may be obtained by multiplying the roof mass m by the acceleration a. When designing a building according to the codes, the lateral force is considered in each of the two orthogonal horizontal directions of the structure. For structures having lateral force-resisting elements (e.g. frames, shear walls) in both directions, the design lateral force is considered along one direction at a time, and not in both the directions simultaneously.

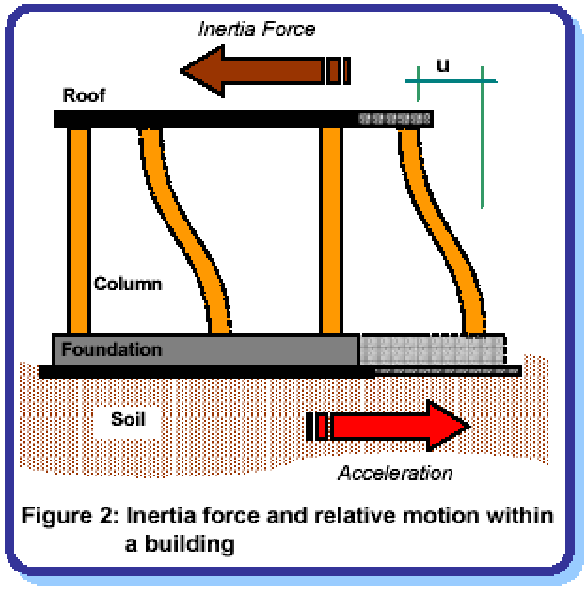

Figure 4: Inertia force and relative motion within a building (internet sources)

3.2 TERMINOLOGY

3.2.1 TERMINOLOGY FOR EARTHQUAKE ENGINEE

For the purpose of standard, the following definitions shall apply which are applicable generally to all structures as per IS 1893 (Part 1) : 2002

3.2.1.1 CLOSELY-SPACED MODES

Closely-spaced modes of a structure are those of its natural modes of vibration whose natural frequencies differ from each other by 10 percent or less of the lower frequency.

3.2.1.2CRITICAL DAMPING

The damping beyond which the free vibration motion will not be oscillatory.

3.2.1.3 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 of critical damping.

3.2.1.4 DESIGN ACCELERATION SPECTRUM

Design acceleration spectrum refers to an average smoothened plot of maximum acceleration as a function of frequency or time period of vibration for a specified damping ratio for earthquake excitations at the base of a single degree of freedom system.

3.2.1.5 DESIGN BASIS EARTHQUAKE (DBE)

It is the earthquake which can reasonably be expected to occur at least once during the design life of the structure.

3.2.1.6 DESIGN HORIZONTAL ACCELERATION COEFFICIENT (A_h)

It is a horizontal acceleration coefficient that shall be used for design of structures.

3.2.1.7 DESIGN LATERAL FORCE

It is the horizontal seismic force prescribed by this standard that shall be used to design a structure.

3.2.1.8 DUCTILITY

Ductility of a structure, or its members, is the capacity to undergo-large inelastic deformations without significant-loss of strength or stiffness.

3.2.1.9 EPICENTRE

The geographical point on the surface of earth vertically above the focus of the earthquake.

3.2.1.10 EFFECTIVE PEAK GROUND ACCELERATION (EPGA)

It is a 0.4 times the 5 percent damped average spectral acceleration between period 0.1 to 0.3 s. This shall be taken as Zero Period Acceleration (ZPA).

3.2.1.11 FLOOR RESPONSE SPECTRA

A floor response spectrum is the response spectra for a time history motion of a floor. This floor motion time history is obtained by an analysis of multi-story building for appropriate material damping values subjected to a specified earthquake motion at the base of structure.

3.2.1.12 FOCUS

The originating earthquake source of the elastic waves inside the earth which cause shaking of ground due to earthquake.

3.2.1.13 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 consequence of its failure, its post-earthquake functional need, historic value, or economic importance.

3.2.1.14 INTENSITY OF EARTHQUAKE

The intensity of an earthquake at a place is a measure of the strength of shaking during the earthquake, and is indicated by a number according to the modified Mercalli Scale of M.S.K. Scale of seismic intensities.

3.2.1.15 LIQUEFACTION

Liquefaction is a state in saturated cohesionless soil where the effective shear strength is reduced to negligible value for all engineering purpose due to pore pressure caused by vibrations during an earthquake when they approach the total confining pressure. In this condition the soil tends to behave like a fluid mass.

3.2.1.16 LITHOLOGICAL FEATURES

The nature of the geological formation of the earths' crust above bed rock on the basis of such characteristics as colour, structure, mineralogical composition and grain size.

3.2.1.17 MAGNITUDE OF EARTHQUAKE (RICHTER'S -MAGNITUDE)

The magnitude of earthquake is a number, which is a measure of energy released in an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer (with a period of 0.8 s, magnification 2800 and damping nearly critical) would register due to the earthquake at an epicenter distance of 100 km.

3.2.1.18 MAXIMUM CONSIDERED EARTHQUAKE (MCE)

The most severe earthquake effects considered by this standard.

3.2.1.19 MODAL MASS (M_K)

Modal mass of a structure subjected to horizontal or vertical, as the case may be, ground motion is a part of the total seismic mass of the structure that is effective in mode k of vibration. The modal mass for a given mode has a unique value irrespective of scaling of the mode shape.

3.2.1.20 MODAL PARTICIPATION FACTOR (P_K)

Modal participation factor of mode k of vibration is the amount by which mode k contribution to the overall vibration of the structure under horizontal and vertical earthquake ground motion. Since the amplitudes of 95 percent mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for mode shapes.

3.2.1.21 MODE SHAPE COEFFICIENT (ϕ_{ik})

When a system is vibrating in normal mode k, at any particular instant of time, the amplitude of mass I expressed as a ratio of the amplitude of one of the masses of the system, is known as mode shape coefficient (ϕ_{ik}).

3.2.1.22 NATURAL PERIOD (T)

Natural period of structure is its time period of undamped free vibration.

3.2.1.22.1 FUNDAMENTAL NATURAL PERIOD (T_l)

It is the first (longest) modal time period of vibration.

3.2.1.22.2 MODAL NATURAL PERIOD (T_k)

The modal natural period of mode k is the time period of vibration in mode k.

3.2.1.23 NORMAL MODE

A system is said to be vibrating in a normal mode when all its masses attain maximum values of displacements and rotations simultaneously, and pass through equilibrium positions simultaneously.

3.2.1.24 RESPONSE REDUCTION FACTOR (R)

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 be reduced to obtain the design lateral force.

3.2.1.25 RESPONSE SPECTRUM

The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

3.2.1.26 SEISMIC MASS

It is the seismic weight divided by acceleration due to gravity.

3.2.1.27 SEISMIC WEIGHT (W)

It is the total dead load plus appropriate amounts of specified imposed load.

3.2.1.28 STRUCTURAL RESPONSE FACTOR (S_a/g)

It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure.

3.2.1.29 TIME HISTORY ANALYSIS

It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion time history.

3.2.1.30 ZONE FACTOR (Z)

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

3.2.1.31 ZERO PERIOD ACCELERATION (ZPA)

It is a value of acceleration response spectrum for period below 0.03 s (frequencies above 33 Hz).

3.2.2 TERMINOLOGY FOR EARTHQUAKE ENGINEERING OF BUILDINGS

For the purpose of earthquake resistant design of building in this standard, the following definitions shall apply.

3.2.2.1 BASE

It is the level at which inertia forces generated in the structure are transferred to the foundation, which then transfers these forces to the ground.

3.2.2.2 BASE DIMENSION (D)

Base dimension of the building along a direction is the dimension at its base, in meter, along that direction.

3.2.2.3 CENTER OF MASS

The point through which the resultant of the masses of a system acts. This point corresponds to the center of gravity of masses of system.

3.2.2.4 CENTER OF STIFFNESS

The point through which the resultant of the restoring forces of system acts.

3.2.2.5 DESIGN ECCENTRICITY (e_{di})

It is the value of eccentricity to be used at floor *i* in torsion calculations for design.

3.2.2.6 DESIGN SEISMIC BASE SHEAR (V_b)

It is the total design lateral force at the base of a structure.

3.2.2.7 DIAPHRAGM

It is a horizontal, or nearly horizontal system, which transmits lateral forces to the vertical resisting elements, for example, reinforced concrete floors and horizontal bracing systems.

3.2.2.8 DUAL SYSTEM

Buildings with dual system consist of shear walls (or braced frames) and moment resisting frames such that:

1. The two systems are designed to resist the total design lateral force in proportion to their lateral stiffness considering the interaction of the dual system at all floor level; and
2. The moment resisting frames are designed to independently resist at least 25 percent of the design base shear.

3.2.2.9 HEIGHT OF FLOOR (h_i)

It is the difference in levels between the base of the building and that of floor *i*.

3.2.2.10 HEIGHT OF STRUCTURE (h)

It is the difference in levels, in meters, between its base and its highest level.

3.2.2.11 HORIZONTAL BRACING SYSTEM

It is a horizontal truss system that serves the same function as a diaphragm.

3.2.2.12 JOINT

It is the portion of the column that is common to other members, for example, beams, framing into it.

3.2.2.13 LATERAL FORCE RESISTING ELEMENT

It is part of the structural system assigned to resist lateral forces.

3.2.2.14 MOMENT-RESISTING FRAME

It is a frame in which members and joints are capable of resisting forces primarily by flexure.

3.2.2.15 ORDINARY MOMENT-RESISTING FRAME

It is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

3.2.2.16 SPECIAL MOMENT-RESISTING FRAME

It is a moment-resisting frame specially detailed to provide ductile behavior and comply with the requirements given in IS 4326 or IS 13920 or SP 6(6).

3.2.2.17 NUMBER OF STOREYS (n)

Number of Storeys of a building is the number of levels above the base. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected.

3.2.2.18 PRINCIPAL AXES

Principal axes of a building are generally two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

3.2.2.19 P-Δ Effect

It is the secondary effect on shears and moments of frame members due to action of the vertical loads, interacting with the lateral displacement of building resulting from seismic forces.

3.2.2.20 SHEAR WALL

It is the wall designed to resist lateral forces acting in its own plane.

3.2.2.21 SOFT STOREY

It is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

3.2.2.22 STATIC ECCENTRICITY (e_{si})

It is the distance between center of mass and center of rigidity of floor i.

3.2.2.23 STOREY

It is the space between two adjacent floors.

3.2.2.24 STOREY DRIFT

It is the displacement of one level relative of the other level above or below.

3.2.2.25 STOREY SHEAR (V_i)

It is the sum of design lateral forces at all levels above the storey under consideration.

CHAPTER 4

SEISMIC DESIGN PHILOSOPHY FOR BUILDINGS

4.1 EARTHQUAKE DESIGN PHILOSOPHY

The earthquake design philosophy may be summarized as follows:

a) Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damage, however building parts that do not carry load may sustain repairable damage.
b) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake and
c) Under strong but rare shaking, the main members may sustain, severe (even irreparable) damage, but the building should not collapse.

Thus, after minor shaking, the building will be fully operational within a short time and the repair costs will be small, and after moderate shaking, the building will be operational once the repair and strengthening of the damaged main members is completed. But after a strong earthquake, the building may become dysfunctional for further use, but will stand so that people can be evacuated and property recovered.

The consequences of damage have to the kept in view in the design philosophy. For example, important buildings, like hospitals and fire stations, play a critical role in post-earthquake activities and must remain functional immediately after the earthquake. These structures must sustain very little damage and should be designed for a higher level of earthquake protection. Collapse of dams during earthquakes can cause flooding in the downstream reaches, which itself can be a secondary disaster. Therefore, dams (and similarly, nuclear power plants) should be designed for still higher level of earthquake motion.

4.2 DAMAGE IN BUILDINGS

Design of buildings to resist earthquakes involves controlling the damage to acceptable levels at a reasonable cost. Contrary to the common thinking that any crack in the building after an earthquake means the building is unsafe for habitation, engineers designing is safe for habitation, engineers designing earthquake-resistant buildings recognize that some damage is unavoidable. Different types of damage (mainly visualized though crakes, especially so in concrete and masonry buildings) occur in buildings during earthquakes. Some of these cracks are acceptable (in terms of both their size and location) while others are not. For instance, in a reinforced concrete frame building with masonry filler walls between columns, the cracks between vertical columns and masonry filler walls are acceptable, but diagonal cracks running through the columns are not. In general, qualified technical professionals are knowledgeable of the causes and severity of damage in earthquake-resistant buildings.

Earthquake-resistant design in therefore concerned about ensuring that the damages in buildings during earthquakes are of the acceptable variety, and also that they occur at the right places and in right amounts. This approach of earthquake-resistant design in much like the use of electrical fuses in houses: to protect the entire electrical wiring and appliances in the house, you sacrifice some small parts of the electrical circuit, called fuses; these fuses are easily replaced after the electrical over current. Likewise, to save the building from collapsing, you need to allow some pre-determined parts to undergo the acceptable type and level of damage.

4.3 DESIGN EARTHQUAKE RESISTANT CRITERIA

4.3.1 ZONE FACTOR

Seismic zoning assesses the maximum severity of shaking that is anticipated in a particular region. The zone factor (Z), thus is defined as a factor to obtain the design spectrum depending on the perceived seismic hazard in the zone in which the structure is located. The basis zone factors included in the code are reasonable estimate of effective peak ground acceleration. Zone factors as per IS 1893 (Part 1) : 2002 are given in Table 1.

Table 1: Zone Factor (Z) as per the zone of the building:

Seismic zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Sever	Very Sever
Z	0.10	0.16	0.24	0.36

4.3.2 IMPORTANCE FACTOR

The importance factor is a factor used to obtain the design seismic force depending upon the functional use of the structure.It is customary to recognize that certain categories of building use should be designed for greater levels design forces. Such categories are:

a) Buildings which are essential after an earthquake – hospitals, fire stations, etc.
b) Places of assembly – schools, theatres, etc.
c) Structures the collapse of which may endanger lives – nuclear plants, dams, etc.

Table 2: Importance factor (I) as per the functional use of the buiding (IS 1893:2002)

Structure	Importance Factor (*I*)
Important service and community buildings, which as hospitals, schools; monumental structure; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire stations, buildings; large community halls like cinemas, assembly halls; and subway stations, power stations	1.5
all other buildings	1.0

4.3.3 RESPONSE REDUCTION FACTOR

The basic principle of designing a structure for strong ground motion is that the structure should not collapse but damage to the structural elements is permitted. Since a structure is allowed to be damaged in case of severe shaking, the structure should be designed for seismic force much less than what is expected under strong shaking, if the structure there to remain linearly elastic. Response reduction factor (R) is the factor by which the actual base shear force should be reduced, to obtain the design lateral force.Response reduction factor for building systems are given below table as per IS 1893(Part 1) : 2002

Table 3: Response reduction factors (R)

Lateral load-resisting system	Response reduction factor (R)
Building frame systems	
Ordinary RCC moment-resisting frame (OMRF)	3.0
Special RCC moment-resisting frame (SMRF)	5.0
Steel frame with	
(a) Concentric braces	4.0
(b) Eccentric braces	5.0
Steel moment-resisting frame designed as per SP 6(6)	5.0
Building with shear walls	
Load bearing masonry wall buildings	
(a) Unreinforced	1.5

(b) Reinforced with horizontal RCC bands	2.5
(c) Reinforced with horizontal RCC bands and Vertical bars at corers of rooms and jambs of openings	3.0
Ordinary RCC shear walls	3.0
Ductile shear walls	4.0
Buildings with dual systems	
Ordinary shear wall with OMRF	3.0
Ordinary shear wall with SMRF	4.0
Ductile shear wall with OMRF	4.5
Ductile shear wall with SMRF	5.0

4.3.4 FUNDAMENTAL NATURAL PERIOD

This fundamental natural period is the first (longest) model time period of vibration of the structure. Because the design loading depend on the building period, and the building period cannot be calculated until a design has been prepared, IS 1893 (Part 1): 2002 provides formulas from which T may be calculated for a moments resisting frame building without brick infill panels, T_a may be estimated by the empirical expressions.

$T = 0.075h^{0.75}$ for RC frame building ……….. …………. (1)

$T = 0.085h^{0.75}$ for steel frame building …………………… (2)

For all other buildings including moment-resisting frame building with brick infill panels, T_a may be estimated by the empirical expression

$$T_a = \frac{0.09\,h}{\sqrt{d}} \qquad \text{………………………. (3)}$$

Where h is height of building in meters (this excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected), and d is the base dimension of the building the plinth level, in meter, along the considered direction of the lateral force.

4.3.5 RESPONSE SPECTRUM METHOD

The design response spectrum is a smooth response spectrum specifying the level of seismic resistance required for a design. Seismic analysis requires that the design spectrum be specified IS 1893 (Part 1): 2002 stipulates a design acceleration spectrum or base shear coefficients as a function of natural period. These coefficients are ordinates of the acceleration spectrum, divided by acceleration due to gravity. This relationship works in SDOF systems. The spectral ordinates are used for the computation of inertia forces. Figure relates to the proposed 5 percent damping for rocky or hard soils sites and Table gives the multiplying factors for obtaining spectral values for various other damping (note that the multiplication is not be done for zero period acceleration). The design spectrum ordinates are independent of the amounts of damping (multiplication factor of 1.0) and their variations from one material or one structural solution to another.

Multiple factor for obtaining spectral values for damping (other than 5 per cent damping)

Table 4: For multiple factor for obtaining spectral values for damping (other than 5 per cent damping)

Damping (per cent)	0	2	5	7	10	15	20	25	30
Factors	3.20	1.40	1.00	0.90	0.80	0.70	0.06	0.55	0.50

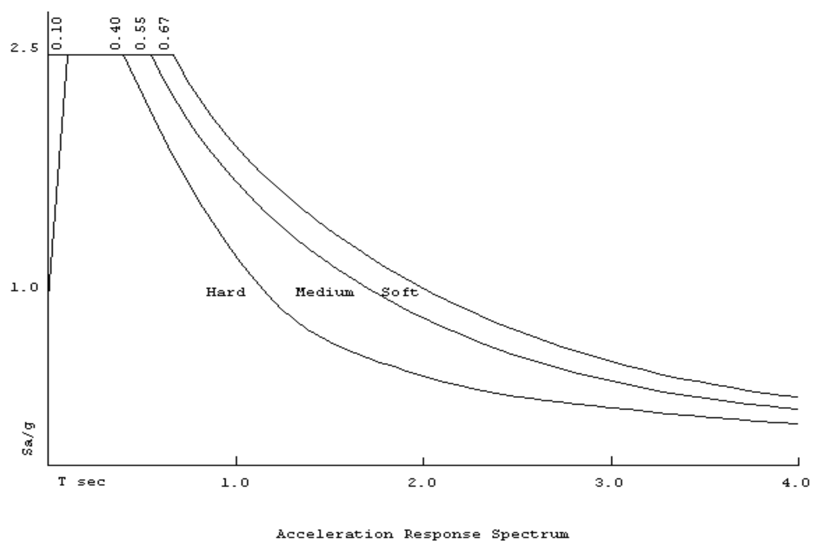

Figure 5: Response Spectra For Rock and Soil Sites for 5 % damping by IS 1893(Part 1): 2002

4.4 IDENTIFICATION OF DAMAGE IN RC BUILDINGS

Reinforced concrete buildings have been damaged on a very large scale in Bhuj earthquake of January 26, 2001. These buildings have been damaged due to various reasons. Identification of a single cause of damage to building is not possible. There are combined reasons, which are responsible for multiple damages. It is difficult to classify the damage, and even more difficult to relate it in quantitative manner. This is because of the dynamic character of the seismic action and the inelastic response of the structures. The principal cause of damage to building are soft stories, floating columns, mass irregularities, poor quality of material, faulty construction practices, inconsistence seismic performance, soil and foundation effect, pounding of adjacent structures and inadequate ductile detailing in structural components, which have been described in detail subsequently.

4.4.1 SOFT STOREY FAILURE

In general, multi-storeyed buildings in metropolitan cities require open taller first storey for parking of vehicles and/or for retail shopping, large space for meeting room or a banking hall owing to lack of horizontal space and high cost. Due to this functional requirement, the first storey has lesser strength and stiffness as compared to upper stories, which are stiffened by masonry infill walls. This characteristic of building construction creates " "soft" storey problems in multi story buildings. Increased flexibility of first story results in extreme deflections, which in turn, leads to concentration of forces at the second storey connections accompanied by large plastic deformations. In addition, most of the energy developed during the earthquake is dissipated by the columns of the soft stories. In this process the plastic hinges are formed at the ends of columns, which transform the soft storey into a mechanism. In such cases the collapse in unavoidable. Therefore, the soft stories deserve a special consideration in analysis and design.

4.4.2 FLOATING COLUMNS

Most of the buildings in Ahmedabad and Gandhidham, are covering the maximum possible area of a plot within the available bylaws. Since balconies are not counted in Floor Space Index. Buildings have balconies overhanging in the upper stories beyond the column footprint area at the ground storey, overhangs up to 1.2m to 1.5 m in plan are usually provided on each side of the building. In the upper stories, the perimeter columns of the ground storey are discontinued, and floating columns are provided along the overhanging perimeter of the building. These floating columns rest at the tip of the taper overhanging beams without considering the increased vulnerability of lateral load resisting system due to vertical discontinuity. This type construction does not create any problem under vertical loading conditions. But during an earthquake a clear load path is not available for transferring the lateral forces to the foundation. Lateral forces accumulated in upper floors during the earthquake have to be transmitted by the projected cantilever beams.

4.4.3 POOR QUALITY OF CONSTRUCTION MATERIAL AND CORROSION OF REINFORCEMENT.

There are numerous instances in which faulty construction practices and lack of quality control contributed to the damage. In the cement-sand ratio, the ratio of sand was dangerously high. It also appeared that recycled steel was used as reinforcement. Himgiri Apartment is now a pile of rubble as a result of poor quality of construction materials. Many buildings are damaged due to spalling of concrete by the corrosion of embedded reinforcing bars. The corrosion is related to insufficient concrete cover, poor concrete placement and porous concrete. Several buildings constructed about 5 to 10 years ago were damaged due to lack of quality control. It is reported that the water supply in the outer part of the city is through ground water, which is salty in taste and the same water is used in preparing the concrete mix for construction. The presence of slats may also have affected the quality of concrete.

4.4.4 POUNDING OF BUILDINGS

Pounding is the result of irregular response of adjacent buildings of different heights and of different dynamic characteristics. When the floors of adjacent buildings are at different elevations, the floor of each buildings acts like rams, battering the columns of the other building. When one of the buildings in higher than the other, the building of lower height acts as a base Earthquakes for the upper part of the adjacent taller building. The low height building receives an unexpected load while the taller building suffers from a major stiffness discontinuity at the level of the top of the lower building. Pounding may also occur because of non-compliance of codal provisions particularly for lateral and torsional stiffness and cumulative tilting due to foundation movement damage due to pounding can be minimized by drift control, building separation, and aligning poor in adjacent buildings.

4.4.5 INCONSISTENT SEISMIC PERFORMANCE OF BUILDINGS

It is evident that the earthquake did not affect all the structures uniformly. The dynamic characteristics of buildings are one of the predominant factors. The severity of damage varied systematically, with total collapse of buildings in some cases to minor damage in nearby buildings. Higher Secondary School in Mani Nagar at Ahmedabad, a four – storey RC building, collapsed while nearby buildings suffered minor damage, Similarly block of Mansi Complex in satellite town sustained only minor damage while the adjacent portion or the A-Block completely collapsed.

A multi-storeyed RC building, under construction, across the road from Shikhar Apartment escaped damage, while D-Block of Shikhar Apartment collapsed. In some cases the buildings appeared to be identical but the degree of damage varied significantly. Possible explanations for the behaviors could be workmanship, detailing practices, quality of material, design, etc.

CHAPTER 5

METHODS OF ELASTIC ANALYSIS

5.0 GENRAL

The most commonly used methods of analysis are based on the approximation that the effects of yielding can be accounted for by linear analysis of the building, using the design spectrum for inelastic system. Forces and displacements due to each horizontal component of ground motion are separately determined by analysis of an idealized building having one lateral degree of freedom per floor in the direction of the ground motion component being considered. Such analysis may be carried out by the equivalent lateral force procedure (static method) or response spectrum analysis procedure (dynamic method), another refined method of dynamic analysis is the elastic time – history method.

5.1 EQUIVALENT LATERAL FORCE METHOD (SEISMIC COEFFICIENT METHOD)

Seismic analysis of most structures is still carried out on the assumption that the lateral (horizontal) force is equivalent to the actual (dynamic) loading. This method requires less effort because, except for the fundamental period, the periods and shapes of higher natural modes of vibration are not required. The base shear which is the total horizontal force on the structure is calculated on the basis of the structure's mass, its fundamental period of vibration, and corresponding shape. The base end shear is distributed along the height of the structure, in terms of lateral force, according to the code formula. Planar models appropriate for each of the two orthogonal lateral directions are analyzed separately, the results of the two analyses and the various effects, including those due to torsional motions of the structure, are combined. This method is usually conservative for low to medium-height buildings with a regular conformation.

5.2 RESPONSE SPECTRUM ANALYSIS

This method is also known as modal method or mode super position method. The method is applicable to those structures where modes other than the fundamental one significantly affect the response of structures. Generally, the method is applicable to analysis of the dynamic response of structure, which are asymmetrical or have areas of discontinuity or irregularity, in their linear range of behavior. In particular, it is applicable to analysis of forces and deformation in multi-storey buildings due to medium intensity ground shaking, which causes a moderately large but essentially linear response in the structure. This method is based on the fact that, for certain forms of damping – which are reasonable models for many buildings-the response in each natural mode of vibration can be computed independently of the others, and the modal responses can be combined to determine the total response. Each mode responds with its own particular pattern of deformation (mode shape), with its own frequency (the modal frequency), and with its own modal damping. The time history of each modal response can be computed by analysis of a SDOF oscillator with properties chosen to be representative of the particular mode and the degree to which it is excited by the earthquake motion. In general, the responses need to be determined only in the first few modes, because response to earthquake in primarily due to lower modes of vibration.

A complete modal analysis provides the history of response – forces, displacements, and deformation – of a structure to a specified ground acceleration history. However, the complete response history is rarely needed for design, the maximum values of response over the duration of the earthquake usually suffice. Because the response in each vibration mode can be modelled by the response of a SDOF oscillator, the maximum response in the mode can be directly computed from the earthquake response spectrum. Procedures for combining the modal maxima to obtain estimates (but not the exact value) of the maximum of total response are available. In its most general from, the modal method for linear response analysis is applicable to arbitrary three-dimensional structural systems. However, for the purpose of design of buildings, it can often be simplified from the general case by restricting its application to the lateral motion in a plane. Planar models appropriate for each of two orthogonal lateral directions are analyzed separately, and the result of the two analyses and the effects of torisonal motions to the structures are combined.

5.3 ELASTIC TIME HISTORY METHOD

A linear time history analysis overcomes all the disadvantages of a modal response spectrum analysis provided non-linear behavior is not involved. This method requires greater computational efforts for calculating the response at discrete times. One interesting advantage of such a procedure is that the relative signs of response quantities are preserved in the response histories. This is important when interaction effects are considered among stress resultants.

5.4 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 behaviour 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 storeys 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 that about 30 percent in adjacent storeys. 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 step 3.
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.

5.5 EXPLANATION OF SEISMIC METHODS

5.5.1 EQUIVALENT LATERAL FORCE METHOD

This method of finding design lateral forces is also known as the static method or the equivalent static method or the seismic coefficient method. This procedure does not require dynamic analysis, however, it accounts for the dynamics of building in an approximate manner. The static method is the simplest one – it requires less computational effort and is based on formulae given in the code of practice. First, the design base shear is computed for the whole building, and it is then distributed along the height of the building. The lateral forces at each floor level thus obtained are distributed to individual lateral load resisting elements.

5.5.1.1 SEISMIC BASE SHEAR

The total design lateral force or design seismic base shear (V_B) along any principal direction is determined by:

$$V_B = A_h W \quad \ldots\ldots\ldots\ldots \qquad (4)$$

Where A_h is the design horizontal acceleration spectrum value, using the fundamental natural period, T, in the considered direction of vibration and W is the seismic weight of the building. The design horizontal seismic coefficient A_h for a structure is determined by the expression.

$$A_h = \frac{ZIS_a}{2\,R\,g} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.(5)$$

For any structure with T≤0.1 s, the value of A_h will not be taken less than Z/2 whatever be the value of I/R. In Equation, Z is the zone factor given in table for the maximum considered earthquake (MCE). The factor 2 in the denominator is used so as to reduce the maximum considered earthquake (MCE) zone factor to the factor for design-basis earthquake (DBE). I is the importance factor given in table, and depends upon the functional use of the structure, the hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance. R is the response reduction factor given in Table, and depends on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. This factor is used to decide what building materials are used, the type of construction, and the type of lateral bracing system. Sa/g is the response acceleration coefficient as given by fig. For 5 percent damping based on appropriate natural periods. The curves of figure Represent free-field ground motion. For other damping values of the structure, multiplying factors given in table should be used.

For rocky or hard soil sites.

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

For medium soil sites

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

For soft soil sites

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

5.5.1.2 SEISMIC WEIGHT

The seismic weight of the whole building is the sum of the seismic weights of all the floors. The seismic weight of each floor is its full dead load plus the appropriate amount of imposed load, the latter being that part of the imposed loads that may reasonably be expected to be attached to the structure at the time of earthquake shaking. It includes the weight of permanent and movable partitions, permanent equipment, a part of the live load, etc. While computing the seismic weight of each floor, the weight of columns and walls in any storey should be equally distributed to the floors above and below the storey. Any weight supported in between storeys should be distributed to the floors above and below in inverse proportion to its distance from the floors.

As per IS 1893: (Part 1), the percentage of imposed load as given in Table should be used. For calculating the design seismic forces of the structure, the imposed load on the roof need not be considered.

Table 5: For Percentage of imposed load to be considered in seismic weight calculation

Imposed uniformly distributed floor load(KN/m^2)	Percentage of imposed load
Upto and including 3.0	25
Above 3.0	50

5.5.1.3 DISTRIBUTION OF DESIGN FORCE

Buildings and their elements should be designed and constructed to resist the effects of design lateral force. The design lateral force is first computed for the building as a whole and then distributed to various floor levels. The overall design seismic force thus obtained at each floor level is then distributed to individual lateral load-resisting elements, depending on the floor diaphragm action.

Vertical distribution of base shear to different floor levels The design base shear (V_B) is distributed along the height of the building as per the following expression

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Where Q_i is the design lateral force at floor i, W_i is the seismic weight of floor i, h_i is the height of floor i measured from the base, and n is the number of storeys in the building i.e., the number of levels at which the masses are located.

Distribution of horizontal design lateral force to different lateral force resisting elements. In the case of buildings in which floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane is distributed to the various vertical elements of the lateral force-resisting system, assuming the floors to be infinitely rigid in the horizontal plane. For buildings in which floor diaphragms cannot be treated as infinitely rigid in their own plane, the lateral shear at each floor is distributed to the vertical element resisting the lateral forces, accounting for the in-plane flexibility of the diaphragms.

A floor diaphragm is considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.5 times the average displacement of the entire diaphragm. Reinforced concrete monolithic slab-beam floors or those consisting of prefabricate/precast elements with topping of reinforced screed can be taken as rigid diaphragms.

5.5.2 RESPONSE SPECTRUM METHOD

In the response spectrum method, the response of a structure during an earthquake is obtained directly from the earthquake response (or design) spectrum. This procedure gives an approximate peak response, but this is quite accurate for structural design applications. In this approach, the multiple modes of response of a building to an earthquake are taken into account. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass. The responses of different modes are combined to provide an estimate of total response of the structure using modal combination methods such as complete quadratic combination (CQC), square root of sum of squares (SRSS), or absolute sum (ABS) method.

Response spectrum method of analysis should be performed using the design spectrum specified or by a site – specific design spectrum, which is specifically prepared for a structure at a particular project site. The same may be used for the design at the discretion of the project authorities.

5.5.2.1 FREE – VIBRATION ANALYSIS

Undamped free-vibration analysis of the entire building is performed as per established methods of mechanics, using the appropriate masses and elastic stiffness of the structural system to obtain natural periods (T) and mode shapes (Φ) of those of its modes of vibration that need to be considered. The number of modes to be used in the analysis should be such that the total sum of modal masses of all modes considered is at least 90 percent of the total seismic mass. If modes with natural frequency beyond 33HZ are to be considered, modal combination should be carried out only for modes up to 33Hz. The effect of modes with natural frequency beyond 33Hz should be included by considering the missing mass correction following established procedure.

5.5.2.2 MODAL COMBINATION

The peak response quantities (e.g., member forces, displacement, storey forces, storey shears, and base reactions) should be combined as per the complete quadratic combination (CQC) method.

$$\lambda = \sqrt{\sum_{i=1}^{r}\sum_{j=1}^{r}\lambda_i \rho_{ij} \lambda_j} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7)$$

Where r is the number of modes being considered. ρ_{ij} is the cross-modal coefficient given by Eqn. λ_i is the response quantity in mode I (including sign), and λ_j is the response quantity in mode j (including sign).

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

Where ζ is the modal damping ratio (in fraction), β is the frequency ratio and is equal to ω_j/ω_i, ω_i is the circular frequency in the i^{th} mode, and ω_j is the circular frequency in the j^{th} mode.

Alternatively, the peak response quantities may be combined by SRSS method as in case I and by ABS method as in case 2 below.

Case 1 : If the building does not have closely-spaced modes, then the peak response quantity λ, due to all modes considered should be obtained as

$$\lambda = \sqrt{\sum_{k=1}^{r}\lambda^2{}_k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(9)$$

Where λ_k is the absolute value of the quantity in mode k and r is the number of modes being considered.

Case 2 : If the building has a few closely-spaced modes, then the peak response quantity λ^*, due to these modes should be obtained as

$$\lambda^* = \sum_{c}^{r} \lambda_c \quad \text{……………………(10)}$$

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.

5.5.2.3 MODAL ANALYSIS

Building with regular, or nominally irregular, plan configurations, 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:

(a) **Modal mass :** The modal mass (M_k) of mode k is given by

$$M_k = \frac{\left[\sum_{i=1}^{n} W_i \phi_{ik}\right]^2}{g \sum_{i=1}^{n} W_i (\phi_{ik})^2} \quad \text{…………………..(11)}$$

Where g is the acceleration due to gravity. Φ_{ik} is the mode shape coefficient at floor i in mode k, and W_i is the seismic weight of floor i.

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

$$P_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i (\phi_{ik})^2} \quad \text{…………………..(12)}$$

(c) **Design lateral force at each floor in each mode :** The peak lateral force (Q_{ik}) at floor i in k^{th} mode is given by

$$\mathbf{Q_{ik} = A_k \Phi_{ik} P_k W_i} \quad \text{…………………(13)}$$

Where A_k is the design horizontal acceleration spectrum value using the natural period of vibration (T_k) of k^{th} mode.

(d) **Storey shear forces in each mode** : The peak shear (V_{ik}) acting in storey i in mode k is given by

$$V_{ik} = \sum_{j=i+1}^{n} \phi_{ik} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(14)$$

(e) **Storey shear forces due to all modes considered :** The peak storey shear force (V_i) in storey i due to all modes considered is obtained by combining those due to each modes as explained above.

(f) **Lateral forces at each storey due to all modes considered:** The design lateral forces, F_{roof} and F_i at roof and at floor i are given by

$F_{roof} = V_{roof}$

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

CHAPTER 6

ANALYSIS OF BUILDING FOR DIFFERENT CONFIGURATIONS

A hypothetical building is assumed for seismic analysis that consists of a G+3 R.C.C. public cum-office building. The plan of the building is regular in nature as it has all columns at equal spacing. The building is located in Seismic Zone IV and is founded on medium type soil. The building is 16.00 m in height 40.70 m in length and 11.5m in width. The important features of this building are shown in Table 6.

Table 6: Important features of building

1.	Type of Structure	Multi-storey rigid jointed frame
2.	Zone	IV
3.	Layout	As shown in Figure no 6
4.	Number of stories	Four (G + 3)
5.	Ground storey height	4.0 m
6.	Floor-to-floor height	4.0 m
7.	External walls	250 mm thick including plaster
8.	Internal walls	150 mm thick including plaster
9.	Live load	3.5 kN/m^2
10.	Materials	M 20 and Fe415
11.	Seismic analysis	Equivalent static method and Response Spectrum Method (IS 1893 (Part 1): 2002
12.	Design Philosophy	Limit state method conforming to IS 456 : 2000
13.	Size of exterior column	300 x 530 mm
14.	Size of interior column	300 x 300 mm
15.	Size of beams in longitudinal and transverse direction	300 x 450 mm
16.	Total thickness of slab	120 mm

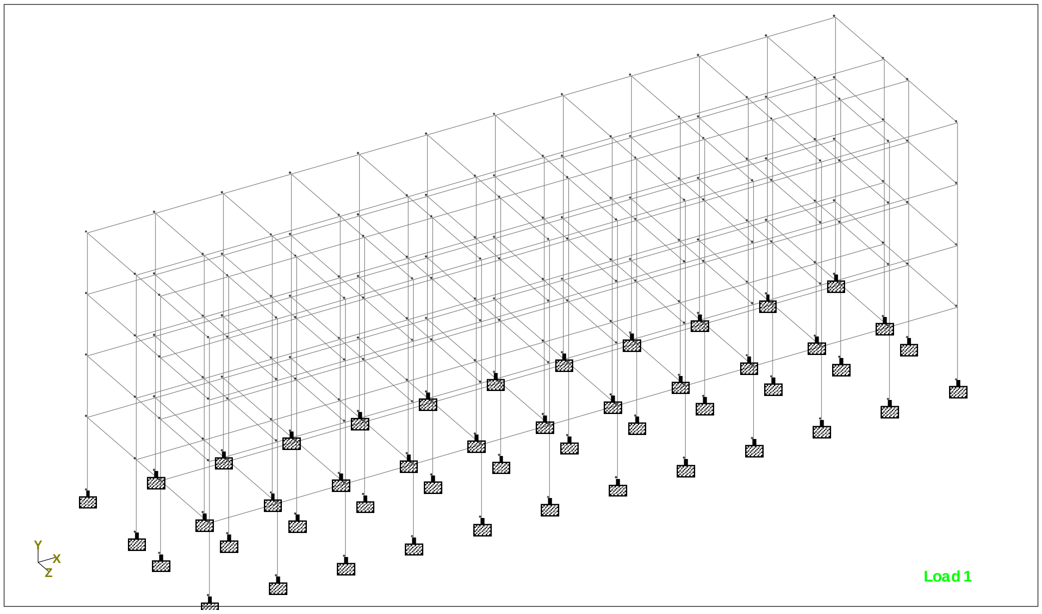

Figure 6: 3D – VIEW OF THE BUILDING

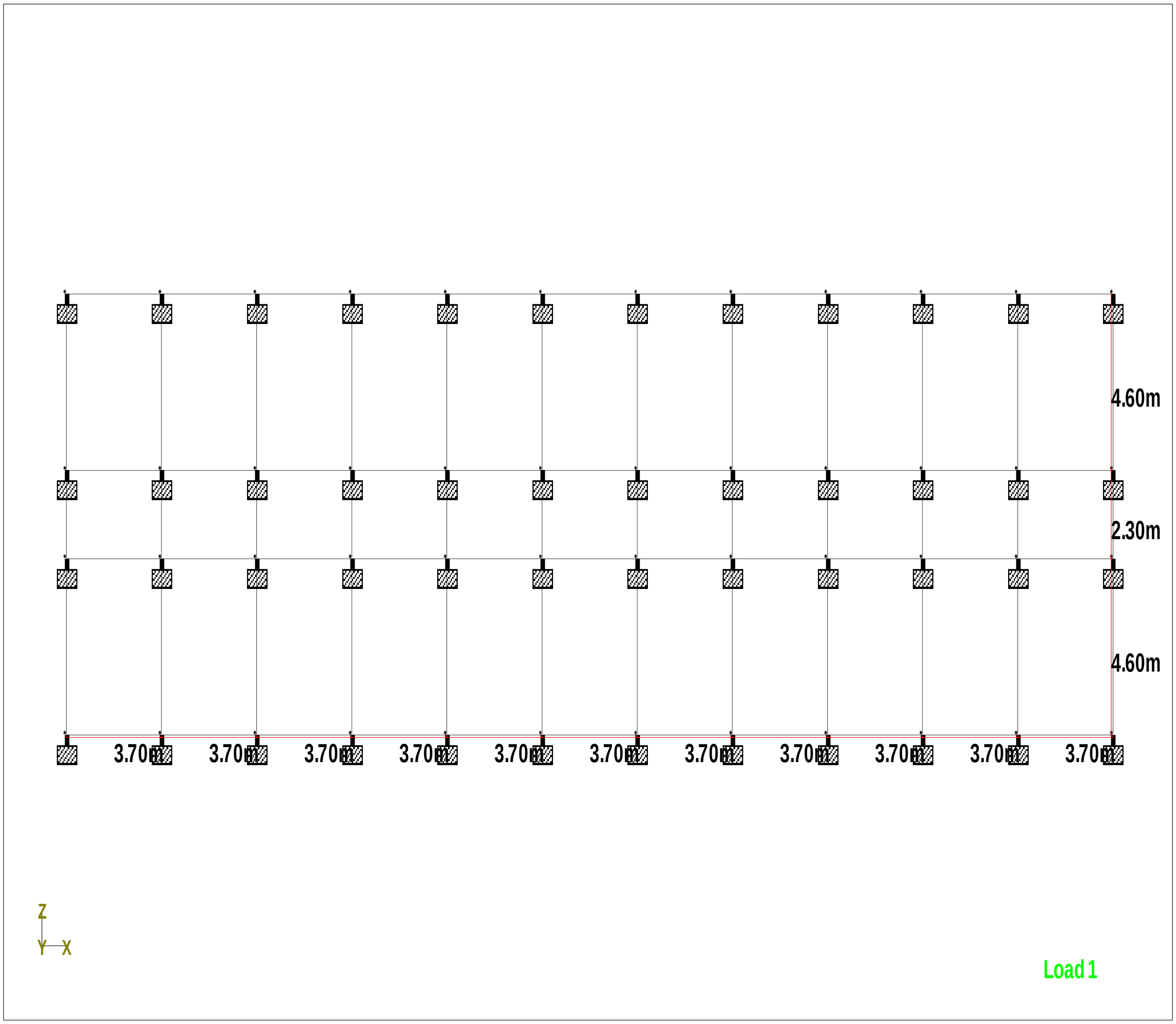

Figure 7: Plan of the building

LUMPED MASS CALCULATION AT VARIOUS FLOOR LEVELS

- Considering slab of dimension 4.6 × 3.7m as Slab Type 1.
- Considering slab of dimension 2.3 × 3.7m as Slab Type 2.

1. Dead load calculation

a) At 1st Floor

Total Weight of one Slab of Type1 = 25 × 0.12 × 4.6 × 3.7 = 51.06 KN

No. of Slabs = 2 × 11 = 22

Total Weight of Slab of Type 1 = 22 × 51.06 = 1123.32 KN

Weight of slabType2 = 25 × 0.12 × 2.3 × 3.7 = 25.53 KN

No of Slab of Type 2 = 11

Total Weight of Slabof Type 2 = 25.53 × 11 = 280.83 KN

Weight of floor finishing = 0.5 KN/m^2

= 0.5 × (40.70 × 11.5)

= 234.025 KN

Self Weight of beams = 25 × 0.30 × (0.45 – 0.12)

= 2.475 KN/m

Total length of beams = 289.3 m

Total Weight of beams = 2.475 × 289.3 = 716.0175 KN

Total dead load at floor 1 = Total Weight of Slab (Type 1 + Type 2) + Weight of floor finish + Self Weight of beams.

= 1123.32 + 280.83 + 234.025 + 716.0175

= 2354.19 KN

b) Dead load at 2nd Floor = Same as calculated for Floor 1

= 2354.19 KN

c) Dead load at 3rd Floor = Same as calculated for Floor 1

= 2354.19 KN

d) Dead load at Roof level = Weight. of Slab Type 1 + Weight. of Slab Type 2 + Self Weight of Beam

= 1123.32 + 280.83 + 716.0175

= 2120.1675 KN

2) Live Load Calculations

a) At 1st , 2nd & 3rd Floor = 50% of live load as per IS 1893-2002

$= 0.50 \times (3.5 \times 40.7 \times 11.5)$

= 819.0875 KN

b) At roof level = 25% of live lad as per 1893-2002

$= 0.25 \times (1.5 \times 40.7 \times 11.5)$

= 175.51 KN

3) Total load (Dead load + Live Load)

a) Lumped mass at floor 1, 2, 3 = m1 = m2 = m3

= 2354.19 + 819.0875

= 3173.28 KN

= 3180 KN

b) Lumped mass at roof level m4 = 2120.1675 + 175.51

= 2295.68 KN

= 2300 KN

CALCULATION OF STIFFNESS AT VARIOUS FLOOR LEVEL

- Considering column of dimension 300mm x 300mm Type 1
- And column of dimension 300 mm x 530mm Type 2

Stiffness of column Type 1

$$K_A = \frac{12EI}{L^3}$$

E	$= 5000 \sqrt{Fck}$	$= 5000 \sqrt{20}$	$= 22360.6798 \times 10^3$ K N/m^2
I	$= bd^3 / 12$	$= 0.300 \times (0.300)^3 / 12$	$= 6.75 \times 10^{-4}$ m^4
L	= 4m		

$K_A = 12 \times 22360.6798 \times 10^3 \times 6.75 \times 10^{-4} / (4)^3$

= 2830.0235 KN/m

Stiffness of column Type 2

$$K_B = \frac{12EI}{L^3}$$

E = 22360.6798×10^3 K N/m^2

I = $bd^3 / 12$ =0. 300 × (0.530)3 / 12 = 3.7219×10^{-3} m^4

L = 4m

K_B = $12 \times x\ 22360.6798 \times 10^3 \times 3.7219 \times 10^{-3} / (4)^3$

= 15604.54 KN/m

Stiffness at floor level 1, 2, 3

K = $24 \times K_A + 24 \times K_B$ = 24× 2830.0235 + 24 × 15604.54=442429.524 KN/m

K1 = K2 = K3=K

Stiffness at roof level K4 = 0.72 × 442429.524

= 318549.2573 KN/m

Note : Factor 0.72 is taken into account because the mass at roof level is 0.72 times the mass of floor level.

LUMPED MASS MODEL

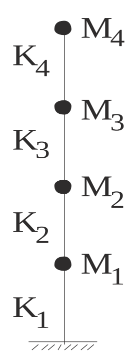

Figure 8: Lumped mass model

CASE I : WHEN NO INFILL WALL IS PROVIDED

A given building is analyzed by two different methods when no infill wall is provided at any floor 1st method is Seismic coefficient method also known as equivalent static lateral force method and 2nd method is response spectrum method.

Seismic coefficient method

```
n=4;
W=[3180 3180 3180 2300];
h=[4.0 4.0 4.0 4.0];
M=(1.0/9.81)*W;
Z=0.24;
I=1.5;
R=5;
g=9.81;
type=2;
TW=0.0;
H=0.0;
for i=1:4
   TW=TW+W(i);
   H=H+h(i);
end
% TO FIND OUT FUNDAMENTAL NATURAL PERIOD OF VIBRATION T
T=0.075*(H^0.75);
T
% TO FIND OUT Sa/g VALUE
for i=1:n
if type==1
   if T<=0.1
      sag=1+15*T;
   elseif T<=0.4
      sag=2.50;
   else
      sag=1.0/T;
   end
elseif type==2
   if T<=0.1
      sag=1+15*T;
   elseif T<=0.55
      sag=2.5;
   else
      sag=1.36/T;
   end
else
   if type==3
      if T<=0.1
         sag=1+15*T;
      elseif T<=0.67
         sag=2.5;
      else
         sag=1.67/T;
```

```
        end
    end
end
end
sag
AH=(Z/2)*(I/R)*sag
VB=AH*TW;
VB
%VERTICAL DISTRIBUTION OF BASE SHEAR TO DIFFERENT FLOOR LEVEL
sumsquare=0.0;
%d=h(1)
d=0.0;
for i=1:n
    d=d+h(i)
    sumsquare=sumsquare+W(i)*(d^2);
    end
dd=0;
for i=1:n
    dd=dd+h(i);
    C(i)=(W(i)*(dd^2))/sumsquare;
        Q(i)=VB*C(i);
end
Q
```

Fundamental Period T_a = 0.6 sec

Sa/g = 2.2667

A_h =0.0816

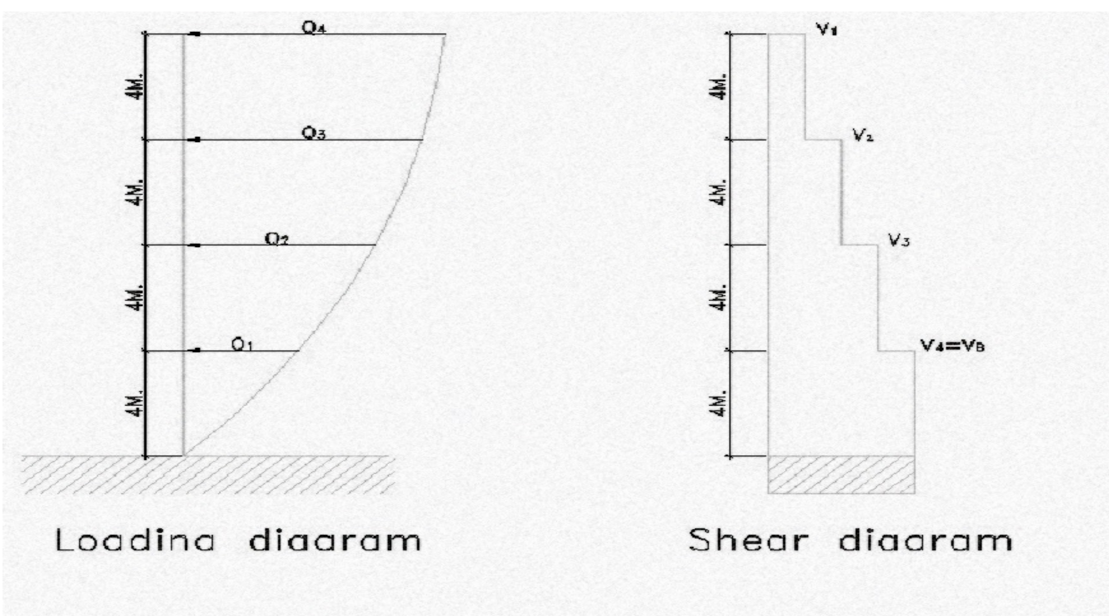

Figure 9: Loading diagram and Shear Diagram of case 1 sesmic coefficient method

Q_1 = 37.7808 KN

V_1 = 437.2122 KN

Q2 = 151.1234 KN

V_2 = 777.2398 KN

Q3 = 340.0276 KN

V_3 = 928.3632 KN

Q4 = 437.2122 KN

V_B = 966.1440 KN

Response Spectrum Method

```
%RESPONSE SPECTRUM METHOD
n=4;
w=[3180 3180 3180 2300];
k=[442429.524 442429.524 442429.524 318549.2573];
m=(1.0/9.81)*w;
M=zeros(n,n);
K=zeros(n,n);
W=zeros(n,n);
evp=zeros(n);
Q=zeros(4);
for i=1:n
   M(i,i)=m(i);
   W(i,i)=w(i);
end
M
for i=1
   K(i,i)=k(i)+k(i+1);
      K(i,i+1)=-k(i+1);
end
for i=2:n
   if i<n
      K(i,i)=k(i)+k(i+1);
      K(i,i+1)=-k(i+1);
      K(i,i-1)=-k(i);
   else
      K(i,i)=k(i);
      K(i,i-1)=-k(i);
   end
end
for i=1:n
   for j=i+1:n
   K(j,i)=K(i,j);
   end
end
K
%CALCULATION OF TIME EIGEN VECTOR, EIGEN VALUE, TIME PERIOD
for i = 1:n
   for j = 1:n
      evp(i,j)= K(i,j)/M(i,i)
   end
```

```
end
[Evect,Evalue]=eig(evp)
%CALCULATION OF EIGEN VALUE (FREQUENCY)
sqrt(Evalue)
%CALCULATION OF EIGEN VECTOR
Evect
%CALCULATION OF NORMALISED EIGEN VECTOR
c1=(Evect')*M;
c2=c1*Evect;
c3=sqrt(c2);
c4=abs(c3);
xtmx=Evect/c4;
xtmx
%CAQLCULATION OF TIME PERIODS
T = 2*pi*eye(n)/sqrt(Evalue)
%CALCULATION OF PARTICIPATION FACTOR
for i=1:n
   sum=0.0;
sumsquare=0.0;
   for j=1:n
      sum=sum+M(j,j)*xtmx(j,i);
      sumsquare=sumsquare+M(j,j)*(xtmx(j,i))^2;
   end
   p(i)=sum/sumsquare;
end
p
%DETERMINATION OF MODAL MASS(MM)
for i=1:n
   sum=0.0;
   sumsquare=0.0;
   for j =1:n
   sum=sum+W(j,j)*xtmx(j,i);
   sumsquare=sumsquare+W(j,j)*(xtmx(j,i))^2;
   end
   MM(i,i)=(sum)^2/(9.81*sumsquare);
end
MM
%MODAL CONTRIBUTION OF VARIOUS MODES IN PERCENTAGE
sumMM =0.0;
for i =1:n
   sumMM =sumMM+MM(i,i);
end
for i=1:n
   MCM(i)=(MM(i,i)/sumMM)*100;
end
MCM
%CALCULATION OF LATERAL FORCES IN VARIOUS MODES
type=2;
for i=1:n
if type==1
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.4
```

```
        sag(i)=2.50;
    else
        sag(i)=1.0/T(i,i);
    end
elseif type==2
    if T(i,i)<=0.1
        sag(i)=1+15*T(i,i);
    elseif T(i,i)<=0.55
        sag(i)=2.5;
    else
        sag(i)=1.36/T(i,i);
    end
else
    if type==3
        if T(i,i)<=0.1
            sag(i)=1+15*T(i,i);
        elseif T(i,i)<=0.67
            sag(i)=2.5;
        else
            sag(i)=1.67/T(i,i);
        end
    end
end
end
sag
Z=0.24;I=1.5;R=5;
for i=1:n
    for j =1:n
    A(i)=(Z/2)*(I/R)*sag(i);
            Q(j,i)=A(i)*p(i)*xtmx(j,i)*M(j,j)*9.81;
    end
end
Q
%DETERMINATION OF STORY SHEAR FORCES IN EACH MODE
QQ=zeros(n);
for i=1:n
    sum=0.0;
    for j=1:n
        for k=j:n
            sum=sum+Q(k,i);
        end
            QQ(j,i)=sum;
            sum=0.0;
    end
end
QQ
%DETERMINATION OF STOREY SHEAR FORCE DUE TO ALL MODES(SRSS)
srssv=0.0;
laforce=zeros(n);
for i=1:n
    for j =1:n
        srssv=srssv+(QQ(i,j))^2;
        laforce(i,i)=(srssv)^0.5;
```

```
    end
    srssv =0.0;
end
laforce
%DETERMINATION OF BASE SHEAR
sum=0.0;
for i=1:n
    sum=sum+laforce(i,i);
end
    bshear=sum;
bshear


M =

  324.1590         0         0         0
         0  324.1590         0         0
         0         0  324.1590         0
         0         0         0  234.4546


K =

  1.0e+005 *

    8.8486   -4.4243         0         0
   -4.4243    8.8486   -4.4243         0
         0   -4.4243    7.6098   -3.1855
         0         0   -3.1855    3.1855


evp =

  1.0e+003 *

    2.7297   -1.3649         0         0
   -1.3649    2.7297   -1.3649         0
         0   -1.3649    2.3475   -0.9827
         0         0   -1.3587    1.3587


ans =

   68.3033         0         0         0
         0   54.3506         0         0
         0         0   36.9096         0
         0         0         0   13.5645


Evect =

   -0.4876   -0.6131   -0.5045    0.2304
    0.6915    0.1007   -0.5054    0.4297
   -0.4931    0.5965   -0.0019    0.5710
```

```
    0.2026   -0.5081    0.7001    0.6605


xtmx =

   -0.0272   -0.0353   -0.0301    0.0136
    0.0386    0.0058   -0.0302    0.0254
   -0.0275    0.0344   -0.0001    0.0338
    0.0113   -0.0293    0.0418    0.0391


T =

    0.0920         0         0         0
         0    0.1156         0         0
         0         0    0.1702         0
         0         0         0    0.4632


p =

   -2.5828   -5.2926   -9.7873   32.8094


MM =

  1.0e+003 *

    0.0067         0         0         0
         0    0.0280         0         0
         0         0    0.0958         0
         0         0         0    1.0765


MCM =

    0.5527    2.3209    7.9367   89.1896


sag =

    2.3798    2.5000    2.5000    2.5000


Q =

   19.1647   53.5278   84.4169  128.1231
  -27.1794   -8.7959   84.5736  238.9740
   19.3812  -52.0824    0.3137  317.6091
   -5.7598   32.0824  -84.7302  265.6985


QQ =
```

5.6066	24.7319	84.5738	950.4046
-13.5580	-28.7959	0.1570	822.2815
13.6213	-20.0000	-84.4166	583.3075
-5.7598	32.0824	-84.7302	265.6985

laforce =

954.4971	0	0	0
0	822.8973	0	0
0	0	589.8808	0
0	0	0	280.7799

bshear =

2.6481e+003

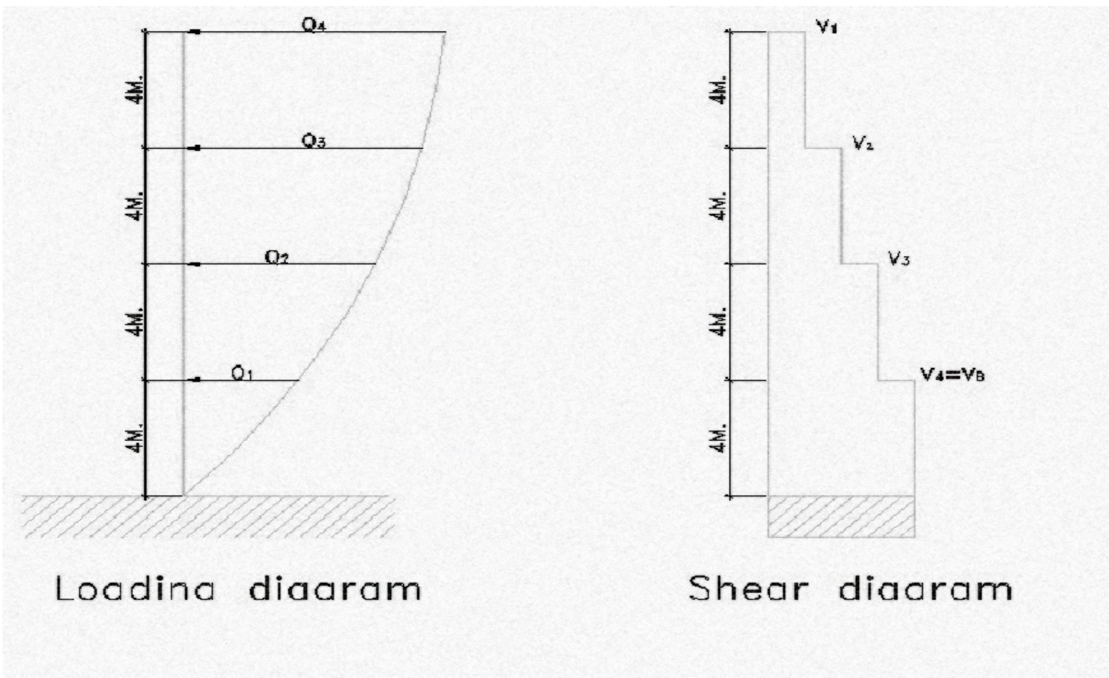

Figure 10: Loading diagram and Shear Diagram of case 1 response spectrum method

Q_1 = 280.7799 KN

Q_2 = 589.8808 KN

Q_3 = 822.8973 KN

Q_4 = 954.4971 KN

V_1 = 954.4971 KN

V_2 = 1777.3944 KN

V_3 = 2367.2752 KN

V_B =2.6481e+003 KN

Case II : **When stiffness of each floor is same.**

Same given building is analyzed by two same different methods as above when no infill wall is provided at any floor, in this case stiffness of all the floor is same so result will be same for seismic coefficient method but for response spectrum method they will differ.

Seismic coefficient method

Fundamental Period T_a = 0.6 sec

Sa/g = 2.2667

A_h =0.0816

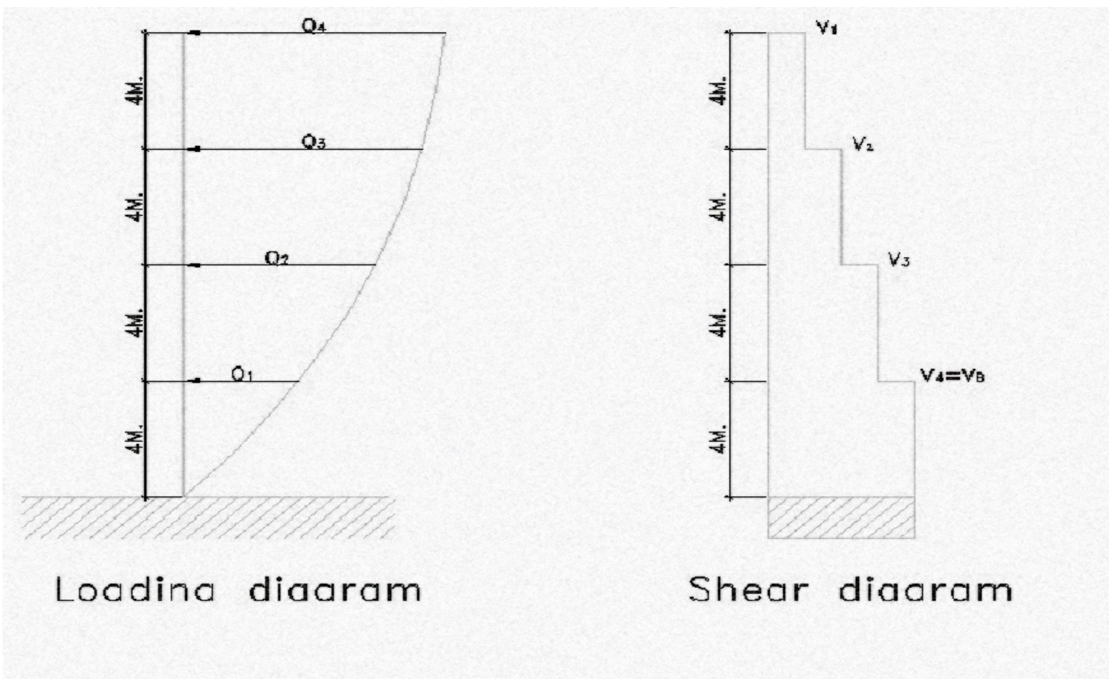

Figure 11: Loading diagram and Shear Diagram of case 2 sesmic coefficient method

V_B = 966.1440 KN

Q_1 = 37.7808 KN V_1 = 437.2122 KN

Q_2 = 151.1234 KN V_2 = 777.2398 KN

Q_3 = 340.0276 KN V_3 = 928.3632 KN

$Q_4 = 437.2122$ KN $V_B = 966.1440$ KN

Response Spectrum Method

```
%RESPONSE SPECTRUM METHOD
n=4;
w=[3180 3180 3180 2300];
k=[442429.524 442429.524 442429.524 442429.524];
m=(1.0/9.81)*w;
M=zeros(n,n);
K=zeros(n,n);
W=zeros(n,n);
evp=zeros(n);
Q=zeros(4);
for i=1:n
   M(i,i)=m(i);
   W(i,i)=w(i);
end
M
for i=1
   K(i,i)=k(i)+k(i+1);
      K(i,i+1)=-k(i+1);
end
for i=2:n
   if i<n
      K(i,i)=k(i)+k(i+1);
      K(i,i+1)=-k(i+1);
      K(i,i-1)=-k(i);
   else
      K(i,i)=k(i);
      K(i,i-1)=-k(i);
   end
end
for i=1:n
   for j=i+1:n
   K(j,i)=K(i,j);
   end
end
K
%CALCULATION OF TIME EIGEN VECTOR, EIGEN VALUE, TIME PERIOD
for i = 1:n
   for j = 1:n
      evp(i,j)= K(i,j)/M(i,i)
   end
end
[Evect,Evalue]=eig(evp);
%CALCULATION OF EIGEN VALUE (FREQUENCY)
sqrt(Evalue)
%CALCULATION OF EIGEN VECTOR
Evect
%CALCULATION OF NORMALISED EIGEN VECTOR
c1=(Evect')*M;
c2=c1*Evect;
```

```
c3=sqrt(c2);
c4=abs(c3);
xtmx=Evect/c4;
xtmx
%CAQLCULATION OF TIME PERIODS
T = 2*pi*eye(n)/sqrt(Evalue)
%CALCULATION OF PARTICIPATION FACTOR
for i=1:n
   sum=0.0;
sumsquare=0.0;
   for j=1:n
      sum=sum+M(j,j)*xtmx(j,i);
      sumsquare=sumsquare+M(j,j)*(xtmx(j,i))^2;
   end
   p(i)=sum/sumsquare;
end
p
%DETERMINATION OF MODAL MASS(MM)
for i=1:n
   sum=0.0;
   sumsquare=0.0;
   for j =1:n
   sum=sum+W(j,j)*xtmx(j,i);
   sumsquare=sumsquare+W(j,j)*(xtmx(j,i))^2;
   end
   MM(i,i)=(sum)^2/(9.81*sumsquare);
end
MM
%MODAL CONTRIBUTION OF VARIOUS MODES IN PERCENTAGE
sumMM =0.0;
for i =1:n
   sumMM =sumMM+MM(i,i);
end
for i=1:n
   MCM(i)=(MM(i,i)/sumMM)*100;
end
MCM
%CALCULATION OF LATERAL FORCES IN VARIOUS MODES
type=2;
for i=1:n
if type==1
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.4
      sag(i)=2.50;
   else
      sag(i)=1.0/T(i,i);
   end
elseif type==2
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.55
      sag(i)=2.5;
```

```
    else
        sag(i)=1.36/T(i,i);
    end
else
    if type==3
        if T(i,i)<=0.1
            sag(i)=1+15*T(i,i);
        elseif T(i,i)<=0.67
            sag(i)=2.5;
        else
            sag(i)=1.67/T(i,i);
        end
    end
end
end
sag
Z=0.24;I=1.5;R=5;
for i=1:n
    for j =1:n
    A(i)=(Z/2)*(I/R)*sag(i);
            Q(j,i)=A(i)*p(i)*xtmx(j,i)*M(j,j)*9.81;
    end
end
Q
%DETERMINATION OF STORY SHEAR FORCES IN EACH MODE
QQ=zeros(n);
for i=1:n
    sum=0.0;
    for j=1:n
        for k=j:n
            sum=sum+Q(k,i);
        end
            QQ(j,i)=sum;
            sum=0.0;
    end
end
QQ
%DETERMINATION OF STOREY SHEAR FORCE DUE TO ALL MODES(SRSS)
srssv=0.0;
laforce=zeros(n);
for i=1:n
    for j =1:n
        srssv=srssv+(QQ(i,j))^2;
        laforce(i,i)=(srssv)^0.5;
    end
    srssv =0.0;
end
laforce
%DETERMINATION OF BASE SHEAR
sum=0.0;
for i=1:n
    sum=sum+laforce(i,i);
end
```

```
   bshear=sum;
bshear


M =

  324.1590         0         0         0
         0  324.1590         0         0
         0         0  324.1590         0
         0         0         0  234.4546


K =

  1.0e+005 *

    8.8486   -4.4243         0         0
   -4.4243    8.8486   -4.4243         0
         0   -4.4243    8.8486   -4.4243
         0         0   -4.4243    4.4243


evp =

  1.0e+003 *

    2.7297   -1.3649         0         0
   -1.3649    2.7297   -1.3649         0
         0   -1.3649    2.7297   -1.3649
         0         0   -1.8871    1.8871


ans =

   70.2732         0         0         0
         0   58.6018         0         0
         0         0   38.9515         0
         0         0         0   13.6552


Evect =

   -0.3716   -0.6196   -0.5783    0.2350
    0.6014    0.3198   -0.5137    0.4379
   -0.6015    0.4545    0.1219    0.5809
    0.3720   -0.5544    0.6220    0.6446


xtmx =

   -0.0210   -0.0360   -0.0340    0.0139
    0.0341    0.0186   -0.0302    0.0259
   -0.0341    0.0264    0.0072    0.0343
    0.0211   -0.0322    0.0366    0.0381
```

```
T =

    0.0894         0         0         0
         0    0.1072         0         0
         0         0    0.1613         0
         0         0         0    0.4601


p =

   -1.8857   -4.6347   -9.9111   32.9191


MM =

  1.0e+003 *

    0.0036         0         0         0
         0    0.0215         0         0
         0         0    0.0982         0
         0         0         0    1.0837


MCM =

    0.2946    1.7798    8.1389   89.7867


sag =

    2.3412    2.5000    2.5000    2.5000


Q =

   10.6379   47.7191   96.4099  130.7114
  -17.2144  -24.6300   85.6470  243.5653
   17.2186  -35.0064  -20.3242  323.1439
   -7.7020   30.8825  -75.0048  259.3469


QQ =

    2.9401   18.9651   86.7279  956.7674
   -7.6978  -28.7539   -9.6820  826.0560
    9.5166   -4.1239  -95.3290  582.4907
   -7.7020   30.8825  -75.0048  259.3469


laforce =
```

960.8818	0	0	0
0	826.6488	0	0
0	0	590.3309	0
0	0	0	271.8447

bshear =

2.6497e+003

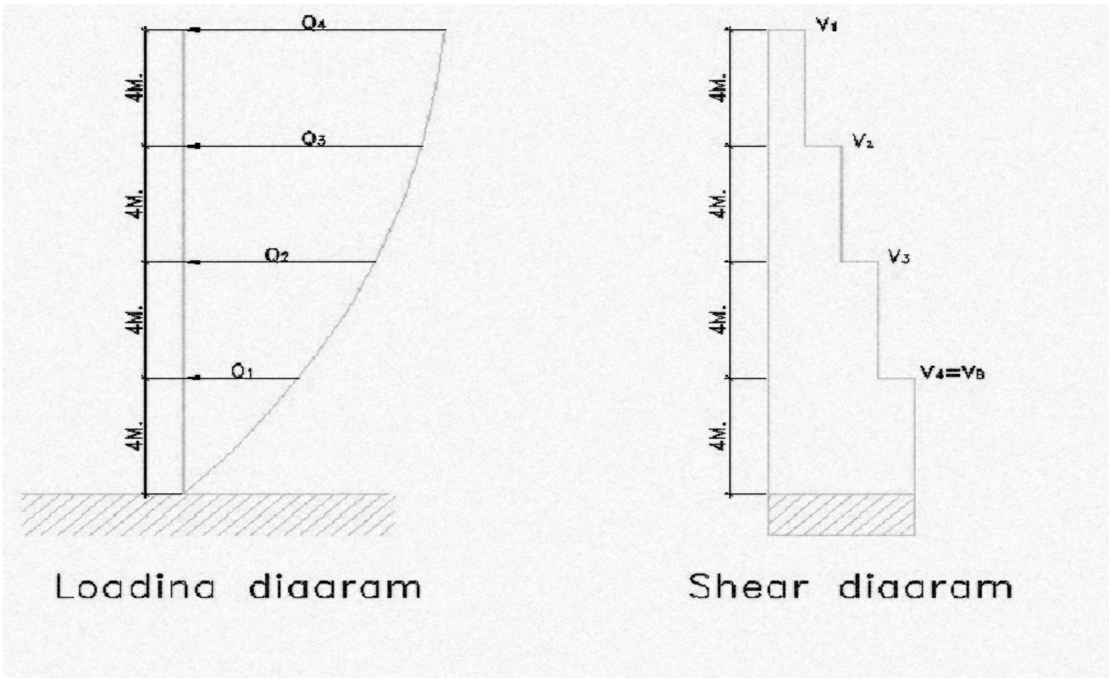

Figure 12: Loading diagram and Shear Diagram of case 2 response spectrum method

$Q_1 =$ 271.8447KN

$Q_2 =$ 590.3309 KN

$Q_3 =$ 826.6488 KN

$Q_4 =$ 960.8818 KN

$V_1 =$ 960.8818 KN

$V_2 =$ 1787.5306 KN

$V_3 =$ 2377.8615 KN

$V_B =$2.6497e+003 KN

Case III: Considering the stiffness of infill from top to bottom.

Same given building is analyzed by two same different methods as above when infill wall is provided at every floor, in this case stiffness of infill wall is calculated as stated below and after adding stiffness of columns as per the floor analyses is made by same different methods.

CALCULATION OF STIFFNESS FOR THE INFILL WALL

Stiffness of infill is determined by modeling the infill as in equivalent diagonal strut in which ,

width of strut, $W = \frac{1}{2}\sqrt{\alpha_h{}^2 + \alpha_l{}^2}$

$$\alpha_h = \frac{\pi}{2}[\frac{E_f I_c h}{2E_m t \, sin2\theta}]^{1/4}$$

$$\alpha_l = \pi[\frac{E_f I_b l}{E_m t \, sin2\theta}]^{1/4}$$

$$\theta = \tan^{-1}\frac{h}{l}$$

E_f = Elastic Modulus of frame material = $5000\sqrt{f_{ck}}$ = 22360.6798 x 10^3 KN/m^2

E_m = Elastic Modulus of masonary wall=13800000 KN/m^2

t= Thickness of infill wall=0.250m

h=height of infill wall=4.0 m

l=length of infill wall=3.7m

I_c = Moment of inertia of column=$\frac{.300\text{x}0.530^3}{12}$=3.7219 x$10^{-3}$ m^4

I_b = Moment of inertia of beam=$\frac{0.300\text{x}0.450^3}{12}$ = 2.278x$10^{-3}m^4$

$$\theta = \tan^{-1}\frac{h}{l} = 47.23^\circ$$

$$\alpha_h = \frac{\pi}{2}[\frac{22360\text{x}10^3\text{x}3.7219\text{x}10^{-3}\text{x}4}{2\text{x}13800\text{x}10^3\text{x}.25\text{x}0.9970}]^{1/4}$$

=.7367

$$\alpha_l = \pi[\frac{22360\text{x}10^3\text{x}2.2781\text{x}10^{-3}\text{x}3.7]^{1/4}}{2\text{x}13800\text{x}10^3\text{x}.25\text{x}0.9970}$$

=1.2781

$$W = \frac{1}{2}\sqrt{.7267^2 + 1.2781^2}$$

=.7376

A=W x t

=.7376 x0.25

=0.1844

$l_d = \sqrt{h^2 + l^2}$ = 5.449

\ Stiffness of infill is :

$$\frac{A\,E_m}{l_d}\cos^2\theta$$

=215372.7018 KN/m

Total Stiffness at floor level 1st ,2nd and 3rd =22 x 215372.7018+total stiffness of column at 1st or 2nd 3rd level

=4738199.44+442429.524

=5180628.964 KN/m

Total Stiffness at 4th floor level = 22 x 215372.7018+total stiffness of column at 4th floor

=4738199.44+318549.2573

= 5056748.697 KN/m

SEISMIC COEFFICIENT METHOD

```
n=4;
W=[3180 3180 3180 2300];
h=[4.0 4.0 4.0 4.0];
M=(1.0/9.81)*W;
Z=0.24;
I=1.5;
R=5;
g=9.81;
type=2;
TW=0.0;
H=0.0;
B1=40.70
for i=2:4

   H=H+h(i);
end
for i=1:4
   TW=TW+W(i);
```

```
end

% TO FIND OUT FUNDAMENTAL NATURAL PERIOD OF VIBRATION T
T=0.09*(H)/sqrt(B1);
T
% TO FIND OUT Sa/g VALUE
for i=1:n
if type==1
   if T<=0.1
      sag=1+15*T;
   elseif T<=0.4
      sag=2.50;
   else
      sag=1.0/T;
   end
elseif type==2
   if T<=0.1
      sag=1+15*T;
   elseif T<=0.55
      sag=2.5;
   else
      sag=1.36/T;
   end
else
   if type==3
      if T<=0.1
         sag=1+15*T;
      elseif T<=0.67
         sag=2.5;
      else
         sag=1.67/T;
      end
   end
end
end
sag
AH=(Z/2)*(I/R)*sag
VB=AH*TW;
VB
%VERTICAL DISTRIBUTION OF BASE SHEAR TO DIFFERENT FLOOR LEVEL
sumsquare=0.0;
%d=h(1)
d=0.0;
for i=1:n
   d=d+h(i)
   sumsquare=sumsquare+W(i)*(d^2);
   end
dd=0;
for i=1:n
   dd=dd+h(i);
   C(i)=(W(i)*(dd^2))/sumsquare;
      Q(i)=VB*C(i);
end
```

Q

B1 =Base dimension of the building at the plinth level along the considered direction of EQ=40.7000m

Fundamental Period T_a = 0.1693 sec

Sa/g = 2.5000

A_h = 0.0900

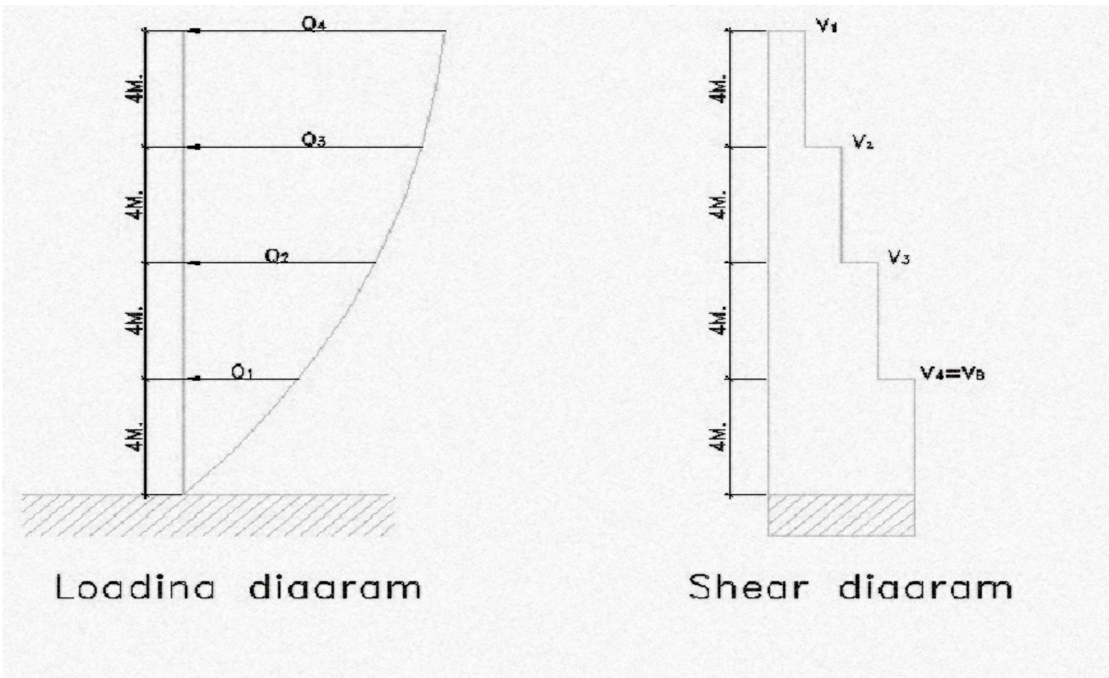

Figure 13: Loading diagram and Shear Diagram of case 3 sesmic coefficient method

V_B = 1.0656e+003 KN

Q_1 = 41.6700 KN

$Q_{2=}$ 166.6802 KN

$Q_{3=}$ 375.0304 KN

$Q_{4=}$ 482.2194 KN

V_1 = 482.2194 KN

V_2 = 857.2498 KN

V_3 = 1023.93 KN

V_B = 1.0656e+003 KN

RESPONSE SPECTRUM METHOD

```
%RESPONSE SPECTRUM METHOD
n=4;
w=[3180 3180 3180 2300];
k=[5180628.964 5180628.964 5180628.964  5056748.697];
m=(1.0/9.81)*w;
M=zeros(n,n);
K=zeros(n,n);
W=zeros(n,n);
evp=zeros(n);
Q=zeros(4);
for i=1:n
   M(i,i)=m(i);
   W(i,i)=w(i);
end
M
for i=1
   K(i,i)=k(i)+k(i+1);
      K(i,i+1)=-k(i+1);
end
for i=2:n
   if i<n
      K(i,i)=k(i)+k(i+1);
      K(i,i+1)=-k(i+1);
      K(i,i-1)=-k(i);
   else
      K(i,i)=k(i);
      K(i,i-1)=-k(i);
   end
end
for i=1:n
   for j=i+1:n
   K(j,i)=K(i,j);
   end
end
K
%CALCULATION OF TIME EIGEN VECTOR, EIGEN VALUE, TIME PERIOD
for i = 1:n
   for j = 1:n
      evp(i,j)= K(i,j)/M(i,i)
   end
end
[Evect,Evalue]=eig(evp);
%CALCULATION OF EIGEN VALUE (FREQUENCY)
sqrt(Evalue)
%CALCULATION OF EIGEN VECTOR
Evect
%CALCULATION OF NORMALISED EIGEN VECTOR
c1=(Evect')*M;
c2=c1*Evect;
c3=sqrt(c2);
c4=abs(c3);
```

```
xtmx=Evect/c4;
xtmx
%CAQLCULATION OF TIME PERIODS
T = 2*pi*eye(n)/sqrt(Evalue)
%CALCULATION OF PARTICIPATION FACTOR
for i=1:n
   sum=0.0;
sumsquare=0.0;
   for j=1:n
      sum=sum+M(j,j)*xtmx(j,i);
      sumsquare=sumsquare+M(j,j)*(xtmx(j,i))^2;
   end
   p(i)=sum/sumsquare;
end
p
%DETERMINATION OF MODAL MASS(MM)
for i=1:n
   sum=0.0;
   sumsquare=0.0;
   for j =1:n
   sum=sum+W(j,j)*xtmx(j,i);
   sumsquare=sumsquare+W(j,j)*(xtmx(j,i))^2;
   end
   MM(i,i)=(sum)^2/(9.81*sumsquare);
end
MM
%MODAL CONTRIBUTION OF VARIOUS MODES IN PERCENTAGE
sumMM =0.0;
for i =1:n
   sumMM =sumMM+MM(i,i);
end
for i=1:n
   MCM(i)=(MM(i,i)/sumMM)*100;
end
MCM
%CALCULATION OF LATERAL FORCES IN VARIOUS MODES
type=2;
for i=1:n
if type==1
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.4
      sag(i)=2.50;
   else
      sag(i)=1.0/T(i,i);
   end
elseif type==2
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.55
      sag(i)=2.5;
   else
      sag(i)=1.36/T(i,i);
```

```
    end
else
    if type==3
        if T(i,i)<=0.1
            sag(i)=1+15*T(i,i);
        elseif T(i,i)<=0.67
            sag(i)=2.5;
        else
            sag(i)=1.67/T(i,i);
        end
    end
end
end
sag
Z=0.24;I=1.5;R=5;
for i=1:n
    for j =1:n
    A(i)=(Z/2)*(I/R)*sag(i);
            Q(j,i)=A(i)*p(i)*xtmx(j,i)*M(j,j)*9.81;
    end
end
Q
%DETERMINATION OF STORY SHEAR FORCES IN EACH MODE
QQ=zeros(n);
for i=1:n
    sum=0.0;
    for j=1:n
        for k=j:n
            sum=sum+Q(k,i);
        end
            QQ(j,i)=sum;
            sum=0.0;
    end
end
QQ
%DETERMINATION OF STOREY SHEAR FORCE DUE TO ALL MODES(SRSS)
srssv=0.0;
laforce=zeros(n);
for i=1:n
    for j =1:n
        srssv=srssv+(QQ(i,j))^2;
        laforce(i,i)=(srssv)^0.5;
    end
    srssv =0.0;
end
laforce
%DETERMINATION OF BASE SHEAR
sum=0.0;
for i=1:n
    sum=sum+laforce(i,i);
end
    bshear=sum;
bshear
```

```
M =

  324.1590         0         0         0
         0  324.1590         0         0
         0         0  324.1590         0
         0         0         0  234.4546


K =

  1.0e+007 *

    1.0361   -0.5181         0         0
   -0.5181    1.0361   -0.5181         0
         0   -0.5181    1.0237   -0.5057
         0         0   -0.5057    0.5057


evp =

  1.0e+004 *

    3.1964   -1.5982         0         0
   -1.5982    3.1964   -1.5982         0
         0   -1.5982    3.1581   -1.5600
         0         0   -2.1568    2.1568


ans =

  239.7055         0         0         0
         0  199.4502         0         0
         0         0  132.8750         0
         0         0         0   46.7075


Evect =

   -0.3837   -0.6167   -0.5737    0.2347
    0.6121    0.3016   -0.5136    0.4374
   -0.5927    0.4691    0.1139    0.5803
    0.3562   -0.5556    0.6278    0.6456


xtmx =

   -0.0217   -0.0358   -0.0338    0.0139
    0.0346    0.0175   -0.0302    0.0258
   -0.0335    0.0272    0.0067    0.0343
    0.0201   -0.0323    0.0369    0.0381
```

```
T =

    0.0262         0         0         0
         0    0.0315         0         0
         0         0    0.0473         0
         0         0         0    0.1345


p =

   -1.9560   -4.6641   -9.9051   32.9126


MM =

  1.0e+003 *

    0.0038         0         0         0
         0    0.0218         0         0
         0         0    0.0981         0
         0         0         0    1.0832


MCM =

    0.3170    1.8024    8.1290   89.7516


sag =

    1.3932    1.4725    1.7093    2.5000


Q =

    6.7676   28.1592   65.4290  130.5528
  -10.7962  -13.7731   58.5756  243.2844
   10.4552  -21.4226  -12.9889  322.8064
   -4.5443   18.3494  -51.7903  259.7497


QQ =

    1.8824   11.3130   59.2254  956.3932
   -4.8852  -16.8463   -6.2036  825.8404
    5.9109   -3.0732  -64.7792  582.5561
   -4.5443   18.3494  -51.7903  259.7497


laforce =

  958.2938         0         0         0
         0  826.0500         0         0
```

0   0 586.1845   0
0   0   0 265.5362

bshear =

2.6361e+003

Q_1 = 265.5362 KN

V_1 = 958.2938 KN

Q_2=586.1845 KN

V_2 = 1784.3438 KN

Q_3 =826.0500 KN

V_3 = 2370.5283 KN

Q_4 =958.2938 KN

V_B = 2.6361e+003KN

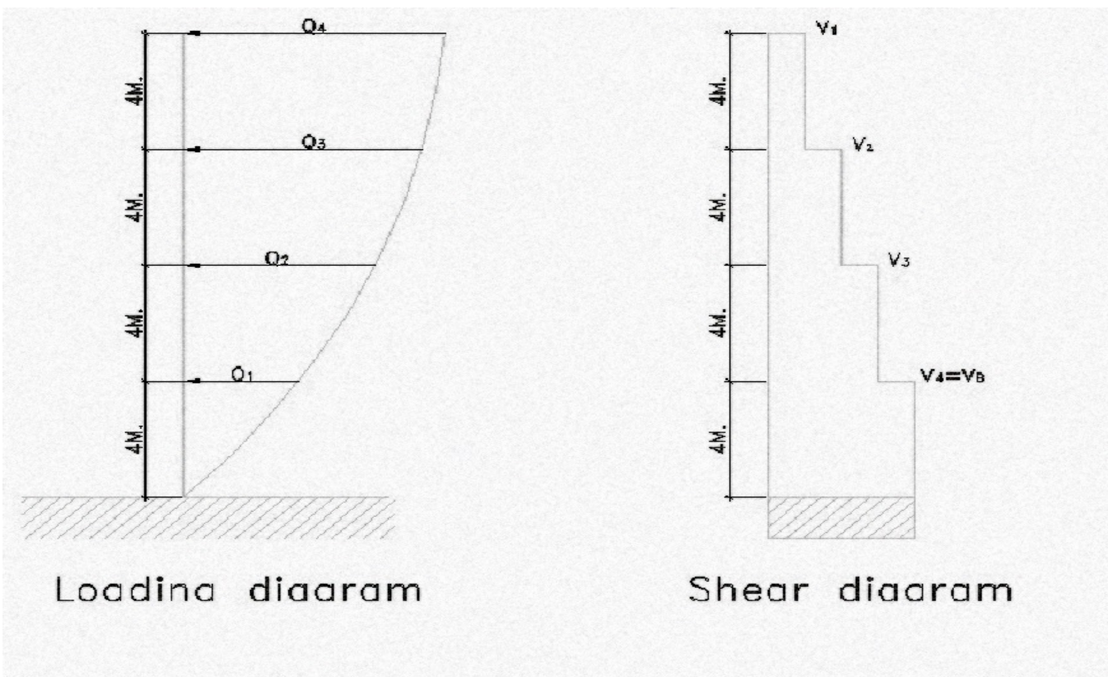

Figure 14: Loading diagram and Shear Diagram of case 3 response spectrum method

CASE IV : THE GROUND FLOOR IS USED FOR PARKING AND THE ABOVE STORIES ARE PROVIDED WITH INFILL WALLS.

Same given building is analyzed by two same different methods as above when infill wall is provided at every floor excluding ground floor this is also known as soft storey, analyses is made by same different methods.

SEISMIC COEFFICIENT METHOD

Fundamental Period T_a = 0.6 sec

Sa/g = 2.2667

A_h =0.0816

Q_1 =37.7808 KN

$Q_{2=}$151.1234 KN

$Q_{3=}$340.0276 KN

$Q_{4=}$437.2122 KN

V_1 = 437.2122 KN

V_2 = 777.2398 KN

V_3 = 928.3632 KN

V_B =966.1440 KN

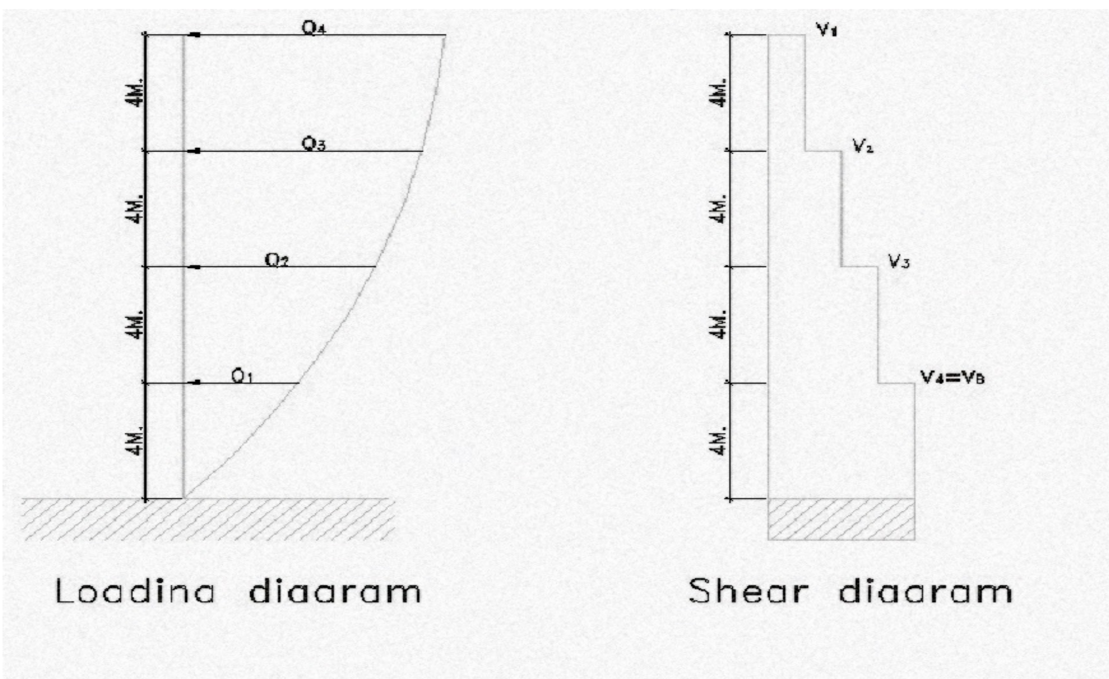

Figure 15: Loading diagram and Shear Diagram of case 4 sesmic coefficient method

RESPONSE SPECTRUM METHOD

```
%RESPONSE SPECTRUM METHOD
n=4;
w=[3180 3180 3180 2300];
```

```
k=[442429.524 5180628.964 5180628.964  5056748.697];
m=(1.0/9.81)*w;
M=zeros(n,n);
K=zeros(n,n);
W=zeros(n,n);
evp=zeros(n);
Q=zeros(4);
for i=1:n
   M(i,i)=m(i);
   W(i,i)=w(i);
end
M
for i=1
   K(i,i)=k(i)+k(i+1);
       K(i,i+1)=-k(i+1);
end
for i=2:n
   if i<n
       K(i,i)=k(i)+k(i+1);
       K(i,i+1)=-k(i+1);
       K(i,i-1)=-k(i);
   else
       K(i,i)=k(i);
       K(i,i-1)=-k(i);
   end
end
for i=1:n
   for j=i+1:n
   K(j,i)=K(i,j);
   end
end
K
%CALCULATION OF TIME EIGEN VECTOR, EIGEN VALUE, TIME PERIOD
for i = 1:n
   for j = 1:n
       evp(i,j)= K(i,j)/M(i,i)
   end
end
[Evect,Evalue]=eig(evp);
%CALCULATION OF EIGEN VALUE (FREQUENCY)
sqrt(Evalue)
%CALCULATION OF EIGEN VECTOR
Evect
%CALCULATION OF NORMALISED EIGEN VECTOR
c1=(Evect')*M;
c2=c1*Evect;
c3=sqrt(c2);
c4=abs(c3);
xtmx=Evect/c4;
xtmx
%CAQLCULATION OF TIME PERIODS
T = 2*pi*eye(n)/sqrt(Evalue)
%CALCULATION OF PARTICIPATION FACTOR
```

```
for i=1:n
   sum=0.0;
sumsquare=0.0;
   for j=1:n
      sum=sum+M(j,j)*xtmx(j,i);
      sumsquare=sumsquare+M(j,j)*(xtmx(j,i))^2;
   end
   p(i)=sum/sumsquare;
end
p
%DETERMINATION OF MODAL MASS(MM)
for i=1:n
   sum=0.0;
   sumsquare=0.0;
   for j =1:n
   sum=sum+W(j,j)*xtmx(j,i);
   sumsquare=sumsquare+W(j,j)*(xtmx(j,i))^2;
   end
   MM(i,i)=(sum)^2/(9.81*sumsquare);
end
MM
%MODAL CONTRIBUTION OF VARIOUS MODES IN PERCENTAGE
sumMM =0.0;
for i =1:n
   sumMM =sumMM+MM(i,i);
end
for i=1:n
   MCM(i)=(MM(i,i)/sumMM)*100;
end
MCM
%CALCULATION OF LATERAL FORCES IN VARIOUS MODES
type=2;
for i=1:n
if type==1
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.4
      sag(i)=2.50;
   else
      sag(i)=1.0/T(i,i);
   end
elseif type==2
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.55
      sag(i)=2.5;
   else
      sag(i)=1.36/T(i,i);
   end
else
   if type==3
      if T(i,i)<=0.1
         sag(i)=1+15*T(i,i);
```

```
        elseif T(i,i)<=0.67
            sag(i)=2.5;
        else
            sag(i)=1.67/T(i,i);
        end
    end
end
end
sag
Z=0.24;I=1.5;R=5;
for i=1:n
    for j =1:n
    A(i)=(Z/2)*(I/R)*sag(i);
            Q(j,i)=A(i)*p(i)*xtmx(j,i)*M(j,j)*9.81;
    end
end
Q
%DETERMINATION OF STORY SHEAR FORCES IN EACH MODE
QQ=zeros(n);
for i=1:n
    sum=0.0;
    for j=1:n
        for k=j:n
            sum=sum+Q(k,i);
        end
            QQ(j,i)=sum;
            sum=0.0;
    end
end
QQ
%DETERMINATION OF STOREY SHEAR FORCE DUE TO ALL MODES(SRSS)
srssv=0.0;
laforce=zeros(n);
for i=1:n
    for j =1:n
        srssv=srssv+(QQ(i,j))^2;
        laforce(i,i)=(srssv)^0.5;
    end
    srssv =0.0;
end
laforce
%DETERMINATION OF BASE SHEAR
sum=0.0;
for i=1:n
    sum=sum+laforce(i,i);
end
    bshear=sum;
bshear
```

M =

324.1590	0	0	0
0	324.1590	0	0
0	0	324.1590	0
0	0	0	234.4546

K =

1.0e+007 *

0.5623	-0.5181	0	0
-0.5181	1.0361	-0.5181	0
0	-0.5181	1.0237	-0.5057
0	0	-0.5057	0.505

evp =

1.0e+004 *

1.7347	-1.5982	0	0
-1.5982	3.1964	-1.5982	0
0	-1.5982	3.1581	-1.5600
0	0	-2.1568	2.1568

```
ans =

  236.6506         0         0         0
         0  186.7845         0         0
         0         0   18.5155         0
         0         0         0  105.9472


Evect =

   -0.2459   -0.4705   -0.4655   -0.6325
    0.5947    0.5164   -0.4953   -0.2423
   -0.6487    0.3760   -0.5145    0.3181
    0.4063   -0.6088   -0.5228    0.6633


xtmx =

   -0.0140   -0.0276   -0.0269   -0.0375
    0.0338    0.0303   -0.0286   -0.0144
   -0.0369    0.0220   -0.0297    0.0189
    0.0231   -0.0357   -0.0302    0.0393
```

T =

0.0266	0	0	0
0	0.0336	0	0
0	0	0.3393	0
0	0	0	0.0593

p =

-0.1104	-0.3498	-34.7075	-1.4776

MM =

1.0e+003 *

0.0000	0	0	0
0	0.0001	0	0
0	0	1.2046	0
0	0	0	0.0022

MCM =

```
    0.0010    0.0101   99.8080    0.1809
```

sag =

```
    1.3983    1.5046    2.5000    1.8896
```

Q =

```
    0.2471    1.6620  267.1436   11.9821
   -0.5977   -1.8243  284.2274    4.5897
    0.6520   -1.3281  295.2142   -6.0263
   -0.2953    1.5554  216.9684   -9.0887
```

QQ =

```
  1.0e+003 *

    0.0000    0.0001    1.0636    0.0015
```

-0.0002	-0.0016	0.7964	-0.0105
0.0004	0.0002	0.5122	-0.0151
-0.0003	0.0016	0.2170	-0.0091

laforce =

1.0e+003 *

1.0636	0	0	0
0	0.7965	0	0
0	0	0.5124	0
0	0	0	0.2172

bshear = 2.5896e+003

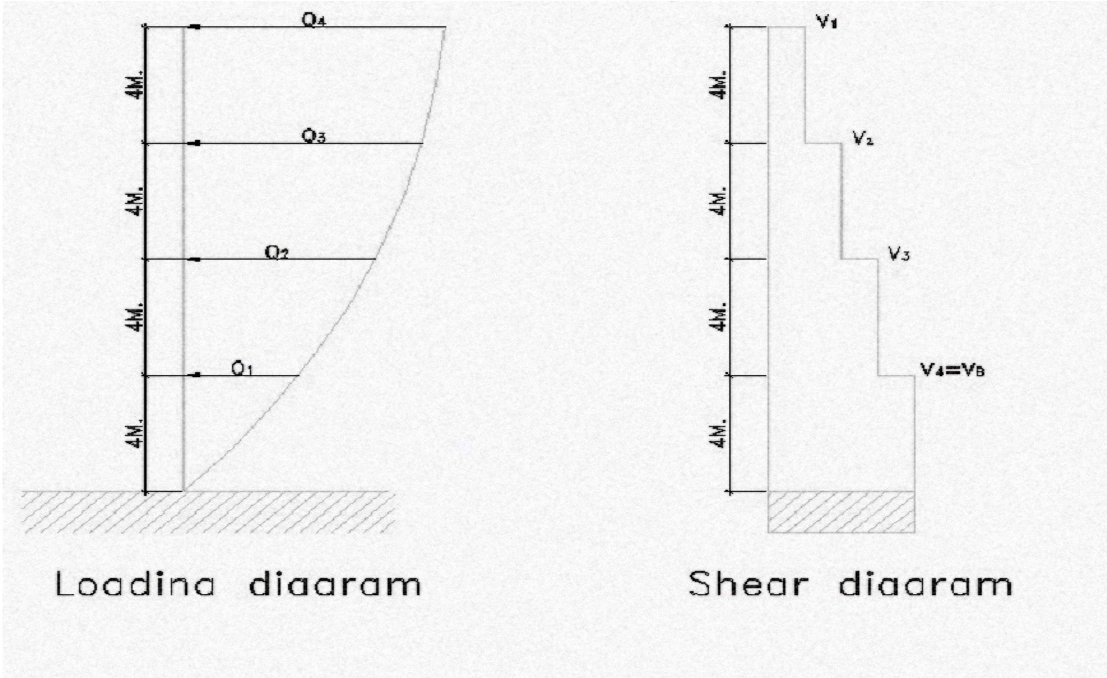

Figure 16: Loading diagram and Shear Diagram of case 4 response spectrum method

Q_1 = 217.2 KN

$Q_{2=}$ 512.4 KN

$Q_{3=}$ 796.5 KN

$Q_{4=}$1063.6 KN

V_1 = 1063.6 KN

V_2 = 1860.1 KN

V_3 = 2372.5 KN

V_B = 2.5896e+003 KN

CASE V : INFILL WALL IS PROVIDED AT ALTERNATE FLOOR LEVELS

Same given building is analyzed by two same different methods as above when infill wall is provided at alternate level in this case it assumed that in 1st floor infill wall is provided , analyses is made by same different methods

SEISMIC COEFFICIENT METHOD

B1 =Base dimension of the building at the plinth level along the considered direction of EQ=40.7000m

Fundamental Period T_a = 0.1693 sec

Sa/g = 2.5000

A_h = 0.0900

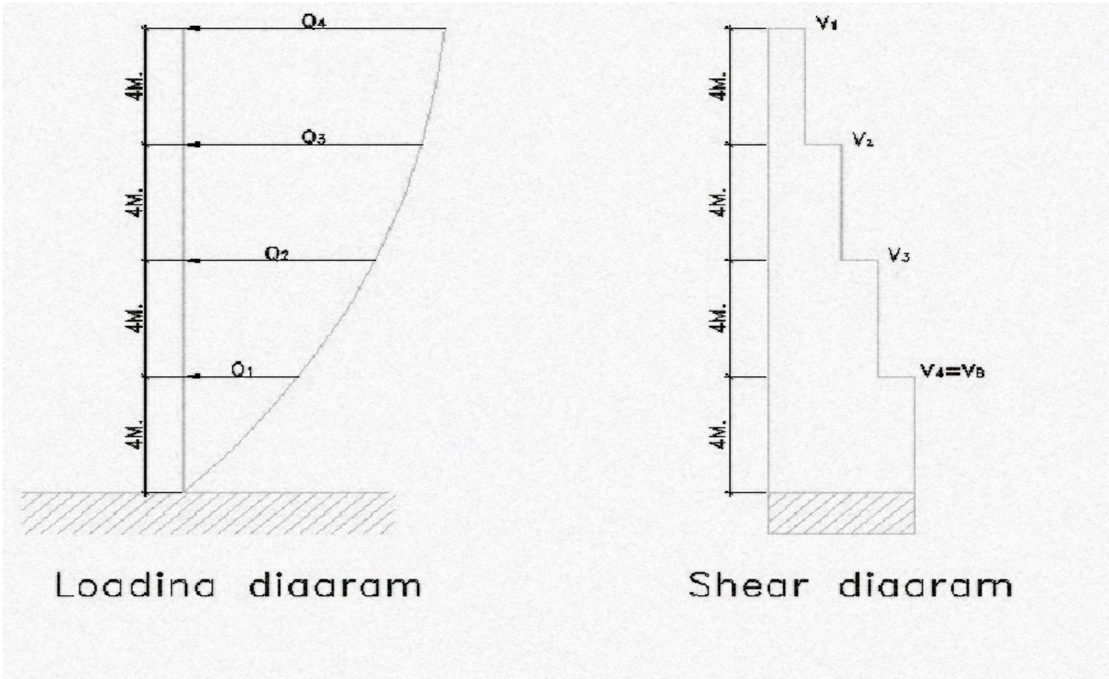

Figure 17: Loading diagram and Shear Diagram of case 5 sesmic coefficient method

Q_1 = 41.6700 KN

Q_2 = 166.6802 KN

Q_3 = 375.0304 KN

$Q_{4=}$ 482.2194 KN

V_1 = 482.2194 KN

V_2 = 857.2498 KN

V_3 = 1023.93 KN

V_B = 1.0656e+003 KN

RESPONSE SPECTRUM METHOD

```
%RESPONSE SPECTRUM METHOD
n=4;
w=[3180 3180 3180 2300];
k=[5180628.964 442429.524 5180628.964 318549.2573];
m=(1.0/9.81)*w;
M=zeros(n,n);
K=zeros(n,n);
W=zeros(n,n);
evp=zeros(n);
Q=zeros(4);
for i=1:n
   M(i,i)=m(i);
```

```
    W(i,i)=w(i);
end
M
for i=1
    K(i,i)=k(i)+k(i+1);
        K(i,i+1)=-k(i+1);
end
for i=2:n
    if i<n
        K(i,i)=k(i)+k(i+1);
        K(i,i+1)=-k(i+1);
        K(i,i-1)=-k(i);
    else
        K(i,i)=k(i);
        K(i,i-1)=-k(i);
    end
end
for i=1:n
    for j=i+1:n
    K(j,i)=K(i,j);
    end
end
K
%CALCULATION OF TIME EIGEN VECTOR, EIGEN VALUE, TIME PERIOD
for i = 1:n
    for j = 1:n
        evp(i,j)= K(i,j)/M(i,i)
    end
end
[Evect,Evalue]=eig(evp);
%CALCULATION OF EIGEN VALUE (FREQUENCY)
sqrt(Evalue)
%CALCULATION OF EIGEN VECTOR
Evect
%CALCULATION OF NORMALISED EIGEN VECTOR
c1=(Evect')*M;
c2=c1*Evect;
c3=sqrt(c2);
c4=abs(c3);
xtmx=Evect/c4;
xtmx
%CAQLCULATION OF TIME PERIODS
T = 2*pi*eye(n)/sqrt(Evalue)
%CALCULATION OF PARTICIPATION FACTOR
for i=1:n
    sum=0.0;
sumsquare=0.0;
    for j=1:n
        sum=sum+M(j,j)*xtmx(j,i);
        sumsquare=sumsquare+M(j,j)*(xtmx(j,i))^2;
    end
    p(i)=sum/sumsquare;
end
```

```
p
%DETERMINATION OF MODAL MASS(MM)
for i=1:n
   sum=0.0;
   sumsquare=0.0;
   for j =1:n
   sum=sum+W(j,j)*xtmx(j,i);
   sumsquare=sumsquare+W(j,j)*(xtmx(j,i))^2;
   end
   MM(i,i)=(sum)^2/(9.81*sumsquare);
end
MM
%MODAL CONTRIBUTION OF VARIOUS MODES IN PERCENTAGE
sumMM =0.0;
for i =1:n
   sumMM =sumMM+MM(i,i);
end
for i=1:n
   MCM(i)=(MM(i,i)/sumMM)*100;
end
MCM
%CALCULATION OF LATERAL FORCES IN VARIOUS MODES
type=2;
for i=1:n
if type==1
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.4
      sag(i)=2.50;
   else
      sag(i)=1.0/T(i,i);
   end
elseif type==2
   if T(i,i)<=0.1
      sag(i)=1+15*T(i,i);
   elseif T(i,i)<=0.55
      sag(i)=2.5;
   else
      sag(i)=1.36/T(i,i);
   end
else
   if type==3
      if T(i,i)<=0.1
         sag(i)=1+15*T(i,i);
      elseif T(i,i)<=0.67
         sag(i)=2.5;
      else
         sag(i)=1.67/T(i,i);
      end
   end
end
end
sag
```

```
Z=0.24;I=1.5;R=5;
for i=1:n
    for j =1:n
    A(i)=(Z/2)*(I/R)*sag(i);
            Q(j,i)=A(i)*p(i)*xtmx(j,i)*M(j,j)*9.81;
    end
end
Q
%DETERMINATION OF STORY SHEAR FORCES IN EACH MODE
QQ=zeros(n);
for i=1:n
    sum=0.0;
    for j=1:n
        for k=j:n
            sum=sum+Q(k,i);
        end
            QQ(j,i)=sum;
            sum=0.0;
    end
end
QQ
%DETERMINATION OF STOREY SHEAR FORCE DUE TO ALL MODES(SRSS)
srssv=0.0;
laforce=zeros(n);
for i=1:n
    for j =1:n
        srssv=srssv+(QQ(i,j))^2;
        laforce(i,i)=(srssv)^0.5;
    end
    srssv =0.0;
end
laforce
%DETERMINATION OF BASE SHEAR
sum=0.0;
for i=1:n
    sum=sum+laforce(i,i);
end
    bshear=sum;
bshear



M =

  324.1590         0         0         0
         0  324.1590         0         0
         0         0  324.1590         0
         0         0         0  234.4546


K =

  1.0e+006 *
```

```
    5.6231   -0.4424         0         0
   -0.4424    5.6231   -5.1806         0
         0   -5.1806    5.4992   -0.3185
         0         0   -0.3185    0.3185


evp =

  1.0e+004 *

    1.7347   -0.1365         0         0
   -0.1365    1.7347   -1.5982         0
         0   -1.5982    1.6964   -0.0983
         0         0   -0.1359    0.1359


ans =

  182.2596         0         0         0
         0  131.6983         0         0
         0         0   20.0112         0
         0         0         0   45.3089


Evect =

    0.0611    0.9963   -0.0386   -0.0385
   -0.7105    0.0016   -0.4796   -0.4319
    0.7004   -0.0851   -0.5052   -0.4100
   -0.0299    0.0072   -0.7164    0.8024


xtmx =

    0.0034    0.0553   -0.0023   -0.0024
   -0.0395    0.0001   -0.0288   -0.0265
    0.0389   -0.0047   -0.0303   -0.0251
   -0.0017    0.0004   -0.0430    0.0492


T =

    0.0345         0         0         0
         0    0.0477         0         0
         0         0    0.3140         0
```

```
         0         0         0    0.1387


p =

    0.5293   16.5294  -29.9653   -5.9592


MM =

    0.2802         0         0         0
         0  273.2202         0         0
         0         0  897.9191         0
         0         0         0   35.5123


MCM =

    0.0232   22.6376   74.3968    2.9424


sag =

    1.5171    1.7156    2.5000    2.5000


Q =

    0.3120  179.6573   19.8641    4.0275
   -3.6281    0.2846  246.6345   45.1294
    3.5766  -15.3428  259.8211   42.8425
   -0.1103    0.9432  266.4530  -60.6456


QQ =

    0.1501  165.5423  792.7728   31.3538
   -0.1619  -14.1150  772.9087   27.3263
    3.4662  -14.3997  526.2742  -17.8031
   -0.1103    0.9432  266.4530  -60.6456


laforce =

  810.4789         0         0         0
         0  773.5204         0         0
         0         0  526.7835         0
         0         0         0  273.2691


bshear =

  2.3841e+003
```

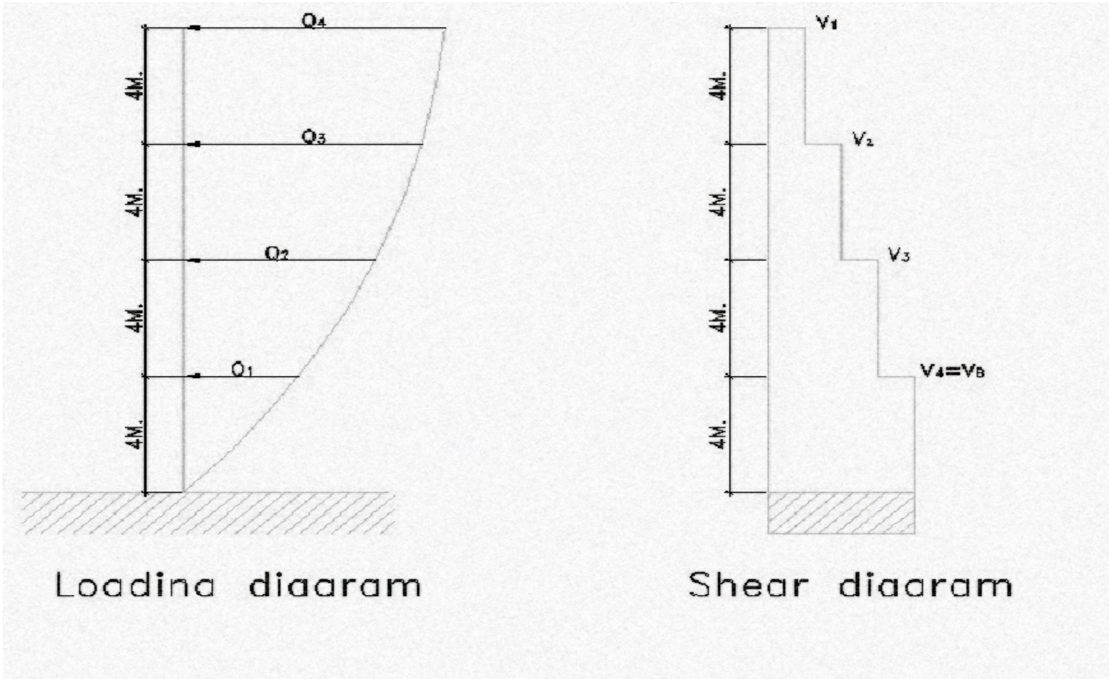

Figure 18: Loading diagram and Shear Diagram of case 5 response spectrum method

Q_1 = 273.2691 KN

Q_2 = 526.7835 KN

Q_3 =773.5204 KN

Q_4 = 810.4789 KN

V_1 = 810.4789 KN

V_2 = 1583.9993 KN

V_3 = 2110.7828 KN

V_B = 2.3841e+003 KN

Chapter 7

RESULTS & CONCLUSION

TABULAR COMPARISON FOR BASE SHEAR & LATERAL FORCES FOR ALL CASES:

LATERAL FORCES	CASE1	CASE1	CASE2	CASE2	CASE3	CASE 3	CASE 4	CASE 4	CASE 5	CASE5
METHOD USED	S.C.M	R.S.M	S.C.M	R.S.M	S.C.M	R.S.M	S.C.M	R.S.M	S.C.M	R.S.M
STOREY I (KN)	37.7808	280.7799	37.7808	271.8447	41.6700	265.5362	37.7808	217.2	41.67	273.2691
STOREY II (KN)	151.1234	589.8808	151.1234	590.3309	166.6802	586.1845	151.1234	512.4	166.6802	526.7835
STOREY III (KN)	340.0276	822.8973	340.0276	826.6488	375.0304	826.0500	340.0276	796.5	375.0304	773.5204
STOREY IV (KN)	437.2122	954.4971	437.2122	960.8818	482.2194	958.2938	437.2122	1063.6	482.2194	810.4789
BASE SHEAR (KN)	966.144	2648.055	966.144	2649.7062	1065.60	2636.0645	966.144	2589.7	1065.60	2384.0519

Case 1: When no infill wall is provided.

Case 2: When stiffness of each floor is same.

Case 3: Considering the stiffness of infill from top to bottom.

Case 4: The ground floor is used for parking and the above stories are provided with infill walls.

Case 5: Infill wall is provided at alternate floor levels.

FROM THE RESULTS SHOWN IN TABULAR FORM THE FOLLOWING CONCLUSIONS ARE DRAWN :

1. There is substantial change in the lateral forces when any type of unsymmetrical like soft or weak storey, Infill wall etc. is incorporated in the building.

2. Base shear is more or less of the same magnitude irrespective of change in configuration of the building if we analysis by response spectrum method.

3. The most optimum case is observed when the infill walls are provided at alternate floor level.

4. The most severe case is observed when the Ground floor is devoid of infill walls.

5. Calculation reveals that, when RC framed buildings having brick masonry infill on upper floor with soft ground floor is subjected to earthquake loading, base shear can be more than twice to that predicted by equivalent earthquake force method with or without infill. Since response spectrum method is seldom used in practice for the design of such buildings, it can be suggested that the base shear calculated by equivalent static method may at least be doubled for the safer design of the columns of soft ground floor.

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