COMPARISON OF SEISMIC PERFORMANCES OF RCC BUILDING BY PUSHOVER ANALYSIS AND NON-LINEAR DYNAMIC ANALYSIS OF MDOF SYSTEM

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Abstract: The prediction of inelastic seismic responses and the evaluation of seismic performance of a building structure are very important subjects in performance-based seismic design. The seismic performances of reinforced-concrete buildings evaluated by nonlinear static analysis (pushover analysis) and nonlinear time history analysis are compared in this research. A finite element model that can accurately simulate nonlinear behaviour of building is formulated by considering several important effects such as p-delta can be considered rigid zones with joint failure due to poor detailing of joints. Both global response such as system ductility demand and local response such as inter-story drift is investigated in this research. A numerical example is performed on a 20-story reinforced concrete building in ZONE V. Finally, the global and local responses obtained from the pushover analysis are compared with those obtained from the nonlinear dynamic analysis of MDOF system. The results show that the PA is accurate enough for practical applications in seismic performance evaluation when compared with the nonlinear dynamic analysis of MDOF system.

Keywords: Performance Based Design, Pushover Analysis, Building Performance Levels.

Introduction

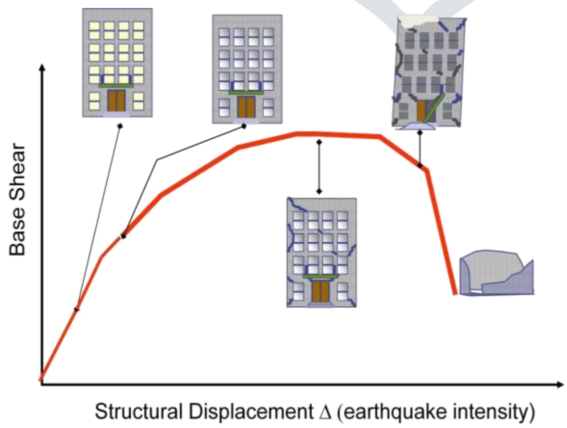
1.1 Performance Based Design

The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance. This approach is not new using this approach/model Turbines, Airplanes & Automobiles are made. In these applications one or more prototype is built and subjected to extensive testing. To incorporate the lessons learned from the experimental evaluations the design and manufacturing process is then revised, Once the cycle of design, prototype manufacturing, testing and redesign is successfully completed, the product is manufactured in a massive scale. In the automotive industry, for example, millions of automobiles which are virtually identical in their mechanical characteristics are produced following each performance-based design exercise. Performance Based Earthquake Engineering/Design is not that popular because the scale of output is not large in comparison to the Automobile industry and others. Each building designed by this process is virtually unique and the experience obtained is not directly transferable to buildings of other types, sizes, and performance objectives. Therefore, up to now PBSE has not been an economically feasible alternative to conventional prescriptive code design practices. In coming few years, we can say that Performance Based Design will become the standard method of delivering Earthquake resistant designs. The facts are clear – We cannot prevent big, destructive earthquakes from occurring. These pose a continuing threat to lives and property in more than 55% of the area of this country. However, it is possible to avoid the disastrous consequences of an earthquake and that precisely is the objective of every seismic design code practice. The seismic codes are framed primarily with the objective of prevention of loss of life. In order to meet this objective, it is essential that the structures/constructed facilities respond to the expected earthquake ground motions at the site in a designated manner, which in turn depends on the nature of ground motion exciting the structure. Thus the reliability of achieving the life safety performance objective of any constructed facility is governed by the most uncertain element in the chain- expected ground motion. Seismic hazard and Damage state are the two essential parts of a Performance Objective. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). The target Performance level is split into two levels Non-structural damage and Structural damage, the combination of the two gives the building a combined performance level. The various Performance levels are described in detail in the next section 1.2. In increasing order of structural displacement, the various Performance levels shown here are Operational, Immediate Occupancy, Life Safety and Last one is Collapse Prevention.

1.2 BUILDING PERFORMANCE LEVELS

The Various Performance levels are tabulated below with their effect on both Structural and Non-structural elements.

Building Performance Levels



Performance Level	Structural Performance	Non Structural Performance
Operational (O)	Very light damage No permanent drift Substantially original strength and stiffness	Negligible damage. Power & other utilities are Available
Immediate Occupancy (IO)	Light damage No permanent drift Substantially original strength & stiffness Minor cracking Elevators can be Restarted Fire protection operable	Equipment & content secure but may not operate due to Mechanical /utility failure
Life Safety (LS)	Moderate damage Some permanent drift Residual strength & stiffness in all stories Gravity elements Function Building may be beyond economical repair	Falling hazard mitigated but extensive systems Damage
Collapse Prevention (CP)	Severe damage Large permanent drifts Little residual strength & Stiffness Gravity elements Function Some exits blocked Building near collapse	Extensive damage

Nonlinear static analysis, or pushover analysis, has been developed over the past twenty years and has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post-elastic behaviour. 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. Although, in literature, pushover analysis has been shown to capture essential structural response characteristics under seismic action, the accuracy and the reliability of pushover analysis in predicting global and local seismic demands for all structures have been a subject of discussion and improved pushover procedures have been proposed to overcome the certain limitations of traditional pushover procedures. However, the improved procedures are mostly computationally demanding and conceptually complex that use of such procedures are impractical in engineering profession and codes. As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitations, weaknesses and the accuracy of its predictions in routine application should be identified by studying the factors affecting the pushover predictions. In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high-rise structures by identifying certain issues such as modelling nonlinear member behaviour, computational scheme of the procedure, variations in the predictions of various lateral load patterns utilized in traditional pushover analysis, efficiency of invariant lateral load patterns in 1 representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed.

1.3 METHODS OF ANALYSIS

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structures.

1.3.1 Elastic Methods of Analysis

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes. In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R". In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding. In code dynamic procedure, force demands on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis. Any effect of higher modes are automatically included in time history analysis. In demand/capacity ratio (DCR) procedure, the force actions are compared to corresponding capacities as demand/capacity ratios. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. DCRs approaching 1.0 (or higher) may indicate potential deficiencies. Although force-based procedures are well known by engineering profession and easy to apply, they have certain drawbacks. Structural components are evaluated for serviceability in the elastic range of strength and deformation. Post-elastic behaviour of 2 structures could not be identified by an elastic analysis. However, post-elastic behaviour should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "R" is utilized to account for inelastic behaviour indirectly by reducing elastic forces to inelastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it has been shown that this factor is a function of the period and ductility ratio of the structure as well. Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don’t predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses. Real deficiencies present in the structure could be missed. Moreover, force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair. The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation. Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly.

1.3.2 Inelastic Methods of Analysis

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behaviour of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis. The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modelling and ground motion characteristics. It requires proper modelling of cyclic load deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and 3 interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method. The theoretical background, reliability and the accuracy of inelastic static analysis procedure is discussed in detail in the following sections.

Literature review

General

The Earthquakes affect buildings in several ways. As we have seen in past earthquakes that there was devastating damage to life and structure. So, safety is a must for them. There have been a number of reports on damage to structures in past earthquakes which have demonstrated the seismic vulnerability of structure and the damage. The accelerating structure and rigid masses, induces substantial pressures on the wall of RC frame which in turn generates lateral pressures (i.e. base shear) and overturning moment. The failure occurs also as the RC structure buckles due to axial compression, toppling of the frame structure, failure of floors, failure if roof and uplift of the anchorage system.

"Earthquake proof structures" generally mean the structures which resist the earthquake and save and maintain their functions. The key points for their design includes select good ground for the site, make them light, make them strong, make them ductile, shift the natural period of the structures from the predominant period of earthquake motion, heighten the damping capacity.

To provide a detailed review of the literature related to modelling of structures in its entirety would be difficult to address in this chapter. A brief review of previous studies on the application of the linear and non-linear time history and response spectrum method of analysis is presented in this section.

Virote Boonyapinyo 1 , Norathape Choopool 2 and Pennung Warnitchai 3 in their study involves The seismic performances of reinforced-concrete buildings evaluated by nonlinear static analysis (pushover analysis and modal pushover analysis) and nonlinear time history analysis are compared in this research. A finite element model that can accurately simulate nonlinear behaviour of building is formulated by considering several important effects such as p-delta, and beam-column joints that can be considered rigid zones with joint failure due to poor detailing of joints. Both global response such as system ductility demand and local response such as inter-story drift are investigated in this research. A numerical example is performed on a 9-story reinforced concrete building. Because Bangkok is located in soft to medium soils, response of studied building under a simulated earthquake ground motion at Bangkok site is compared with that under a measured earthquake ground motion of EI-Centro. Finally, the global and local responses obtained from the modal pushover analysis are compared with those obtained from the nonlinear dynamic analysis of MDOF system. The results show that the MPA is accurate enough for practical applications in seismic performance evaluation when compared with the nonlinear dynamic analysis of MDOF system. The results also show that ductility of the studied building can be estimated to 2.40, 2.02 and 1.65 by Fajfar, Chopra and Lee methods, respectively.

Eduardo A FIERROAnd Cynthia L PERRY in their study involved nonlinear analyses of a family of single-degree-of-freedom (SDOF) nonlinear systems subjected to a series of earthquake time histories. The SDOF systems were selected to represent a range of stiffness (initial elastic periods of 0.2, 0.5, 1.0, 2.0 and 3.0 seconds); a range of strengths (each system yields at 10%, 20%, 40%, 60%, 80%, or 100% of the maximum elastic strength required to resist a particular ground motion record); and a range of damping values (2% and 5% of critical damping). Each system was subjected to a series of ground motion recordings measured at soft soil sites (6 records), intermediate soil sites (14 records), and rock sites (10 records). These combinations resulted in 300 linear and 1500 nonlinear computer runs using the program NONLIN. The results were averaged for each soil type and plotted as a percent of the maximum elastic strength versus the ductility demands.

Dimpleben P. Sonwan et al., in their paper presents an effective computer based technique that incorporates pushover analysis together with pushover drift performance design of RC buildings is carried out. The study begins with the selection of performance objectives, followed by development of preliminary design, an assessment whether design meets performance objectives or not, finally redesign and reassessment, if required, until the desired performance level is achieved. In present study RC framed building example (Designed according to IS 456:2000) analysed using pushover analysis and redesigning by changing the main reinforcement of various frame elevations at different storey level and analysing. The pushover analysis has been carried out using SAP 2000, product of computers and structures international. The building is considered as special moment resisting framed building and the main objective of this study is to check kind of performance a building can give when designed as per IS. The best possible combination of reinforcement that is economical, effective and having minimum damage to enable immediate occupancy is determined and is termed as performance based design.

A.Kadid et al., in their paper conducted static pushover analysis. They considered three framed buildings with 5,8 and 12 stories and analysed these.

Mehmed Causevic · Sasa Mitrovic in their studies present Several procedures for non-linear static and dynamic analysis of structures have been developed in recent years. This paper discusses those procedures that have been implemented into the latest European and US seismic provisions: non-linear dynamic time-history analysis; N2 non-linear static method (Eurocode 8); non-linear static procedure NSP (FEMA 356) and improved capacity spectrum method CSM (FEMA 440). The presented methods differ in respect to accuracy, simplicity, transparency and clarity of theoretical background. Non-linear static procedures were developed with the aim of overcoming the insufficiency and limitations of linear methods, whilst at the same time maintaining a relatively simple application. All procedures incorporate performance-based concepts paying more attention to damage control. Application of the presented procedures is illustrated by means of an example of an eight-storey reinforced concrete frame building. The results obtained by non-linear dynamic time-history analysis and non-linear static procedures are compared. It is concluded that these non-linear static procedures are sustainable for application. Additionally, this paper discusses a recommendation in the Eurocode 8/1 that the capacity curve should be determined by pushover analysis for values of the control displacement ranging between zero and 150% of the target displacement. Maximum top displacement of the analysed structure obtained by using dynamic method with real time-history records corresponds to 145% of the target displacement obtained using the non-linear static N2 procedure.

Seismic analysis of structures

3.1 GENERAL

For the determination of seismic responses there is necessary to carry out seismic analysis of structures. The analysis can be performed on the basis of external action, the behaviour of structure or structural materials, and the type of structural models selected. Based on the type of external action and the type of structural behaviour, the analysis can be further classified as :1) Linear Static analysis, (2) Nonlinear Static Analysis, (3) Linear Dynamic Analysis; and (4) Nonlinear Dynamic Analysis. Linear Static Analysis can be used for structures with limited height. Linear dynamic analysis can be performed by response spectrum method. The significant difference between the linear static and linear dynamic analysis is the level of forces and their distribution along the height of the structure. Nonlinear static analysis is an improvement over linear static or dynamic analysis in the sense that it allows inelastic behaviour of structure. A non-linear dynamic analysis is the only method to describe the actual behaviour of the structure during an earthquake. The method is based on direct numerical integration of the differential equation of motion by considering the elasto-plastic deformation of the structural material.

Same magnitude earthquakes can cause dissimilar damaging effects in different regions. It is therefore necessary to study variations in seismic behaviour of multistoried RCC framed buildings for different seismic intensities in terms of various responses such as base shear, displacements at various levels, etc. for determination of seismic behaviour it is necessary to carry out dynamic analysis.

The introduction of response spectrum method of analysis which provides convenient means for representing the elastic behaviour of simple structures, was later identified that the forces predicted by such spectra exceeded normal design requirements. Because structures having much less strength that is predetermined by the spectral values were observed to have performed satisfactorily in earthquakes, it became apparent that elastic response spectrum is not a direct measure of the significant behaviour of many structures. Even moderate earthquakes can be expected to produce inelastic deformations. Therefore, inelastic analysis is important and plastic energy absorbed by the structure has a controlling effect on deformations of the structure.

3.2 METHODS OF ANALYSIS

i) Equivalent Static Analysis

This procedure does not require dynamic analysis, however, it accounts for dynamics of the building in an approximate manner. This is the simplest one-requiring less computational efforts and is based on formulae given in code of practice. Firstly, the design base shear is computed for the whole building, and it is then distributed along the height of the structure. The lateral forces at each floor levels thus obtained are distributed to individuals lateral load resisting elements.

ii)Linear Dynamic Analysis

It can be performed in two ways either by mode superposition method or response spectrum method of analysis and elastic time history methods. This analysis will produce effects of higher modes of vibration and actual distribution of forces in the elastic range in a better way. The significant difference between the linear static and linear dynamic analysis is the level of forces and their distribution along the height of the structure.

iii)Non-linear Static Analysis

Non-linear static analysis is an improvement over linear static or dynamic analysis as it allows the inelastic behaviour of the structure. The method still assumes a set of incremental lateral loads over the height of the structure. The method is relatively simple to be implements and provides information on the strength, deformation and ductility of the structure and the distribution demands. This permits identification of critical members likely to reach limit states during the earthquake, for which attention should be paid to designing and detailing process. But this method contains limited assumptions, which includes neglecting effects of higher modes, behaviour of loading patterns and the effect of resonance.

iv)Non-linear Dynamic Analysis

A nonlinear dynamic analysis is the true presentation of behaviour of the structure during earthquake. It is a step by step analysis of the dynamic response of a structure to a specified loading that may be varying with time.The method is based on direct numerical integration of the differential equation of motion by considering the elasto-plastic deformation of the structural material. It captures effect of resonance, variations at diverse levels of frame, among other advantages.

3.3 SEISMIC DESIGN METHODS

Conventional civil engineering structures are designed on the basis of two main criteria that are strength and rigidity. The strength is related to damageability or the ultimate limit state, assuring that the force level developed in the structure remains in the elastic range, or some limited plastic deformation. The rigidity is related to serviceability limit state, for which the structural displacements must remain in some limits. This assures that no damage occurs in the non-structural elements. In case of earthquake resistant design of structures, a new demand must be added to the above-mentioned ones, that is the ductility demand. Ductility is an essential attribute of a structure that must respond to strong ground motions. Ductility serves as a shock absorber in a building, for it reduces the transmitted force to one that is sustainable.

3.3.1 Code-based methods for seismic design

Lateral strength-based design:

This is the most common seismic design approach used today and IS 13290:1993 code is based on this approach. It is based on providing the structure with minimum lateral strength to resist seismic loads, assuming adequate behavior of structure in nonlinear range.

Displacement or ductility-based design:

It is very well recognized now that the because of economic reasons the structure is not designed to have sufficient strength to remain elastic in extreme earthquakes. The structure is designed to have adequate ductility so that it can dissipate energy by yielding and survive the shock. This method deals with deformation quantities which gives better insight on the expected behaviour of the structure, rather than simply providing strength.

Capacity based Design:

In this the structure is designed in such a way so that hinges can only form in predetermined positions and sequences. It is a design process in which strengths and ductility’s are allocated and analysis is independent. Its procedure stipulates the margin of strength that is necessary for elements to ensure that their behaviour remains elastic.

Energy based design:

One of the promising approaches is energy approach. In this approach, it is recognized that the total energy input, ET can be resisted by the sum of the kinetic energy EK, the elastic strain energy EES, energy dissipated through plastic deformations (hysteretic damping) EH, and equivalent viscous damping Eε.

The energy equation for a single mass vibrating system is the energy balance between total input energy and the energies dissipated by a viscous damping and inelastic deformations and can be written as,

$$ET = EK + EES + EH + E\varepsilon$$

In order to perform the seismic analysis and design of a structure to be built at a particular location, the actual time history record is required. However, it is not possible to have such records at each and every location. Further, the seismic analysis of structures cannot be carried out simply based on the peak value of the ground acceleration as the response of the structure depend upon the frequency content of ground motion and its own dynamic properties. To overcome the above difficulties, earthquake response spectrum is the most popular tool in the seismic analysis of structures. There are computational advantages in using the response spectrum method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves the calculation of only the maximum values of the displacements and member forces in each mode of vibration using smooth design spectra that are the average of several earthquake motions. This chapter deals with response spectrum method and its application to various types of the structures. The codal provisions as per IS:1893 (Part 1)-2002 code for response spectrum analysis of multi-story building is also summarized.

3.4 RESPONSE SPECTRA

Response spectrum is a plot of peak response vs modal frequency for a given damping of various single degree of freedom systems (representing various modes of vibrations of system) subjected to same loading. Response spectra thus helps in obtaining the peak structural responses under linear range, which can be used for obtaining lateral forces developed in structure due to earthquake thus facilitates in earthquake-resistant design of structures.

3.4.1 RESPONSE SPECTRUM METHOD OF ANALYSIS

Response spectrum method of analysis is a linear dynamic method of analysis which measures contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response spectrum provides insight into dynamic behaviour by measuring pseudo acceleration, velocity or displacement as a function of structural period for a given time history and level of damping. Principle of superposition is valid for this analysis. Thus analysis is done to get the dynamic characteristics of the building (natural frequency and mode shape) with which a statistical analysis is performed for each mode, the results of which are then combined to get design forces.

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. El Centro response spectrum is used in this analysis.

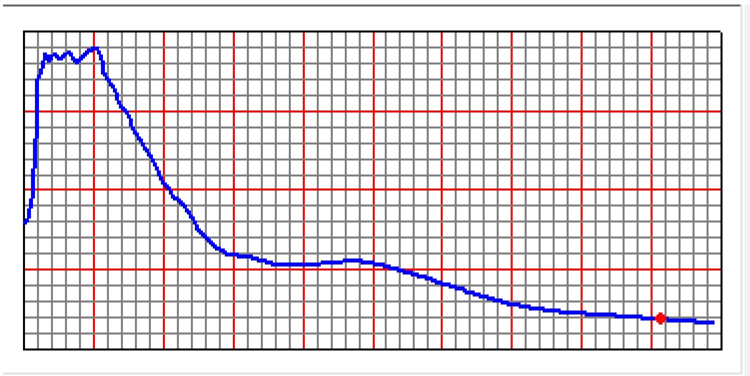


FIG. 3.1: EL-CENTRO RESPONSE SPECTRA

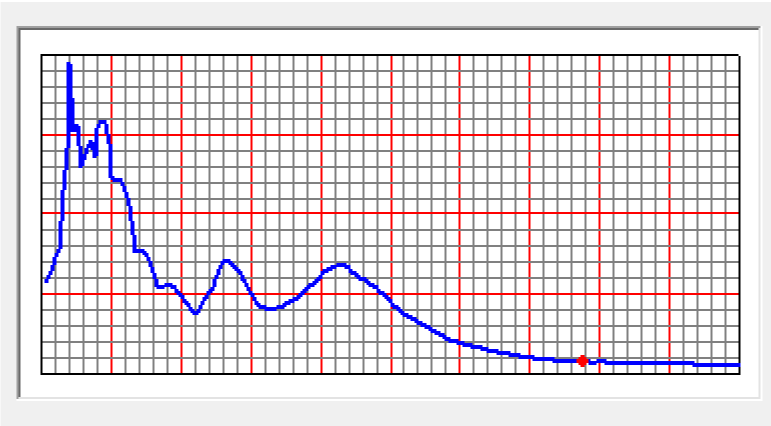


FIG. 3.2: KOBE'S RESPONSE SPECTRA

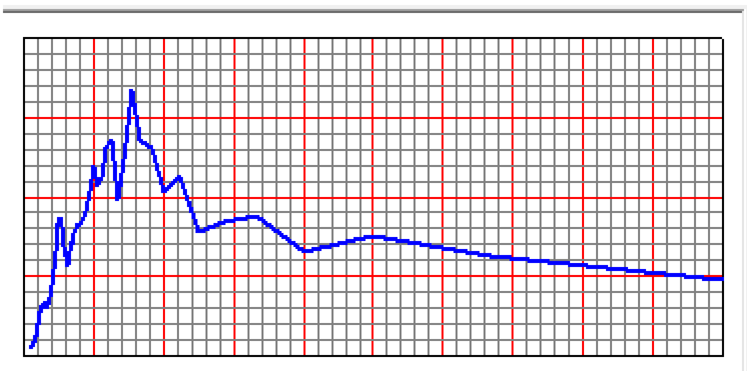


FIG. 3.3: NORTHRIDGE'S RESPONSE SPECTRA

3.5 Linear Time History Analysis:

The complete time history of response to an earthquake can be obtained by calculating the response at successive discrete times, with the time step (interval between calculation times) sufficiently short to allow extrapolation from one calculation time to the next. Where a linear analysis is performed the time step should not exceed a quarter of the period of highest structural mode of interest. The method involves higher computational efforts than corresponding RSA and at least three representative earthquake motions must be considered to allow for the uncertainty in the precise frequency content of the design motion at a site. Linear time history analysis can be performed as modal time history analysis and direct integration time history analysis. Modal time history analysis solves the full equations of motion for each time step with the use of modal superposition. Direct integration time history solves the full equation of motion for each time step without using modal superposition. Direct integration generally gives more accurate results than the modal time history analysis for nonlinear analysis but are computationally expensive. For the following analysis, the Hilber-Hughes-Taylor alpha(HHT) method is used.

3.6 Non-Linear Time History Analysis:

In this method, the seismic response of the structure is evaluated using step by step time history analysis. The main methodology of this procedure is almost similar to the static method of analysis. However, these approached differ in the concept that the design displacements are not established using the target displacement; but, are estimated through dynamic analysis by subjecting the building model to an ensemble of ground motions. The calculated seismic response is very sensitive to the ground motion characteristics, and the analysis is carried out for more than one ground motion record. To perform non-linear dynamic analysis, the equation given by Newmark's method can be extended. Non-linear analysis id adopted for analytical study due to its accuracy and efficiency in determining the inelastic seismic response of a system subjected to the ground motion data. The combination of seismic response with the onset of plasticity and variation in time dependent parameters such as possible loss of strength and stiffness of the plastic hinge regions under repeated cyclic strains, etc. is accounted in nonlinear time history analysis. The time history procedure is used if it is important to represent inelastic response characteristics or to incorporate time dependent effects when computing the structure's dynamic response. In general, time history record is the most common way to describe a ground motion. The motion parameters may be acceleration, velocity, or displacement, or all the three combined together. Generally, the direct measured quantity is acceleration and others are derived quantities. However, others can be measured directly too. The measured time history record includes errors resulting from many sources, such as noises at high and low frequencies, base line error, and instrumental error. All these errors are removed from data before they are used. Further, the measured data is in analogue form, which are digitized before they are used as seismic inputs. At any measuring station, ground motions are recorded in three orthogonal directions; two of them are in horizontal direction and third is in vertical direction. Thus three components are available in any measuring station.

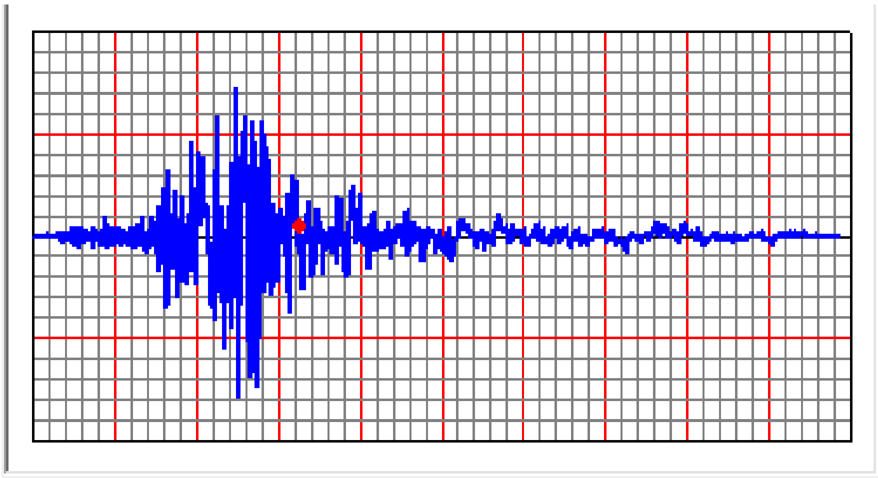


FIG. 3.4: EL-CENTRO TIME HISTORY

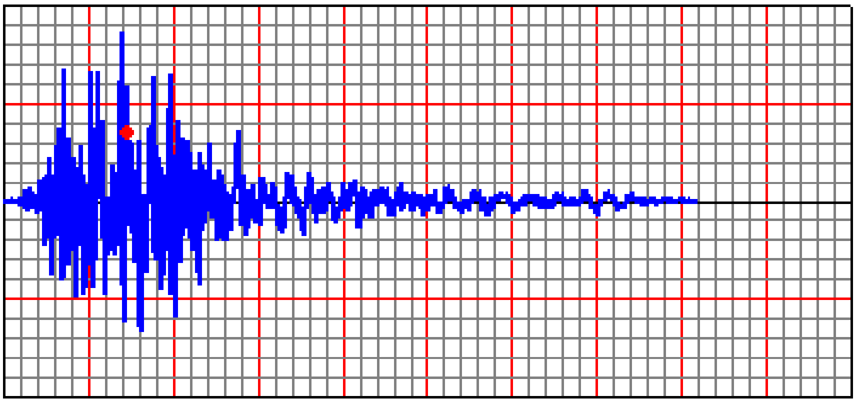


FIG. 3.5: KOBE’S TIME HISTORY

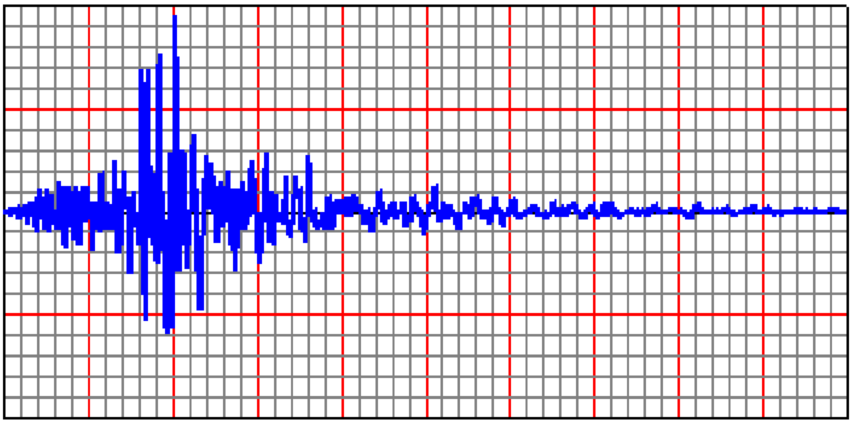


FIG. 3.6: NORTHRIGDE’S TIME HISTORY

3.7 PUSHOVER ANALYSIS:

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two- or three-dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied.

The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve. Pushover analysis can be performed as force-controlled or displacement controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e, force-controlled procedure should be used when the load is known (such as gravity loading). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

Use of Pushover Results

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analysis provide information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis. These are estimates of inter-story drifts and its distribution along the height • determination of force demands on brittle members, such as axial force demands on columns, moment demands on beam-column connections determination of deformation demands for ductile members • identification of location of weak points in the structure (or potential failure modes) • consequences of strength deterioration of individual members on the behaviour of structural system • identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range • verification of the completeness and adequacy of load path. Pushover analysis also expose design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

TARGET DISPLACEMENT:

The fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which seismic performance evaluation of the structure is to be performed. The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the center of mass of the structure. In the pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the SDOF domain through the use of a shape factor. This assumption, which is always an approximation, can only be accepted within limitations and only be accepted within limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics that significantly affect the maximum displacement of the MDOF structure. Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape factor without regards to higher mode effects. Under the Non-linear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.

Limitations of Pushover Analysis

There are many unsolved issues that need to be addressed through more research and development. Examples of the important issues that need to be investigated are:

1. Incorporation of torsional effects (due to mass, stiffness and strength irregularities). 2. 3-D problems (orthogonality effects, direction of loading, semi-rigid diaphragms, etc) 3. Use of site specific spectra. 4. Cumulative damage issues. 5. Most importantly, the consideration of higher mode effects once a local mechanism has formed. Since the pushover analysis is approximate in nature and is based on static loading, as such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that occur in a structure subjected to severe earthquakes, and it may significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

METHODOLOGY AND STRUCTURAL MODELLING ON SAP2000

4.1 GENERAL OVERVIEW

In this study performance of a twenty storey buildings subjected to severe earthquake loads was evaluated using elastic and inelastic analysis. Based on the findings from the analysis, a performance point was obtained for the structure.

4.2 METHODOLOGY

The general methodology adopted for this study was as follows:

- A model of 20 storey RCC frame was made using the structural analysis software SAP2000 V14. For this study, code design methods of IS Code 456:2000 were used.
- Modal analysis was performed and actual fundamental period of the structure found out.
- Static response spectrum analysis was carried out in accordance with code methods of IS1893:2002.
- Dynamic time history analysis of the structure was performed. El Centro, Kobe and Northridge was used as input earthquake.

- Pushover analysis of the structure was performed. El Centro, Kobe and Northridge was used as input response spectra.
- Non-linear static Analysis and Non-Linear Time History Analysis was performed on the structures.

4.3 GENERAL DESCRIPTION OF BUILDING MODELS:

The building is a 20 storey Reinforced Concrete Frame, with bays along X-direction, Y directions. The concrete floors are modelled as rigid. It is composed of storeys having 3.0 m height.

Type of frame = Special RC moment resisting frames

Number of storeys = 20

Number of bays along X-direction = 4

Number of bays along Y-direction = 4

Bay width along X-direction = 4 m

Bay width along Y-direction = 4 m

Columns dimensions: 950 mm* 750 mm

Beam dimensions: 550 mm * 350 mm

Thickness of slab = 200 mm

Seismic Zone V

Type of soil: Hard Soil(Type 1)

Importance factor = 1

Response Reduction Factor =5

Live load = 2.5 kN/mm^2

The concrete mix used is M25 for beams and slabs and M30 for columns.

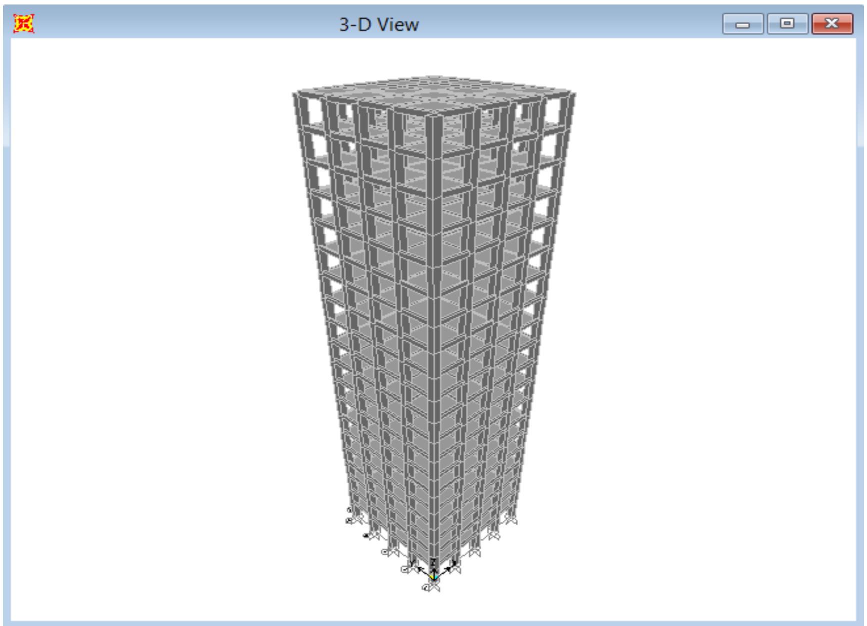


FIG 4.1. -3D elevational view of buildings

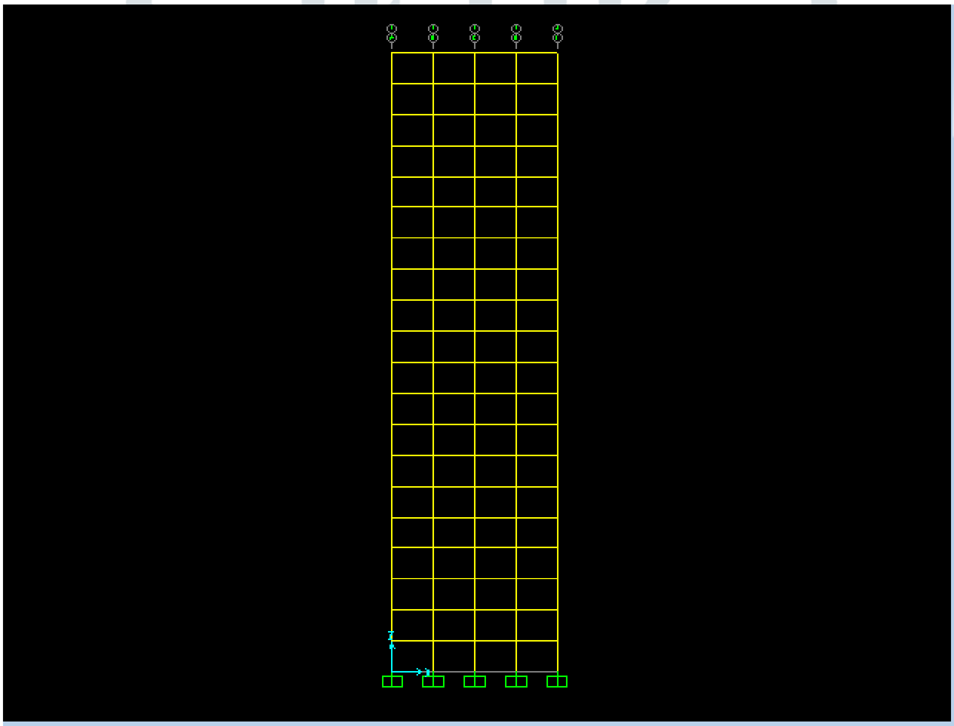


FIG.4.2. 2D Elevation view of buildings

Analysis Results

Displacements(mm) obtained from non-linear time history analysis:

Table.5.1. Displacements(mm) obtained from non-linear time history analysis

Storey	NORTHRIDGE	KOBE	ELCENTRO
GF	2	4.3	4.6
1	12.9	14.8	14.32
2	23.9	25.4	26.37
3	36.4	38.1	38.6
4	49.1	51.2	50.7
5	62.0	64.2	62.1
6	74.8	77.2	72.7
7	87.5	90.2	82.4
8	100.0	103.3	91.3
9	112.3	116.1	99.5
10	124.3	128.9	107.2
11	136.3	141.4	114.4
12	149.1	153.4	120.8
13	162.8	165.1	126.3
14	174.9	176.3	131
15	186	186.8	135.3
16	195.9	196.7	139.3
17	204.4	205.7	143
18	211.5	213.8	146
19	217.4	221.5	149.2

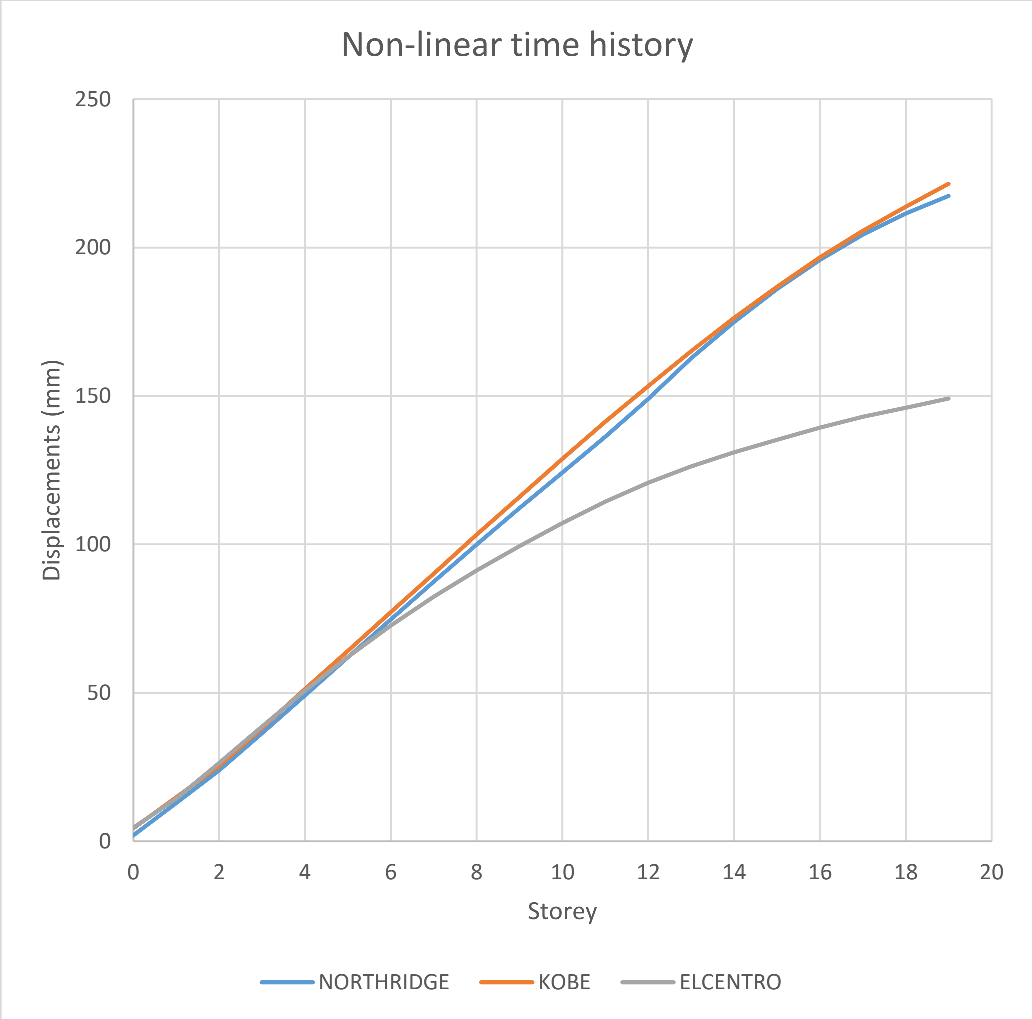


FIG.5.1: DISPLACEMENTS FROM NON-LINEAR TIME HISTORY

PLOT DISPLAY FUNCTIONS FOR THE KOBE'S TIME HISTORY:

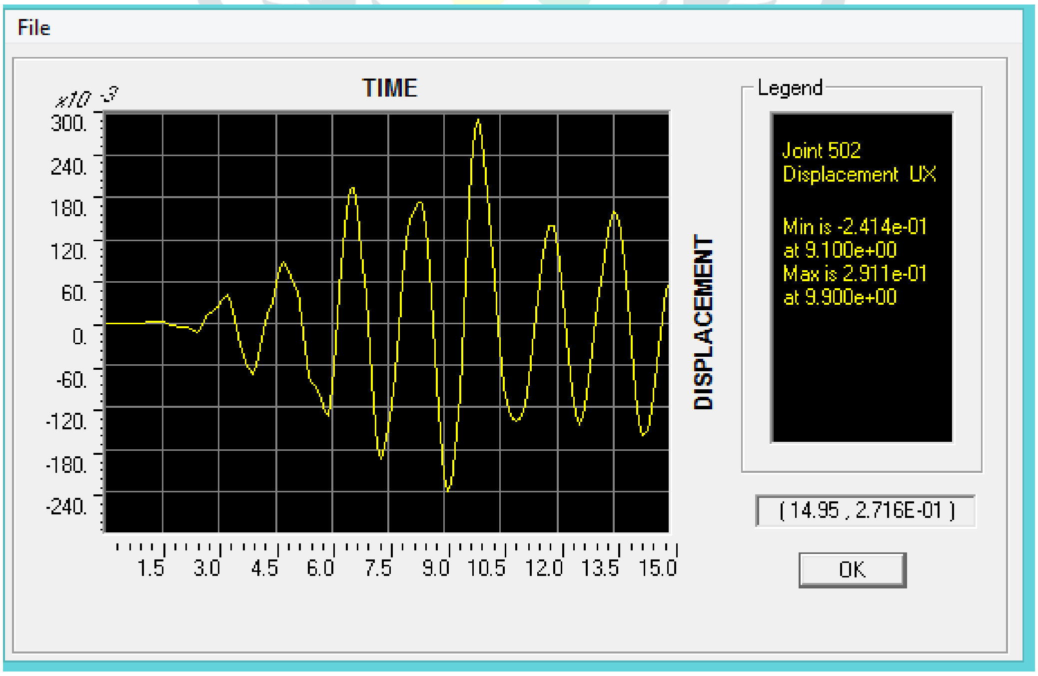


FIG..5.2: DISPLACEMENT AT 20 th FLOOR USING KOBES EARTHQUAKE TIME HISTORY

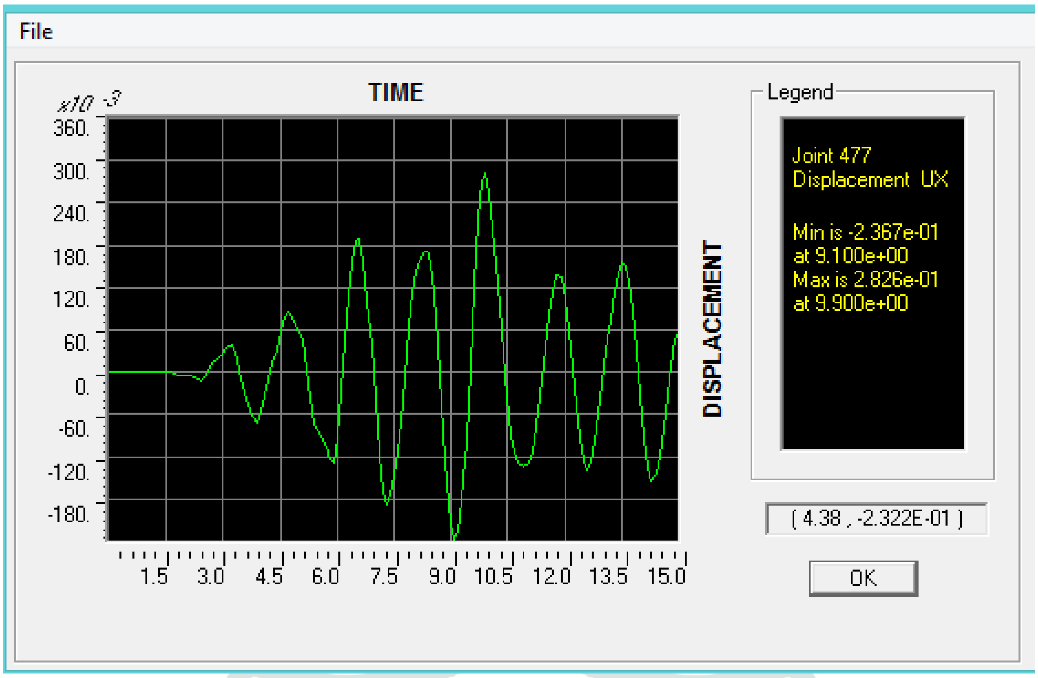


FIG.5.3.: DISPLACEMENT AT 19 th FLOOR USING KOBE'S EARTHQUAKE TIME HISTORY

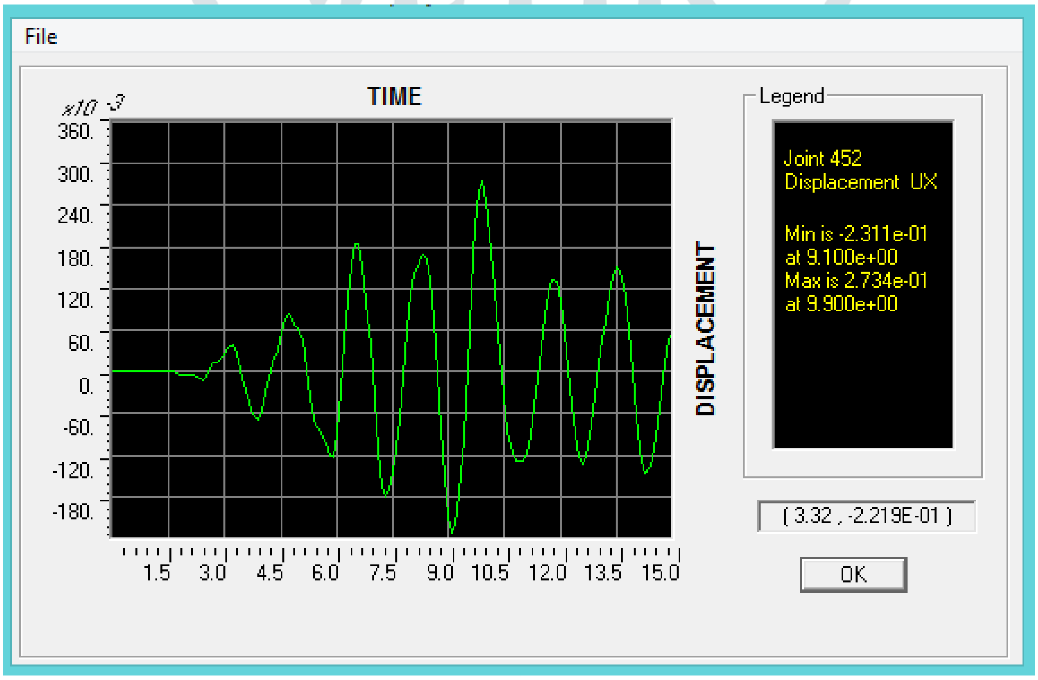


FIG.5.4: DISPLACEMENT AT 18 th FLOOR USING KOBE'S EARTHQUAKE TIME HISTORY

PLOT DISPLAY FUNCTIONS FOR THE KOBE'S TIME HISTORY:

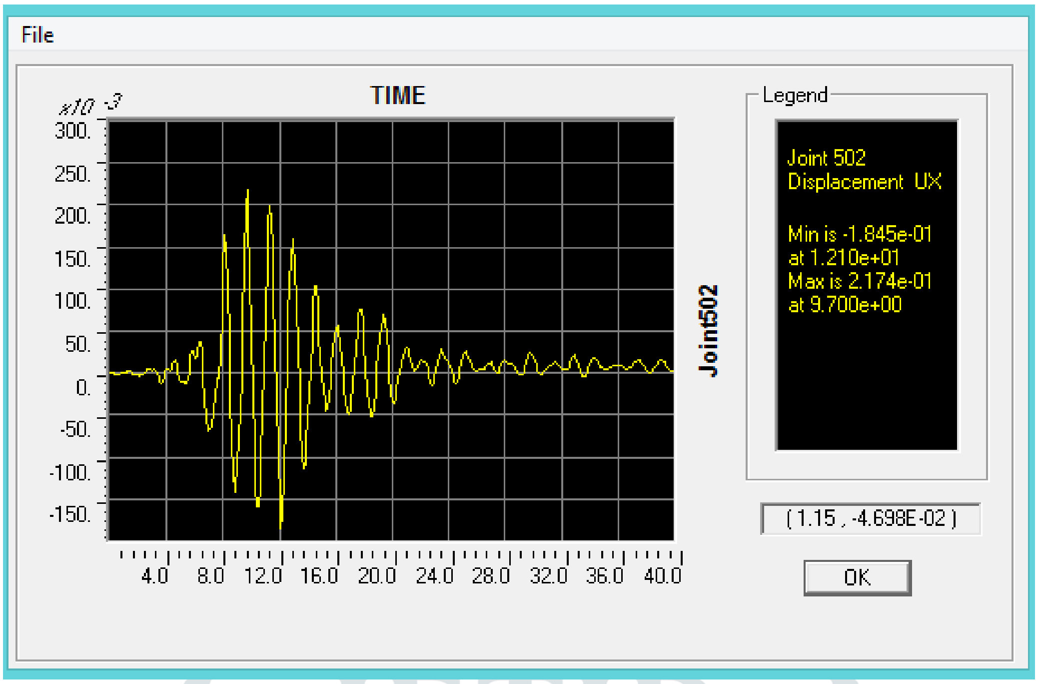


FIG.5.5: DISPLACEMENT AT 20 th FLOOR USING NORTHRIDGE EARTHQUAKE TIME HISTORY

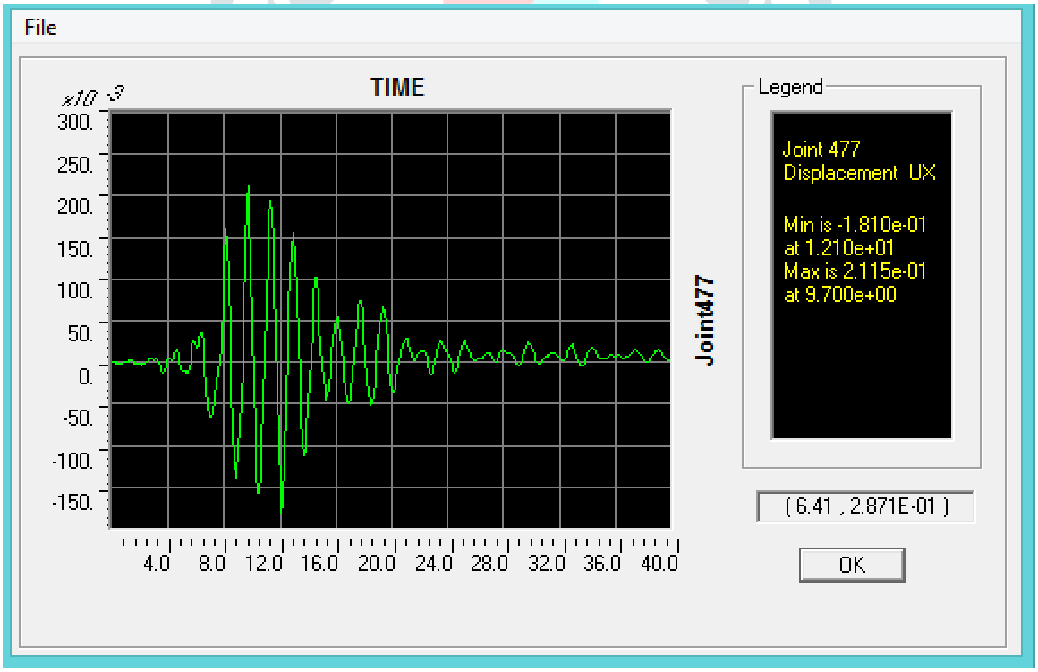


FIG.5.6.: DISPLACEMENT AT 19 th FLOOR USING NORTHRIDGE EARTHQUAKE TIME HISTORY

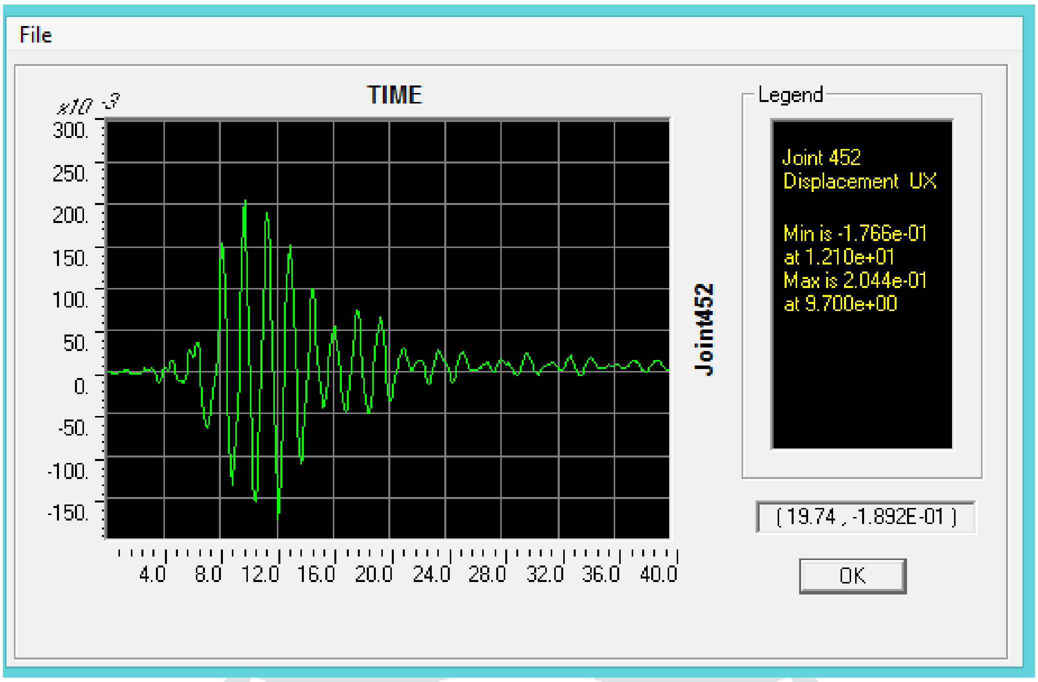


FIG.5.7.: DISPLACEMENT AT 18 th FLOOR USING NORTHRIDGE EARTHQUAKE TIME HISTORY

PLOT FUNCTIONS FOR EL-CENTRO EARTHQUAKE

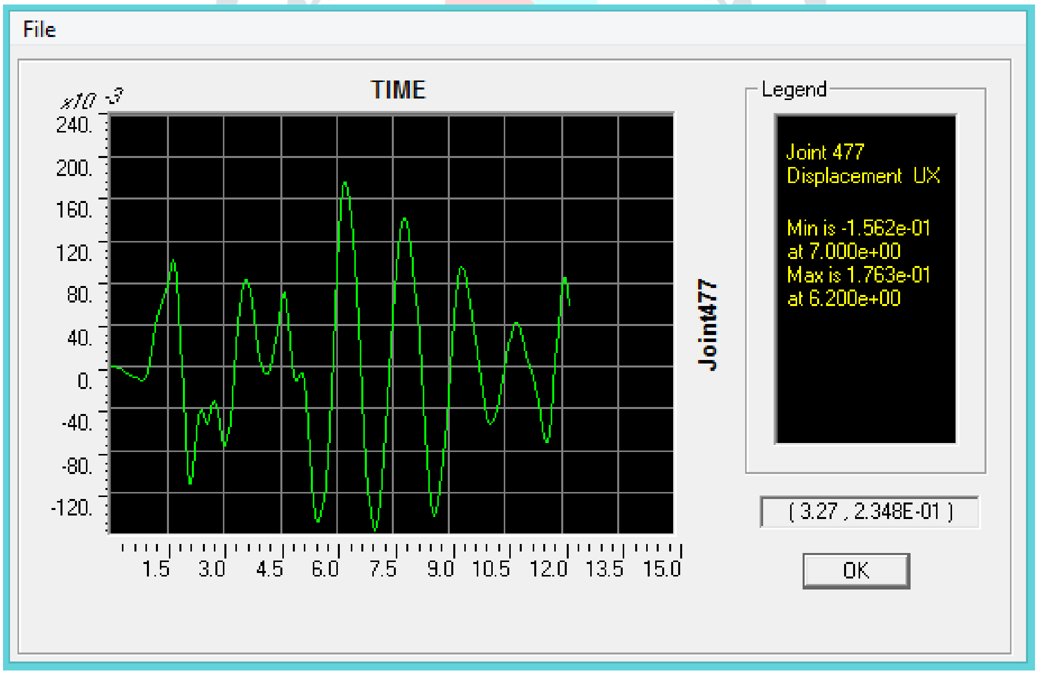


FIG.5.8.: DISPLACEMENT AT 20th FLOOR USING EL-CENTRO EARTHQUAKE TIME HISTORY

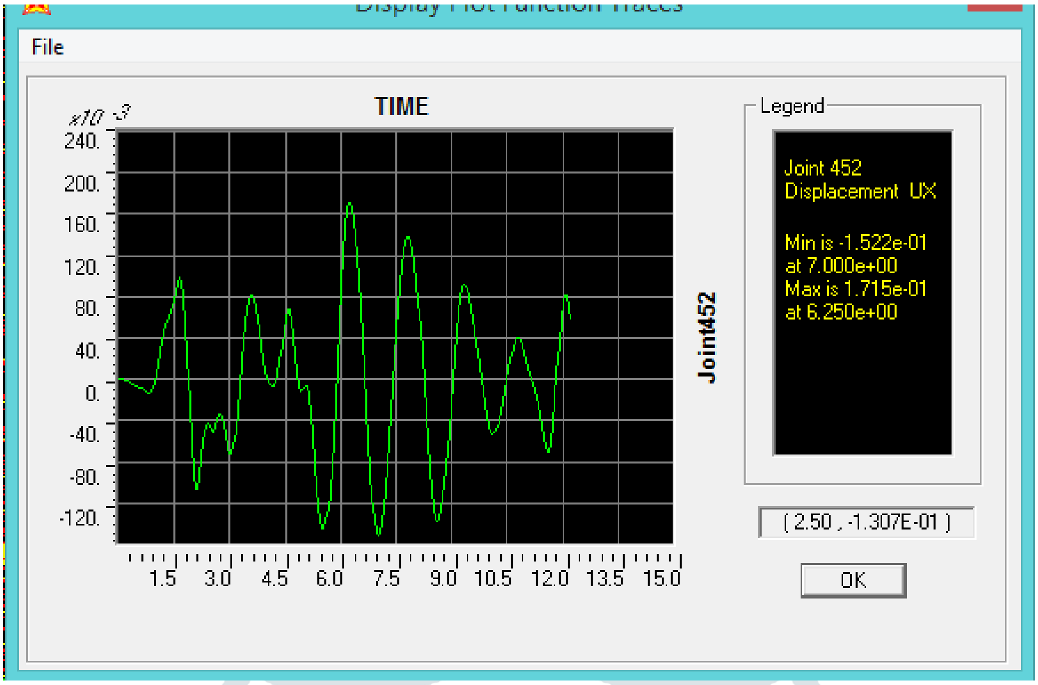


FIG.5.9.: DISPLACEMENT AT 19th FLOOR USING EL-CENTRO EARTHQUAKE TIME HISTORY

INTER STOREY DRIFTS FROM NON-LINEAR TIME HISTORY ANALYSIS:

Table.5.2. INTER STOREY DRIFTS FROM NON-LINEAR TIME HISTORY ANALYSIS

Storey	**NORTHRIDGE**	**KOBE**	**ELCENTRO**
GF	2	4.3	4.6
1	10.9	10.5	9.7
2	11.0	10.6	12.1
3	12.5	12.7	12.2
4	12.8	13.1	12.1
5	12.9	13	11.4
6	12.8	13	10.6
7	12.7	13	9.7
8	12.5	13.1	8.9
9	12.3	12.8	8.2
10	11.9	12.8	7.7
11	12.0	12.5	7.2
12	13.5	12	6.4
13	12.9	11.7	5.5
14	12.2	11.2	4.7
15	11.0	10.5	4.3
16	9.9	9.9	4.0
17	8.5	9	3.7
18	7.1	8.1	3.0
19	5.9	7.7	3.0

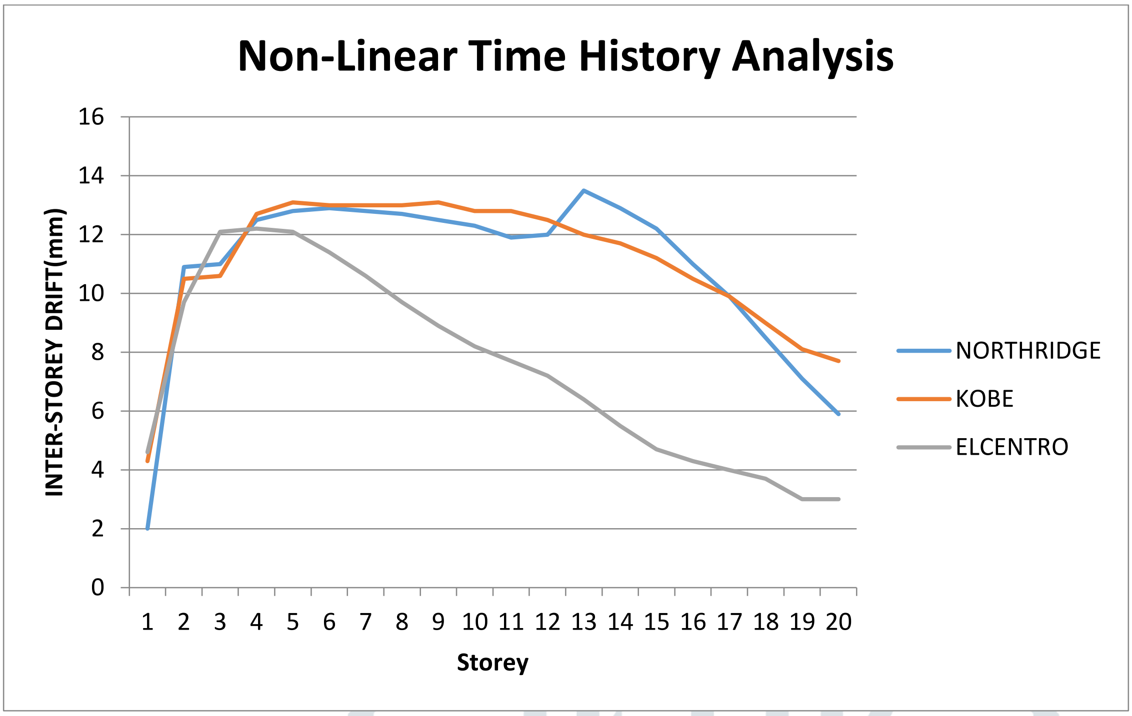


FIG.5.10.: INTER-STOREY DRIFT FROM NON-LINEAR TIME HISTORY

BASE SHEAR FROM NON-LINEAR TIME HISTORY ANALYSIS

Table.5.3. BASE SHEAR FROM NON-LINEAR TIME HISTORY ANALYSIS

S.NO	EARTHQUAKE	BASE SHEAR(KN)
1.	NORTHRIDGE	12744
2.	KOBE	16659
3.	EL-CENTRO	12798

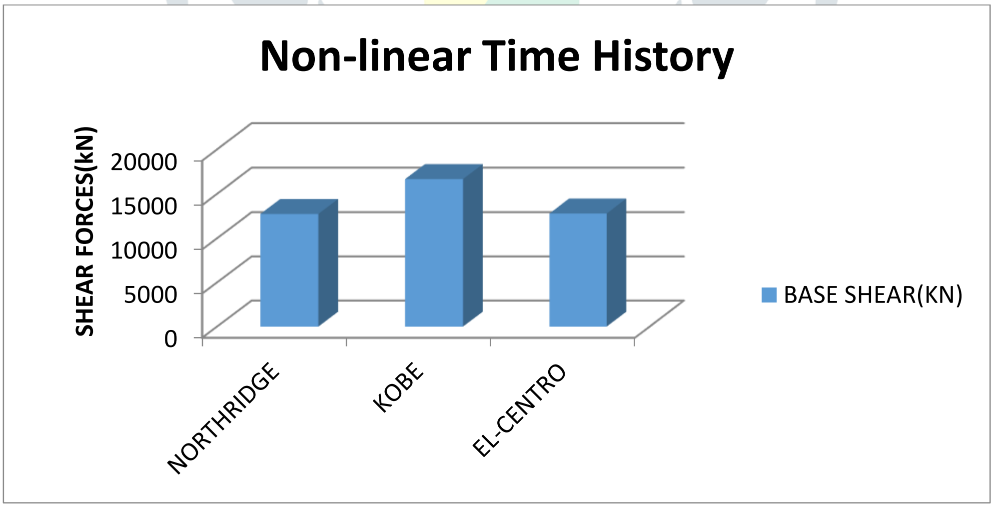


FIG.5.11.: BASE SHEAR FROM DIFFERENT TIME HISTORIES

PLOT OF BASE SHEAR FROM NON-LINEAR TIME HISTORY:

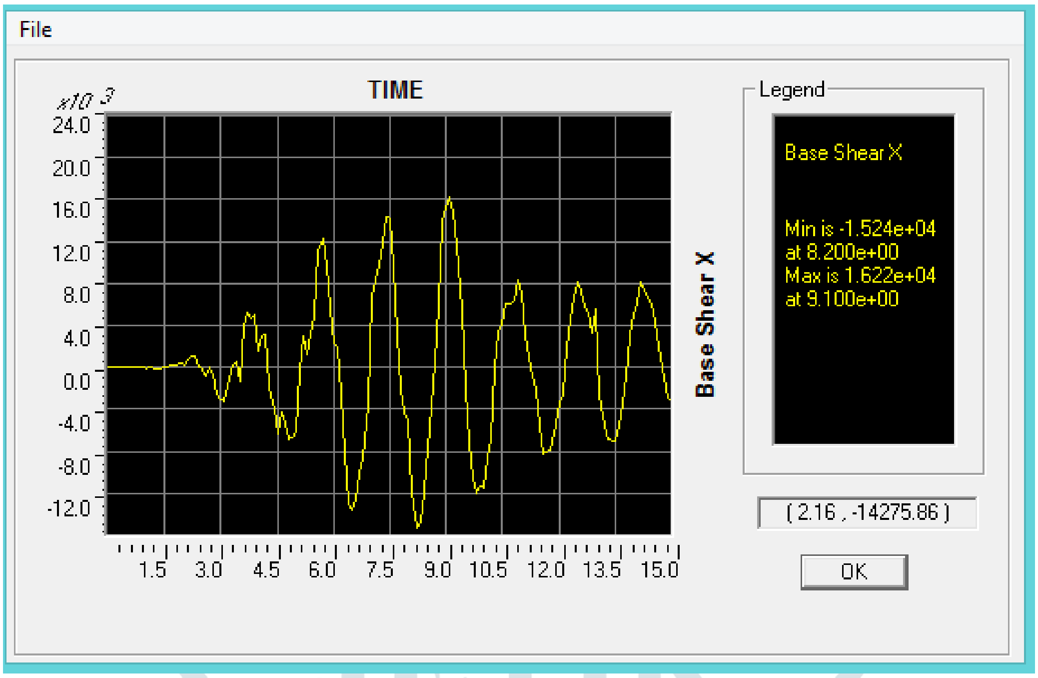


FIG.5.12.: BASE SHEAR FROM KOBE'S EARTHQUAKE

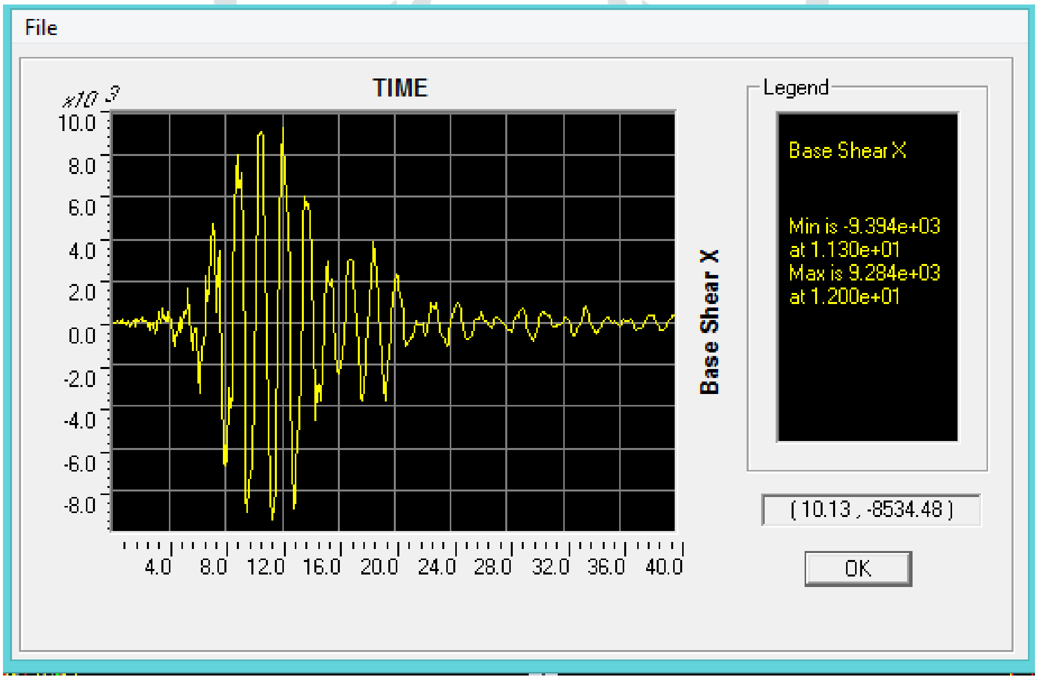


FIG.5.13.: BASE SHEAR FROM NORTHRIDGE EARTHQUAKE

RESULTS FOR PUSHOVER ANALYSIS FROM CAPACITY SPECTRUM:

1.PERFORMANCE POINT AS BASE SHEAR VS TOP ROOF DISPLACEMENT

Table.5.4. PERFORMANCE POINT AS BASE SHEAR VS TOP ROOF DISPLACEMENT

S.NO	RESPONSE SPECTRUM	BASE SHEAR,V(KN)	TOP ROOF DISPLACEMENT,D (mm)
1.	NORTHRIDGE	13599.3	152
2.	KOBE	13924.6	156
3.	EL-CENTRO	11815.9	132

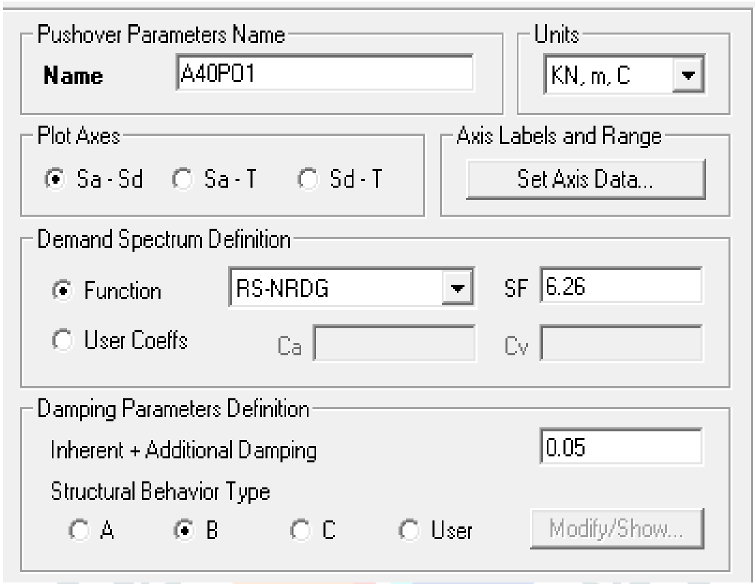


Fig.5.14. Parameters details

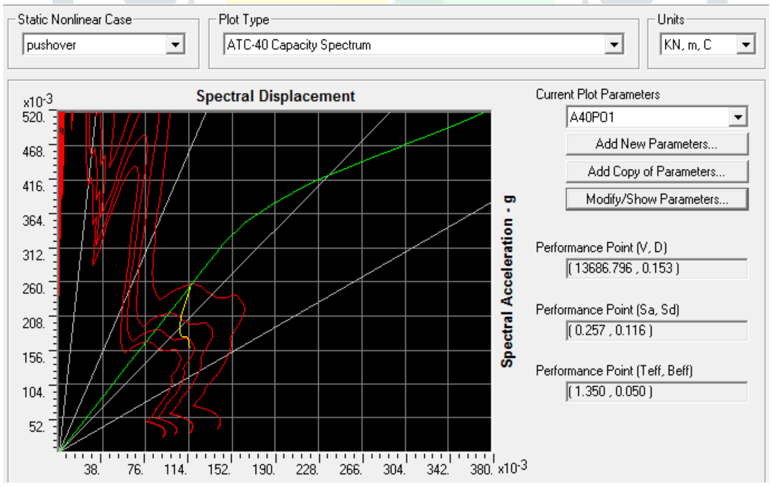


Fig.5.15. Spectral Displacement-Acceleration Graph

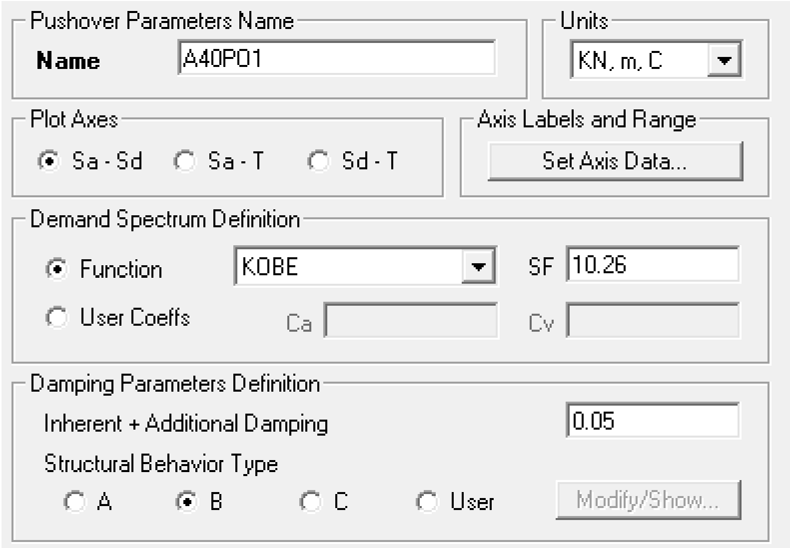


Fig.5.16. Pushover Parameters

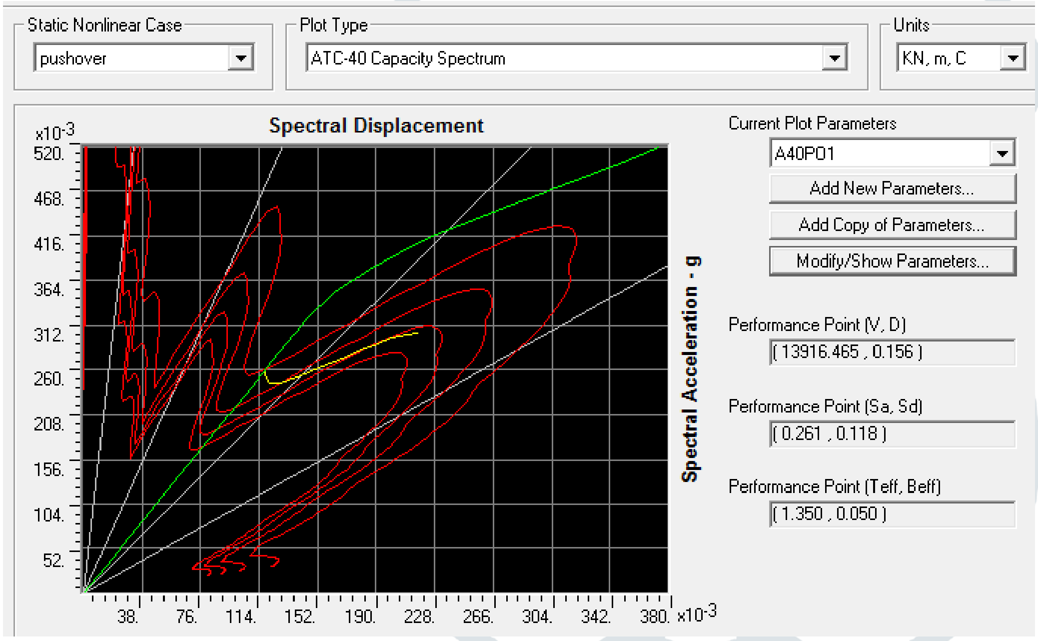


Fig.5.17. Spectral Displacement-Acceleration Graph

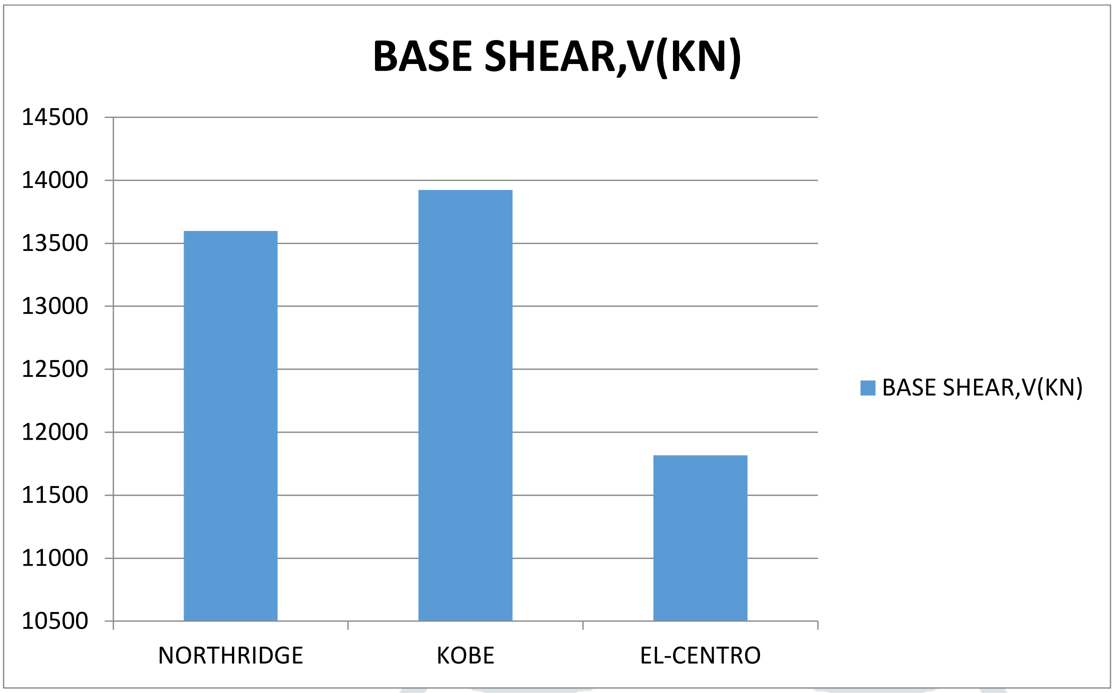


Fig.5.18.: Base Shear from Pushover Analysis Results For Different Response Spectra

2.PERFORMANCE POINT AS SPECTRAL ACCELERATION VS SPECTRAL DISPLACEMENT

Table.5.5. Performance Point as Spectral Acceleration Vs Spectral Displacement

S.NO	RESPONSE SPECTRUM	SPECTRAL ACCELERATION, Sa(g)	SPECTRAL DISPLACEMENT, Sd(mm)
1.	NORTHRIDGE	0.255	115
2.	KOBE	0.261	118
3.	EL-CENTRO	0.22	100

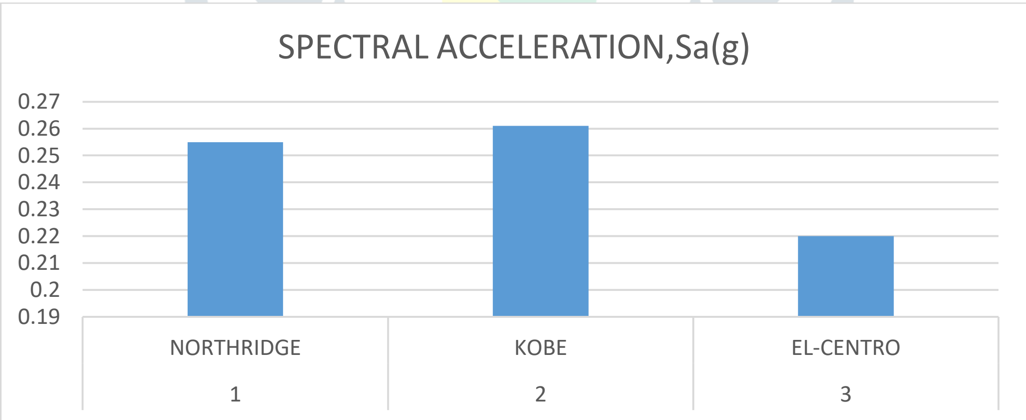


FIG.5.19.: Performance Point in Terms Of Spectral Acceleration

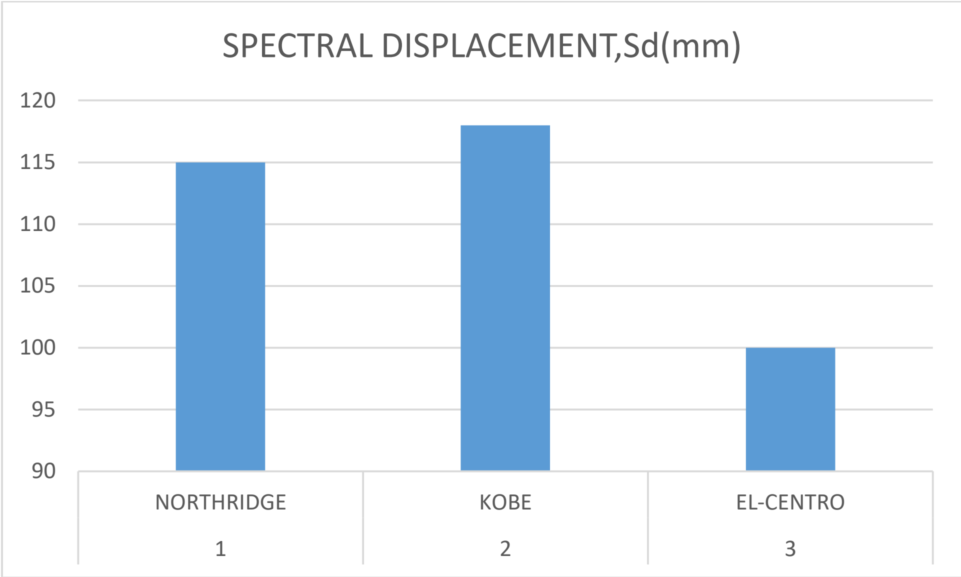


Fig.5.20.: Performance Point in Terms Of Spectral Displacement

Comparison of base shear by pushover and time history analysis

Table.5.6. Comparison of Base Sear by Pushover & Time History Analysis

EARTHQUAKE	TIME HISTORY-BASE SHEAR(KN)	RESPONSE SPECTRA-BASE SHEAR(KN)
NORTHRIDGE	12744	13599.3
KOBE	16659	13924.6
EL-CENTRO	12798	11815.9

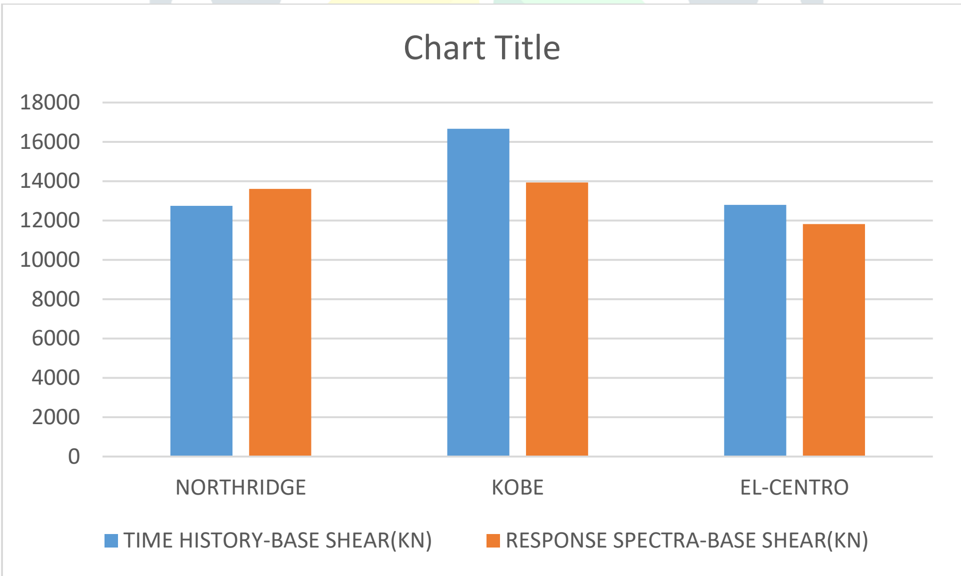


Fig.5.21.: Comparison of Base Shear By Time History And Pushover

- Maximum base shear was observed for kobes earthquake and was more than electros earthquake by 15.14% and was more than Northridge by 2.34% for pushover analysis.
- Maximum top floor displacement was observed for Kobe's earthquake and was more than elcentros earthquake by 15.13% for pushover analysis.

- Performance point in terms of spectral acceleration was observed maximum for kobes and varied from el-centro by 15.71% and by 2.3% from North-ridge earthquake.
- Performance point in terms of spectral displacement was observed minimum in case of El Centro and was lesser than Kobe's by 18% and by 15% for Northridge.
- Maximum base shear was observed Kobe's earthquake and was more than el centro's earthquake by 23.17% and was more than Northridge by 23.5% for non-linear time history analysis.
- Maximum top floor displacement obtained for Kobe's earthquake and varied with elcentro by 32.64% for non-linear time history analysis.
- Maximum inter storey drift is in case of Northridge and had value of 13.5mm while in case of el centro it was observed to be 12.2 mm and for Kobe's it was 13.1 mm.
- The base shear was found to be more for pushover analysis than non-linear time history analysis by 6.3% when Northridge earthquake was applied.
- The base shear was found to be more for non-linear time history analysis by 16.4% than pushover analysis when Kobe earthquake was applied.
- The base shear was found to be more for non-linear time history analysis than pushover analysis by 7.67% when El Centro earthquake was applied.

Conclusion

1. The 1st mode alone provides adequate estimates of floor displacements but it is inadequate especially in estimating the storey drift.

2. First mode pushover analysis is unable to identify the plastic hinges in upper stories where higher contribution of response is known to be more significant.

The higher modes are necessary to identify hinges in upper stories.

3. The selection of an appropriate load shape for any non-linear static procedure is the key issue in accurate prediction of the structural responses.

4. The 20 storey Reinforced concrete building deforms into the inelastic range which leads to yielding of some of the beams and columns for seismic intensity of 0.36 peak ground acceleration.

5. To evaluate the seismic behaviour of structure with significance higher modes effects, the non-linear dynamic analysis method generally provides more reliable assessment of earthquake performance than the other methods.

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