NON LINEAR CONSIDERING MASS AND Stiffness IRREGULARITIES ANALYSIS

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ABSTRACT

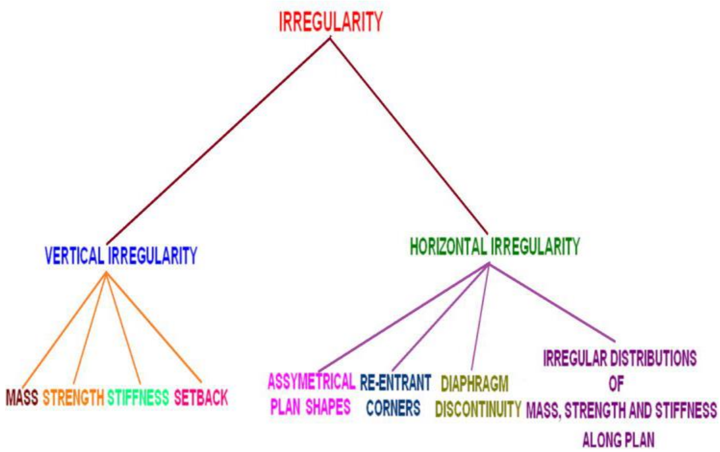
A structure can be classified as vertically irregular if it contains irregular distribution of mass, strength and stiffness along the building height. As per IS 1893:2002, a storey in a building is said to contain mass irregularity if its mass exceeds 200% than that of the adjacent storey. If stiffness of a storey is less than 60% of the adjacent storey, then a storey is termed as „weak storey". If stiffness of a storey is less than 70% or above as compared to the adjacent storey, then the storey is termed as „soft storey". In reality, many existing buildings contain irregularity, and some of them have been designed initially to be irregular to fulfill different functions e.g. basements for commercial purposes created by eliminating central columns. Also, reduction of size of beams and columns in the upper storeys to fulfill functional requirements and for other commercial purposes like storing heavy mechanical appliances etc. This difference in usage of a specific floor with respect to the adjacent floors results in irregular distributions of mass, stiffness and strength along the building height. In addition, many other buildings are accidentally rendered irregular due to variety of reasons like non-uniformity in construction practices and material used. The building can have irregular distributions of mass, strength and stiffness along plan also. In such a case it can be said that the building has a horizontal irregularity.

INTRODUCTION

1.1 GENERAL

The component of the building, which resists the seismic forces, is known as lateral force resisting system (L.F.R.S). The L.F.R.S of the building may be of different types. The most common forms of these systems in a structure are special moment resisting frames, shear walls and frame-shear wall dual systems. The damage in a structure generally initiates at location of the structural weak planes present in the building systems. These weaknesses trigger further structural deterioration which leads to the structural collapse. These weaknesses often occur due to presence of the structural irregularities in stiffness, strength and mass in a building system. The structural Irregularity can be broadly classified as plan and vertical irregularities.

ed classification of structural irregularity is presented in Figure shown in Figures 1.2 to 1.4. The past earthquake records show poor seismic performance of these structures during earthquakes as discussed in the next section. The different types of irregularities are presented from Figures 1.5 to 1.8.



India has experienced many major earthquakes in the past century, as high as 8.7 magnitudes and has lost huge number of lives and properties. Keeping in view there is an urgency to recheck the safety of the structure which still exists. There are many analytical methods developed in the past few decades to evaluate the seismic demand of the structure so as to predict their behavior during the earthquake and thereby take up the suitable retrofitting measures and hence assure safety against earthquake and minimise the losses. During an earthquake the principal attack on a structure is by transient horizontal forces. The earthquake resistance of the structure depends on a combination of elastic strength, inelastic deformability and damping capacity. The relative effectiveness of these three factors depends in part on the character of the attacking earthquake.

Typical earthquakes have one of four characteristics:

Type 1 earthquakes are impulsive with a single dominant lurch in one direction. Structures may resist these earthquakes by a combination of elastic strength and inelastic deformability.

Type 2 earthquakes are of long duration with irregular "noise-like" ground motions. Structures resist them by elastic strength, inelastic deformability.

Type 3 earthquakes are of long duration and have regular motions with one or more dominant periods. They are a consequent of the partial resonance of flexible ground and are therefore a micro zone effect. Avoidance of such near-coincidence of periods may be impeded by architectural requirements, by difficulty in predicting earthquake periods, and by increases in ground period with increasing ground strains.

Type 4 earthquakes are those which include severe ground damage in addition to the inertia attack; Economy of design is achieved by allowing a structure to deform well into the inelastic range during severe earthquakes. Effective earthquake resistance may be obtained provided the structure has adequate capacity for inelastic deformation. The capacity of a structure to deform in elastically is expressed as a ductility factor, defined as the ratio of 'the maximum deflection.

The methods like elastic and nonlinear methods are in use in the seismic evaluation of the structures. The non linear methods can determine the post elastic behavior of the members and hence it is most preferred method of analysis.

1.2 THE ROLE AND USE OF NONLINEAR ANALYSIS IN SEISMIC DESIGN

While buildings are usually designed for seismic resistance using elastic analysis, most will experience significant inelastic deformations under large earthquakes. Modern performance-based design methods require ways to determine the realistic behavior of structures under such conditions. Enabled by advancements in computing technologies and available test data, nonlinear analyses provide the means for calculating structural response beyond the elastic range, including strength and stiffness deterioration associated with inelastic material behavior and large displacements. As such, nonlinear analysis can play an important role in the design of new and existing buildings.

Nonlinear analyses involve significantly more effort to perform and should be approached with specific objectives in mind. Typical instances where nonlinear analysis is applied in structural earthquake engineering practice are to: (1) assess and design seismic retrofit solutions for existing buildings; (2) Design new buildings that employ structural materials, systems, or other features that do not conform to current building code requirements; (3) assess the performance of buildings for specific owner/stakeholder requirements. If the intent of using a nonlinear analysis is to justify a design that would not satisfy the prescriptive building code requirements,

it is essential to develop the basis for acceptance with the building code authority at the outset of a project. The design basis should be clearly defined and agreed upon, outlining in specific terms all significant

performance levels and how they will be evaluated.

1.3 REGULAR AND IRREGULAR STRUCTURES

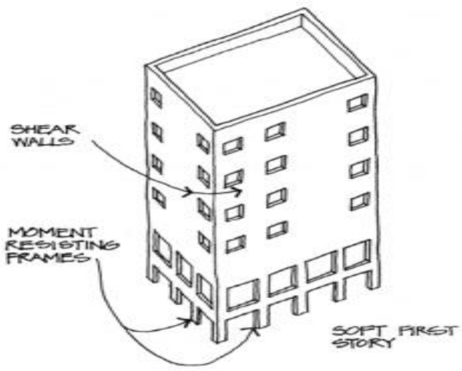
Structures are designated as structurally regular or irregular. A regular structure has no significant discontinuities in plan, vertical configuration, or lateral force resisting systems. An irregular structure, on the other hand, has significant discontinuities such as those in ASCE 7 Tables 12.3-1 (horizontal irregularities) and 12.32 (vertical irregularities).Regular and symmetrical structures exhibitmore favourable and predictable seismic response characteristics than irregular

Structures.Therefore, the use of irregular structures in earthquake-prone areas should be avoided if possible.However, the IBC does not prohibit irregularstructures. Instead, it contains specific design requirements for each type of irregularity. In some cases of irregularity, the static lateral force procedure as described in this lesson is not permitted, and a dynamic procedure is required. The following is a summary of some recommended design practices concerningstructural regularity.

1. Structures should be regular in stiffness and geometry, both in plan andelevation.

2. Abrupt changes of shape, stiffness, or resistance, such as a soft first story, Should be avoided.

3. Portions of a building thSat is different in size, shape, or rigidity seismic separation, as shown in Figures



Stiffness irregularity Vertical geometric irregularity

1.4 OBJECTIVES OF THE STUDY

The present work aims at the following objectives:

- Study of Seismic demands of regular and irregular R.C buildings using the Non-linear time history analysis.
- To assess the building performances in terms of overall deflection, inter-story drift, and joint acceleration.
- To obtain the behaviour of regular and irregular R.C buildings considering mass and stiffness irregularities using NLTH.
- Comparative analysis of Rcc building with and with base isolator

1.5SCOPE OF THE STUDY

Validation of the results are done by comparing the results of Response spectrum and Time history analysis for Regular and Irregular Buildings considering mass and stiffness irregularity were analysed using SAP2000 software

In the present work models of a fifteen storey buildings irregular in the plan and Regular located in Different seismic zones with medium soil are analysed by response spectrum method time history method using SAP2000. It includes finding Base Shear and Time period for given structure from the results of Modal analysis and to find the variation of Relative Results with and without Base Isolator in Different Seismic zones.

1.6 ORGANIZATION OF THESIS

Chapter 1 is the discussion of the importance of earthquake hazards analysis procedures adopted for the structures and a brief introduction to the earthquake design philosophy in the seismic codes of India. The scope and objective of the study has also been discussed.

Chapter 2 deals with the various literatures that have been published on the energy based pushover analysis and seismic evaluation techniques.

Chapter 3 covers the complete study on nonlinear time history analysis which is divides into nonlinear direct integration analysis and nonlinear modal time history analysis and the appropriate procedure to be adopted in the present work

Chapter 4 completely takes care of the case study of a building under consideration and the various building data surveys done to gather the information for modeling of the structure.

Chapter 5 Copes with the numerical study and presentation of results of NLTHA with and without seismic control devices for the current building under study.

Chapter 6 Details about the discussions drawn based on the present work and the scope for the further study.

1.7 SUMMARYIn this chapter, the importance of earthquake and the post disaster effects of itand some light has been thrown on Non-linear Time History Analysis.The objectives are also been discussed.Based on the objective of the present study, research papers were collected and studied thoroughly. The review of research papers is discussed in the next chapter named as literature review.

CHAPTER II

LITERATURE REVIEW

ARMAN CHOWDHURY, WAHID HASSAN (2013) Studied the comparative of the Dynamic Analysis of multi-story irregular Building with or without Base Isolator. The Basic objective ofthe research is to perform nonlinear dynamic analysis of building with Isolated Bearing and non-isolated one. The investigation gives emphasis on the feasibility of incorporationof isolators and its structural implications in buildings in Dhaka. He observed that, the maximum displacements of buildings in different storeys in x and y direction the displacement is significant in first five stories and difference between displacements is decreasing with the increasing storey height. It is observed that the relative displacement between stories after using isolator is much less than before. Finally, he concluded axial force on column will be Reduced which will reducethe Design reinforcement for column. So, Base isolation is Economical under design consideration.

HABIBIANAASADI (2013); Studied the Seismic Performance of RC Frames Irregular in Elevation Designed Based on Iranian Seismic Code. This Paper addresses multistory Reinforced Concrete (RC) frame buildings, regular and irregular in elevation. Several multistory Reinforced Concrete Moment Resisting Frames (RCMRFs) with different types of setbacks, as well as the regular frames in elevation, are designed according to the provisions of the Iranian national building code and Iranianseismic code for the high ductility class. Inelastic dynamic time-history analysis is performed on all frames subjected to ten input motions. The assessment of the seismic performance is done based on both global and local criteria. Results show that when setback occurs in elevation, the requirements of the life safety level are not satisfied. It is also shown that the elements near the setback experience the maximum damage. Therefore it is necessary to strengthen these elements by appropriate method to satisfy the life safety level of the frames.

LADJINOVIC AND FOLIC (2008): Studied the seismic analysis of asymmetric in plan buildings. The good behaviour of the structure can be provided with a well distributed lateral load resisting system. The inelastic seismic behaviour of asymmetric-plan buildings is considered by using the histories of base shear and torque (BST). The procedure to construct the BST surface of the system with an arbitrary number of resisting elements in the direction of asymmetry and of ground motion is proposed. The BST surface describes the inelastic properties of a system.

ZBIGNIEWZEMBATY (2007)studied the spatial seismic excitations and response spectra. The Formal extensions of the response spectrum method to include spatial seismic effects are reviewed. Two approaches are described in detail: the first based on random vibrations of simple oscillator undertow-component excitations, and the second analyzing multi-column building seismic response. The subjective choice of these two complementing approaches aims at analyzing the phenomenon of spatial seismic vibrations of structures from a broader physical perspective of various wave types propagatingamong structural supports, with detailed random vibration sensitivity analysis of a simple structuralsystem still included.

SIGMUND FREEMAN (2007) : Studied the Response spectra as a useful design and analysis toolfor practicing structural engineers. The purpose of this paper is to review the concept of response spectra for design engineers not familiar with their significance and to summarize a variety of uses that can be applied for purposes such as rapid evaluation for a large inventory of buildings, performance verification of new construction, evaluation of existing structures for seismic vulnerability, and post earthquake estimates of potential damage of buildings.

DEVESH AND BHARAT (2006) A review of studies on the seismic behavior of vertically irregular structures along with their findings has been presented. It is observed that building codes provide criteria to classify the vertically irregular structures and suggest dynamic analysis to arrive at design lateral forces. Most of the studies agree on the increase in drift demand in the tower portion of set-back structures and on the increase in seismic demand for buildings with discontinuous distributions in mass, stiffness, and strength. The largestseismic demand is found for the combined-stiffness-and-strength irregularity.

KHOURY ET AL.,(2005), Considered four nine storey asymmetric setback perimeter frame structures designed according to the Israeli steel code SI 1225(1998) – that differed with special attention on the influence of the setback level, non-linear dynamic analyses were performed, and a 3D structural Model was used under bi-directional ground motions. Results showed amplification in response at the upper storeys, thus suggesting that the higher vibration modes have significant influence, Particularly the torsional ones. In this respect, the authors recommended that future research on setback buildings should be conducted on full plan-asymmetric structures.

ATHANASSIADOU AND BERVANAKIS,(2005), Studied the seismic Behaviour of Reinforced concrete Buildings with setbacks designed to capacity design procedure provided by Euro code 8. In their study, two ten storey frames with 2 and 4 legs setbacks in the upper floors respectively, as well as a third one, regular in elevation, have been designed to the provisions of Euro Code 8 for the high(H) ductility class and a common peak ground acceleration(PGA) of 0.25g. All frames were subjected to inelastic dynamic time-history Analyses for selected input motions. They found that the seismic performance of the studied multi-storey reinforced concrete frame Buildings with setbacks in the upper storeys designed to EC8 can be considered as completely satisfactory, not inferior and in some cases even superior of that of the regular ones, even for motions twice as strong as the design earthquake. Inter-storey drif ratios of irregular frames were found to remain quite low even in the case of the 'Collapse Prevention' earthquake with an intensity double that of the design one.

XAVIER ROMAO, ET.AL.,(2004), The purpose of the presented work is the behavior assessment of reinforced concrete frame structures with irregularities in Elevation. 3 6-storey frame structures with different degrees of irregularity in elevation were selected and their response were compared with the ones of a corresponding regular structure. In this study, the effects of the variation of the axial force in the columns and of different contributions of slab width to the beams flexural strength were investigated. In addition, a structure similar in configuration with one of the previously selected was defined and designed using current Portuguese codes to perform a comparison, between different design approaches. Structural performance was assessed by comparing local ductility levels, displacements inter –storey drifts and damage indices of the irregular frames and the regular one.

CROSBIE ET AL., (1997) carried out a research work to study the seismic performance of four, eight and twelve storey masonry shear walls mounted on base isolation devices. The main purpose of this study was to examine an alternative method of limiting the inertia forces in masonry buildings by seismic base isolation and also gave a significant contribution towards later attempts made to investigate the seismic response of base isolated multi storey structures.

CHAPTER III

METHODOLOGY

3.1 SEISMIC ANALYSIS METHODS CAN BE DONE IN 4 WAYS

1) Equivalent Static Analysis

2) Non-linear Static Analysis

3) Linear Dynamic Analysis

4) Non-linear Dynamic Analysis

Suitable for small and at no point the load will reach to collapse load and differs in obtaining level of forces and their distribution along the height of the structure whereas, the non-linear static and Non-linear dynamic analyses are the Improved methods over linear. During Earthquake structural loading will reach to collapse load and the material stresses will be above yield stresses so in that case, material non-linearity and geometrical non-linearity should be incorporated into the analyses to get better Results. These methods provide Information on the strength, deformation and ductility of the structures as well as distribution of demands.

3.1 Equivalent Static Analysis: This Approach defines a series of forces on a building to represent the effect of earthquake ground motion. It assumes that the building responds in its fundamental mode for this to be true, the building must be low rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the Natural frequency of the Building. The Applicability of this method is extended in many Building Codes by applying factors to account for Higher Buildings with some higher mode, and for low level of twisting. To account for effects due to “yielding” of the structure, many codes apply modification factors that reduce the Design forces (Eg: force reduction factors)

3.2 Pushover Analysis or non linear static Analysis: Pushover analysis or non linear static analysis has been developed over the past two decades and has become the preferred analysis procedure for design and seismic performance evaluation purposes, as the procedure is relatively simple and also considers post elastic behav However, the procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis.Pushover analysis is used mainly to evaluate seismic performance of existing buildings and retrofit them. It can also be applied for new structures. Reinforced concrete(RC) framed buildings would become massive if they were to bedesigned to behave elastically during earthquakes, without damage also they would become uneconomical. Therefore the structures must undergo damage to dissipate seismic energy. To design such a structure, it is necessary to know its performance and collapse pattern. To know performance and collapse pattern non linear static procedures are helpful.

3.3 Response Spectrum Method:

This method is applicable for those structures where modes other than the fundamental one affect significantly the response of the structure. In this method, The response of MDOF system is Expressed as the Superposition of modal response, each modal response being determined from the spectral analyses of SDOF system, which are then combined to compute the total response. Modal Analyses leads to the response history of the structure to a specified ground motion; however, the method is usually used in Conjunction with a Response Spectrum.

METHODOLOGIES FOR THE STUDY:

1) Linear Dynamic Analyses – Response Spectrum Analyses as per IS 1893 -2002

2) Nonlinear dynamic Analyses – Time History Analyses3.4 Linear Dynamic Analysis – Response Spectrum Analysis

This method is also known as modal method or mode superposition method. The method is applicable to those structures where modes other than the fundamental one significantly affect the response of the structure. Generally, the method is applicable to analysis of the dynamic response of structures, which are asymmetrical or have the areas of discontinuity or irregularity in their linear range of behavior. In particular, it is applicable to analysis of forces and deformations in multi – storey buildings due to medium intensity ground shaking which causes 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 model responses can be combined to determine the 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 analysis of SDOF oscillator with properties chosen to be representative of the particular method.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 is primarily due to lower modes of vibration.

A complete modal analysis provides the history of response-forces, displacements, and deformations-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 modeled 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 form, 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' results of the two analyses and the effects of torsional motions of the structures are combined.

3.4.1The Equation of Motion

The equation of motion can be derived by using the *d'Alembert's principle* which gives a concept that a mass develops an inertial force proportional to its acceleration and opposing the acceleration of the mass.

The equation of motion of a dynamic system can be formulated by directly expressing its equilibrium of all forces acting on the mass of the system using *d'Alembert's Principle* as:

$$f_I(t) + f_D(t) + f_S(t) = p(t) \ldots \ldots \ldots \quad 1$$

Each of the forces represented on the left hand side of this equation is a function of the displacement *u(t)* or one of its time derivatives. The positive sense of these forces has been deliberately chosen to correspond with the negative-displacement.

In accordance with d'Alembert, inertial force is the product of mass and acceleration

$$f_{i(t)} = m\ddot{u}(t) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2$$

Assuming a viscous damping mechanism, the damping force is the product of the damping constant c and the velocity

$$f_D(t) = c\ (t) \ldots \ldots \ldots \ 3$$

Finally, the elastic force is the product of the spring stiffness and the displacement

B. 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 of safety than the others, and this is achieved by specifying higher lateral 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.

The importance factors are given in Table.

Notes:

- The design engineer may choose values of importance factor I greater than those mentioned above.
- Buildings not covered in the table above may be designed for a higher value, depending on economy and strategy considerations. These could be buildings such as multi-storey buildings having several residential units.
- This table does not apply to temporary structures like excavations, scaffolding, etc.

C. 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 forces much less than what is expected under strong shaking, if the structures were 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. Base shear force is the force that would be generated Response reduction factor for building system.

D. Fundamental Natural Period

The fundamental natural period is the first (longest) modal time period of vibration of the structure. Because the design loading depends on the building period, and the period cannot be calculated until a design has been prepared, IS 1893 (part I): 2002 provides formulae from which Ta may be calculated.

For a moment-resisting frame building without brick infill panels, Ta may be estimated by the empirical expressions

Ta = 0.075ho.75 ---------- For RCC building

Ta = O.085h ---------- For steel building

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

$$T_a = 0.09h/\sqrt{d}$$

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

Seismic Base Shear

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

$$V_B = A_h * W$$

where Ah 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 Ah for a structure is determined by the expression

$$A_h = \frac{ZIS_a}{2Rg}$$

CHAPTER-IV

CASE STUDY

The Layout of plan having 3X4bays of equal length of 6m. The buildings considered are Reinforced concrete special moment resisting space frames of 15 storey Regular as well as Irregular with mass and Regular &Irregular with Stiffness Irregularities of the infill is neglected in order to account the Nonlinear Behavior of Seismic demands. All these buildings have been analyzed by NLTHA method. The storey height is kept uniform of 3m for building models which does not consider stiffness and for the Building models considering stiffness the Bottom storey is of 4m height and the remaining kept as 3m height. The analysis illustrates the step-by-step procedure for determination of forces.

The Plan configuration consists of

1. Model 1 – Building in rectangular shape symmetry, with mass Irregularities Fifteen storey-Number of bays in x-direction=3,Number of bays in y-direction=4.
2. Model 2 – Building in rectangular shape symmetry, with stiffness Irregularities
3. Model 3 -- Building in L shape Asymmetry, Fifteenstoreys.with mass Irregularities
4. Model 4 -- Building in L shape Asymmetry, Fifteen storeys.with stiffness Irregularities

Table 4.1 Assumed Preliminary data required for the Analysis of the frame

Sl.no	Variable	Data
1	Type of structure	Special Moment Resisting Frame

Fig 4.1 Elevation and Isometric View of Model -1

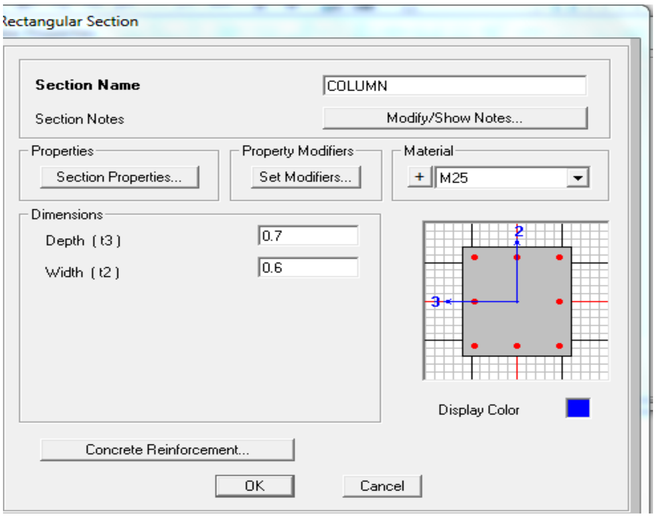
Fig4.1Dimensions Assigned for Beam

Fig4.2 Dimensions Assigned for Column

Fig 4.3 Assigning Thickness for slabs

4.3 ANALYSIS METHODS

Analysis methods are broadly classified as linear static, linear dynamic, nonlinear static and nonlinear dynamic methods. During earthquake loads the structural loading will reach to collapse load and the material stresses will be above yield stresses. so in that case material



nonlinearity and geometric nonlinearity should be incorporated into the analysis to get better results. These methods also provide information on the strength, deformation and ductility of the structures as well as distribution of demands.

4.3.1 Equivalent Static Method

Equivalent static method of analysis is a linear static procedure, in which the response of building is assumed as linearly elastic manner. The analysis is carried out as per IS1893-2002 (Part 1).

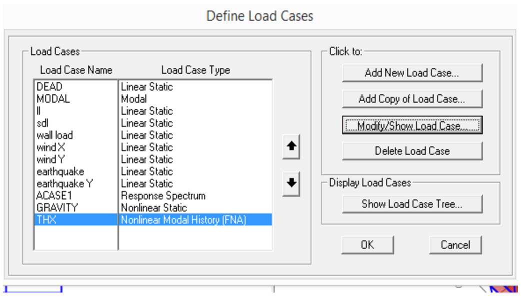


Figure 4.4: Illustrates the Equivalent static gravity & lateral loads Applied on the Structure

4.4 NON-LINEAR TIME HISTORY ANALYSIS

NLTHA is one of the methods and the most accurate method available to understand the behavior of structures subjected to earthquake forces. As the name implies, it is the process of finding out the history of responses throughout the life span of the dynamic loading like an earthquake ground acceleration record until the structure reaches a limit state. The dynamic loading consists of applying a earth-quake ground acceleration Record of lateral loads to a model which captures the material non- linearity of an existing or previously designed structure, and monotonically increasing those loads which vary with time so that the peak response of the structure is evaluated.

For this purpose two earth-quake ground acceleration records namely N-E Bhuj and N-W Bhuj components of the Bhuj Earthquake record have been selected. Bhuj is a place located in the state of Gujarat which is a high intensity earthquake zone of zone factor 0.36 which comes under the Zone-V according to the classification of seismic zones by IS 1893-2002 part-1.The records are defined for the acceleration points with respect to a time-interval of 0.005 second. The acceleration record has units of m/sec^2 and has a total number of 26,706 acceleration data coordinates out of which the most critical data points which are of the highest intensity are the first 10,000 acceleration data coordinates have been considered.

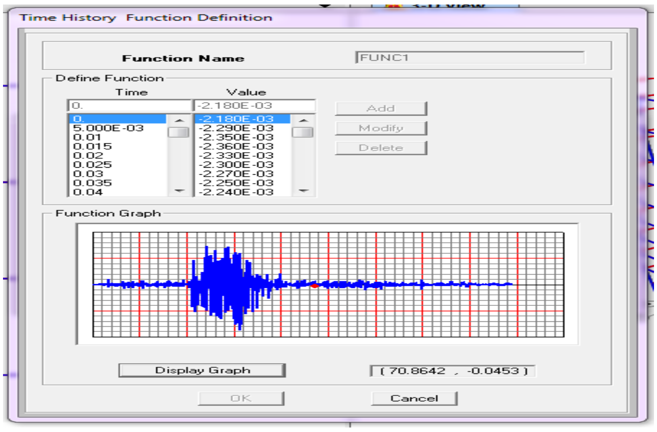


Figure 4.5: Bhuj-N-E component earthquake ground acceleration record.

4.5 NON-LINEAR DIRECT-INTEGRATION TIME HISTORY LOAD CASES

The Non-Linear time history analysis is carried out through the Direct-Integration methods by defining the Non-Linear time history direct integration load cases. The acceleration data points are converted to loading by multiplying the acceleration data points with the mass matrix of the structure. The time history load case is continued from state end of a non-linear load case known as a non-linear static load case. The appropriate additional parameters such as P-Δ effects, damping matrix, selection of time step, number of time steps, time-integration method etc.

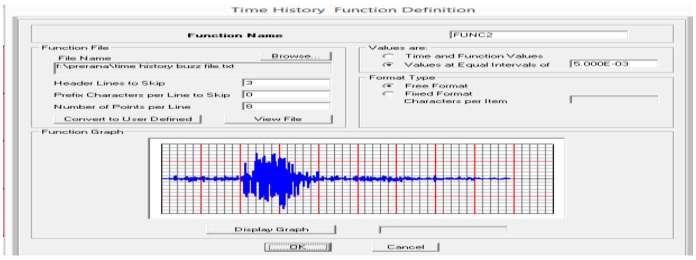


Figure 4.6: Illustrates the non-linear direct integration time history load case using Bhuj-N-E time history function

4.6 PLASTIC (NON-LINEAR) HINGES

Default hinge definitions according to the FEMA-356 guidelines have been provided at the ends, where the formation of the potential plastic hinges is more probable for beams and columns with degree of freedom as M3 and the shear value for the hinge is taken from the Dead load case. The hinges are set so that they drop the load after reaching the point E of the performance level.

Figure 4.7: Performance levels according to hinge states.

Force-displacement or moment-rotation curve for a hinge definition used in SAP2000 [21] is referred to as a plastic deformation curve. Thplastic deformation curve is characterized by the following points as:

1. Point A represents the origin.
2. Point B represents the yielding state. No deformation occurs in the hinge up to point B, regardless of the deformation value specified for point B. The displacement (rotation) at point B will be subtracted from the deformations at points C, D, and E. Only the plastic deformation beyond point B will be exhibited by the hinge.

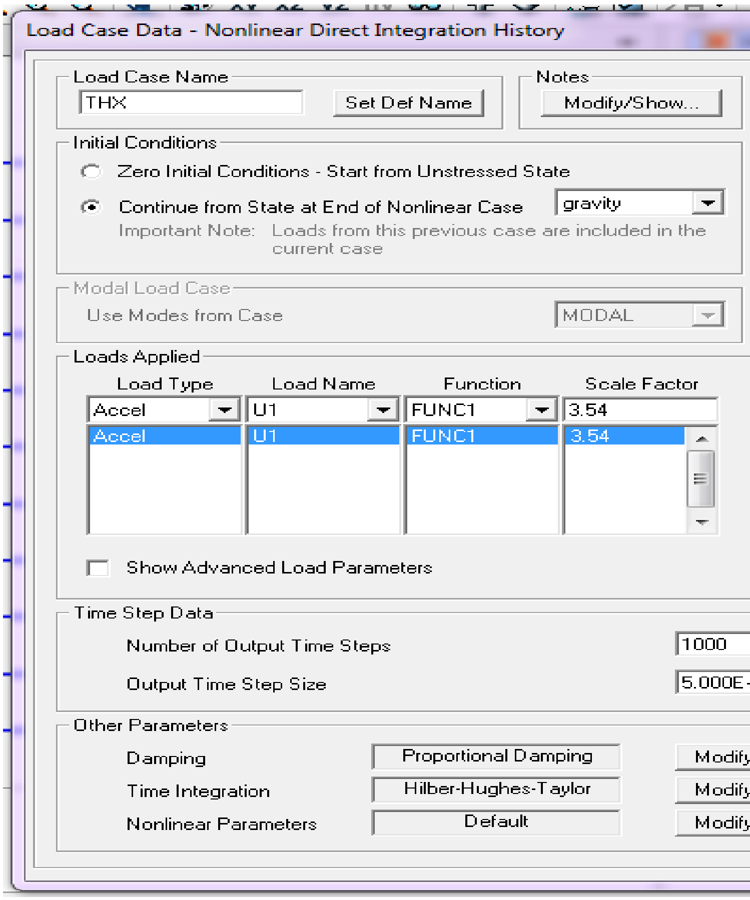


Fig 4.9: Illustrates the non-linear modal time history load caCHAPTER-V

RESULTS AND DISCUSSIONS

The Results obtained are of different parameters such as Storey drifts, Base shear, Modal Periods, Torsion etc. Firstly the results obtained by carrying out Non-Linear Time History Analysis for both Symmetric and Asymmetric Buildings for Fifteen storey Building are listed. Subsequent Discussions are made about the Results Obtained based on the storey drifts, Base shear, Torsion etc. for both Symmetric and Asymmetric buildings individually and also considering the Storey effect of both

Symmetric and Asymmetric buildings by comparing the Response of the structure for fifteen storey Building.

5.1 RESPONSE OF FIFTEEN STOREY SYMMETRIC BUILDING.

Figure 5.1:Storey Drifts in X-Direction comparison for fifteen storey symmetric building.

Figure 5.2:base shear force comparison for fifteen storey for load case th-x.

Figure 5.3: Base shear time history for fifteen storey structure without base isolation for load case th-xFigure 5.4: Base shear time history for fifteen storey structure with base isolation for load case th-x

Figure 5.5: Base moment time history for five storey structure without base isolation for load case th-x

Figure 5.6: Base moment time history for five storey structure with base isolation for load case th-x

Figure 5.7:base moment comparison for fifteen storey for load case th-x.

5.2 BASE ISOLATION OF FIFTEEN STOREY ASYMMETRIC BUILDING.

Figure 5.8:Storey Drifts in X-Direction comparison for fifteen storey asymmetric building.

Figure 5.9:base shear force comparison for fifteen storey for load case th-x.

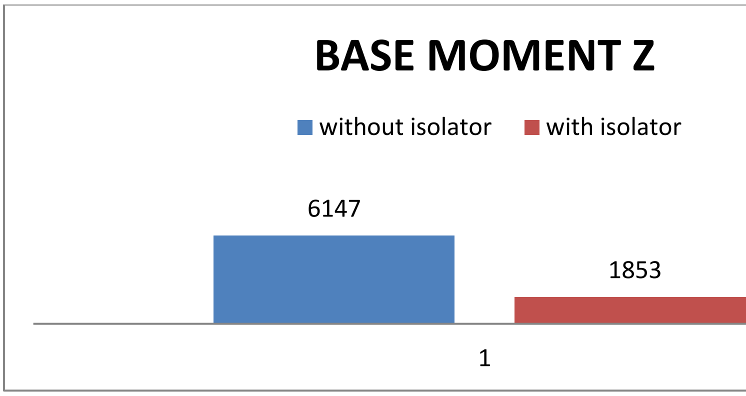


Figure 5.10:base moment comparison for fifteen storey for load case th-x.

5.3 BASE ISOLATION OF FIFTEEN STOREY SYMMETRIC BUILDING WITH STIFFNESS IRREGULRITY.

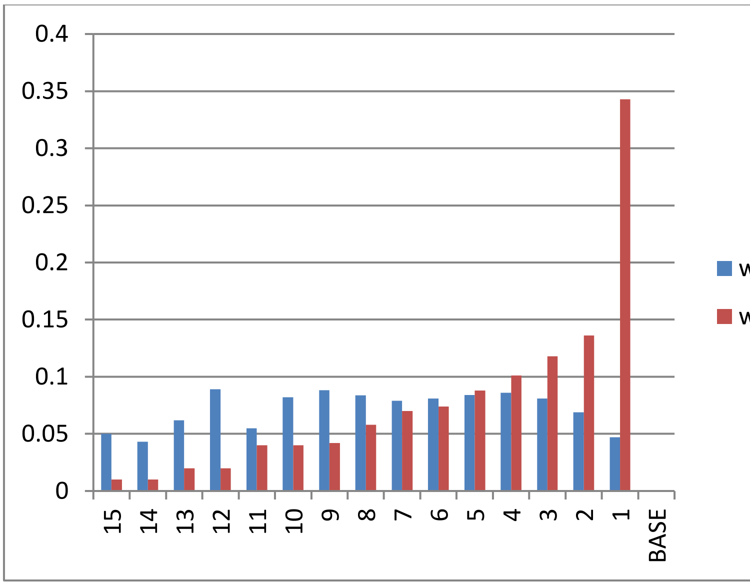


Figure 5.11:Storey Drifts in X-Direction comparison for fifteen storey symmetric building with stiffness irregularity

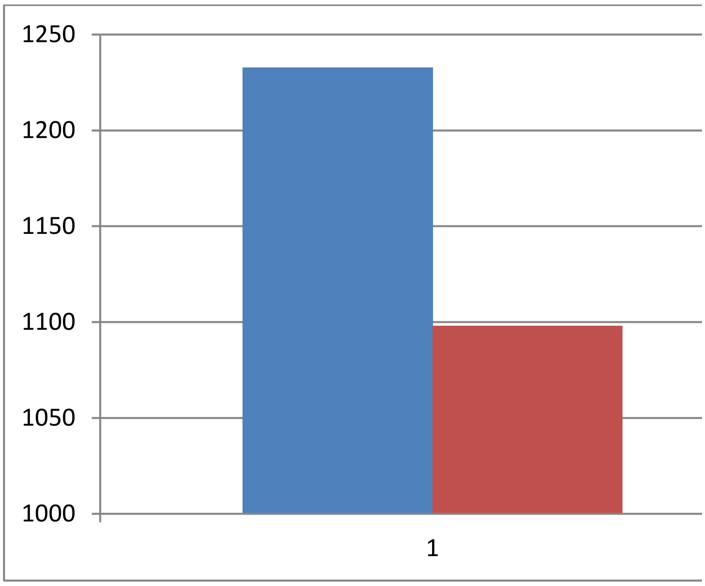


Figure 5.12: base shear force comparison for fifteen storeyRegular stiffness irregularity for load case th-x.

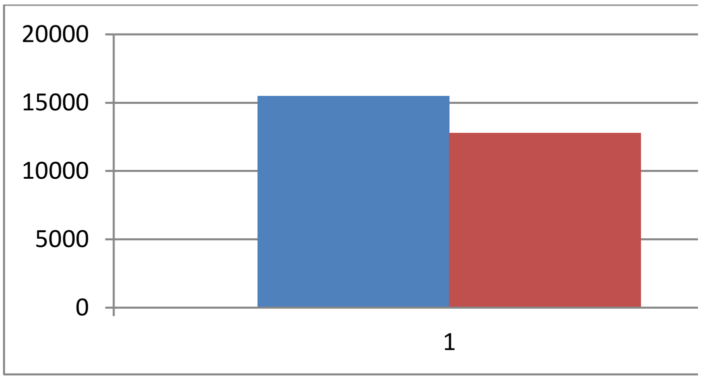


Figure 5.13:base moment force comparison for fifteen storeyRegular stiffness irregularity for load case th-x.

5.4 BASE ISOLATION OF FIFTEEN STOREY ASYMMETRIC BUILDING WITH STIFFNESS IRREGULRITY.

th-x.

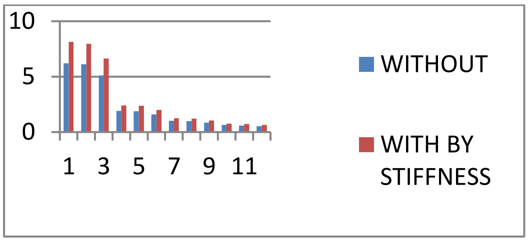


Figure 5.18: Comparison of modal time periods for asymmetric building with and without isolation for stiffness irregularity

5.5DISCUSSIONS OF RESULTS

Results for Base Isolation of Symmetric Buildings:

Figure 5.1 Illustrates the Reduction in Storey Drift using Base Isolation at top storey level from 45mm to 3mm. The storey drifts on an average were reduced by 42% for the fifteen storey structure which is illustrated in figure 5.1.The base shear is reduced by 76% for the fifteen storey structure Illustrated in the Figure 5.2.

Results for Base Isolation of Asymmetric Buildings:

Figure 5.8 illustrates the reduction in storey drifts using base isolators at top storey level from 100 mm to 20 mm. The storey drifts on an average decreased by 80% for the fifteen storey building. Figure 5.10 illustrates that there is a reduction in the base torsion moment using base isolation from 6147 kNm to 1853 kNm which is a reduction by 70%. Reduction in base shear was observed for the fifteen storey building from 4421 kNm to 1103 kNm for the load case Th-x as shown in Figure 5.9

CHAPTER-VI

6.1 CONCLUSIONS

1. The storey drifts were decreased by 42% for fifteen storey symmetric building and decreased

6.2 SCOPE FOR FURTHER STUDY

As the various researchers are getting attracted towards the NLTHA, the scope of the studies under the particular topic can be stretched to wide horizons.

1. Soil structure interaction has always attracted many researchers as an

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