NON-LINEAR STATIC PUSHOVER ANALYSIS OF A G+2 STOREY REGULAR RCC BUILDING

(MAJOR PROJECT REPORT - II)

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CERTIFICATE

This is to certify that the project titled "**NON-LINEAR STATIC PUSHOVER ANALYSIS OF A G+2 STOREY REGULAR RCC BUILDING**" is a bonafide dissertation work carried out by me, Nitin Behl, Roll No. 2010/STR/19, student of Master of Technology in Structures (Civil Engineering) from Delhi Technological University, New Delhi, during the session 2012-2013 towards the partial fulfilment of the requirements for award of the degree of Master of Technology in Structures (Civil Engineering).

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ABSTRACT

When the structures are regular and of small height, simple linear static analysis or dynamic analysis (response spectrum analysis) methods are better to get accurate solutions. But as one aims for high-rise structures and irregularity is introduced within the structure, linear static analysis does not yield optimum results. Moreover linear static analysis method assumes that the material property lies within linearity zone i.e. elastic zone. It does not consider the redistribution of moments and concept of plastic zone. Hence the design is conservative. For small regular structures, it is safer to be conservative in analysing and design. But as we go for high-rises, if we are conservative, this will affect the cost of the project and may work out to be significant. Also the fact remains that the linear static analysis will not be able to predict the actual behaviour of the structure. Hence, to overcome this difficulty of analysing the complex behaviour of such reinforced concrete structures, enhanced analysis methods known as non-linear static analysis and non-linear dynamic analysis methods have been developed. By performing analysis using these methods, nowadays engineers can predict the actual behaviour of the structure and make optimum designs.

Apart from the above, the devastation caused due to collapse of structures during earthquakes was primarily due to constraints in the linear static and dynamic analysis methods. To overcome this limitation, either a non-linear static or dynamic analysis is desirable. The best and most accurate method for this purpose is the non-linear dynamic analysis as it incorporates the non-linearity and the dynamic effects. But the non-linear dynamic analysis which is better known as the Time History Analysis requires the selection and employment of an appropriate set of ground motions followed by effectively analysing the data to produce ready-to-use results. To perform these activities, the time required for even simple structures will be very high. Hence, due to its simplicity and less time requirement, the structural engineering profession has been utilising the non-linear static analysis procedure which is also known as the pushover analysis. Modelling for such analysis requires the determination of the non-linear properties of each component in the structure, quantified by strength and deformation capacities which depend upon modelling assumptions. Pushover Analysis is carried out for either user-defined nonlinear hinge properties or default hinge properties, available in some programs based on FEMA-356 and ATC-40 guidelines. Many papers/journals provide the hinge properties for several ranges of detailing while the programs may implement averaged values. The user needs to be careful; the misuse of default-hinge properties may lead to unreasonable displacement capacities for existing structures. Plastic hinge length and transverse reinforcement spacing are assumed to be effective parameters in the user-defined hinge properties. These parameters have considerable effects on the displacement capacity of the frames. An increase in the amount of transverse reinforcement improves the displacement capacity. This dissertation aims to evaluate the performance of an existing four storey RCC frame hospital building located in zone–V as per parameters given in ATC-40. Since the Hospital building was constructed more than 50 years ago, the construction was guided by the parameters of older prevalent version of the respective codes i.e. IS1893:1984, IS456:1978, etc. which have now been revised and updated. It was required to check the actual behaviour of the structure and point out its performance level.

Accordingly, hinge properties were generated for each member. Also as the structure was a low rise structure and the first mode was the dominating mode, hence the lateral load was applied on the structure in pattern similar to the first mode shape. Finally, the results of pushover analysis viz. pushover curves and capacity spectrum were conducted in both orthogonal directions to obtain the respective performance points.

CONTENTS

OBJECTIVE AND SCOPE

1.1 OBJECTIVE

To carry out non-linear static pushover analysis of an existing RCC hospital building to locate its performance point in ETABSv9.7.4.

1.2 SCOPE

Following is the scope of the present study to achieve the above objective:

- Modal analysis and Response Spectrum analysis of an existing regular G+2 storied RCC frame hospital building in ETABS v9.7.4 to obtain target displacement due to seismic forces.

- Modal and Response spectrum analysis of above building with infill walls to obtain the target displacement due to seismic forces..

- Incorporation of hinge properties of the different members of the above model by using moment-curvature and interaction diagram from SAP2000.

- To perform non-linear static pushover analysis to obtain the capacity curve and performance point for the above models.

INTRODUCTION

Earlier all structures were designed only for vertical loads and forces acting in the direction of gravity. However, the direction of forces need not be acting in the direction of gravity always. This can be illustrated by the action of a Tsunami wave striking the structure where the force is directly acting on the structure or due to the development of inertia forces developed within the structure due to ground movements i.e. indirectly. In both cases the structure is subjected to forces in lateral directions, for which the behaviour of the structures was not known. This has led to catastrophic failure of many structures causing great loss of life and setback to the economy of the area subjected to such hazards.

Seismic hazard in the context of engineering design is generally defined as the predicted level of ground acceleration which would be exceeded with 10% probability at the site under consideration due to the occurrence of an earthquake anywhere in the region, in the next 50 years. A lot of complex scientific perception and analytical modelling is involved in seismic hazard estimation. A computational scheme involves the following steps: delineation of seismic source zones and their characterisation, selection of an appropriate ground motion attenuation relation and a predictive model of seismic hazard. Although these steps are region specific, certain standardisation of the approaches is highly essential so that reasonably comparable estimates of seismic hazard can be made worldwide, which are consistent across the regional boundaries. The **National Geophysical Research Institute (NGRI), Hyderabad,** India was identified as one such center responsible for estimating the seismic hazard for the Indian region. As it is well known, earthquake catalogues and data bases make the first essential input for the delineation of seismic source zones and their characterisation. Thus, preparation of a homogeneous catalogue for a region under consideration is an important task. The data from historic time to recent can broadly be divided in to three temporal categories: 1) since 1964, for which modern instrumentation based data are available 2) 1900-1963, the era of early instrumental data, and 3) pre 1900, consisting of pre- instrumental data, which is based primarily on historical and macro-seismic information. In India, the scenario is somewhat similar. The next key component of seismic hazard assessment is the creation of seismic source models, which demand translating seismo-tectonic information into a spatial approximation of earthquake localisation and temporal recurrence. For this purpose, all the available data on neo-tectonics, geodynamics, morpho structures etc., need to be compiled and viewed, overlain on a

seismicity map. These maps then need to be critically studied for defining aerial seismic source zones and active faults. An earthquake recurrence model is then fitted to these source zones, for defining the parameters that characterise the seismicity of the source region, which go as inputs to the algorithm for the computation of seismic hazard.
The uncertainties involved in accurate determination of material properties, element and structure capacities, the limited prediction of ground motions that the structure is going to experience and the limitations in accurate modelling of structural behaviour make the seismic performance evaluation of structures a complex and difficult process.

To understand the actual behaviour of structures, a method known as performance based design was envisaged. In performance based design, the structure is analysed by a suitable procedure to obtain a target building performance level. Hence, to get in depth knowledge of performance based design, two parameters i.e. the respective building performance levels and the procedures available need to be studied in detail so that a correct choice of analysis procedure is made to achieve the realistic target building performance level. Accordingly, the various building performance levels and the procedures which are there to conduct the analysis are given in the ensuing paragraphs.

2.1 BUILDING PERFORMANCE LEVELS AND RANGES (ATC, 1997A)

As mentioned above, pushover analysis is one of the methods to carry out performance based design. Hence, it becomes imperative to understand the different performance levels and their ranges to decide the level that one wants to achieve before doing the pushover analysis. Accordingly, the terminology used in performance based design is given below:

2.1.1 Performance Level: The intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

2.1.2 Performance Range: a range or band of performance, rather than a discrete level.

2.1.3 Designations of Performance Levels and Ranges: The performance levels are grouped under two heads. One group is named as the structural performance level and ranges and the second group is named as the non-structural performance levels. Structural performance levels are identified by both a name and numerical designator i.e. S-1 through S-5 while the non-structural performance levels are identified by a name and alphabetical designator i.e. N-A through N-D. They are mentioned below:

2.1.3.1 Structural Performance Levels & Ranges:

- S-1: Immediate Occupancy Performance Level: It relates to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.
- S-2: Damage Control Performance Range: It corresponds to the continuous range of damage states that entail less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level. Design for Damage Control performance may be desirable to minimize repair time and operation interruption; as a partial means of protecting valuable equipment and contents; or to preserve important historic features when the cost of design for Immediate Occupancy is excessive. Acceptance criteria for this range may be obtained by interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels.
- S-3: Life Safety Performance Level: It relates to the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical.
- S-4: Limited Safety Performance: It corresponds to the continuous range of damage states between the Life Safety and Collapse Prevention levels. Design parameters for this range may be obtained by interpolating between the values provided for the Life Safety (S-3) and Collapse Prevention (S-5) levels.
- S-5: Collapse Prevention Performance Level: It relates to the post-earthquake damage state where the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure and to more limited extent degradation

in vertical-load-carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.

- S-6: Structural Performance not considered (covering the situation where only non-structural improvements are made)

2.1.3.2 Non-structural Performance Levels:

- N-A:Operational Performance Level: It corresponds to the post-earthquake damage state of the building in which the non-structural components are able to support the building's intended function. At this level, most non-structural systems required for normal use of the building including lighting, plumbing, etc.; are functional, although minor repair of some items may be required. This performance level requires considerations beyond those that are normally within the sole province of the structural engineer.
- N-B: Immediate Occupancy Performance Level: It corresponds to the post-earthquake damage state in which only limited non-structural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable. There could be minor window breakage and slight damage to some components. Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function if necessary utility service is available. However, some components may experience misalignments or internal damage and be non- operable. Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to non-structural damage is very low.
- N-C: Life Safety Performance Level: It corresponds to the post-earthquake damage state in which potentially significant and costly damage has occurred to non-structural components but they have not become dislodged and fallen, threatening life safety either within or outside the building. Egress routes within the building are not extensively blocked. While injuries may occur during the earthquake from the failure of

non-structural components, it is expected that, overall, the risk of life-threatening injury is very low. Restoration of the non-structural components may take extensive effort.

- N-D: Hazards Reduced Performance Level: It corresponds to the post-earthquake damage state level in which extensive damage has occurred to non-structural components, but large or heavy items that pose a falling hazard to a number of people such as parapets, cladding panels, heavy plaster ceilings, or storage racks are prevented from falling. While isolated serious injury could occur from falling debris, failures that could injure large numbers of persons either inside or outside the structure should be avoided. Exits, fire suppression systems, and similar life-safety issues are not addressed in this performance level.
- N-E: Non-structural Performance Not Considered (covering the situation where only structural improvements are made.

2.1.4 Building Performance Level: Target Building performance is a combination of the performance of both structural and non-structural components. Three Structural Performance Levels and four Non-structural Performance Levels are used to form the four basic Building Performance Levels. They are: Collapse Prevention, Life Safety, Immediate Occupancy, and operational. Figure 2.1 shown below, shows these levels as discrete points on a continuous scale describing the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur.

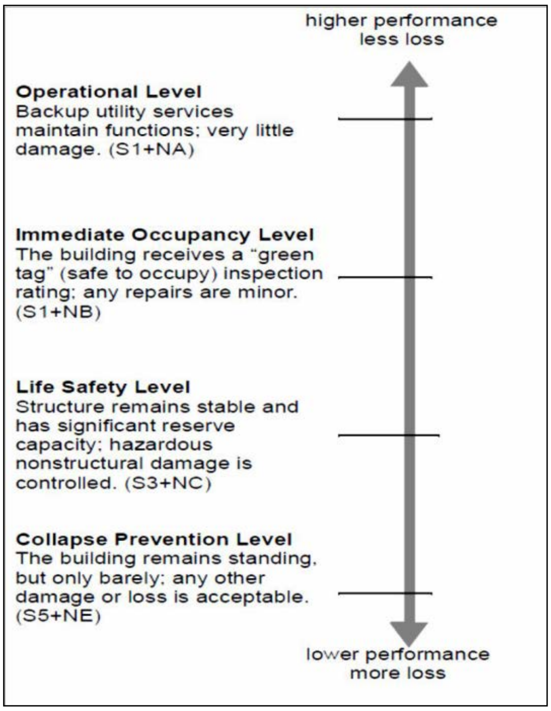


Fig. 2.1 Building Performance Levels (ATC, 1997a)

Based on this method, a performance point for each structure is calculated to know its behaviour and likely loss that would occur under a hazard of known intensity. Fig. 2.2 shows a flow chart that presents the key steps in the performance-based design process. It is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved.

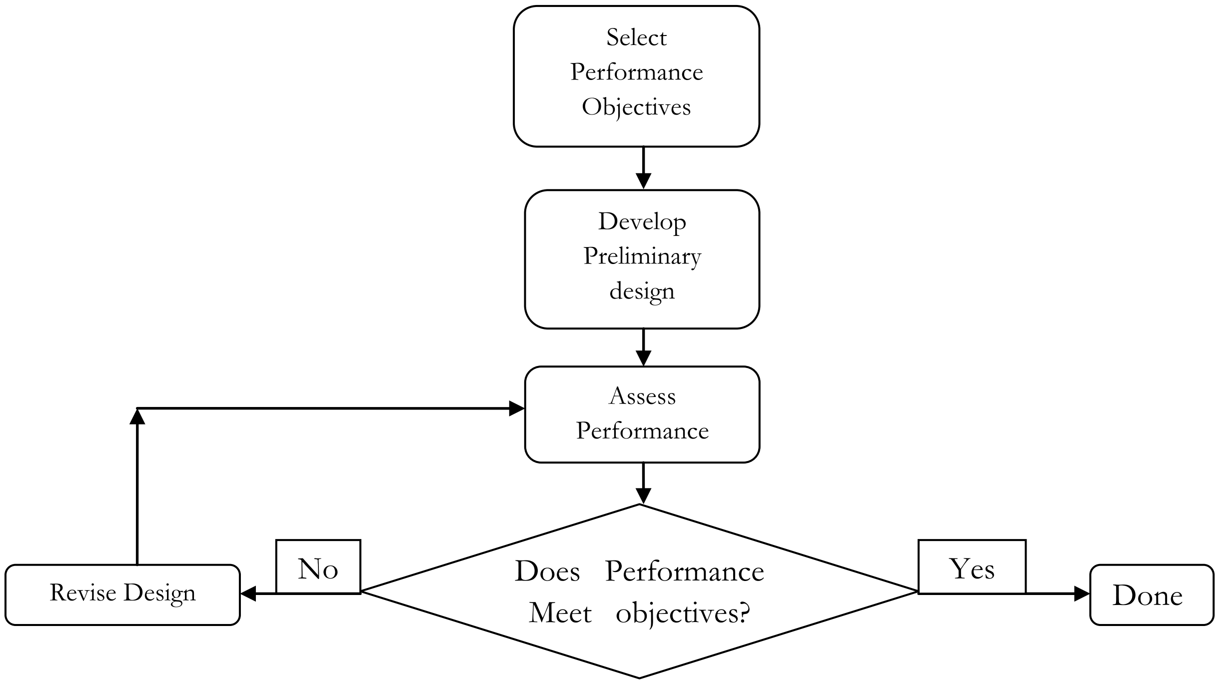


Fig. 2.2 Performance-Based Design Flow Diagram (ATC, 1997a)

2.2 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 linear and non-linear, are available to predict the seismic performance of the structures. Broadly they are classified as follows:

2.2.1 Linear Static Procedures (LSP),
2.2.2 Linear Dynamic Procedures (LDP),
2.2.3 Non-Linear Static Procedures (NLSP),
2.2.4 Non-Linear Dynamic Procedures (NLDP).

The first two methods fall under the linear or elastic analysis procedures while the latter two are non0-linear analysis procedures.

In the linear procedures, the force demand on each component is obtained and compared with the available capacities by performing an linear analysis. Linear 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 effects 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 structures could not be identified by a linear analysis.

However, non-linear 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 inelastic forces to elastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it is known that this factor is a function of the period and ductility ratio of the structure as well.

Linear methods can predict elastic capacity of structure; 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. The dynamic characteristics of the structure change with time so investigating the performance of a structure requires non-linear analytical procedures accounting for these features. Non-linear analytical procedures help to understand the actual behaviour of structures by identifying failure modes and the potential for progressive collapse. Non-linear analysis procedures basically include non-linear time history analysis and non-linear static analysis which is also known as pushover analysis.

The non-linear time history analysis is the most accurate method to predict the force and deformation demands of various components of the structure. However, the use of non-linear 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 interpreting voluminous output make the use of non-linear time history analysis impractical for seismic performance evaluation.

Non-linear 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(10), Displacement Coefficient Method(61) and the Secant Method(65). The theoretical background, reliability and the accuracy of non-linear static analysis procedure is discussed in detail in the following paragraphs.

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 is 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 representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed.

The aim of the present work is to study the behaviour of an existing G+2 storied regular RCC frame structure in ETABS v9.7.4 to study the development of hinges in respective members and finally to locate its performance point. To achieve the above objective, a model was created in ETABS v9.7.4 for simulating the actual ground conditions. After generating the model, modal analysis was performed to obtain the data for the different mode shapes and their patterns. The fundamental mode or first mode shape details i.e. it's time period and total base shear contribution was obtained. Thereafter the hinges were generated for each member and introduced in the model and Pushover Load Case in pattern similar to the first mode shape was applied. Finally the analysis results were checked to see the pattern of formation of hinges and their respective locations. The Acceleration versus Displacement graph was plotted to get the performance point of the structure.

REVIEW OF LITERATURE

Structures are expected to deform in-elastically when subjected to severe earthquakes, so seismic performance evaluation of structures should be conducted considering post-elastic behaviour. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as post-elastic behaviour cannot be determined by an elastic analysis. Moreover, maximum inelastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behaviour.

Various simplified nonlinear analysis procedures and approximate methods to estimate maximum inelastic displacement demand of structures are proposed in literature. The widely used simplified nonlinear analysis procedure, pushover analysis, has also been an attractive subject of study.

The accuracy and reliability of nonlinear time history analysis in simulating the actual behaviour of structure under seismic action has been widely accepted since 1960s. However, the time required for proper modelling, input preparation, computation time, computer costs and the effort for the interpretation of voluminous output make use of such analyses impractical. This led researchers to propose simplified nonlinear analysis procedures and structural models to estimate inelastic seismic demands. The proposed simplified nonlinear analysis procedures and structural models are usually based on the reduction of MDOF model of structures to an equivalent SDOF system.

Rosenblueth and Herrera[(1)] proposed a procedure in which the maximum deformation of inelastic SDOF system is estimated as the maximum deformation of a linear elastic SDOF system with lower lateral stiffness (higher period of vibration, Teq) and higher damping coefficient (ζeq) than those of inelastic system. In this procedure, a sequence of equivalent linear systems with successively updated values of Teq and ζeq provide a basis to estimate the deformation of the inelastic system. Rosenblueth and Herrera[(1)] used the secant stiffness at maximum deformation to represent period shift and equivalent damping ratio is calculated by equating the energy dissipated per cycle in nonlinear and equivalent linear SDOF system subjected to harmonic loading.

Gülkan and Sözen[2] noted that most of the time the displacement would be significantly smaller than the maximum response under earthquake loading. Thus the equivalent damping proposed by Rosenblueth and Herrera[1] would result in an overestimation of equivalent viscous damping that the response would be underestimated. Gülkan and Sözen[2] developed an empirical equation for equivalent damping ratio using secant stiffness Takeda hysteretic model[3] and the results obtained from experiments made on single story, single bay frames supported the proposed procedure.

The empirical procedure proposed by Gülkan and Sözen[2] was later extended to MDOF in the well known substitute structure procedure by Shibata and Sözen[4]. Inelastic seismic design force requirements of a RCC structure can be determined by analysing a substitute structure having the stiffness and damping properties derived from the original frame under an elastic response spectrum. In the procedure, the displacement ductility ratio was replaced with a damage ratio in the equivalent viscous damping ratio equation proposed by Gülkan and Sözen[2]. Only 2D models of structures which are regular in plan and elevation can be analysed by the procedure.

Iwan[5] and Kowalsky[6] developed empirical equations to define the period shift and equivalent viscous damping ratio to estimate maximum displacement demand of inelastic SDOF system from its linear representation.

In 1981, Q-model which is a 'low-cost' analytical model for the calculation of displacement histories of multi-storey reinforced concrete structures subjected to ground motions was proposed by Saiidi and Sözen[7]. Q-model is a SDOF system consisting of an equivalent mass, a viscous damper, a massless rigid bar and a rotational spring. The hysteretic response of the spring was based on force-displacement curve of actual structure under monotonically increasing lateral force with a triangular height-wise distribution. The measured displacement histories of eight 10-story small scale RCC structures with frame and frame-wall structural systems were used to test the Q-model. For structures without abrupt changes in stiffness and mass along their heights, the overall performance of Q-model in simulating earthquake response was satisfactory.

Later, Fajfar and Fischinger[8] proposed the N2 method as a simple nonlinear procedure for seismic damage analysis of reinforced concrete buildings. The method uses response spectrum approach and nonlinear static analysis. The method was applied to three 7-story buildings[9]. The capacity curve of a MDOF system was converted to that of a SDOF and a global demand was obtained. A damage model which includes cumulative damage was determined at global demand. The method yields reasonably accurate results provided that the structure vibrates predominantly in the first mode.

Capacity Spectrum Method[10] is one of the most popular methods utilized for a quick estimate to evaluate the seismic performance of structures. The method is recommended by ATC-40[10] as a displacement-based design and assessment tool for structures. The method was developed by Freeman[11] and it has gone through several modifications since then. The most recent three versions (Procedures A, B and C) of Capacity Spectrum Method[10] are presented in detail in ATC-40[10]. The method requires construction of a structural capacity curve and its comparison with the estimated demand response spectrum, both of which are expressed in Acceleration-Displacement Response Spectrum (ADRS) format. Mahaney et al.[12] introduced the ADRS format that the spectral accelerations are plotted against spectral displacements with radial lines representing the period, T. The demand (inelastic) response spectrum accounting for hysteretic nonlinear behaviour of structure is obtained by reducing elastic response spectrum with spectral reduction factors which depend on effective damping. A performance point that lies on both the capacity spectrum and the demand spectrum (reduced for nonlinear effects) is obtained for performance evaluation of the structure. The dependence of spectral reduction factors on structural behaviour type (hysteretic properties) and ground motion duration and the approximations involved in determination of these characteristics are the main weaknesses of the method.

Similarly, Displacement Coefficient Method described in FEMA-356[13] is a non-iterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients.

Newmark and Hall[14] and Miranda[15] proposed procedures based on displacement modification factors in which the maximum inelastic displacement demand of MDOF system is estimated by applying certain displacement modification factors to maximum deformation of equivalent elastic SDOF system having the same lateral stiffness and damping coefficient as that of MDOF system. The procedure proposed by Newmark and Hall[14] is based on the estimation of inelastic response spectra from elastic response spectra while displacement modification factor varies depending on the spectral region. Miranda[15] conducted a statistical analysis of ratios of maximum inelastic to maximum elastic displacements computed from ground motions recorded on firm soils and proposed a simplified expression which depends on ductility and initial vibration period.

Miranda and Ruiz-García[16] conducted a study to evaluate the accuracy of approximate procedures proposed by Rosenblueth and Herrera[1] , Gülkan and Sözen[2], Iwan[5], Kowalsky[6], Newmark and Hall[14] and Miranda[15]. SDOF systems with elasto-plastic, modified Clough stiffness degrading model[17] and Takeda hysteretic model[3] and periods between 0.05 and 3.0 s undergoing six different levels of maximum displacement ductility demands when subjected to 264 ground motions recorded on firm sites from 12 California were used. For each procedure, mean ratios of approximate to exact displacement and dispersion of relative errors were computed as a function of vibration period and displacement ductility ratio. Despite having relatively small mean errors, dispersion of results, particularly for large levels of inelastic behaviour, is substantial. It is concluded that approximate procedures can lead to significant errors in estimation of maximum displacement demand when applied to individual ground motion records.

Moreover, Chopra and Goel[18] have proposed an improved capacity-demand diagram method that uses constant ductility demand spectrum to estimate seismic deformation of inelastic SDOF systems.

More recently, Bracci, Kunnath and Reinhorn[19], Munshi and Gosh[20], Kappos and Manafpour[21] proposed seismic performance evaluation procedures that utilize the basic principles of aforementioned simplified nonlinear analysis procedures.

Most of the simplified nonlinear analysis procedures utilized for seismic performance evaluation make use of pushover analysis and/or equivalent SDOF representation of actual structure. However, pushover analysis involves certain approximations that the reliability and the accuracy of the procedure should be identified.

For this purpose, researchers investigated various aspects of pushover analysis to identify the limitations and weaknesses of the procedure and proposed improved pushover procedures that consider the effects of lateral load patterns, higher modes, failure mechanisms, etc.

Krawinkler and Seneviratna[22] conducted a detailed study that discusses the advantages, disadvantages and the applicability of pushover analysis by considering various aspects of the procedure. The basic concepts and main assumptions on which the pushover analysis is based, target displacement estimation of MDOF structure through equivalent SDOF domain and the applied modification factors, importance of lateral load pattern on pushover predictions, the conditions under which pushover predictions are adequate or not and the information obtained from pushover analysis were identified. The accuracy of pushover predictions were evaluated on a 4-story steel perimeter frame damaged in 1994 Northridge earthquake. The frame was subjected to nine ground motion records. Local and global seismic demands were calculated from pushover analysis results at the target displacement associated with the individual records. The comparison of pushover and nonlinear dynamic analysis results showed that pushover analysis provides good predictions of seismic demands for low-rise structures having uniform distribution of inelastic behaviour over the height. It was also recommended to implement pushover analysis with caution and judgement considering its many limitations since the method is approximate in nature and it contains many unresolved issues that need to be investigated.

Mwafy and Elnashai[23] performed a series of pushover analyses and incremental dynamic collapse analyses to investigate the validity and the applicability of pushover analysis. Twelve reinforced concrete buildings with different structural systems (four 8- story irregular frame, four 12-story regular frame and four 8-story dual frame-wall), with different design accelerations (0.15g and 0.30g) and with different design ductility levels (low, medium and high) were utilized for the study. Nonlinear dynamic analysis using four natural and four artificial earthquake records scaled to peak ground accelerations of 0.15g and 0.30g were performed on detailed 2D models of the structures considering predefined local and global collapse limits. Then, complete pushover-like load-displacement curves in the form of upper and lower response envelopes as well as the best fit (ideal envelope) were obtained for each structure by performing regression analyses using the results of nonlinear dynamic analyses. Also, pushover analyses using uniform, triangular and multimodal load patterns were conducted and pushover curves were obtained. The results showed that the triangular load pattern outcomes were in good correlation with dynamic analysis results and a conservative prediction of capacity and a reasonable estimation of deformation were obtained using triangular load pattern. It was also noted that pushover analysis is more appropriate for low-rise and short period structures and triangular loading is adequate to predict the response of such structures.

Further developments on accounting the inelasticity of lateral load patterns which would enable more accurate analysis of high-rise and highly irregular structures were recommended. The inability of invariant lateral load patterns to account for the redistribution of inertia forces and to predict higher mode effects in post-elastic range have led many researchers to propose adaptive load patterns. Fajfar and Fischinger(8) suggested using story forces proportional to the deflected shape of the structure, Eberhard and Sozen(24) proposed using load patterns based on mode shapes derived from secant stiffness at each load step and Bracci et. al(19) proposed the use of stiffness-dependent lateral force distributions in which story forces are proportional to story shear resistances at the previous step.

İnel, Tjhin and Aschheim(25) conducted a study to evaluate the accuracy of various lateral load patterns used in current pushover analysis procedures. First mode, inverted triangular, rectangular, "code", adaptive lateral load patterns and multimode pushover analysis were studied. Pushover analyses using the indicated lateral load patterns were performed on four buildings consisting of 3- and 9-story regular steel moment
resisting frames designed as a part of SAC joint venture (FEMA-355C) (26) and modified versions of these buildings with a weak first story. Peak values of story displacement, inter-storey drift, story shear and overturning moment obtained from pushover analyses at different values of peak roof drifts representing elastic and various degrees of nonlinear response were compared to those obtained from nonlinear dynamic analysis. Nonlinear dynamic analyses were performed using 11 ground motion records selected from Pacific Earthquake Research Center (PEER) strong motion database. Approximate upper bounds of error for each lateral load pattern with respect to mean dynamic response were reported to illustrate the trends in the accuracy of load patterns. Simplified inelastic procedures were found to provide very good estimates of peak displacement response for both regular and weak-story buildings. However, the estimates of inter-storey drift, story shear and overturning moment were generally improved when multiple modes were considered. The results also indicated that simplifications in the first mode lateral load pattern can be made without an appreciable loss of accuracy.

R.Abhilash, V.Biju and Rahul Leslie(27) in their paper, studied the effects of lateral load patterns in Pushover analysis. They used different types of lateral load patterns, namely uniform load distribution, equivalent lateral force distribution as per FEMA-257, lateral loads from response spectrum analysis as per IS-1893(2002) and the lateral load pattern as per Upper Bound Pushover analysis method. For all the four type loadings the performance points are very close, specially for Uniform loading from FEMA and IS-1893 loading. Similarly Equivalent Lateral loading (FEMA) and UBPA loading performance appeared to be the same. This was due to the close similarity between the load patterns. Different loading pattern shows only slight change in performance point in regular building. The case may vary for irregular buildings. To select the exact loading, performance of buildings in different configuration have to be studied and should be compared with Non-linear Time history analysis.

Sasaki, Freeman and Paret(28) proposed Multi-Mode Pushover (MMP) procedure to identify failure mechanisms due to higher modes. The procedure uses independent load patterns based on higher modes besides the one based on fundamental mode. A pushover analysis is performed and a capacity curve is obtained for each load pattern considering the modes of interest. Structure's capacity for each mode is compared with earthquake demand by using Capacity Spectrum Method(10). Capacity curves and response spectrum are plotted in ADRS format on the same graph and the intersections of capacity spectra with the response spectrum represent the seismic demand on the structure. A 17-story steel frame damaged by 1994 Northridge earthquake and a 12-story steel frame damaged by 1989 Loma Prieta earthquake were evaluated using MMP. For both frames, pushover analysis based only on first mode load pattern was inadequate to identify the actual damage. However, pushover results of higher modes and/or combined effect of 1st mode and higher modes matched more closely the actual damage distribution. It was concluded that MMP can be useful in identifying failure mechanisms due to higher modes for structures with significant higher-order modal response. Although MMP is very useful to identify the effects of higher modes qualitatively, it cannot provide an estimation of seismic responses and their distribution in the structure.

Dhileep. M et al.(29) explained the practical difficulties associated with the non linear direct numerical integration of the equations of motion leads to the use of non linear static pushover analysis of structures. Pushover analysis is getting popular due to its simplicity. High frequency modes and non linear effects may play an important role in stiff and irregular structures. The contribution of higher modes in pushover analysis is not fully developed. The behaviour of high frequency model responses in non linear seismic analysis of structures is not known. In their paper an attempt is made to study the behaviour of high frequency model responses in non linear seismic analysis of structures. Non linear static pushover analysis used as an approximation to non linear time history analysis is becoming a standard tool among the engineers, researches and professionals worldwide. High frequency modes may contribute significantly in the seismic analysis of irregular and stiff structures. In order to take the contribution of higher modes structural engineers may include high frequency modes in the non linear static pushover analysis. The behaviour of high frequency modes in non linear static pushover analysis of irregular structures is studied. At high frequencies, the responses of non linear dynamic analysis converge to the non linear static pushover analysis. Therefore non linear response of high frequency modes can be evaluated using a non linear static push over analysis with an implemental force pattern given by their modal mass contribution times zero period acceleration. The higher modes with rigid content as a major contributing factor exhibit a better accuracy in non linear pushover analysis of structures when compared to the damped periodic modes.

Moghadam[30] proposed a procedure to quantify the effects of higher mode responses in tall buildings. A series of pushover analysis is performed on the buildings using elastic mode shapes as load pattern. Maximum seismic responses are estimated by combining the responses from the individual pushover analyses. The proposed combination rule is that response for each mode is multiplied by mass participating factor for the mode considered and contribution of each mode is summed. The procedure was applied to a 20-story steel moment resisting frame to assess the accuracy of the procedure. The frame was subjected to six earthquake ground motions and mean of maximum displacements and inter-story drift ratios of each story of the frame in six analyses were calculated. Also, pushover analyses for first three modes were performed on the frame and the responses for each mode were combined to estimate the final response. Comparison of estimated displacements and inter-story drifts with the mean of maximum responses resulted from six nonlinear dynamic analysis indicated a good correlation.

Gupta[31] analysed the recorded responses of eight real buildings that experienced ground accelerations in the excess of 0.25g in 1994 Northridge earthquake to understand the behaviour of real structures and to evaluate the acceptability of pushover analysis. The selected buildings were 5, 7, 10, 13, 14, 17, 19- and 20-story structures having moment resisting and shear wall lateral force resisting systems and were instrumented at the time of the earthquake. The recorded story displacement, inter-story drift, story inertia force and story shear profiles at various instants of time were evaluated. It was observed that the response of buildings were significantly affected by higher modes with the exception of low-rise structures and these effects were better understood by analysing the inertia force and story drift profiles rather than displacements. These observations indicated that the pushover analysis is inadequate and un-conservative.

Hence, Gupta[31] proposed Adaptive Modal Pushover Procedure which accounts for the effects of higher modes and limitations of traditional pushover analysis. The proposed method is, at any step, identical to response spectrum analysis. An incremental static analysis of the structure for story forces corresponding to each mode is performed independently. Any response quantity is calculated by an SRSS combination of respective modal quantities. Whenever some member(s) yield, a new structure is created by changing the stiffness of yielded member(s) and the procedure is repeated. The process is repeated until a specified global drift limit is reached. Any number of modes can be considered by the proposed procedure. The applicability and the accuracy of the procedure were evaluated by applying it to 4, 8, 12, 16- and 20-story frames with a variety of lateral force resisting systems (moment resisting frames, frames with soft first story, frames with weak stories and flexure-controlled isolated shear wall). The results of the proposed adaptive procedure were compared with the ones obtained from nonlinear dynamic analyses and pushover analyses with uniform and "code" lateral load patterns. Fifteen earthquake data from the SAC ground motion records[32] for Los Angeles area were used. PGAs of all ground motions used for nonlinear dynamic analyses of a given structure were scaled to have identical elastic 5 percent damped spectral acceleration at the fundamental period to reduce the variability of nonlinear response and to study the effects of higher modes. Global structure behaviour, inter-story drift distributions and plastic hinge locations were studied in detail. The results of the proposed adaptive procedure were in very good correlation with dynamic analyses while pushover analyses failed to capture the effects of higher modes. The procedure was also validated using an existing multi-storey building for which instrumented data was available. The procedure can use site-specific spectra but it is unable to account for the effects of hysteretic degradation.

Chopra and Goel[33] developed an improved pushover analysis procedure named as Modal Pushover Analysis (MPA) which is based on structural dynamics theory. Firstly, the procedure was applied to linearly elastic buildings and it was shown that the procedure is equivalent to the well known response spectrum analysis. Then, the procedure was extended to estimate the seismic demands of inelastic systems by describing the assumptions and approximations involved. Earthquake induced demands for a 9-story SAC building were determined by MPA, nonlinear dynamic analysis and pushover analysis using uniform, "code" and multi-modal load patterns. The comparison of results indicated that pushover analysis for all load patterns greatly underestimates the story drift demands and lead to large errors in plastic hinge rotations. The MPA was more accurate than all pushover analyses in estimating floor displacements, story drifts, plastic hinge rotations and plastic hinge locations. MPA results were also shown to be weakly dependent on ground motion intensity based on the results obtained from El Centro ground motion scaled by factors varying from 0.25 to 3.0. It was concluded that by including the contributions of a sufficient number of modes (two or three), the height-wise distribution of responses estimated by MPA is generally similar to the 'exact' results from nonlinear dynamic analysis.

Chintanapakdee and Chopra(34) evaluated the accuracy of MPA procedure for a wide range of buildings and ground motions. Generic one-bay frames of 3, 6, 9, 12, 15- and 18-stories with five strength levels corresponding to SDOF-system ductility factors of 1, 1.5, 2, 4 and 6 were utilized. Each frame was analysed by a set of 20 large-magnitude small- distance records obtained from California earthquakes. Median values of story drift demands from MPA and nonlinear dynamic analyses were calculated and compared. It was shown that with two or three modes included, MPA predictions were in good correlation with nonlinear dynamic analyses and MPA predicted the changing height-wise variation of demand with building height and SDOF-system ductility factor accurately. The bias and dispersion in MPA estimates of seismic demands were found to increase for longer-period frames and larger SDOF-system ductility factor although no perfect trends were observed. It was also illustrated that the bias and dispersion in MPA estimates of seismic demand for inelastic frames were larger than those for elastic systems due to additional approximations involved in MPA procedure. Finally, the MPA procedure was extended to estimate seismic demand of inelastic systems with seismic demand being defined by an elastic design spectrum.

Jan, Liu and Kao(35) proposed an upper bound pushover analysis procedure to estimate seismic demands of high-rise buildings by considering higher mode effects. In this procedure, the elastic displacement-response contribution ratios of higher modes with respect to fundamental mode is first obtained for a set of earthquake records and number of modes that dominate the displacement response is determined from the envelope curves of contribution ratios. Then, a pushover analysis using the newly formulated lateral load pattern and target displacement considering the contributions of higher modes as well as fundamental mode is performed to estimate seismic demands. The procedure was applied to 2, 5, 10, 20- and 30-story moment resisting frames of strong column-weak beam systems designed according to seismic code of Taiwan. The elastic displacement-response contribution ratios of higher modes were obtained by subjecting the frames to 13 earthquake records chosen from Chi Chi earthquake. The envelope curves of contribution ratios showed that first two mode contributions were dominant that other higher modes were ignored. The proposed pushover analysis method was performed considering first two modes to estimate floor displacements, story drift ratios and plastic hinge rotations. The accuracy of the procedure was evaluated by comparing the results obtained from pushover analysis with triangular loading, modal pushover analysis and nonlinear dynamic analysis. Seismic predictions of pushover analysis with triangular loading and modal pushover analysis were in good correlation with nonlinear dynamic analysis for frames not taller than 10 stories while only the proposed procedure could predict the seismic demands of 20- and 30-story buildings.

J.P. Moehle(36) presented a performance based seismic design of tall buildings in the U.S. He presented that the building codes in the United States contain prescriptive requirements for seismic design as well as an option for use of alternative provisions. Increasingly these alternative provisions are being applied for the performance-based seismic design of tall buildings. Application of performance-based procedures requires: An understanding of the relation between performance and nonlinear response; selection and manipulation of ground motions appropriate to the seismic hazard; selection of appropriate 21 nonlinear models and analysis procedures; interpretation of results to determine design quantities based on nonlinear dynamic analysis procedures; appropriate structural details; and peer review by independent qualified experts to help assure the building official that the proposed materials and system are acceptable. Both practice- and research-oriented aspects of performance-based seismic design of tall buildings are presented.

He said that the west coast of the United States, a highly seismic region, is seeing a resurgence in the design and construction of tall buildings (defined here as buildings 240 feet (73 meters) or taller). Many of these buildings use high-performance materials and framing systems that are not commonly used for building construction or that fall outside the height limits of current buildings codes. In many cases, prescriptive provisions of governing building codes are found to be overly restrictive, leading to designs that are outside the limits of the code prescriptive provisions. This is allowable through the alternative provisions clause of building codes. When the alternative provisions clause is invoked, this normally leads to a performance-based design involving development of a design-specific criteria, site-specific seismic hazard analysis, selection and modification of ground motions, development of a nonlinear computer analysis model of the building, performance verification analyses, development of building-specific details, and peer review by tall buildings design experts.

His views about the new generation of tall buildings in the western U.S. is that Urban regions along the west coast of the United States are seeing a boom in tall building construction. To meet functional and economic requirements, many of the new buildings are using specialized materials and lateral-force-resisting systems that do not meet the prescriptive definitions and requirements of current building codes.

According to Moehle's a design criteria document generally is developed by the designer to clearly and concisely communicate to the design team, the building official, and the peer reviewers the intent and the process of the building structural design. A well prepared document will likely include data and discussion regarding the building and its location; the seismic and wind force-resisting systems; sample conceptual drawings; codes and references that the design incorporates in part or full; exceptions to aforementioned code prescriptive provisions; performance objectives; gravity, seismic, and wind loading criteria; load combinations; materials; methods of analysis including software and modelling procedures; acceptance criteria; and test data to support use of new components. The document is prepared early for approval by the building official and peer reviewers, and may be modified as the design advances and the building is better understood. The design criteria document must define how the design is intended to meet or exceed the performance expectations inherent in the building code. Performance-based seismic analysis of tall buildings in the U.S. increasingly uses nonlinear analysis of a three-dimensional model of the building. Lateral-force-resisting components of the building are modelled as discrete elements with lumped plasticity or fiber models that represent material nonlinearity and integrate it across the component section and length. Gravity framing elements increasingly are being included in the nonlinear models so that effects of building deformations on the gravity framing as well as effects of the gravity framing on the seismic system Because the behaviour is nonlinear, behaviour at one hazard level cannot be scaled from nonlinear results at another hazard level. Furthermore, conventional capacity design approaches can underestimate internal forces in some structural systems (and overestimate them in others) because lateral force profiles and deformation patterns change as the intensity of ground shaking increases(57). Results of non-linear dynamic analysis are sensitive to modelling assumptions. A significant percentage of recent high-rise building construction in the western U.S. has been for residential and mixed-use occupancies. Thus, much of it has been of reinforced concrete, and the majority of those have used reinforced concrete core walls. Some concrete and steel framing, and some steel walls, also are used. Under design-level earthquake ground motions, the core wall may undergo inelastic deformations near the base (and elsewhere) in the presence of high shear.

Ductile performance requires an effectively continuous tension chord, adequately confined compression zone, and adequate proportions and details for shear resistance. In locations where yielding is anticipated, splices (either mechanical or lapped) must be capable of developing forces approaching the bar strength. Furthermore, longitudinal reinforcement is to be extended a distance $0.8l_w$ past the point where it is no longer required for flexure based on conventional section flexural analysis, where l_w is the (horizontal) wall length. Walls generally are fully confined at the base and extending into subterranean levels. Confinement above the base may be reduced (perhaps by half) where analysis shows reduced strains, though strains calculated by nonlinear analysis software generally should be viewed sceptically as they are strongly dependent on modelling assumptions (modelling procedures should be validated by the engineer of record against strains measured in laboratory tests). The reduced confinement usually continues up the wall height until calculated demands under maximum expected loadings are well below spalling levels. Transverse reinforcement for wall shear generally is developed to the far face of the confined boundary zone; otherwise, the full length of the wall is not effective in resisting shear. Coupled core walls require ductile link beams that can undergo large inelastic rotations. Away from the core walls, gravity loads commonly are supported by post-tensioned floor slabs supported by columns. Slab-column connections are designed considering the effect of lateral drifts on the shear punching tendency of the connection. For post-tensioned slabs, which are most common, at least two of the strands in each direction must pass through the column cage to provide post-punching resistance.

He concluded that Performance-based earthquake engineering increasingly is being used as an approach to the design of tall buildings in the U.S. Available software, research results, and experience gained through real building applications are providing a basis for effective application of nonlinear analysis procedures. Important considerations include definition of performance objectives, selection of input ground motions, construction of an appropriate nonlinear analysis model, and judicious interpretation of the results. Implemented properly, nonlinear dynamic analysis specific to the structural system and seismic environment is the best way to identify nonlinear dynamic response characteristics, including yielding mechanisms, associated internal forces, deformation demands, and detailing requirements. Proportions and details superior to those obtained using the prescriptive requirements of the building code can be determined by such analysis, leading to greater confidence in building performance characteristics including serviceability and safety. Although performance-based designs already are under way and are leading to improved designs, several research needs have been identified, the study of which can further improve design practices.

A. Shuraim et al.[37] summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame, in order to examine its applicability. Potential structural deficiencies in RC frame, when subjected to a moderate seismic loading, were estimated by the code seismic-resistant design and pushover approaches. In the first method the design was evaluated by redesigning under one selected seismic combination in order to show which members would require additional reinforcement. It was shown that most columns required significant additional reinforcement, indicating their vulnerability if subjected to seismic forces. On the other hand, the nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column. Vulnerability locations from the two procedures are significantly different. The paper has discussed the reasons behind the apparent discrepancy which is mainly due to the default assumptions of the method as implemented by the software versus the code assumptions regarding reduction factors and maximum permissible limits. In new building design, the code always maintains certain factor of safety that comes from load factors, materials reduction factors, and ignoring some post yielding characteristics (hardening). In the modelling assumptions of ATC-40, reduction factor is assumed to be one, and hardening is to be taken into consideration. Hence, the paper suggests that engineering judgment should be exercised prudently when using the pushover analysis and that engineer should follow the code limits when designing new buildings and impose certain reductions and limits in case of existing buildings depending on their conditions. In short software should not substitute for code provisions and engineering judgment.

A.Whittaker, Y. N. Huang et al.[38] summarize the next (second) generation tools and procedures for performance-based earthquake engineering in the United States. The methodology, which is described in detail in the draft Guidelines for the Seismic Performance Assessment of Buildings, builds on the first generation deterministic procedures, which were developed in the ATC-33 project in the mid 1990s and in ASCE Standard: ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings.

The procedures and methodologies described in these guidelines include an explicit treatment of the large uncertainties in the prediction of losses due to earthquakes. This formal treatment of uncertainty and randomness represents a substantial advance in performance based engineering and a significant departure from the first generation deterministic procedures.

He identified the five basic steps proposed for a next-generation seismic performance assessment. Unlike prior assessment procedures that addressed either structural damage or repair cost, three measures of seismic performance are proposed in the guidelines: 1) direct economic loss (repair cost), 2) indirect economic loss (downtime or business interruption), and 3) casualties (including injuries and death). Each of three performance measures is treated as a potential loss. Section 2 of the paper introduces the three types of performance assessment that can be performed using the draft Guidelines and identifies the basic procedure for each. In Section 3, he described the five steps for seismic performance assessment.

The procedures set forth in these guidelines represent a substantial departure from the deterministic tools and procedures used at this time because uncertainty and randomness is captured explicitly in every step of the proposed procedures. Fragility functions, damage states and building-level consequence functions, are used in the proposed procedures to compute losses.

Ceroni et al.[39] formulated that ductility of R.C. elements has been widely studied either experimentally and theoretical since its evaluation is basic to carry out a reliable non- linear analysis of structures; post-elastic deformability is a resource for redistributing stresses in a structure to increase the ultimate load but, above all, to absorb and dissipate energy during major earthquakes. However, the problem remains open and models still need an improvement in two directions. On one side, mechanical models can be implemented to take into account constructive details, shear-flexure interaction, size effects as well as non-linear constitutive relationship of materials and steel-concrete bond. On the other side, simplified approaches have to be assessed in order to allow an easy but reliable ductility evaluation without using any sophisticated analytical model, generally not very designer friendly. In this paper a wide parametric analysis with a refined model is carried out in order to build on a reliable formulation for the plastic hinge length of R.C. columns subjected to axial and flexural load. The model used to analyse the non-linear behaviour of the element and to estimate the plastic rotation is a point by point model, including an explicit formulation of the bond slip relationship and capable to take into account the effect of the distributed and concentrated non-linearity, as the spread of plasticity along the member and the fixed end rotation. Its efficiency has been already successfully applied to experimental comparison[56].

The rotational capacity evaluated by the model varying some parameters allows a clear understanding of the futures influence involved in the structural problem. Ductility of RCC elements depends on behaviour of the cracked section, which is well represented by moment-curvature relationship; the ratio of ultimate curvature to the one at first yielding is called section ductility. If the rotational capacity has to be calculated in actual cases, models based on the evaluation of a plastic hinge length are very useful thanks to their procedure simplicity. It is therefore surely interesting to review the evaluation of the plastic hinge length Lp using the detailed model.

$Lp = Lp^{1} + Lp^{11}$

where Lp^{1} is due to the plastic rotation of the column and Lp^{11} to the fixed end rotation at the footing zone of the column. In order to extrapolate a formulation for Lp^{1} and Lp^{11}, a wide parametric analysis has been developed in the same hypothesis explained in the previous paragraph. The column considered has length L equal to 1.5 m, 2 m, 2.5 m, 3 m and a square cross section with side H equal to 30 and 60 cm symmetrically reinforced; the combination of values of L and H gives back, for the ratio L/H, the values of 3.33, 5, 6.67, 8.33 and 10. The concrete strength in compression is fc=30 MPa and the volumetric percentage of stirrups is 0.1%. The ratio ft/fy does vary in the range 1.05-1.45; the ultimate strain of steel εu does vary in the range 0.04-0.16. Three diameters of steel bar, db, (10, 16 and 20 mm) are considered. The values of the ratio N/Nu considered are 0, 0.1, 0.2, 0.3, and 0.4.

$$L_p^1 = 6.1\left(\frac{L}{H}\right)^{0.43}\left(\frac{f_t}{f_y}-1\right)^{0.65}\varepsilon^{-0.32}.\left(1+\frac{N}{N_0}\right)^{-1.83}$$

$$L_p^{II} = 5.d_b\left(\frac{f_t}{f_y}-1\right)^{0.2}$$

The influence on the plastic hinge of the ratio between an element typical length (distance of critical section to the point of contra flexure, shear span…) and the section height has been already pointed out by Baker et. al.[52], who also explicitly introduced the influence of the ratio N/Nu, while the steel properties and concrete strength were considered as factors for mild and cold-worked steel. Since then, laying on experimental results and empirical considerations, other expressions have been proposed aimed to simplify the formulation of Lp reducing the number of parameters and considering only the influence of geometrical properties of an element (length, height of section). The influence of steel bar diameter was taken into account by Park & Priestly[(53)], based on the analysis of experimental tests on 20 columns:

$Lp = 0.08L + 6d_b$

where L is the distance from the point of contra flexure of the column to the section of maximum moment and d_b the bar diameter; the first and second terms of the formulation represent the Lp^1 and Lp^{11} contributions, both independent from the steel characteristics. The variables examined in the experimental tests were the section shape (square, rectangular and circular cross section), the longitudinal and lateral reinforcement content and the loading rate. The effect of axial load and steel properties was not analysed. Later on, in B.I.A.[(54)] a modification of the previous expression was proposed introducing the effect of the steel yielding stress:

$Lp = 0.08L + 0.022f_y\, d_b$

Recently in Fib Bulletin.[(55)] formulations similar to last eqn. for monotonic and cyclic loads have been suggested, as follows:

for monotonic loads: $Lp = 0.18Ls + 0.025f_y \,.\, d_b$

for cyclic loads: $Lp = 0.08Ls + 0.017f_y \,.\, d_b$

where L is the shear span.

At last he concluded in his formulation that the availability of a reliable formulation for the plastic hinge length is a key issue for any analysis of RCC element ductility, i.e. to non-linear behaviour of RCC frames under seismic actions. The proposed formulation is based on a wide numerical analysis developed through a detailed mechanical model which takes into account the non-linear constitutive relationship of material and the steel-concrete bond law. It allows considering the effect of yielding penetration between cracks of the structures but also at the steel anchorage in the foundation.

In particular, the two contributions to the plastic deformability of a column can be separately evaluated multiplying the respective plastic length by the curvature of the section at the element base; thus the element ductility can be easily evaluated knowing the section behaviour. The formulation in terms of plastic rotation takes into account many parameters and shows a low scatter respect to the numerical results; furthermore the influence of parameters appears in agreement with the mechanical behaviour. The range of some parameters considered to assess the proposal is wider than the ones used in experimental tests at the base of other available formulations, but it is limited to cold formed steel and elements without shear-flexure interaction; therefore the analysis has to be extended developing an experimental comparison too.

Chung-Yue Wang et al.[40] in this paper, he presented a method for the determination of the parameters of plastic hinge properties (PHP) for structure containing RC wall in the pushover analysis. Nonlinear relationship between the lateral shear force and lateral deformation of RC wall is calculated first by the Response-2000 and Membrane-2000 code. The PHP (plastic hinge properties) value of each parameter for the pushover analysis function of SAP2000 or ETABS is defined as the product of two parameters α and β. Values of α at states of cracking, ultimate strength and failure of the concrete wall under shear loading can be determined respectively from the calculations by Response-2000. While the corresponding β value of each PHP parameter is obtained from the regression equations calibrated from the experimental results of pushover tests of RC frame-wall specimens. The accuracy of this newly proposed method is verified by other experimental results. It shows that the presented method can effectively assist engineers to conduct the performance design of structure containing RC shear wall using the SAP2000. SAP2000 program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members. These built-in properties can be useful for preliminary analyses, but user-defined properties are recommended for final analyses[41]. Yielding and post-yielding behaviour can be modelled using discrete user-defined hinges. Currently SAP2000 allows hinges can only be introduced into frame elements; the PHP properties can be assigned to a frame element at any location along it. The authors have developed a dual parameters method to define the PHP properties of RC frame structure for the pushover analysis[42]. The purpose of this paper is to extend the application of this method to the RC structures containing RC shear wall. In order to use the functions provided by the SAP2000 code, the RC shear wall is treated as a wide, flat column. Modelling a RC wall as a wide and flat column (frame elements) not only can consider the steel reinforcements in RC elements exactly, but also can assign the PHP of RC walls according to its plastic behaviour. In SAP2000, the default properties are available for hinges in the following degrees of freedom: Axial (P), Major shear (V2), Major moment (M3) and Coupled P-M2-M3 (PMM). The effectiveness of this simple method is verified by the agreement of the prediction curves with some additional test data. This newly proposed method is quite simple and is easy for engineers to link with commercial structural analysis code to conduct the performance design of structure under seismic loading.

Konuralp Girgin et al.[43] explained that structural frames are often filled with infill walls serving as partitions. Although the infill is usually not considered in the structural analysis and design, their influence on the seismic behaviour of the infilled frame structures is considerable. In this study, a parametric study of certain infilled frames, using the strut model to capture the global effects of the infills was carried out. Three concrete planar frames of five-stories and three-bays are considered which have been designed in accordance with Turkish Codes. Pushover analysis is adopted for the evaluation of the seismic response of the frames. Each frame is subjected to four different loading cases. The results of the cases are briefly presented and compared. The effect of infill walls on seismic behaviour of two sample frames with different infill arrangements was investigated. The results yield that it is essential to consider the effect of masonry infills for the seismic evaluation of moment-resisting RC frames, especially for the prediction of its ultimate state, infills having no irregularity in elevation have beneficial effect on buildings and infills appear to have a significant effect on the reduction of global lateral displacements. Infills have been considered as non-structural elements, although there are codes such as the Eurocode-8 that include rather detailed procedures for designing infilled R/C frames. However, even though they are considered non-structural elements the presence of infills in the reinforced concrete frames can substantially change the seismic response of buildings in certain cases producing undesirable effects (torsional effects, dangerous collapse mechanisms, soft storey, variations in the vibration period, etc.) or favourable effects of increasing the seismic resistance capacity of the building.

Mehmet et al.[44] explained that due to its simplicity, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis. Modelling for such analysis requires the determination of the nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modelling assumptions. Pushover analysis is carried out for either user-defined nonlinear hinge properties or default-hinge properties, available in some programs based on the FEMA-356 and ATC-40 guidelines. While such documents provide the hinge properties for several ranges of detailing, programs may implement averaged values. The user needs to be careful; the misuse of default-hinge properties may lead to unreasonable displacement capacities for existing structures. This paper studies the possible differences in the results of pushover analysis due to default and user-defined nonlinear component properties. Four- and seven-story buildings are considered to represent low and medium rise buildings for this study. Plastic hinge length and transverse reinforcement spacing are assumed to be effective parameters in the user-defined hinge properties. Observations show that plastic hinge length and transverse reinforcement spacing have no influence on the base shear capacity, while these parameters have considerable effects on the displacement capacity of the frames. Comparisons point out that an increase in the amount of transverse reinforcement improves the displacement capacity. Although the capacity curve for the default-hinge model is reasonable for modern code compliant buildings, it may not be suitable for others. Considering that most existing buildings in Turkey and in some other countries do not conform to requirements of modern code detailing, the use of default hinges needs special care. The observations clearly show that the user-defined hinge model is better than the default-hinge model in reflecting nonlinear behaviour compatible with the element properties. However, if the default-hinge model is preferred due to simplicity, the user should be aware of what is provided in the program and should avoid the misuse of default-hinge properties. He concluded that the interior frames of 4- and 7-story buildings were considered in pushover analyses to represent low- and medium rise reinforced concrete (RC) buildings for study.

Beam and column elements are modelled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. The frames were modelled with default and user-defined hinge properties to study possible differences in the results of pushover analyses. The following findings were observed:

1. The base shear capacity of models with the default hinges and with the user-defined hinges for different plastic hinge length and transverse reinforcement spacing are similar; the variation in the base shear capacity is less than 5%. Thus, the base shear capacity does not depend on whether the default or user-defined hinge properties are used.
2. Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. Comparisons show that there is a variation of about 30% in displacement capacities due to Lp.
3. Displacement capacity depends on the amount of transverse reinforcement at the potential hinge regions. Comparisons clearly point out that an increase in the amount of transverse reinforcement improves the displacement capacity. The improvement is more effective for smaller spacing. For example, reducing the spacing from 200 mm to 100 mm provides an increase of up to 40% in the displacement capacity, while reducing the spacing from 200 mm to 150 mm provides an increase of only 12% for the 4-story frame.
4. Comparison of hinging patterns indicates that both models with default hinges (Case A) and the user-defined hinges (Case B3) estimate plastic hinge formation at the yielding state quite well. However, there are significant differences in the hinging patterns at the ultimate

state. Although the hinge locations seem to be consistent, the model with default hinges emphasizes a ductile beam mechanism in which the columns are stronger than the beams; damage or failure occurs at the beams. However, this mechanism is not explicitly guaranteed for the structures designed according to the 1975 Turkish Earthquake Code or pre-modern codes in other countries.

5. Time-history results point out that pushover analysis is reasonably successful in capturing hinging patterns for low and medium-rise buildings, except that the plastic hinge formation in the upper levels is not estimated adequately by pushover analysis, as observed by other researchers.
6. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties. Based on the observations in the hinging patterns, it is apparent that the user-defined hinge model is more successful in capturing the hinging mechanism compared to the model with default hinges.
7. Although the capacity curve for the default-hinge model is reasonable for modern code compliant buildings, it may not be suitable for others. Considering that most existing buildings in Turkey and some other countries do not conform to requirements of modern code detailing, the use of default hinges needs special care.

X.-K. Zou et al.[45] presented an effective computer- based technique that incorporates Pushover Analysis together with numerical optimisation procedures to automate the Pushover drift performance design of reinforced concrete buildings. Performance-based design using nonlinear pushover analysis, which generally involves tedious and intensive computational effort, is a highly iterative process needed to meet designer-specified and code requirements. This paper presents an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design of reinforced concrete (RC) buildings. Steel reinforcement, as compared with concrete materials, appears to be the more cost-effective material that can be effectively used to control drift beyond the occurrence of first yielding and to provide the required ductility of RC building frameworks. In this study, steel reinforcement ratios are taken as design variables during the design optimization process. Using the principle of virtual work, the nonlinear inelastic seismic drift responses generated by the pushover analysis can be explicitly expressed in terms of element design variables. An optimality criteria technique is presented in this paper for solving the explicit performance-based seismic design optimization problem for RC buildings.

It has been recognized that the inter-story drift performance of a multi-story building is an important measure of structural and non-structural damage of the building under various levels of earthquake motion(46). In performance based design, inter-story drift performance has become a principal design consideration(10). The system performance levels of a multi-story building are evaluated on the basis of the inter-story drift values along the height of the building under different levels of earthquake motion(47). The control of inter-story drift can also be considered as a means to provide uniform ductility over all stories of the building. A large story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Therefore, a uniform story ductility over all stories for a multi-story building is usually desired in seismic design(48). It has been recognized that there is a pressing need for developing optimized performance-based design procedures for seismic engineering of structures.

In seismic design, it is commonly assumed that a building behaves linear-elastically under minor earthquakes and may respond nonlinear-inelastically when subjected to moderate and severe earthquakes. Under such an assumption, the entire design optimization process can therefore be decomposed into two phases(49). In the first phase, the structural concrete cost is minimized subject to elastic drift responses under minor earthquake loading using elastic response spectrum analysis. In this phase, concrete member sizes are considered as the only design variables since the concrete material plays a more dominant role in improving the elastic drift performance of the building. Once the optimal structural member sizes are determined at the end of the first phase of the optimization, the steel reinforcement quantities can then be considered as design variables in the second phase. In controlling the inelastic drift responses, steel reinforcement is the only effective material that provides ductility to an RC building structure beyond first yielding. In this second design phase, the member sizes are kept unchanged and the cost of the steel reinforcement is minimized subject to design constraints on inelastic inter-story drift produced by the nonlinear pushover analysis.

A.K.Chopra(48) extracted an improved Direct Displacement-Based Design Procedure for Performance-Based seismic design of structures. Direct displacement-based design requires a simplified procedure to estimate the seismic deformation of an inelastic SDF system, representing the first (elastic) mode of vibration of the structure. This step is usually accomplished by analysis of an “equivalent” linear system using elastic design spectra. In their work, an equally simple procedure is developed that is based on the well-known concepts of inelastic design spectra.

This procedure provides: (1) accurate values of displacement and ductility demands, and (2) a structural design that satisfies the design criteria for allowable plastic rotation. In contrast, the existing procedure using elastic design spectra for equivalent linear systems is shown to underestimate significantly the displacement and ductility demands.

In this work, it is demonstrated that the deformation and ductility factor that are estimated in designing the structure by this procedure are much smaller than the deformation and ductility demands determined by nonlinear analysis of the system using inelastic design spectra. Furthermore, it has been shown that the plastic rotation demand on structures designed by this procedure may exceed the acceptable value of the plastic rotation.

PUSHOVER ANALYSIS

4.1 BACKGROUND

The pushover analysis (also named nonlinear static analysis) was introduced back in 1970's and for the last 35-40 years it has been noticed as a powerful engineering tool. Pushover analysis is able to consider the inelastic response characteristics and therefore provide information of performance of a structure in a seismic event, which the linear approach is not capable of. The main purpose of the pushover analysis is to compare the strength and deformation capacity with the demands at the corresponding performance level, by using a static nonlinear analysis algorithm. The analysis considers geometrical non-linearity and material inelasticity, as well as the internal force redistribution.

It is carried out under constant gravity loads and monotonically increased lateral forces, applied at the location of the masses in the structural model, to simulate the inertia forces. The method is able to describe the evaluation of plastic mechanism and structural damage as a function of the lateral forces since they are increased monotonically. The pushover analysis may be described as an extension of the lateral force method of linear analysis in to the nonlinear regime.

However, the method is based on many assumptions and may in some cases provide misleading results, as explained in the end of this section. Pushover analysis may be provided if there is a doubt that simple analysis provides insufficient information on the structural seismic resistance. The pushover analysis provides more relevant information and response characteristics that cannot be obtained from a RSA. Pushover analysis is also feasible for seismic analysis of existing structures and design of retrofit schemes.

Three dimensional analytical model of a structure would be the most preferable one, but earlier only a few adequate analytical tools were available for that purpose. However, the capability of computers is growing fast and for the last few years, sophisticated finite element computer programs from Computer Structure Inc. like instance SAP2000 and ETABS v9.7.4 have introduced pushover analysis of steel and concrete frame structures. In SAP2000 and ETABS, the nonlinear properties of the elements are implemented in the form of yield hinges, chosen and defined by the structural designer. Other finite element programs, for instance ANSYS and Cosmos/M, can perform pushover analysis where the nonlinear material properties are considered. However a three dimensional model of a typical structure would be cumbersome and with few exceptions too time consuming for a typical design process in the consulting engineering field. The basic assumption is that the response of a MDOF (multi-degree-of-freedom) structural system can be related to the response of an equivalent SDOF system. This implies that the response is controlled by a single mode and that the shape of the mode is constant throughout. It is clear that both these assumptions are not correct. However, several pilot studies have indicate that these assumptions result are in fairly good prediction of the maximum seismic response of MDOF structures as long as the response is dominated by a single mode. Several studies have shown that results of experimental tests and nonlinear dynamic analysis are similar to those obtained from the pushover analysis.

4.2 GENERAL

Static Nonlinear Analysis technique, also known as sequential yield analysis, or simply "pushover" analysis has gained significant popularity during the past few years. It is the one of the three analysis techniques recommended by FEMA-273/274 and a main component of the Spectrum Capacity Analysis method (ATC-40). Proper application can provide valuable insights into the expected performance of structural systems and components. Misuse can lead to an erroneous understanding of the performance characteristics. Unfortunately, many engineers are unaware of the details that have to observed in order to obtain useful results from such analysis[(49)].

In this procedure, a computer model of the structure is subjected to a predefined pattern of monotonically increasing lateral forces, to examine the non-linear behaviour of structure, including the deformation and damage pattern. Hence, pushover analysis can provide significant insight into the weak links in seismic performance of a structure.

It consists of two parts. First, a target displacement for the structure is established. The target displacement is an estimate of the seismic top displacement of the building, when it is exposed to the design earthquake excitation.

Then the model is subjected to a predefined lateral force. 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.

The pushover analysis is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

(a) The realistic force demands on potentially brittle elements, axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in reinforced concrete beams, etc.
(b) Estimates of the deformations demands for elements that have to form in-elastically in order to dissipate the energy imparted to the structure.
(c) Consequences of the strength deterioration of individual elements on behaviour of the structural system.
(d) Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
(e) Identification of the strength discontinuity in plan & elevation that will lead to changes in the dynamic characteristics in elastic range.
(f) Estimates of the inter-story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
(g) Verification of the completeness and adequacy of load path, considering all the elements of the structural systems, all the connections, and stiff non-structural elements of significant strength, and the foundation system[50].

4.3 LIMITATIONS

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

Target displacement is the global displacement expected in a design earthquake. The roof displacement at mass centre of the structure is used as target displacement. The accurate estimation of target displacement associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis.

However, in pushover analysis, generally an invariant lateral load pattern is used that the distribution of inertia forces is assumed to be constant during earthquake and the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to that experienced in design earthquake. Thus the capacity curve is very sensitive to the choice of lateral load distribution, selection of lateral load pattern is more critical than the accurate estimation of target displacement.

The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, "code" distributions, a single concentrated horizontal force at the top of structure, triangular loading pattern, etc. These loading patterns usually favour certain deformation modes that are triggered by the load pattern and miss others that are initiated and propagated by the ground motion and inelastic dynamic response characteristics of the structure. Moreover, invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if the structural response is not severely affected by higher modes and the structure has only a single load yielding mechanism that can be captured by an invariant load pattern.

FEMA-273 recommends utilising at least two fixed load patterns that form upper and lower bounds for inertia force distributions to predict likely variations on overall structural behaviour and local demands. The first pattern should be uniform load distribution and the other should be "code" profile or multi-modal load pattern. The 'Code' lateral load pattern is allowed if more than 75% of the total mass participates in the fundamental load. The invariant load patterns cannot account for the redistribution of inertia forces due to progressive yielding and resulting changes in dynamic properties of the structure. Also, fixed load patterns have limited capability to predict higher mode effects in post-elastic range. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. Although some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated. The scale of improvement has been a subject of discussion that simple invariant load patterns are widely preferred at the expense of accuracy. Whether lateral loading is invariant or adaptive, it is applied to the structure statically and a static loading cannot represent inelastic dynamic response with a large degree of accuracy(51).

Hence to summarise, the limitations are as follows:

(a) The pushover analysis is static and cannot predict the dynamic behaviour of the structure with large accuracy.

(b) The pushover analysis could underestimate affects of modes that may occur in a structure subjected to severe seismic events and exaggerate others. This applies in case of higher modes i.e. in tall buildings. Hence, the pushover analysis becomes inaccurate if higher mode effects are important. However pushover analysis procedure where effects of higher modes are considered has been available for the last few years.

(c) The load pattern affects the results dramatically. Each load pattern is likely to favour certain deformation mode. Therefore more than one load pattern should always be considered in the pushover analysis.

(d) Incorporation of torsion effects due to mass, stiffness and strength irregularities could affect the results and also 3-D problems like orthogonality effects, direction of loading and semi rigid diaphragms.

4.4 ANALYSIS PROCEDURE OF PUSHOVER ANALYSIS

The step-wise procedure to do pushover analysis through Capacity Spectrum Method as given in ATC-40 is given below:

1. Create a computer model of the structure following the modeling rules excluding the foundation.
2. Apply lateral storey forces to the structure in any of the following manners:
   a) Simply apply a single concentrated horizontal force at the top of the structure.

b) Apply lateral forces to each storey in proportion to the standard code procedure without the concentrated force at the top i.e. $F_x = \left[\frac{w_x h_x}{\sum w_x h_x} \right] V$

c) Apply lateral forces in proportion to the product of storey masses and first mode shape of the elastic model of the structure i.e. $F_x = \left[\frac{w_x \phi_x}{\sum w_x \phi_x} \right] V$. The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure.

d) Same as Level c until first yielding. For each increment beyond yielding, adjust the forces to be consistent with changing deflected shape.

e) Similar to c and d above, but including the effects of higher modes of vibration in determining yielding in individual structural elements while plotting the capacity curve for the building in terms of first mode lateral forces and displacements. The higher mode effects may be determined by doing higher mode pushover analyses.

3. Calculate member forces for the required combinations of vertical and lateral load.
4. Adjust the lateral force level so that some elements are stressed to within 10% of its member strength.
5. Record the base shear versus the roof displacement.
6. Revise the model using zero stiffness for the yielding elements.
7. Apply a new increment of lateral load to the revised structure such that another element/s yields.
8. Add the increment of lateral load and the corresponding increment of roof displacement to the previous totals to give accumulated values of base shear and roof displacement.
9. Repeat steps 6, 7 and 8 until the structure reaches an ultimate limit, such as: instability from $P-\Delta$ effects; distortions considerably beyond the desired performance level; an element group reaching a lateral deformation level at which significant strength degradation begins.

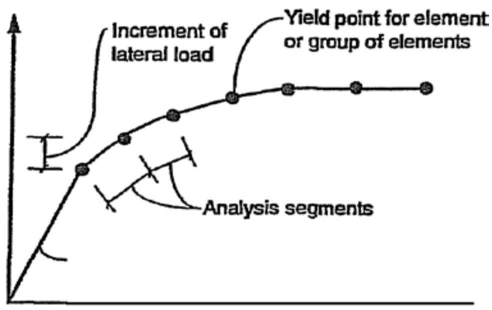


Fig.4.1- Capacity Curve showing plot of Base shear vs Roof Displacement

10. Explicitly model global strength degradation. If incremental loading was stopped in step 9 as a result of reaching a lateral deformation level at which all or a significant portion of an element/s loads can no longer be resisted i.e. its strength has significantly degraded, then the stiffness of those element/s is reduced, or eliminated. A new capacity curve is created starting with step c of this step-by-step process.

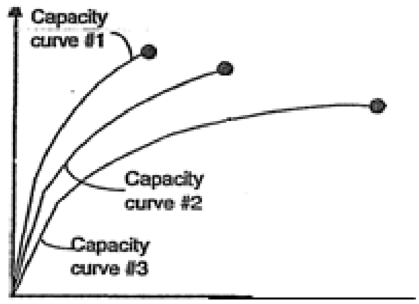


Fig.4.2.- Capacity Curves showing plot of Base shear vs Roof Displacement

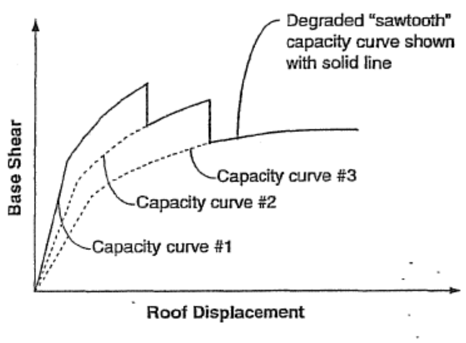


Fig.4.3 – Capacity curve with Global strength Degradation modelled

Once this capacity curve has been obtained, it is converted into capacity spectrum curve in Acceleration-Displacement response Spectra (ADRS) system by using the following formulae:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} \frac{(w_i \phi_{i1})}{g}}{\sum_{i=1}^{N} \frac{(w_i \phi_{i1}^2)}{g}} \right] \quad \& \quad \alpha_1 = \frac{\left[\sum_{i=1}^{N} \frac{(w_i \phi_{i1})}{g} \right]^2}{\left[\sum_{i=1}^{N} \frac{w_i}{g} \right] \left[\sum_{i=1}^{N} \frac{(w_i \phi_{i1}^2)}{g} \right]}$$

$$S_a = \frac{(V/W)}{\alpha_1} \quad \& \quad S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}}$$

Where:

PF_1 =modal participation factor for the first natural mode.

α_1 =modal mass coefficient for the first natural mode.

$\frac{w_i}{g}$ = mass assigned to level i

ϕ_{i1} =amplitude of mode 1 at level i

N= Level N, the level which is the uppermost in the main portion of the structure

V=base shear

W=building dead weight plus likely live loads.

From IS1893 (Part I):2002, we already have the demand spectra plotted in form of (Sa/g) vs T. To make comparison, this demand spectra is also converted into ADRS system by using the following formulae:

$S_d = \frac{1}{4\prod^2} S_a T^2$ & the spectral acceleration $\left(\frac{S_a}{g}\right)g$.

Now that both the capacity spectrum and demand spectrum are in same ADRS system, the two graphs are plotted together. The point where the capacity spectrum meets with the demand spectrum is the Performance Point of the structure. The same can be understood from the following figure shown below

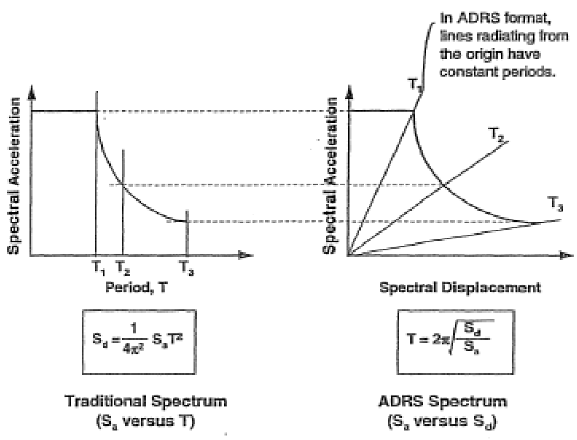


Figure 8-6. Response Spectra in Traditional and ADRS Formats

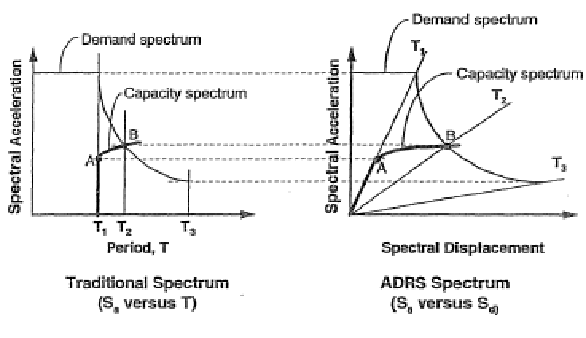


Fig.4.4-Capacity Spectrum Superimposed over Response Spectra *in* ADRS Formats

WORK CARRIED OUT (A CASE STUDY)

5.1 OVERVIEW

As mentioned earlier, non-linear static pushover analysis procedure can be applied to new structures after designing afresh and then performing the pushover analysis, or for existing structures by first modelling the actual properties of the structure as on site in a computer model and then applying the pushover loads to obtain the behaviour of the structure.

In this instance, the objective was to do pushover analysis of an existing hospital RC frame structure/building located in earthquake zone-V. It is a well known fact that steel being homogeneous, the non-linear behaviour of such members can be more accurately and easily modelled unlike that for RCC, which constitutes of both concrete and reinforcement and though considered homogeneous in theory is not actually in practical, hence the hinge properties given as default in the programs is not very accurate and can give misleading results.

Accordingly, to model the realistic behaviour of the non-linearity of the members, hinges based on the moment curvature and interaction diagram was generated for each member and incorporated in the model to perform the pushover analysis.

5.2 BUILDING DETAILS

The structural details that was observed and measured on site are as mentioned below:

1. The structure was a Ground plus two storey (G+2) building, located in Nongpoh, Ri-Bhoi district Meghalaya (Seismic Zone-V).
2. It was constructed as a Reinforced Concrete (RC) Ordinary Moment resisting frame (OMRF) structure.
3. It was symmetrical in plan with 7 bays of 3.6meters each along the x-axis, while along the y-axis, there were three bays. The first and last bays were 5meters each, while the intermediate bay was 2meters.
4. The height of each floor to floor was 3.5meters, making the total height of the building as 10.5meters.
5. RCC slab of 100mm thickness was provided at the first floor and second floor levels.

6. The infill walls were observed to be 230mm thick in exterior bay and 150mm thick for all interior bays.
7. At roof level, only the beams were constructed. Trusses were rested on the beams to give sloping roof. Hence the roof was inaccessible.
8. The base of the foundations of the structure was located at a depth of 1.5meters below the ground level.
9. All beams including the plinth beams were 250mmX400mm and all columns were 250mmX400mm in size. Only the reinforcement percentage was varied for the beams. Figure 5.1 presents the sections of the beams and column of the structure.

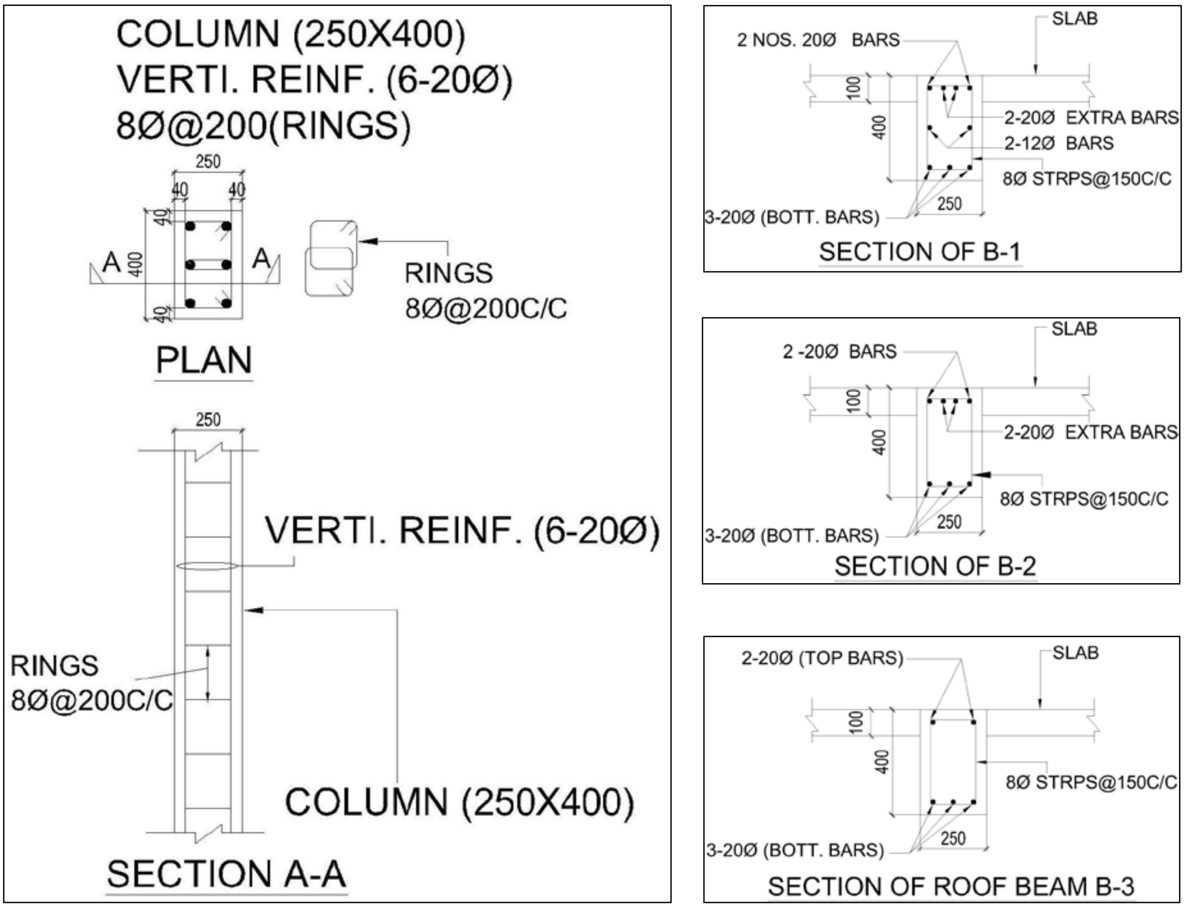


Fig 5.1- Sections of Column and Beams

10. The plinth beams were raised to a height of 450mm above the existing ground level.
11. The hospital was constructed approximately 25-30 years ago. Hence IS13920:1993 was not followed.
12. As the structural drawings of the Hospital Building were available, hence the same values were used in creating the model of the structure after physical verification of the same on site. Figure 6.2 presents the typical frame plan showing column and beam layout.

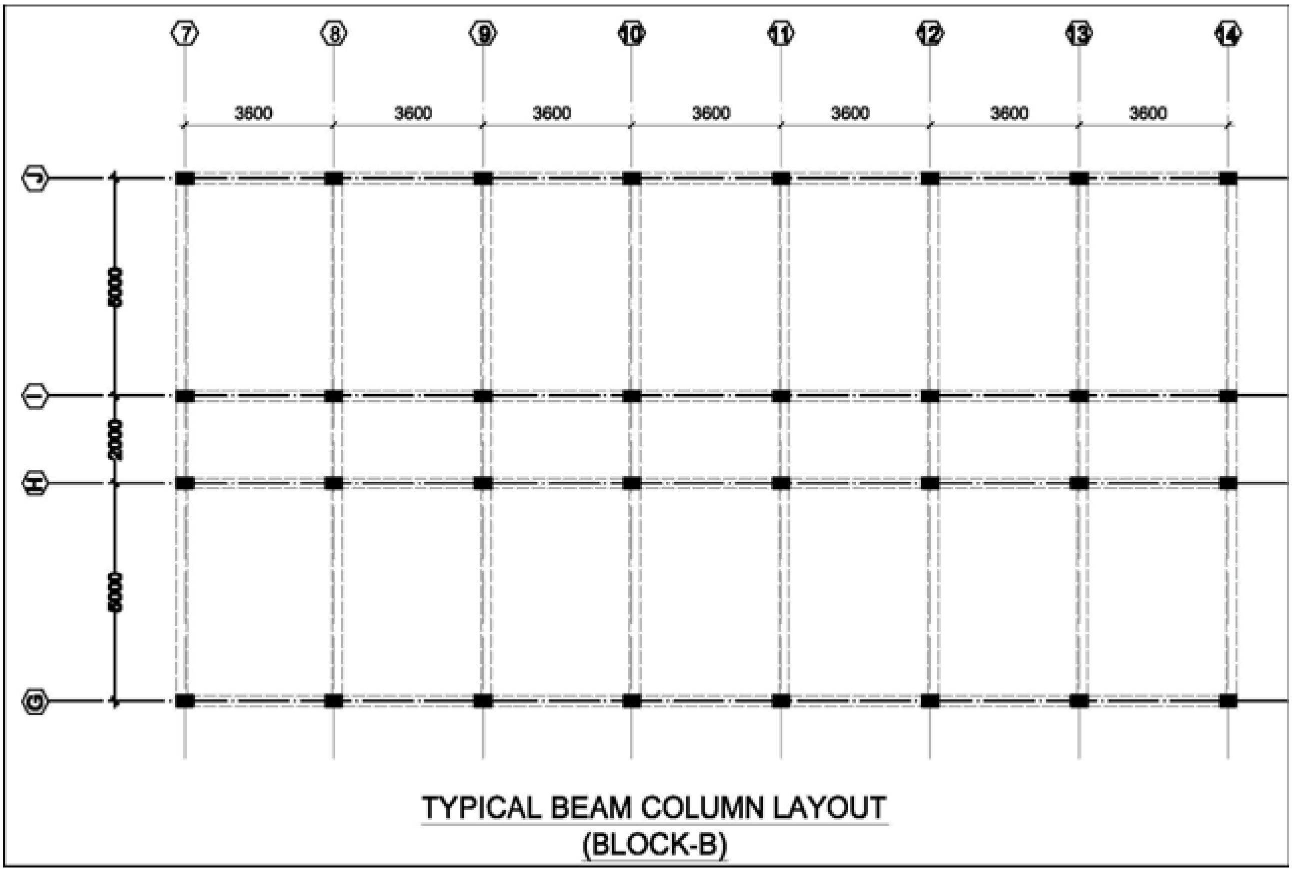


Fig.5.2

5.3 LOADING

1. Floor Finish load of slab has been taken as 1.8 KN/m^2 considering 75mm Floor Finish for intermediate floors.
2. Live load has been taken as 4KN/m^2 and since it is only a G+2 structure, no reductions in accordance with Clause 3.2.1 of IS875 (Part 2): 1987 has been considered.
3. DL of exterior walls with openings = 7.6kN/m, interior walls is taken as 9.69KN/m.

5.4 LOAD COMBINATIONS

The Load Combinations taken for analyzing of the structure was as per IS456:2000 & Clause 6.3.1.2 of IS1893:2002.

5.5 RESPONSE SPECTRUM ANALYSIS

5.5.1 For Model with infill walls:

$A_h = (Z.I/2.R)\ (S_a/g)$ (*As per Clause 6.4.2 of IS1893:2002*)

Where:

Seismic Zone = V, therefore Zone Factor Z = 0.36 (*Table 2 Clause 6.4.2 of IS1893:2002*)

Importance Factor I =1.5 (*Importance of structure is high as it is a Hospital Building*)

Response Reduction factor R=3 (*Table 7 Clause 6.4.2 of IS1893:2002*)

Soil type = Medium.

Now,

$T_a = 0.09h/\sqrt{d}$ *(Clause 7.6.2 of IS 1893:2002)*

In <u>X-Direction</u>: h = 10.95 mts.; d = 25.2mts.; T_a = 0.196secs; Therefore $(S_a/g) = 2.5$

In <u>Z-Direction</u>: h = 10.95 mts.: d = 12 mts.; T_a = 0.2845secs; Therefore $(S_a/g) = 2.5$

Therefore, $A_h = 0.225$ (*in both directions*)

5.5.2 For Model without infill walls:

$A_h = (Z.I/2.R)\ (S_a/g)$ (*As per Clause 6.4.2 of IS1893:2002*)

Where:

Seismic Zone = V, therefore Zone Factor Z = 0.36 (*Table 2 Clause 6.4.2 of IS1893:2002*)

Importance Factor I =1.5 (*Importance of structure is high as it is a Hospital Building*)

Response Reduction factor R=3 (*Table 7 Clause 6.4.2 of IS1893:2002*)

Soil type = Medium.

Now,

$T_a = 0.075.h^{(0.75)}$ *(Clause 7.6.2 of IS 1893:2002)*

In <u>X-Direction</u>: h = 10.95 mts.; T_a = 0.4515secs; Therefore $(S_a/g) = 2.5$

In <u>Z-Direction</u>: h = 10.95 mts.; T_a = 0.4515secs; Therefore $(S_a/g) = 2.5$

Therefore, $A_h = 0.225$ (*in both directions*)

5.6 STRUCTURAL MODELLING

Modelling a building involves the modelling and assemblage of its various load carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modelling of the material properties and geometric details is as per details mentioned below.

5.6.1 Material Properties

The material properties used in creating the model were as follows:

1. Grade of Concrete – M15,
2. Grade of Reinforcement used – Fe415,
3. Poisson Ratio of Concrete – 0.2
4. Poisson Ratio of Reinforcement – 0.3
5. Poisson Ratio of Brick Masonry – 0.15[44]
6. Density of Concrete – 25KN/m^2
7. Density of Reinforcement – 78.5KN/m^2
8. Density of Brick masonry – 21KN/m^2
9. Young's Modulus of concrete – 22076005.07 KN/m^2 ($5700.\sqrt{fck}$)
10. Young's Modulus of reinforcement – $2.0X10^8$ KN/m^2
11. Young's Modulus of Brick Masonry – $6.3X10^6$ KN/m^2
12. Damping Factor – 0.05 (As per Clause 7.8.2.1 of IS1893(Part 1):2002

5.6.2 Geometrical Properties

The geometrical properties measured are as follows:

1. The slab thickness – 0.1m
2. Beam cross sections on all floors – 0.25mX0.40m
3. Column cross section on all floors – 0.25mX0.40m
4. Outer wall thickness – 0.23m
5. Inner wall thickness – 0.15m
6. Foundation depth – 1.5m
7. Height of Plinth level – 0.45m from G.L.
8. Nos. of bays along length i.e. X-Axis – 7
9. Nos. of bays along width i.e. Y-Axis – 3
10. Nos. of floors Along Z-Axis – 3
11. Storey Height – 3.5m

5.6.3 Structural Elements

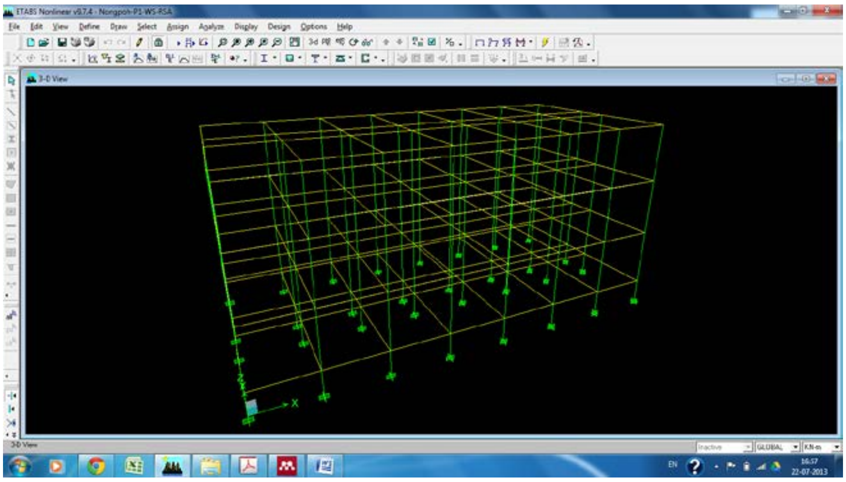
The structural elements were modelled as follows:

1. Beams and columns are modelled by 3D frame elements. The beam-column joints are assumed to be rigid. Beams and columns in the present study were modelled as frame elements

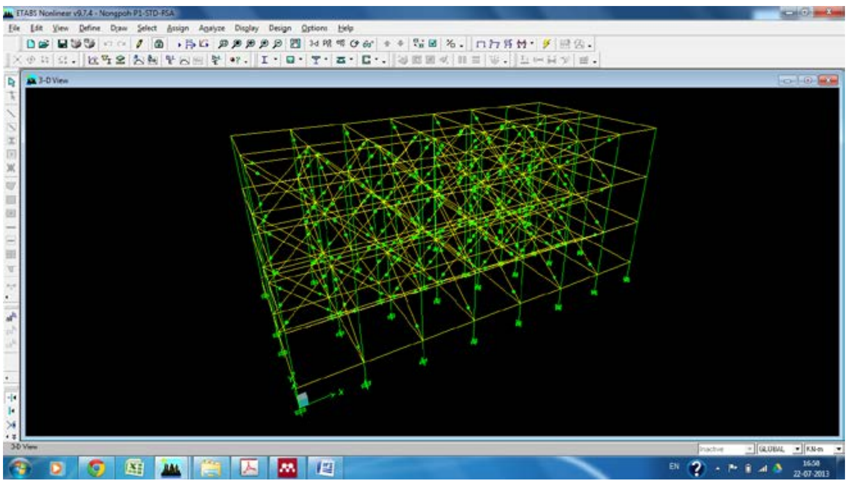
with the centrelines joined at nodes using commercial software ETABS v9.7.4. The dead weight of the beams and columns was calculated by the program using the material densities and the geometrical dimensions of the respective members.

2. The floor slabs were modelled to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was taken into account by self weight calculation by the program using the material properties and geometrical dimensions.

3. Infill walls were modelled as struts as per Clause 7.10 of IS1893 (Part 1) Draft Indian Standard in, both directions. The ends were released for moments and torsion and only axial force was allowed to be borne by the struts. Moreover the struts were assigned '0' in tension and 10N/mm^2 as compressive strength of the brick masonry (assumed) so that the strut would fail when the compressive strength would exceed this value. The 3D model with and without infill is as shown in figure below.



(a) Without Infill



(b) With Infill

Fig.5.3 – 3D Computer Model of the building with and without Infill stiffness considerations respectively.

5.6.4 Modelling of Flexural Plastic Hinges

In the implementation of pushover analysis, the model must account for the nonlinear behaviour of the structural elements. In the present study, a point-plasticity approach is considered for modelling nonlinearity, wherein the plastic hinge is assumed to be concentrated at a specific point in the frame member under consideration. Beam and column elements in this study were modelled with flexure (M3 for beams and P-M2-M3 for columns) hinges at possible plastic regions under lateral load (i.e., both ends of the beams and columns). Refer Fig. 6.4 for the local axis system considered. Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load.

A generalized force-displacement characteristic of a non-degrading frame element (or hinge properties) is shown in figure 5.4 below.

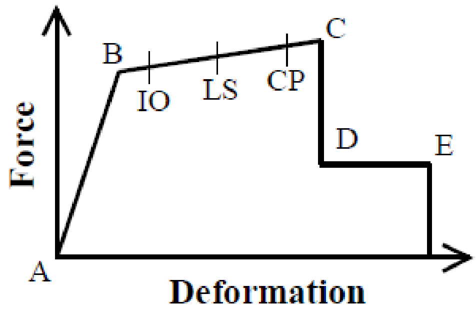


Fig. 5.4 Force-Deformation for Pushover Hinge

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained. Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P- M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, more than one type of hinge can be assigned at the same location of a frame element. There are three types of hinge properties in SAP2000 and ETABS v9.7.4. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements.

When these hinge properties (default and user-defined) are assigned to a frame element, the program automatically creates a new generated hinge property for each and every hinge.

Default hinge properties cannot be modified and they are section dependent. When default hinge properties are used, the program combines its built-in default criteria with the defined section properties for each element to generate the final hinge properties. The built-in default hinge properties for steel and concrete members are based on ATC-40 and FEMA-273 criteria.

User-defined hinge properties can be based on default properties or they can be fully user-defined. When user-defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They could be viewed, but they could not be modified.

However, in the present study the plastic hinge properties are generated in SAP2000. This has been done as the ETABS v9.7.4 does not have the feature of generating the Interaction Diagram and the Moment-Curvature Graph to obtain the flexural hinge property for each member by taking into consideration the confined strength of concrete and the plastic hinge length, both important parameters required for generating a realistic non-linear hinge property for RC members. The analytical procedure used to model the flexural plastic hinges are explained below.

When the section of the frame member is created using section designer, it also asks for the details of the confined concrete strength parameters, apart from the stirrup details and plastic hinge length.

With all these parameters as input, the moment-curvature and interaction diagram are both generated automatically by the software. The values thus obtained can be used to generate the hinge properties for pushover analysis.

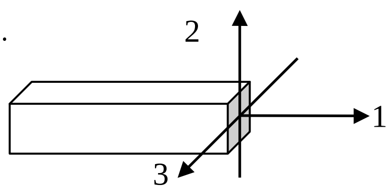


Fig.5.5 – The co-ordinate system used to define the flexure and shear hinges

Flexural hinges in this study are defined by moment-rotation curves calculated based on the cross-section and reinforcement details at the possible hinge locations. For calculating hinge properties it is required to carry out moment–curvature analysis of each element. Constitutive relations for concrete and reinforcing steel, plastic hinge length in structural element are required for this purpose. The flexural hinges in beams are modelled with uncoupled moment (M3) hinges whereas for column elements the flexural hinges are modelled with coupled P-M2-M3 properties that include the interaction of axial force and bi-axial bending moments at the hinge location. Although the axial force interaction is considered for column flexural hinges the rotation values were considered only for axial force associated with gravity load.

5.6.4.1 Stress-Strain Characteristics for Concrete

The stress-strain curve of concrete in compression forms the basis for analysis of any reinforced concrete section. The characteristic and design stress-strain curves specified in most of design codes (IS 456: 2000, BS 8110) do not truly reflect the actual stress-strain behaviour in the post-peak region, as (for convenience in calculations) it assumes a constant stress in this region (strains between 0.002 and 0.0035). In reality, as evidenced by experimental testing, the post-peak behaviour is characterised by a descending branch, which is attributed to 'softening' and micro-cracking in the concrete. Also, models as per these codes do not account for strength enhancement and ductility due to confinement. However, the stress-strain relation specified in ACI 318M-02 consider some of the important features from actual behaviour. A previous study [45] on stress-strain relation of reinforced concrete section concludes that the model proposed by Panagiotakos and Fardis[46] represents the actual behaviour best for normal-strength concrete. Accordingly, this model has been selected in the present study for calculating the hinge properties. This model is a modified version of Mander's model[47] where a single equation can generate the stress f_c corresponding to any given strain ε_c:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r}$$

where: $x = \frac{\varepsilon_c}{\varepsilon_{cc}}$; $r = \frac{E_c}{E_c - E_{sec}}$; $E_c = 5700\sqrt{f'_{co}}$; $E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$ and f'_{cc} is the peak strength expressed as follows:

$$f'_{cc} = f'_{co}\left[1 + 3.7\left(\frac{0.5 k_e \rho_s f_y h}{f'_{co}}\right)^{0.85}\right]$$

The expressions for critical compressive strains are expressed in this model as follows:

$$\varepsilon_{cu} = 0.004 + \frac{0.6\rho_s f_{yh}\varepsilon_{sm}}{f_{cc}^{'}}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f_{cc}^{'}}{f_{co}^{'}} - 1\right)\right]$$

The unconfined compressive strength $\left(f_{co}^{'}\right)$ is $0.75 f_{ck}$, k_e having a typical value of 0.95 for circular sections and 0.75 for rectangular sections.

Figure 5.6 shows a typical plot of stress-strain characteristics for M-20 grade of concrete as per Modified Mander's model[46]. The advantage of using this model can be summarized as follows:

- A single equation defines the stress-strain curve (both the ascending and descending branches) in this model.
- The same equation can be used for confined as well as unconfined concrete sections.
- The model can be applied to any shape of concrete member section confined by any kind of transverse reinforcement (spirals, cross ties, circular or rectangular hoops).
- The validation of this model is established in many literatures (e.g. Pam & Ho, 2001)

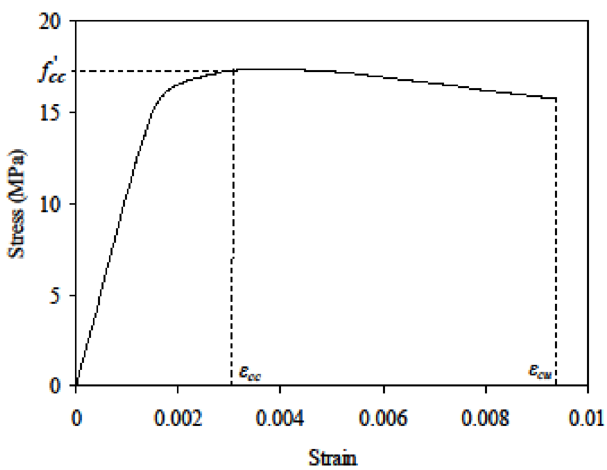


Fig. 5.6 - Typical stress-strain curve for M-20 grade concrete[46]

5.6.4.2 Stress-Strain Characteristics for Reinforcing Steel

The constitutive relation for reinforcing steel given in IS 456 (2000) is well accepted in literature and hence considered for the present study. The 'characteristic' and 'design' stress-strain curves specified by the Code for Fe-415 grade of reinforcing steel (in tension or compression) are shown in figure 5.7.

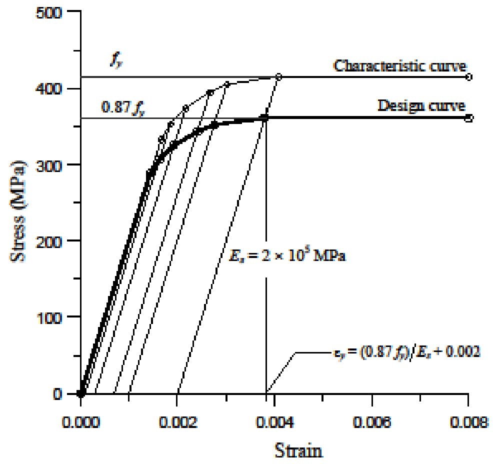


Fig.5.7 – Typical stress-strain curve for reinforcement – IS456(2000)

5.6.4.3 Moment-Curvature Relationship

Moment-curvature relation is a basic tool in the calculation of deformations in flexural members. It has an important role to play in predicting the behaviour of reinforced concrete (RC) members under flexure. In nonlinear analysis, it is used to consider secondary effects and to model plastic hinge behaviour. Curvature (φ) is defined as the reciprocal of the radius of curvature (R) at any point along a curved line. When an initial straight beam segment is subject to a uniform bending moment throughout its length, it is expected to bend into a segment of a circle with a curvature φ that increases in some manner with increase in the applied moment (M). Curvature φ may be alternatively defined as the angle change in the slope of the elastic curve per unit length $(\varphi = 1/R = d\theta/ds)$. At any section, using the 'plane sections remain plane' hypothesis under pure bending, the curvature can be computed as the ratio of the normal strain at any point across the depth to the distance measured from the neutral axis at that section. If the bending produces extreme fibre strains of ε_1 and ε_2 at the top and bottom at any section (it is assumed that compression is on top and tension is in the bottom), then, for small deformations, it can be shown that $\varphi = (\varepsilon_1 + \varepsilon_2)/D$. If the beam behaviour is linear elastic, then the moment-curvature relationship is linear and the curvature is obtained as $\varphi = M / EI$ .The flexural rigidity (EI) of the beam is obtained as a product of the modulus of elasticity E and the second moment of area of the section I.

When a RC flexural member is subjected to a gradually increasing moment, its behaviour transits through various stages, starting from the initial un-cracked state to the ultimate limit state of collapse. The stresses in the tension steel and concrete go on increasing as the moment increases.

The behaviour at the ultimate limit state depends on the percentage of steel provided, i.e., on whether the section is 'under-reinforced' or 'over-reinforced'. In the case of under-reinforced sections, failure is triggered by yielding of tension steel whereas in over-reinforced section the steel does not yield at the limit state of failure. In both cases, the failure eventually occurs due to crushing of concrete at the extreme compression fibre, when the ultimate strain in concrete reaches its limit. Under-reinforced beams are characterised by 'ductile' failure, accompanied by large deflections and significant flexural cracking. On the other hand, over-reinforced beams have practically no ductility, and the failure occurs suddenly, without the warning signs of wide cracking and large deflections.

In the case of a short column subject to uni-axial bending combined with axial compression, it is assumed that $\varphi = M / EI$ remains valid and that "plane sections before bending remain plane". However, the ultimate curvature (and hence, ductility) of the section is reduced as the compression strain in the concrete contributes to resisting axial compression in addition to flexural compression.

5.6.4.4 Modelling of Moment-Curvature in RC Sections

Using the Modified Mander model of stress-strain curves for concrete[46] and Indian Standard IS456:2000 stress-strain curve for reinforcing steel, for a specific confining steel, moment curvature relations can be generated for beams and columns (for different axial load levels). The assumptions and procedure used in generating the moment-curvature curves are outlined below.

1. The strain is linear across the depth of the section ('plane sections remain plane').
2. The tensile strength of the concrete is ignored.
3. The concrete spalls off at a strain of 0.0035.
4. The initial tangent modulus of the concrete, E_c is adopted from IS 456:1978, as 5700 $\sqrt{f_{ck}}$
5. In determining the location of the neutral axis, convergence is assumed to be reached within an acceptable tolerance of 1%.

Apart from the above explanation of obtaining the confined strength and graph of the concrete to develop the moment-curvature and interaction graphs, the procedure to obtain the same in SAP2000 is shown in pictorial format for better understanding of the procedure involved. The same is shown below.

5.7 STEP-BY-STEP PROCEDURE OF ANALYSIS IN ETABS V 9.7.4

1. Create the basic frame of the building. This is done by filling in the details in the pop-up box shown below.

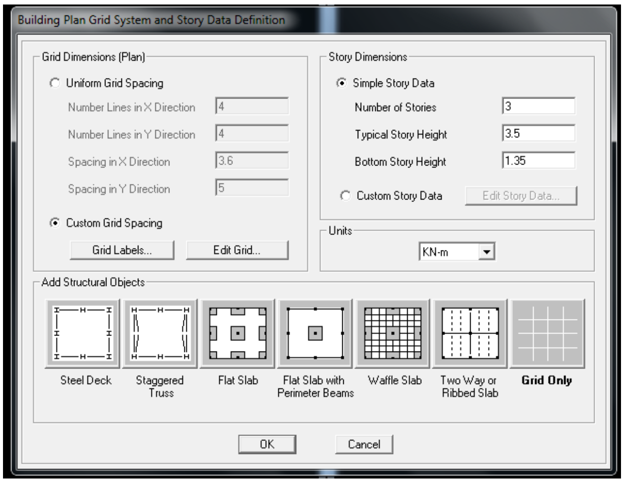


Fig.5.8 – 3D Frame Grid Line Generation Popup Box

2. Next we correct the spacing of the respective gridlines as per the requirement by clicking on the Custom Grid Spacing shown in the picture above.

3. Once the grid lines have been formed, the next step is to provide the material properties, which are the density, Poisson ratio, yield strength, etc. Properties for all varieties of concrete, reinforcement grades, etc. are entered. The same is done through the pop-up boxes shown below.

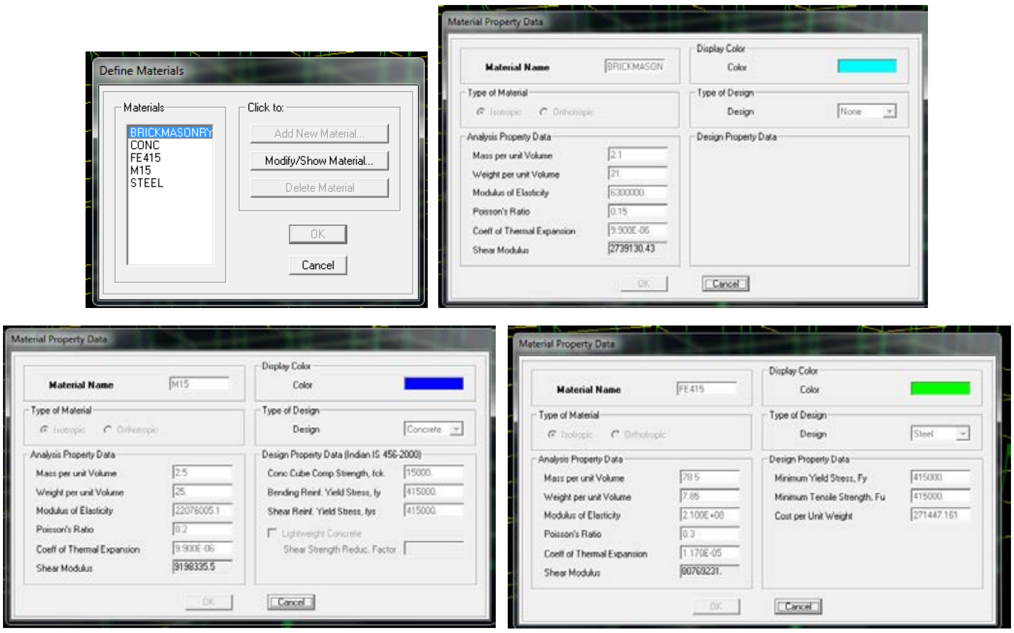


Fig.5.9 – Material Properties Input Pop-Up

4. The entry of the material properties is followed by the geometrical properties of all beams, columns, etc. For model with struts, the properties of the struts are also entered here. The pictures of the various pop-up boxes for the same are shown below:

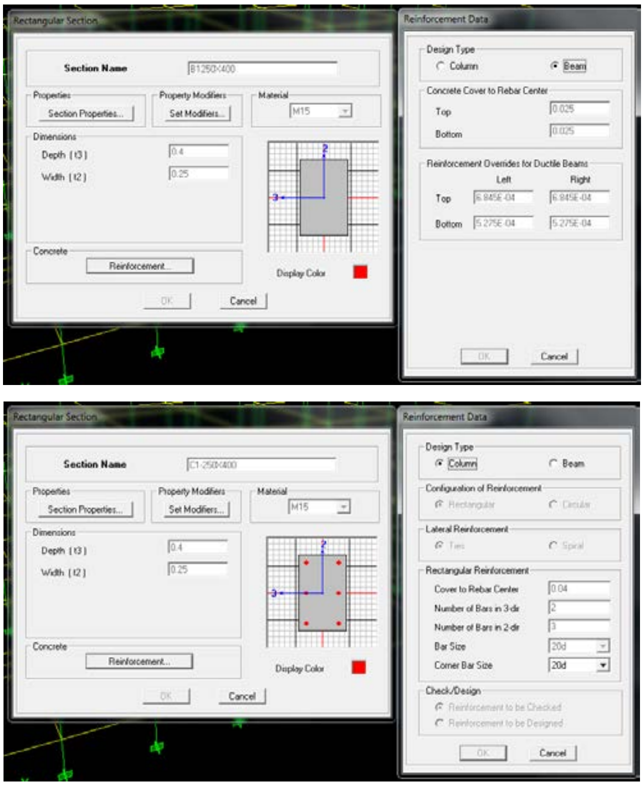


Fig.5.10 – Pop-up boxes for entering the beam and column properties

5. Thereafter, the slab properties are created along with the diaphragm modelling of the slab at the various vertical levels. The same is entered through the pop-up box shown below.

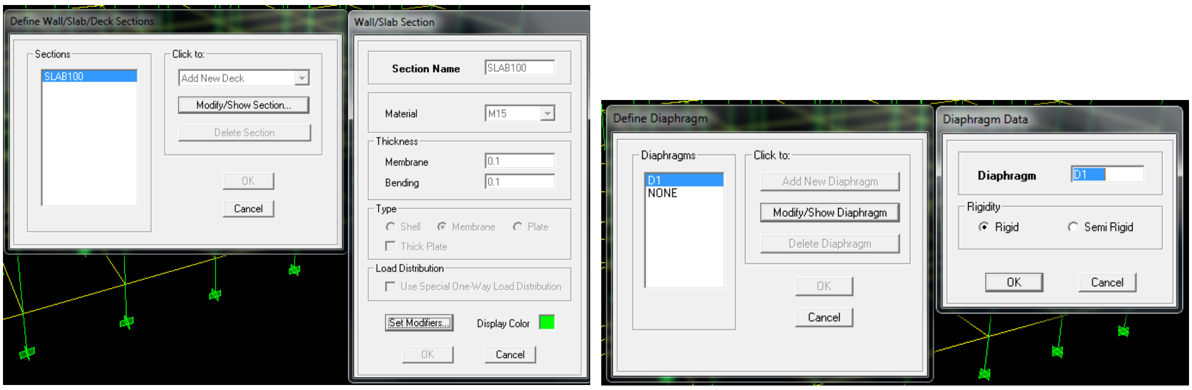


Fig.5.11 – Pop-up boxes showing the input parameters required for slab and diaphragm.

6. Now that the material properties with members sections have all been created, we "DRAW" the frame and area members in the grid lines created at the start and simultaneously assign the

respective properties of the beam or column at that location. The same is done through pop-up box shown below.

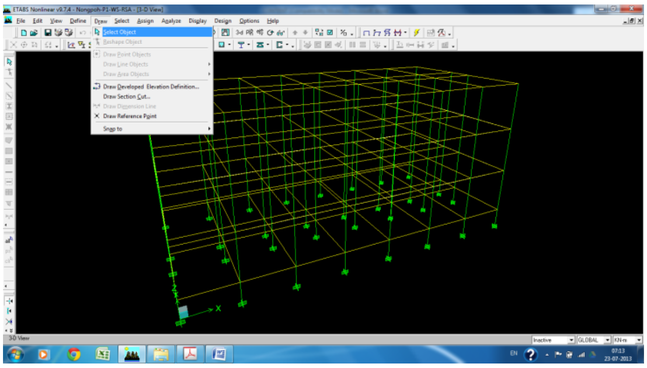


Fig.5.12 – Pop-up shown for drawing of the various structural members onto the grid lines.

7. Thereafter, we define the load cases and the response spectrum parameters. Finally using the load cases, we create the load combinations required for performing the respective analysis. The following pop-up boxes show how this is done in ETABS v9.7.4.

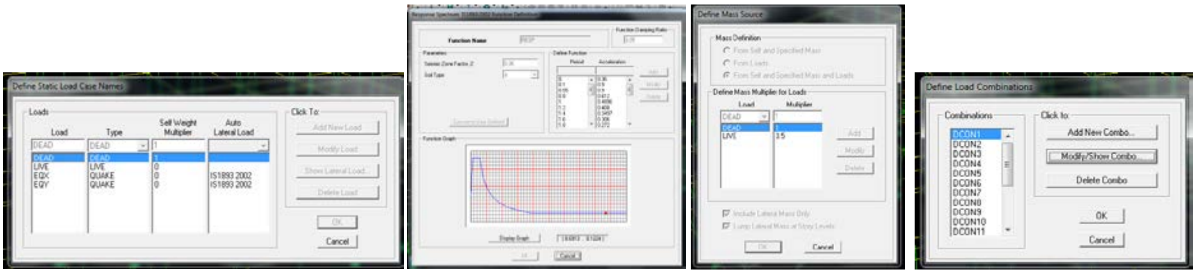


Fig.5.13–Pop-ups for generating the load Cases, Response Spectrum Parameters & Load Combinations

8. After the load combinations have been generated, we apply the loads through the following pop-up boxes under the respective load cases.

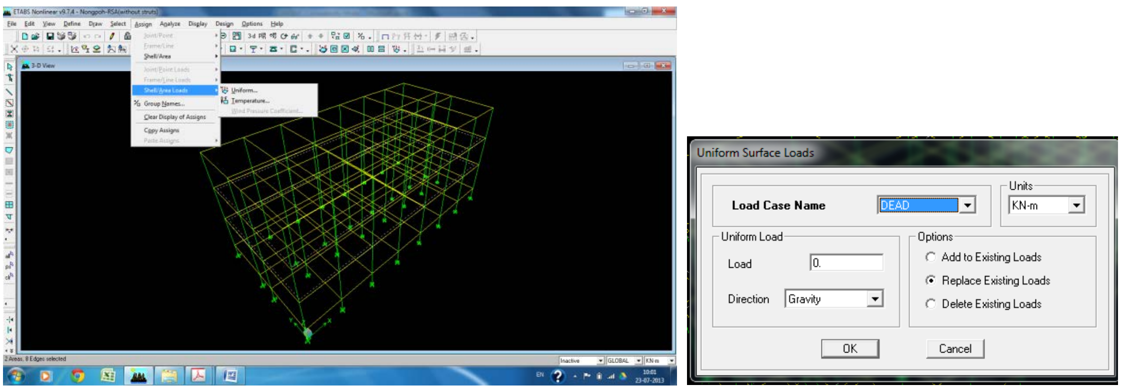


Fig.5.14 – Pop-ups showing entry of Load values for the model.

9. Finally we perform analysis and obtain the fundamental mode base shear. The parameters such as 90% mass participation, spectrum base shear should be at least equal to the fundamental base shear calculated from formula, etc. that are required to be satisfied as per IS1893:2002 are checked and alterations made to accomplish these stipulations.

10. The above steps are followed, this time with struts modelled in the frame to study the affect of the stiffness of the infill walls in the model.

11. Thereafter, we generate hinge properties for each member by entering the stirrups spacing and bar diameters and the confined strength of concrete to get the moment-curvature and interaction diagram in SAP2000. The various pop-up boxes to obtain the graphs in SAP2000 are shown below.

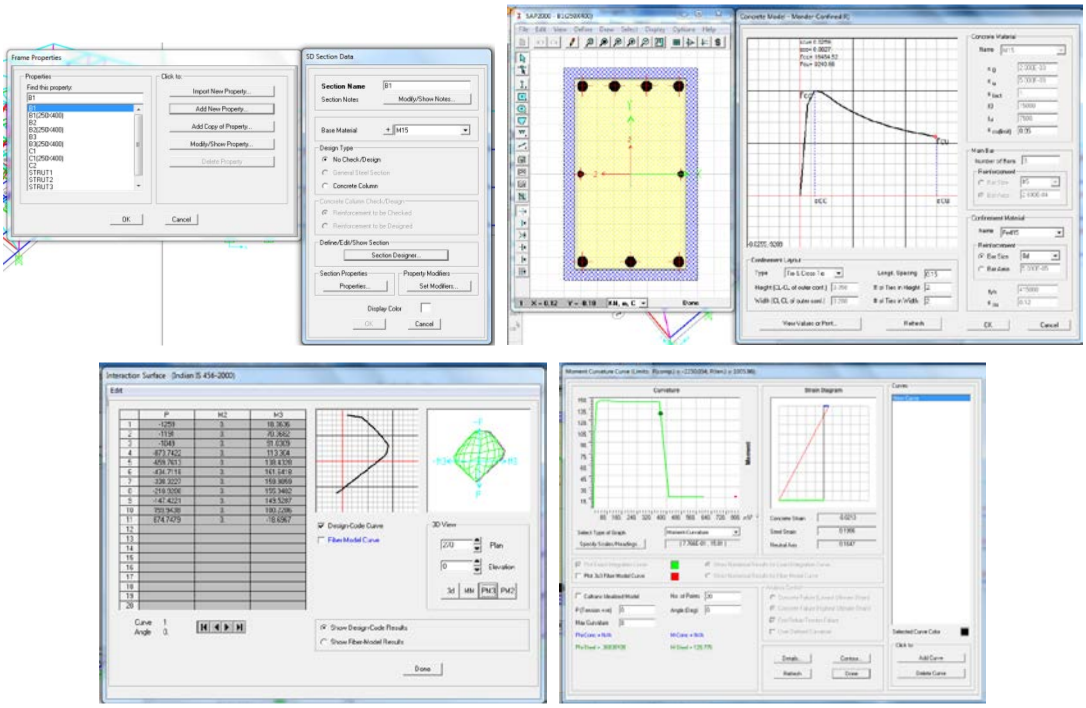


Fig.5.15 – After entering the input parameters in the windows shown in first layer, the interaction diagram and the moment-curvature graphs are obtained as shown in the second layer windows.

12. These graphs are then used to provide the user-defined hinge properties in the model in ETABS. Finally these hinges are assigned for each member at 0.05 and 0.95 meters at the start and end of the member length as it corresponds to the joints where the hinge is most likely to form first.

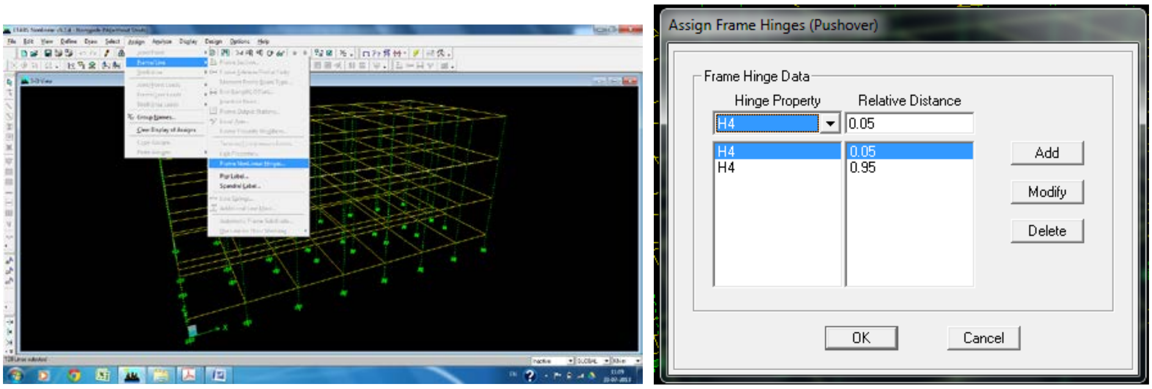


Fig.5.16 – Windows showing method to provide hinge at specific location of the respective members

13. We create the non-linear static pushover loads as per predefined pattern, in this instance, the structure being a low rise building, the first mode was the most apt to represent the behaviour of the building. The windows wherein the parameters are entered in the model is shown below.

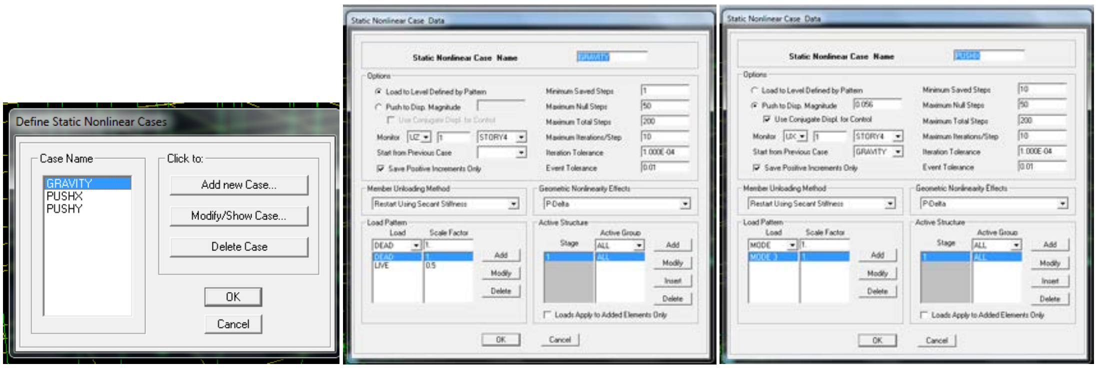


Fig.5.17 – Windows wherein the input parameters are entered to generate the pushover load cases.

14. Finally, we carry-out the analysis to obtain the capacity curve, performance point and step wise manner in the formation of hinges due to the pushover loads.

RESULTS AND DISCUSSIONS

6.1 TARGET DISPLACEMENT OF BUILDING

The target displacements were achieved from the response spectrum analysis of the models i.e. with infill walls and without infill walls. The same is shown in tabulated format below:

	Axis	Dominating Mode	Maximum Displacement
Without Struts	**X**	**III**	**0.056m**
	Y	**I**	**0.12m**
Including Struts	**X**	**III**	**0.017m**
	Y	**II**	**0.015m**

Thereafter, after obtaining the target displacements from the output of the Response Spectrum Analysis, they are used as inputs for the pushover analysis to obtain the following results. The same is shown in a tabulated format below:

	Axis	Performance Point Lies Between Steps	Performance Point Parameter Values
Model Without Struts	X	7 & 8	$T_{eff} = 0.463$
Model Without Struts	X	7 & 8	$\beta_{eff} = 0.121$
Model Without Struts	X	7 & 8	$V = 2106.199$
Model Without Struts	X	7 & 8	$D = 0.043$
Model Without Struts	X	7 & 8	$S_a = 0.642$
Model Without Struts	X	7 & 8	$S_v = 0.034$
Model Without Struts	Y	7 & 8	$T_{eff} = 0.702$
Model Without Struts	Y	7 & 8	$\beta_{eff} = 0.156$
Model Without Struts	Y	7 & 8	$V = 1670.803$
Model Without Struts	Y	7 & 8	$D = 0.076$
Model Without Struts	Y	7 & 8	$S_a = 0.500$
Model Without Struts	Y	7 & 8	$S_v = 0.061$
Model Including Struts	X	4 & 5	$T_{eff} = 0.172$
Model Including Struts	X	4 & 5	$\beta_{eff} = 0.050$
Model Including Struts	X	4 & 5	$V = 1721.786$
Model Including Struts	X	4 & 5	$D = 6.348\text{E-}03$
Model Including Struts	X	4 & 5	$S_a = 0.898$
Model Including Struts	X	4 & 5	$S_v = 6.607\text{E-}03$
Model Including Struts	Y	No Performance Point Achieved.	$T_{eff} =$ Not obtained
Model Including Struts	Y	No Performance Point Achieved.	$\beta_{eff} =$ Not obtained
Model Including Struts	Y	No Performance Point Achieved.	$V =$ Not obtained
Model Including Struts	Y	No Performance Point Achieved.	$D =$ Not obtained
Model Including Struts	Y	No Performance Point Achieved.	$S_a =$ Not obtained
Model Including Struts	Y	No Performance Point Achieved.	$S_v =$ Not obtained

6.2 CAPACITY CURVES & PERFORMANCE POINTS

The following pictures show the capacity curves obtained in respective directions for both the models i.e. with and without infill walls.

6.2.1 Model without Infill walls in X-direction:

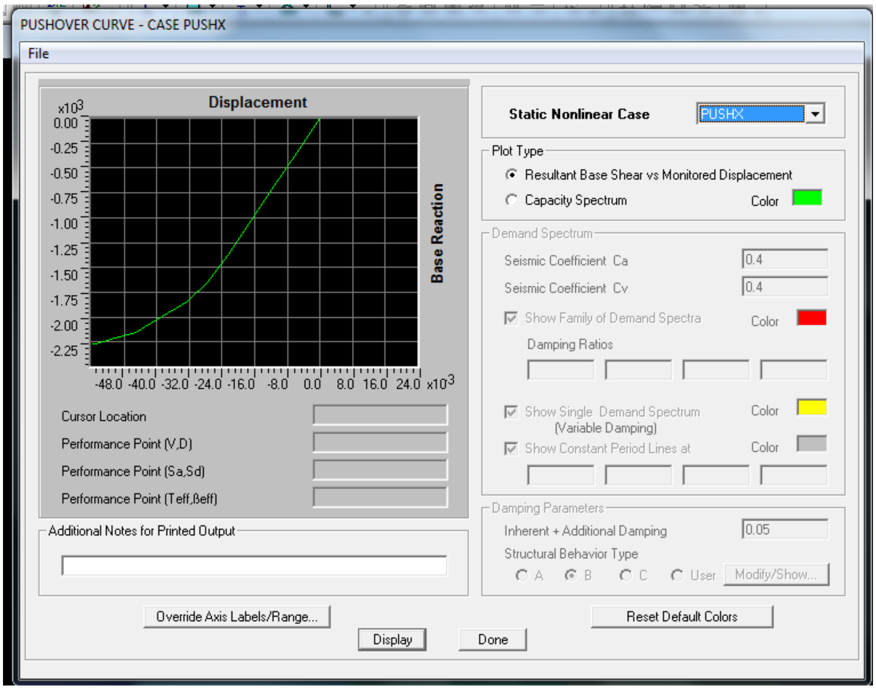


Fig.6.1 – Capacity Curve

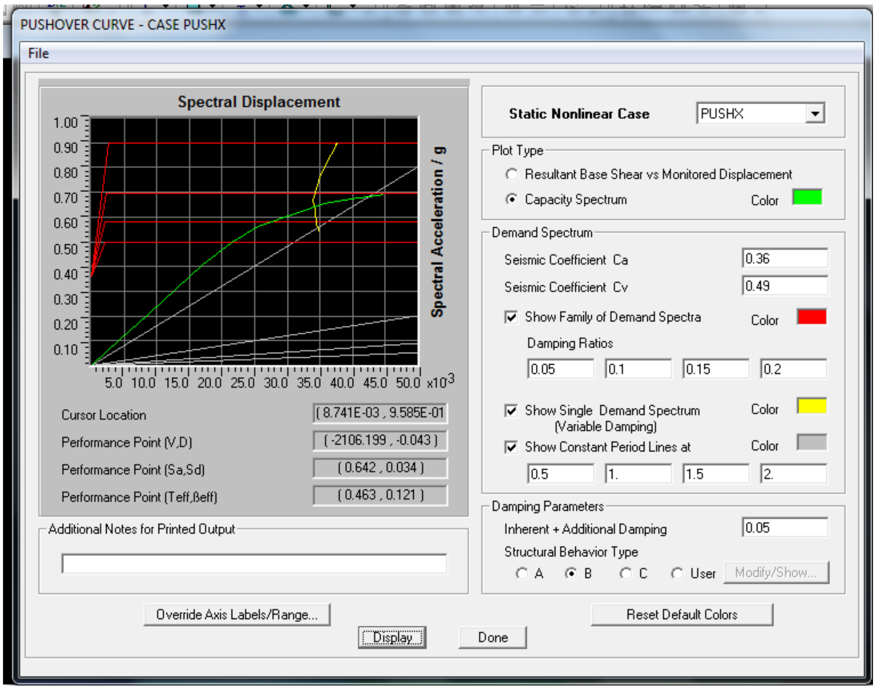


Fig.6.2–Capacity Curve superimposed on Demand Curve.

6.2.2 Model without Infill walls in Y-direction:

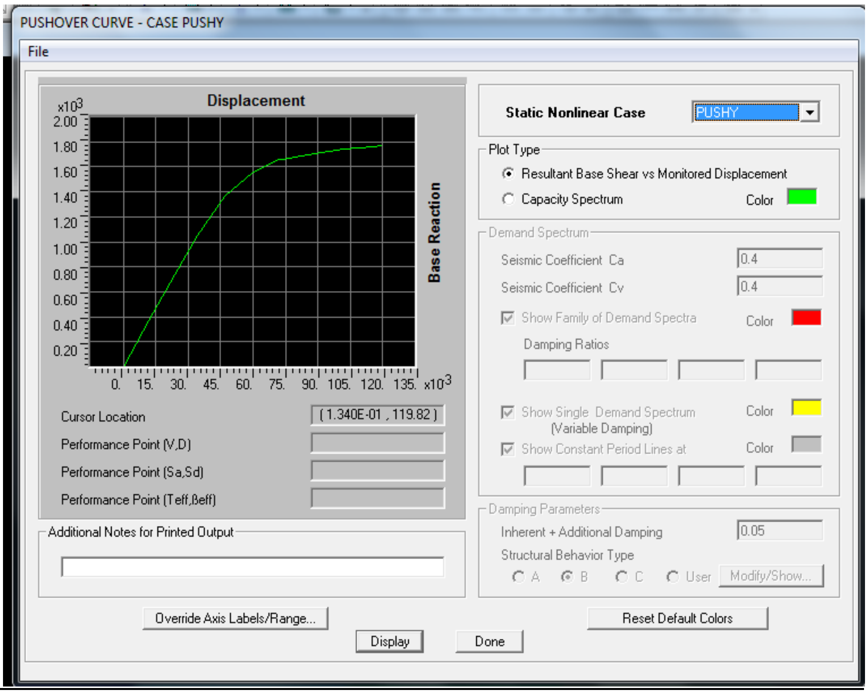


Fig.6.3 – Capacity Curve

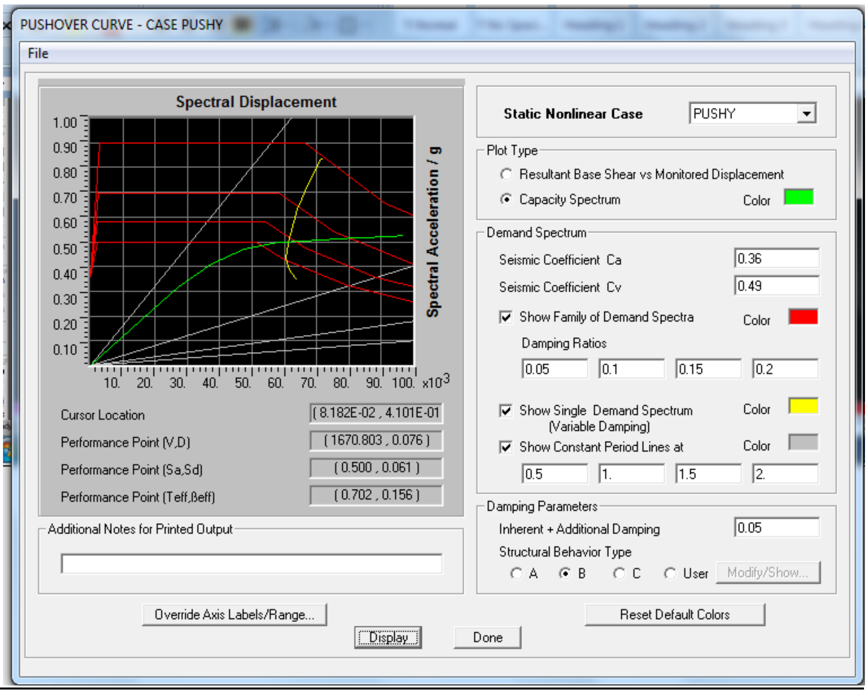


Fig.6.4–Capacity Curve superimposed on Demand Curve.

6.2.3 Model with Infill walls in X-direction:

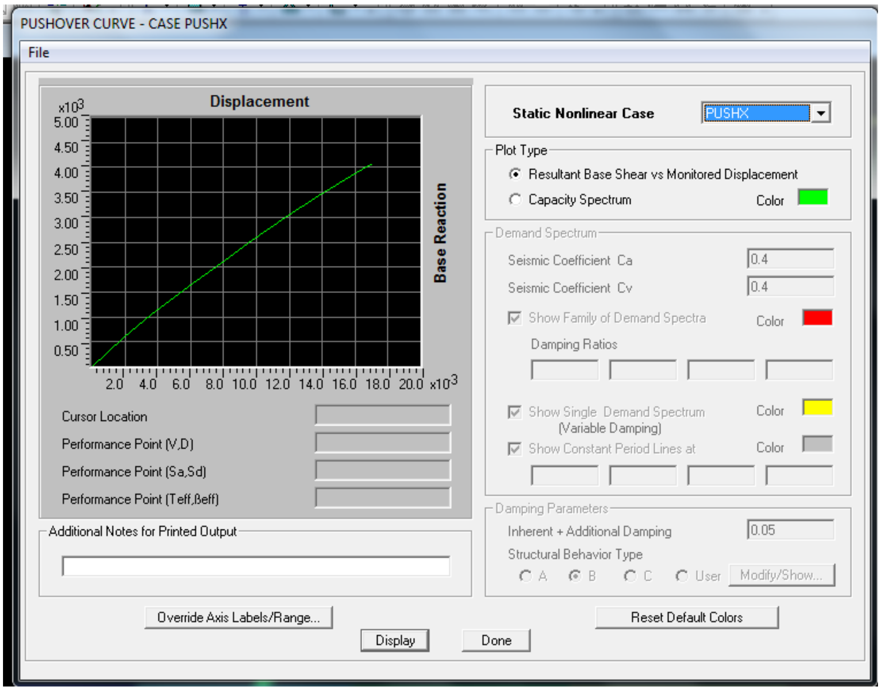


Fig.6.5 – Capacity Curve

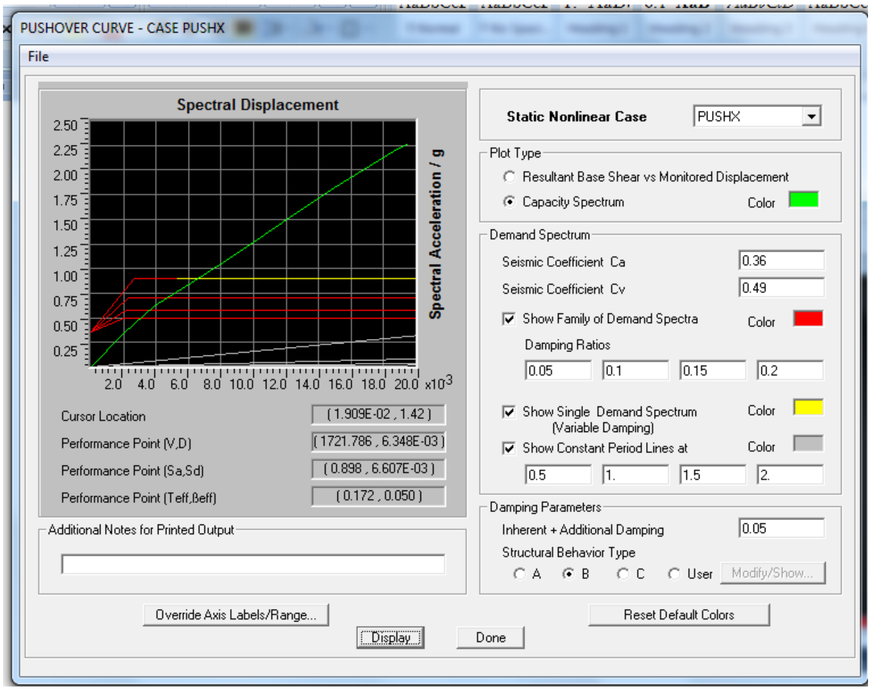


Fig.6.6–Capacity Curve superimposed on Demand Curve.

6.2.4 Model with Infill walls in Y-direction:

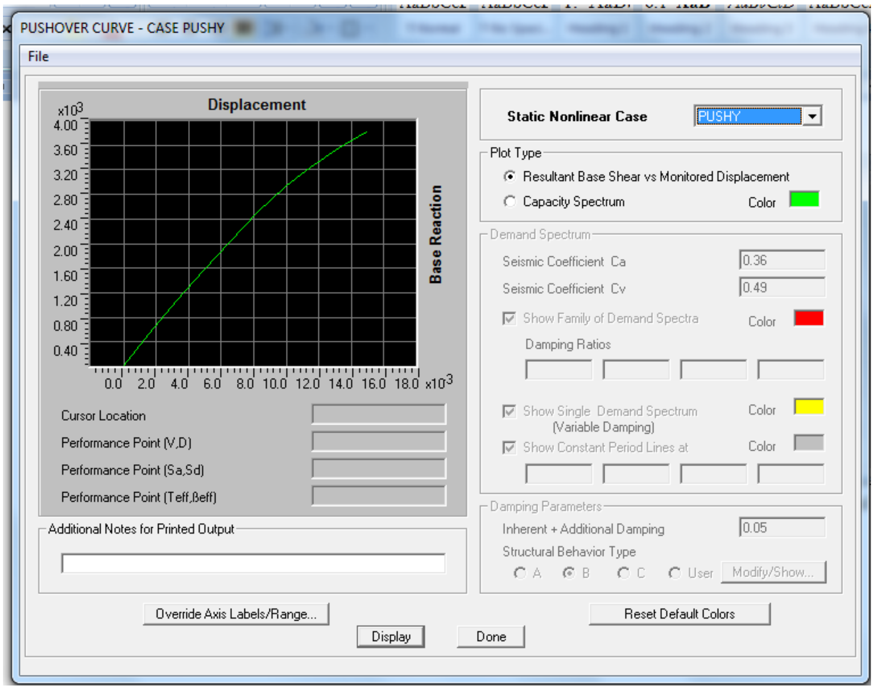


Fig.6.7 – Capacity Curve

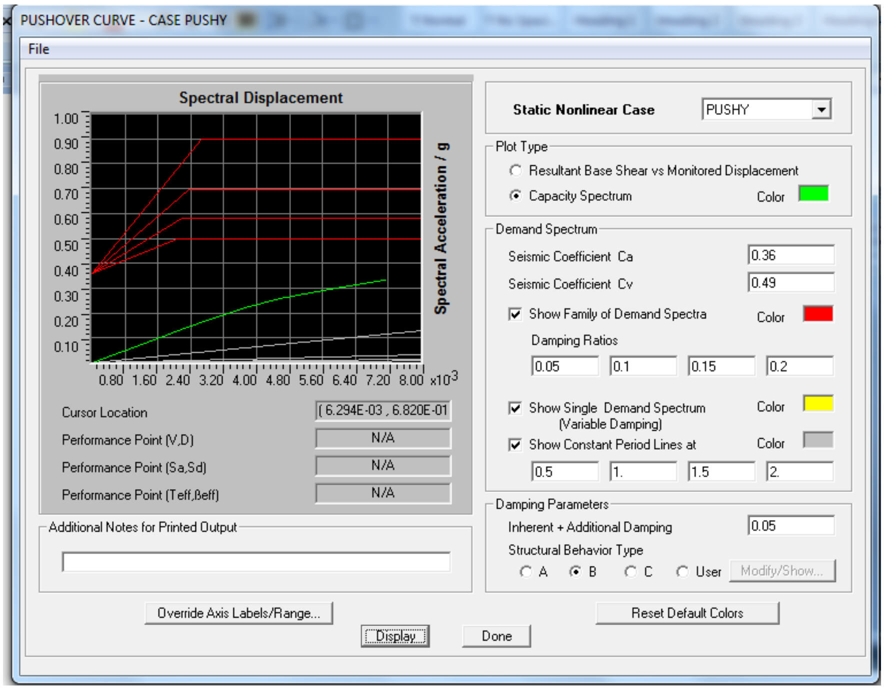
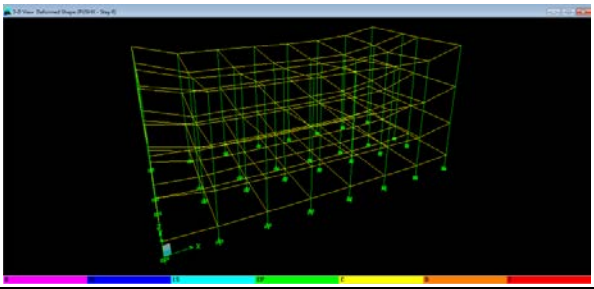


Fig.6.8–Capacity Curve superimposed on Demand Curve.

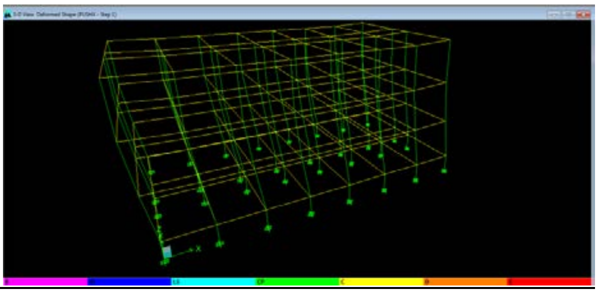
6.3 HINGE FORMATION OBSERVATION

Step-wise formation of hinges under the application of the pushover loads in respective directions is shown in the following pictures.

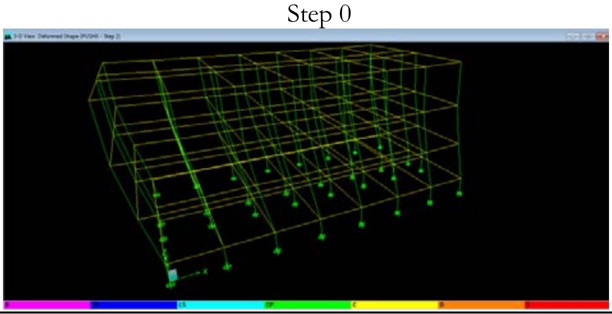
6.3.1 Model without Struts with pushover load application along X-axis:



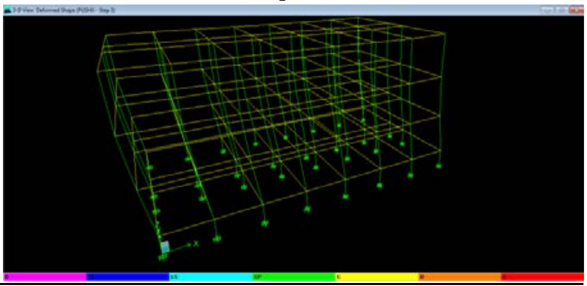
Step 0



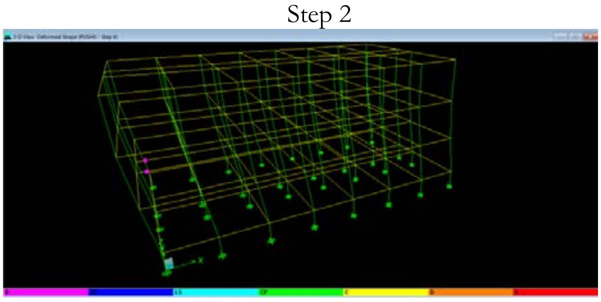
Step 1



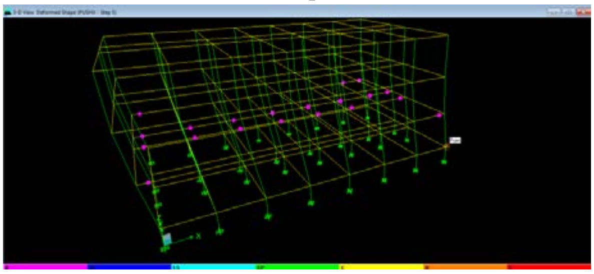
Step 2



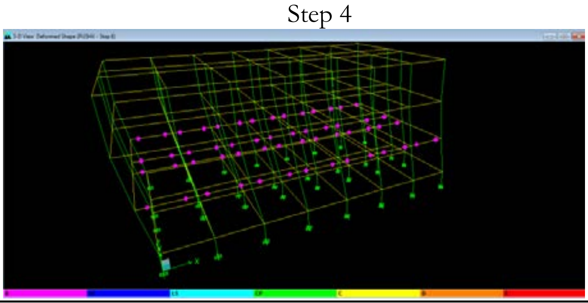
Step 3



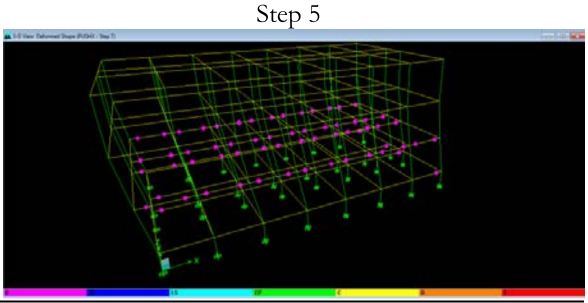
Step 4



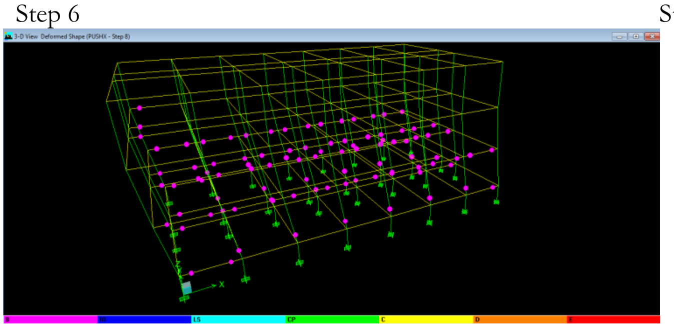
Step 5



Step 6

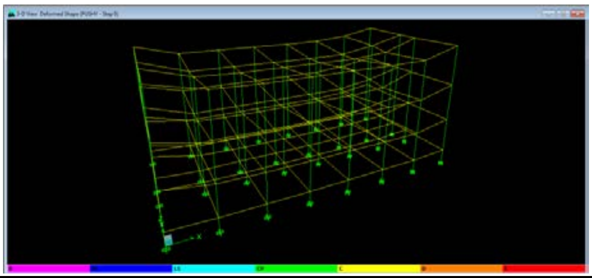


Step 7



Step 8

6.3.2 Model without Struts with pushover load application along Y-axis:



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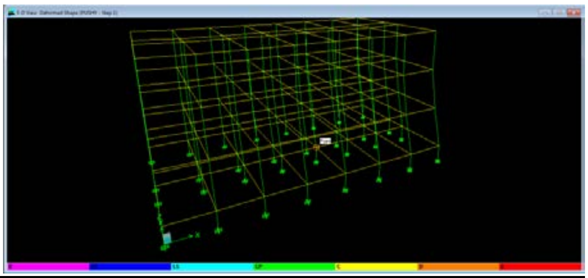
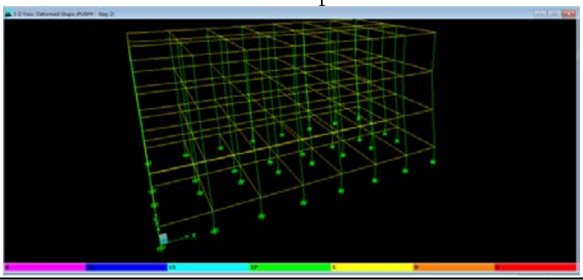
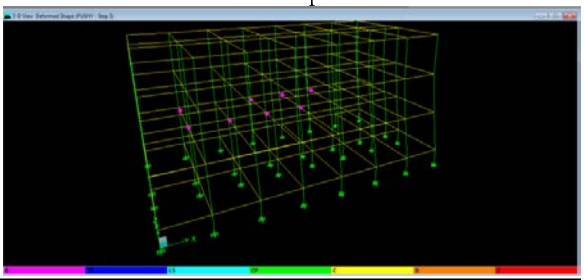


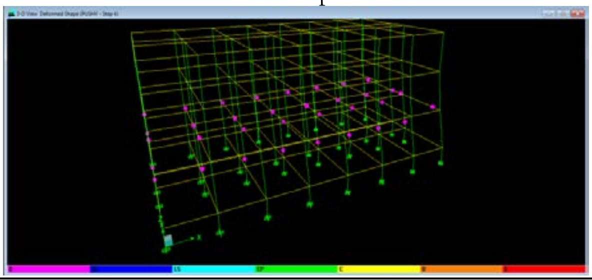
Step 1



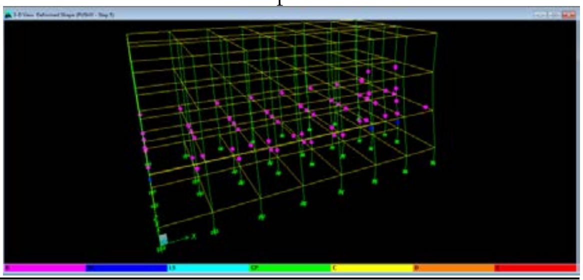
Step 2



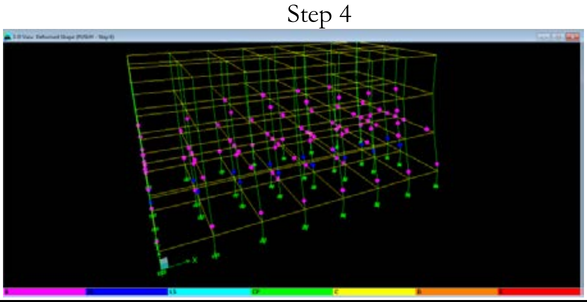
Step 3



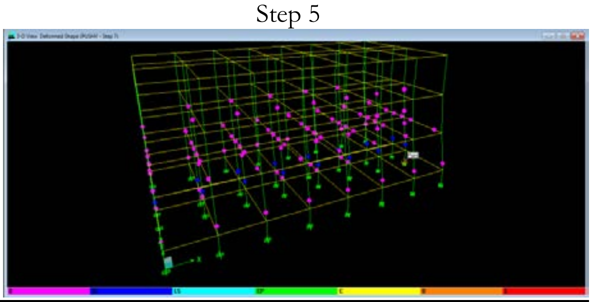
Step 4



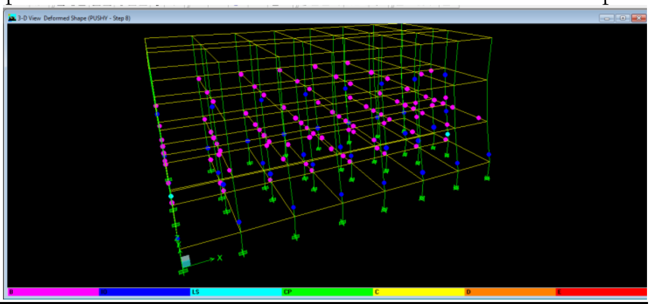
Step 5



Step 6

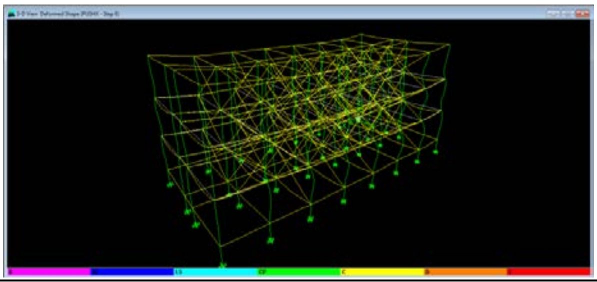


Step 7

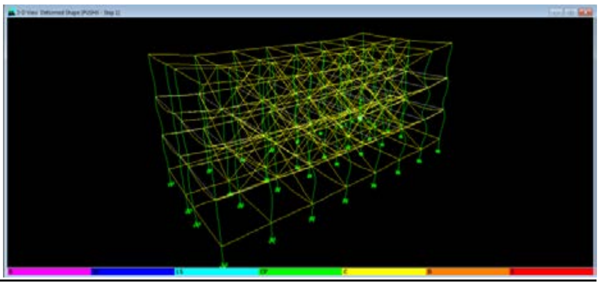


Step 8

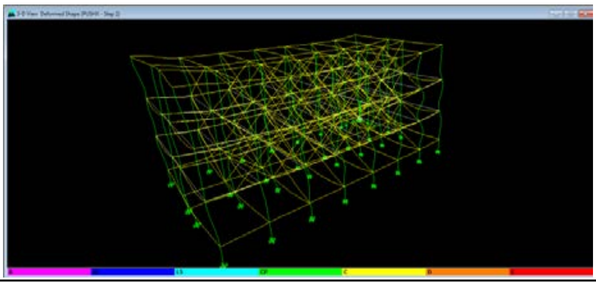
6.3.3 Model including Struts with pushover load application along X-axis:



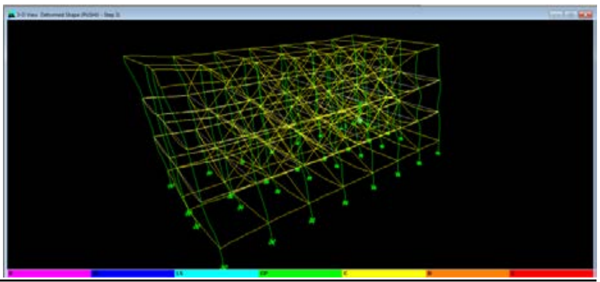
Step 0



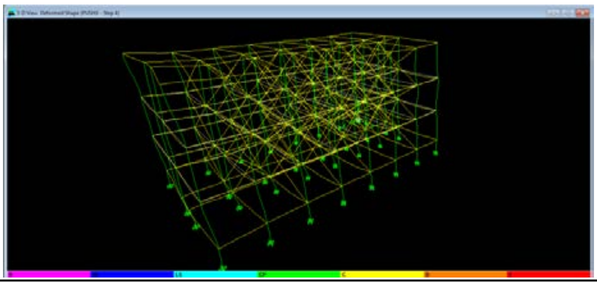
Step 1



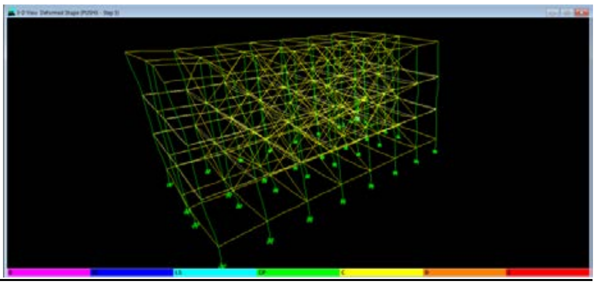
Step 2



Step 3



Step 4



Step 5

6.3.4 Model including Struts with pushover load application along Y-axis:

Performance point not achieved.

SUMMARY

(a) In model without struts with pushover loads in x-direction: Hinge formation started forming from Step 4 onwards and spread throughout the first floor level. Thereafter the hinge formation spread in the adjacent floor levels and columns. They all satisfied the criteria of 'Immediate Occupancy' till the performance point of the building was achieved.

(b) In model without struts with pushover loads in y-direction: Hinges started to form from step 3 onwards. Only 8 hinges formed at the first floor level followed by more hinge formations in the first floor level and inner columns at ground floor level. The hinges in columns at ground floor level started yielding at a faster rate and reached the condition of 'Life safety' till the performance point of the building was achieved.

(c) In model with struts with pushover loads in x-direction: Hinge formation did not start until the 6^{th} step. Thereafter, the hinge started forming in the columns at foundation base level and rapidly started yielding and passed the 'Immediate Occupancy' zone. However, the performance point of the building was between the 4^{th} and 5^{th} Steps.

(d) In model with struts with pushover loads in y-direction: No hinge formation occurred until the 6^{th} step. However, no performance point was achieved of the building in the Y-direction.

CONCLUSIONS

7.1 GENERAL

In the present study, the response spectrum analysis of a three storey reinforced concrete frame Hospital Building was carried out in ETABS v9.7.4 for two different cases. The first model was simulated as a bare frame under dead and live loads, whereas the second model was with infill walls. Target building displacement in both orthogonal axes was obtained for both models. Thereafter, user-defined hinge properties based on the moment-curvature and interaction diagram of respective members was derived and incorporated in the model for the respective members. Non-linear static pushover analysis was carried out for both models to study the different structural behaviour of the building through the various parameters such as the formation of the hinges, their rate of formation, members which were yielding, performance point of the building, etc.

The hinge formation in model without infill wall shows ductile behaviour with the increase of lateral displacement. Simultaneously, the formation of number of hinges increases and spreads throughout the structure. The performance of the structure with infill wall shows less lateral displacement, which is as expected. Moreover, the hinge formation is these models are fewer in number.

The present derived hinge properties shows better results as compared to the default hinge properties inbuilt within the used software.

For infill wall structure, the rate of formation of plastic hinge between two performance points i.e. Immediate Occupancy (IO) and Life Safety (LS) is faster than the structure with no infill wall.

7.2 RECOMMENDATIONS

It is concluded from the literature review that the Time-History analysis consumes more time and is tedious compared to the present non-linear static analysis (pushover analysis). Pushover analysis may be recommended for accurately predicting the behaviour of the structure as it gives the actual mechanism of the structure.

7.3 FUTURE SCOPE

Effect of infill wall stiffness may be studied for medium and high rise structures.

Higher mode shape loads and other types of loading patterns may be incorporated for pushover analysis studies.

In the present study, the effect of the passive earth pressure created by the soil from ground level till depth of foundation has not been modelled. Passive pressure causes strong resistance to the building for lateral displacement. Hence, the study can be done with simulation of the earth pressure.

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