PUSHOVER ANALYSIS OF MULTISTOREY BUILDING

Dessertation Submitted in the partial fulfilment of the requirement for the award of

MASTER OF ENGINEERING

(STRUCTURAL ENGINEERING)

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CERTIFICATE

This is to be certify that the project entitle **'PUSHOVER ANALYSIS OF MULTISTOREY BUILDING'**, being submitted by me, is a bonafied record of my own work carried by me under the guidance and supervision of Asst. Professor, Mr. G P Awadhiya in partial fulfilment of the requirement for the award of the Degree of Master of engineering (Structure Engineering) in civil Engineering, from Delhi College of Engineering, Delhi.

The matter embodied in this project has not been submitted for the award of any other degree.

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Any accomplishment requires the effort of many people and this work is no exception. I appreciate the contribution and support, which various individuals have provided for the successful completion of this to mention all by name but the following were singled out for their exception help.

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ABSTRACT

In the practical design applications the evaluation of seismic response is usually based on linear elastic structural behaviour. However this approach may be not sufficient in limiting the damage levels of the buildings. To this purpose more accurate methods of analyses, which can predict the real behaviour under strong seismic actions, are required. The non-linear dynamic analysis is the most rigorous method.

The non-linear static pushover analysis seems to be a more rational method for estimating the lateral strength and the distribution of inelastic deformations. In this thesis Pushover analyses were performed by ETABS to predict the behaviour under strong seismic action and comparing the forces in static linear and nonlinear analysis.

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LIST OF ABBREVIATION

English Symbols

$\mathbf{C}$ - Classical damping

$\mathbf{C_0}$ - Factor for MDOF displacement

$\mathbf{C_1}$ - Factor for inelastic displacement

$\mathbf{C_2}$ - Factor for strength and stiffness degradation

$\mathbf{C_3}$ - Factor for geometric nonlinearity

$\mathbf{E_c}$ - Short-term modulus of elasticity of concrete

$\mathbf{E_D}$ - Energy dissipated by damping

$\mathbf{E_s}$ - Modulus of elasticity of steel rebar

$\mathbf{E_S}$ - Maximum strain energy

$\mathbf{E_I}$ - Flexural rigidity of beam

$\mathbf{f_y}$ - Yield stress of steel rebar

$\mathbf{F_y}$ - Defines the yield strength capacity of the SDOF

$\mathbf{K_{eq}}$ - Equivalent stiffness

Ki - Initial stiffness

$\mathbf{m}$ - Storey mass

$\mathbf{n_M}$ - Modal mass for nth mode

$\mathbf{N}$ - Number of modes considered

$\mathbf{q(t)_n}$ - The modal coordinate for nth mode

$\mathbf{R}$ - Normalized lateral strength ratio

$\mathbf{S_a}$ - Spectral acceleration

$\mathbf{S_d}$ - Spectral displacement

$\mathbf{SR_A}$ - Spectral reduction factor at constant acceleration region

$\mathbf{SR_V}$ - Spectral reduction factor at constant velocity region

$\mathbf{T}$ - Fundamental natural period of vibration

$\mathbf{T_{eq}}$ - Equivalent time period

$\mathbf{T_i}$ - Initial elastic period of the structure

$\mathbf{T_n}$ - Nth mode natural period

Greek Symbols

$\boldsymbol{\alpha}$ - Post-yield stiffness ratio

$\boldsymbol{\beta_{eq}}$ - Equivalent damping

$\boldsymbol{\beta_i}$ - Initial elastic damping

$\boldsymbol{\beta_s}$ - Damping due to structural yielding

$\mathbf{t}\ \boldsymbol{_\delta}$ - Target displacement

$\boldsymbol{\phi_u}$ - Ultimate curvature

$\boldsymbol{\phi}\ \boldsymbol{_y}$ - Yield curvature

$\boldsymbol{\Gamma}$ - Mode participation factor

$\boldsymbol{\mu}$ - Displacement ductility ratio

$\boldsymbol{\omega_n}$ - Nth mode natural frequency

$\boldsymbol{\xi_n}$ – Nth mode damping ratio

<u>CHAPTER – 1</u>

INTRODUCTION

PUSHOVER ANALYSIS OF MULTISTOREY BUILDING

CHAPTER - 1

1.0 INTRODUCTION

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.

In literature, pushover analysis has been shown to capture essential structural response characteristics under seismic action.

1.1 Definition: As per ATC 40

Pushover analysis is a static, nonlinear procedure uses simplified nonlinear technique to estimate seismic structural deformations. It is an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element.

The analysis involves applying horizontal loads, in a prescribed pattern, to the structure incrementally, i.e. pushing the structure and plotting the total applied shear force and associated lateral displacement at each increment, until the collapse condition.

1.2 Purpose of Pushover Analysis

It is expected that most buildings rehabilitated in accordance with a standard, would perform within the desired levels when subjected to the design earthquakes. Structures designed according to the existing seismic codes provide minimum safety to preserve life and in a major earthquake, they assure at least gravity-load-bearing elements of non-essential facilities will still function and provide some margin of safety.

However, compliance with the standard does not guarantee such performance. They typically do not address performance of non-structural components neither provide differences in performance between different structural systems. This is because it cannot accurately estimate the inelastic strength and deformation of each member due to linear elastic analysis. Although an elastic analysis gives a good indication of elastic capacity of structures and indicates where first yielding will occur, it cannot predict failure mechanisms and account for redistribution of forces during progressive yielding.

To overcome this disadvantage different nonlinear static analysis method is used to estimate the inelastic seismic performance of structures, and as a result, the structural safety can be secured against an earthquake. Inelastic analyses procedures help demonstrate how buildings really work by identifying modes of failure and the potential for progressive collapse. The use of inelastic procedures for design and evaluation helps engineers to understand how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded. This resolves some of the uncertainties associated with code and elastic procedures. The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limit some form of nonlinear analysis, like Pushover Analysis, is required

1.3 Objective of Pushover Analysis

Pushover analysis is a performance-based analysis that refers to a methodology in which structural criteria are expressed in terms of achieving a performance objective. A performance level describes a limiting damage condition which may be considered satisfactory for a giving building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building’s occupants created by the damage, and the post earthquake serviceability of the building. The basic approach is to improve the probable seismic performance of the building or to otherwise reduce the existing risk to an acceptable level.

The seismic evaluation of existing buildings is a more difficult task than the seismic design of new buildings. Non-linear methods are needed if realistic results are to be obtained.

Structural response to strong earthquake ground motion cannot be accurately predicted due to large uncertainties and the randomness of structural properties and ground motion parameters. Consequently, excessive sophistication in structural analysis is not warranted. For the time being, the most rational analysis and performance evaluation methods for practical applications seem to be simplified non-linear procedures, which combine the non-linear static (pushover) analysis of a relatively simple mathematical model and the response spectrum approach.

Practically all structural damage and a large portion of the non-structural damage sustained in buildings as a result of earthquake ground motions are produced by lateral displacements. Thus, the estimation of lateral displacement demands is of primary importance in performance based earthquake resistant design and in general when damage control is of interest.

Furthermore, most structures will experience inelastic deformations when subjected to severe earthquake ground motions. Thus, of special interest is an adequate estimation of lateral displacement demands in structures that exhibit non-linear behaviour. This is particularly true in performance-based design in which a better prediction of seismic performance is desired.

The structural analysis in earthquake engineering is a complex task because (a) the problem is dynamic and usually non-linear, (b) the structural system is usually complex, and (c) input data (structural properties and ground motions) are random and uncertain .However, such an approach, for the time being, is not practical for everyday design use. It requires additional input data (time history of ground motions and detailed hysteretic behaviour of structural members) which cannot be reliably predicted.

Non-linear dynamic analysis is, at present, appropriate for research and for design of important structures. It represents a long-term trend. On the other hand, the methods applied in the great majority of existing building codes are based on the assumption of linear elastic structural behaviour and do not provide information about real strength, ductility and energy dissipation. They also fail to predict expected damage in quantitative terms. For the time being, the most rational analysis and performance evaluation methods for practical applications seem to be simplified inelastic procedures, which combine the non-linear static (pushover) analysis of a relatively simple mathematical model and the response spectrum approach.

1.4 Earthquake Resistant Design Technique

The design seismic forces acting on a structure as a result of ground shaking are usually determined by one of the following methods:

- Static analysis, using equivalent seismic forces obtained from response spectra for horizontal earthquake motions.
- Dynamic analysis, either modal response spectrum analysis or time history analysis with numerical integration using earthquake records.

1.4.1 Static Analysis

Although earthquake forces are of dynamic nature, for majority of buildings, equivalent static analysis procedure can be used. These have been developed on the basis of considerable amount of research conducted on the structural behaviour of structures subjected to base movements. These methods generally determine the shear acting due to an earthquake as equivalent static base shear. It depends on the weight of the structure, the dynamic characteristics of the building as expressed in the form of natural period or natural frequency, the seismic risk zone, and the type of structure the geology of the site and importance of the building.

1.4.1.1 Linear Static Procedure (LSP)

Under the linear static procedure (LSP), design seismic forces their distribution over the height of the building and the corresponding internal forces and the system displacements are determined using a linearly elastic static analysis. In the LSP, the building is modelled with linearly elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Design earthquake demands for the LSP are represented by static lateral forces whose sum is equal to the pseudo lateral forces. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linear elastic model of the building it will result in design displacement amplitude approximating maximum displacements that are expected during the design earthquake. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximation of those expected during the design earthquake. If the building responds in-elastically to the design earthquake, as will commonly be the case the internal forces that would develop yielding in the building, will be less than the internal forces calculated on an elastic basis.

1.4.1.2 Non Linear Static Procedure (NSP)

Under the non linear static procedure (NSP), a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformation and forces are determined. The non-linear deformation characteristics of individual components and elements of the building are modelled directly. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is expected or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude.

Because the mathematical model accounts directly for the material inelastic response, the calculated internal forces will be reasonable approximation of those expected during the design earthquake. Results of the NSP are to be checked using the applicable acceptance criteria. Calculated Displacement and internal forces are compared directly with allowable values.

1.4.2 Dynamic Analysis

1.4.2.1 Linear Dynamic Procedure (LDP)

Under the LDP design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly elastic dynamic analysis. The basis modelling approaches and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using either modal spectral analysis or time history analysis. Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacement that are approximately correct, but will produce internal forces that exceed those that would be obtained in a yielding building. Results of the LDP are to be checked using the applicable acceptance criteria. Calculated displacements are compared directly with allowable values. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of component and elements. These obtained design forces are evaluated through the acceptance criteria, which include modification factors and alternative analysis procedure to account for anticipated inelastic response.

1.4.2.2 Nonlinear Dynamic Procedure (NDP)

Under the non linear Dynamic Procedure (NDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis. The basis, modelling approaches, and acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using Time-History Analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground motion histories. Calculations response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. Results of the NDP are to be checked using the applicable acceptance criteria. Calculated displacements and internal forces are compared directly with allowable values.

CHAPTER – 2

LITERATURE REVIEW

CHAPTER – 2

2.0 LITERATURE REVIEW

2.1 Brief Overview of Simplified Non-Linear Methods

In 1975, Freeman at al developed a rapid evaluation method, which can be considered as a forerunner of the today's "Capacity spectrum method". In 1981, Saiidi and Sozen proposed to perform non-linear dynamic analyses on an equivalent SDOF system. Based on this idea, Fajfar and Fischinger developed in mid-1980s the first version of the N2 method (N stands for Non-linear and 2 for two mathematical models – a SDOF and a MDOF model). However, the earthquake engineering community has not paid much attention to simplified non-linear approaches until mid-1990s, when a breakthrough of this approaches occurred. All methods combine the pushover analysis of a multi-degree-of freedom model (MDOF) with the response spectrum analysis of an equivalent single degree-of-freedom (SDOF) system. Inelastic spectra or elastic spectra with equivalent damping and period are applied. As an alternative representation of inelastic spectrum the Yield point spectrum has been developed.

This chapter presents an overview on related topics that provide the necessary background for this research. For the understanding of seismic capacity, a review of literature is required in experimental testing, current design practice, theoretical strength evaluation and modelling technique such as finite element modelling. The literature review begins with a coverage of general earthquake engineering topics, which serves to the context of the research.

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 elastic and inelastic, are available to predict the seismic performance of the structures.

Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, it cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. Inelastic analysis procedures help demonstrate how buildings really work by identifying modes of failure and the potential for progressive collapse. The use of inelastic procedures for design and evaluation is an attempt to help engineers better understand how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded. This resolves some of the uncertainties associated with code and elastic procedures.

2.3 Elastic Methods of Analysis

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes.

In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum. 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.

Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don’t predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses. Real deficiencies present in the structure could be missed. Moreover, force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair. The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation. Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly.

2.4 Inelastic Methods of Analysis

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behaviour of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modelling and ground motion characteristics. It requires proper modelling of cyclic load deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method [20] and the Secant Method .The theoretical background, reliability and the accuracy of inelastic static analysis procedure is discussed in detail in the following sections.

2.4.1 Summary

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.

Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

2.5 DESCRIPTION OF PUSHOVER ANALYSIS

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve (Fig.2.1).

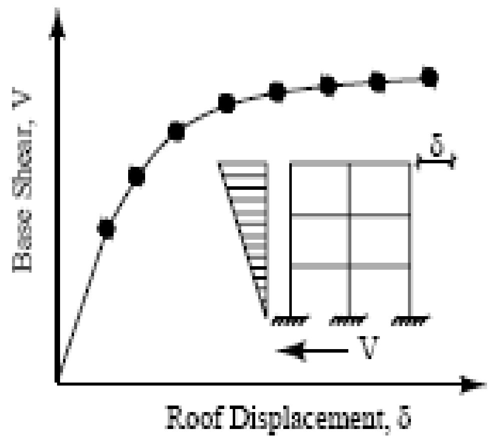


FIG 2.1 GLOBAL CAPACITY (PUSHOVER) CURVE OF A STRUCTURE

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. Generally, pushover analysis is performed as displacement-controlled.

The objective of a performance-based design is achieved after the user and the designer collectively select a target performance for the structure in question. The engineer carries out the conventional design and subsequently performs a pushover (elasto-plastic) analysis to evaluate if the selected performance objective has been met.

2.5.1 Force-Based Design Method

When equivalent static design loads are computed in a typical seismic design, the method illustrated in Fig. 2.2 is generally used. The engineer applies appropriate response force modification factors (R) to compute the design loads and ensures that the structure is capable of resisting the design loads. The significance of using the R factors here is that the structure exhibits inelastic behaviours during an earthquake.

That is, the structure is inflicted with material damage due to the earthquake loads. Depending on the energy absorption capability of the structure, the response force modification factors vary. The design method described herein is relative to loads and as such it is termed as “force-based design method”. However, a simple comparison of the strengths cannot predict the true behaviour of a structure. As a result, it is highly likely that a structure may be designed without a clear knowledge of the structural performance characteristics.

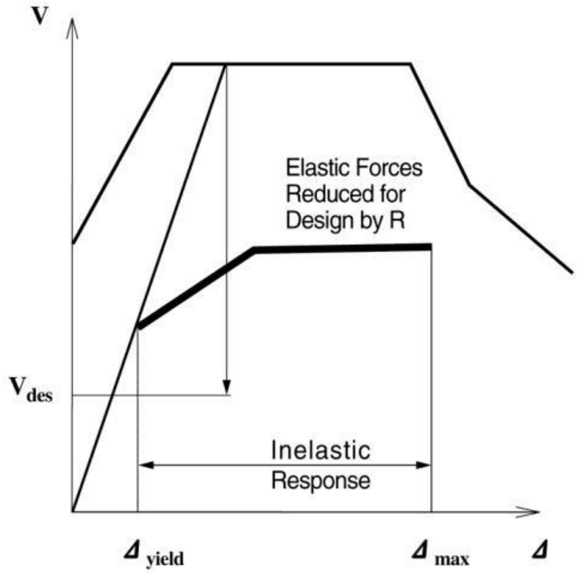


FIG. 2.2 CALCULATION OF EARTHQUAKE LOADS AS PER FORCE-BASED DESIGN METHOD

2.5.2 Displacement-Based Design Method

Where a performance-based design method is adopted, the project owner and the engineer pre-select a target performance. This reflects the intent of the project team to allow an appropriate level of structural damage or select the level of energy absorption capability due to anticipated seismic loads in a given circumstance. In order to achieve the objective, we need to be able to predict the deformation performance of the structure to the point of ultimate failure. The Eigen values change with the level of energy absorption capability. If the performance criteria are evaluated on the basis of the structure’s displacements, it is termed as “displacement-based design method”.

Where pushover analysis is carried out as one of the means of evaluating the structure’s deformability, a load-displacement spectrum is created as illustrated in Fig.

2.3. A demand spectrum is also constructed depending on the level of energy absorption capability of the structure. The intersection (performance point) of the two curves is thus obtained. If the point is within the range of the target performance, the acceptance criteria are considered to have been satisfied. That is, the performance point is evaluated against the acceptance criteria or vice versa.

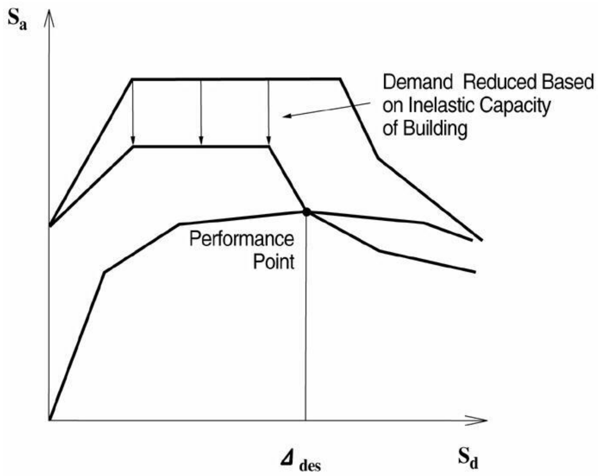


FIG. 2.3 SEISMIC DESIGNS BY PERFORMANCE-BASED DESIGN METHOD

The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

2.5.3 Analysis Method

Pushover analysis creates a capacity spectrum expressed in terms of a lateral load-displacement relationship by incrementally increasing static forces to the point of the ultimate performance. The capacity spectrum is then compared with the demand spectrum, which is expressed in the form of a response spectrum to seismic loads, to examine if the structure is capable of achieving the target performance. Accordingly, pushover analysis is often referred to as the second stage analysis, which is subsequently carried out after the initial structural analysis and design.

Pushover analysis can provide the following advantages:

- It allows us to evaluate overall structural behaviours and performance characteristics.

- It enables us to investigate the sequential formation of plastic hinges in the individual structural elements constituting the entire structure.
- When a structure is to be strengthened through a rehabilitation process, it allows us to selectively reinforce only the required members, thereby maximizing the cost efficiency.
- Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.

2.5.4 Primary Elements of the Pushover Analysis

Simplified non-linear analysis procedures using pushover method, such as capacity spectrum method, and displacement coefficient method, requires determination of three primary elements are briefly discussed below:
Capacity: Capacity is a representation of the structure's ability to resist the seismic demand the overall capacity of the structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine the capacities beyond the elastic limits, some form of non-linear analysis, such as the pushover procedure, is required.

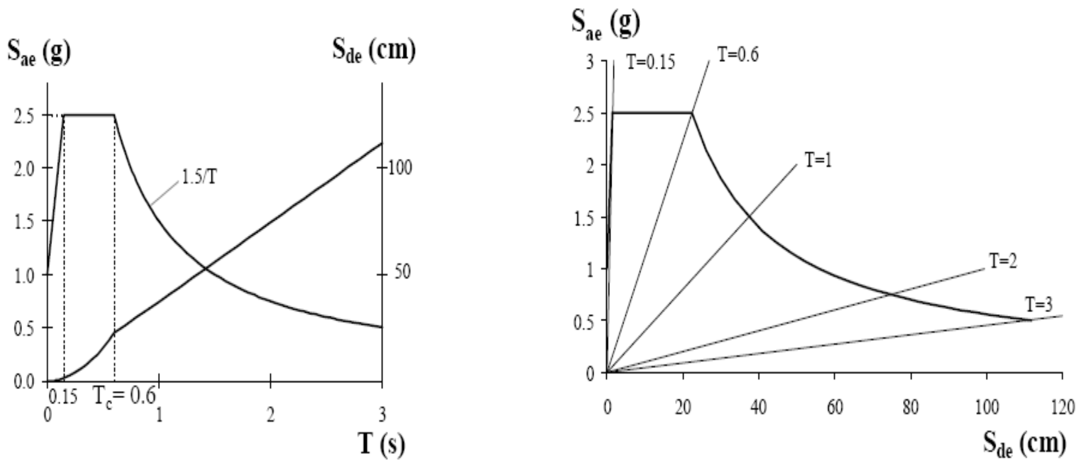


FIG. 2.4 TYPICAL ELASTIC ACCELERATION (S_{AE}) AND DISPLACEMENT SPECTRUM (S_{DE}) FOR 5 PER CENT DAMPING NORMALISED TO 0.1 G PEAK GROUND ACCELERATION A) TRADITIONAL FORMAT B) ADRD FORMAT

Capacity Spectrum

It is the capacity curve transformed from shear forces vs. roof displacement (V vs. d) coordinate into acceleration vs. spectral displacement (S_a Vs S_d) coordinates.

Demand (displacement): Demand is the representation of earthquake ground motion or shaking that the building is subjected to. In nonlinear static analysis procedures, demand is represented by an estimation of the displacements or deformations that the structure is expected to undergo.

Demand spectrum

It is the reduced response spectrum used to represent the earthquake ground motion in the capacity spectrum method.

Performance

Performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist demands of the earthquake such that the performance of the structure is compatible with the objectives of the design. Performance objective is to obtain a desired level of seismic performance of the building, generally described by specifying maximum allowable (or acceptable) structural or non-structural damage, for a specified level of seismic hazard.

Once a capacity curve and demand displacement is defined a performance check can be done. A performance check verifies that structural and non-structural components are not damaged beyond the acceptable limits of the performance objective for the force and displacement demand.

Performance point

The intersection of the capacity and the demand spectrum in the capacity spectrum method (the displacement at the performance point i.e. equivalent to the target displacement in the coefficient method)

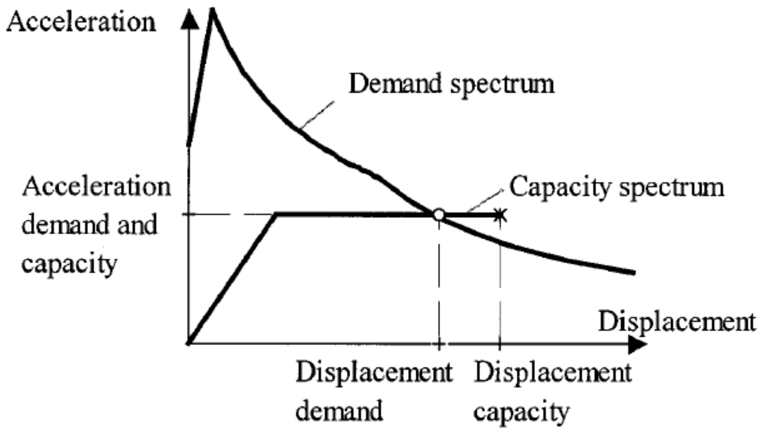


Figure 1. Capacity spectrum method

FIG. 2.5 CAPACITY SPECTRUM METHOD DEVELOPMENT OF PUSHOVER CURVE

Force Controlled Actions

According to ATC 40, force-controlled refers to components, elements, actions, or systems which are not permitted to exceed their elastic limits. This category of elements generally referred to as brittle or non ductile, experiences significant degradation after only limited post-yield deformation.

Deformation controlled actions

Deformation-controlled refers to components, elements, actions, or systems which can, and are permitted to, exceed their elastic limit in a ductile manner. Force or stress levels for these components are of lesser importance than the amount of deformation beyond the yield point.

Overview of Nonlinear Analysis

When a structure is analyzed for linear elastic behaviours, the analysis is carried out on the premise that a proportional relationship exists between loads and displacements. This assumes a linear material stress-strain relationship and small geometric displacements.

The assumption of linear behaviours is valid in most structures. However, nonlinear analysis is necessary when stresses are excessive, or large displacements exist in the structure. Construction stage analyses for suspension and cable stayed bridges are some of large displacement structure examples. Nonlinear analysis can be classified into 3 main categories.

First, material nonlinear behaviours are encountered when relatively big loadings are applied to a structure thereby resulting in high stresses in the range of nonlinear stress-strain relationship. The relationship, which is typically represented as in Fig. 2.6, widely varies with loading methods and material properties.

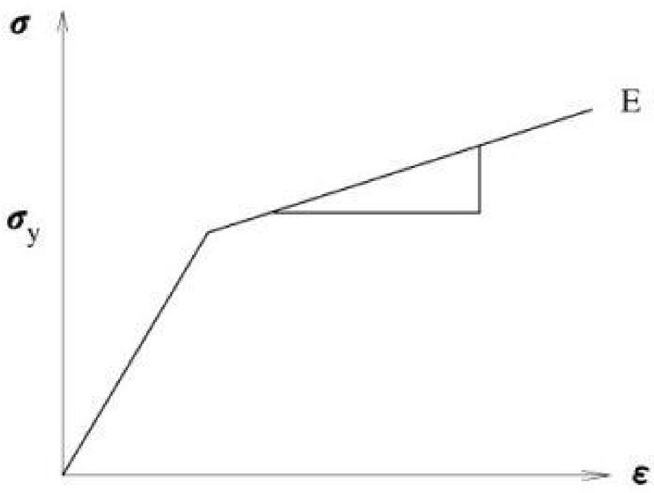
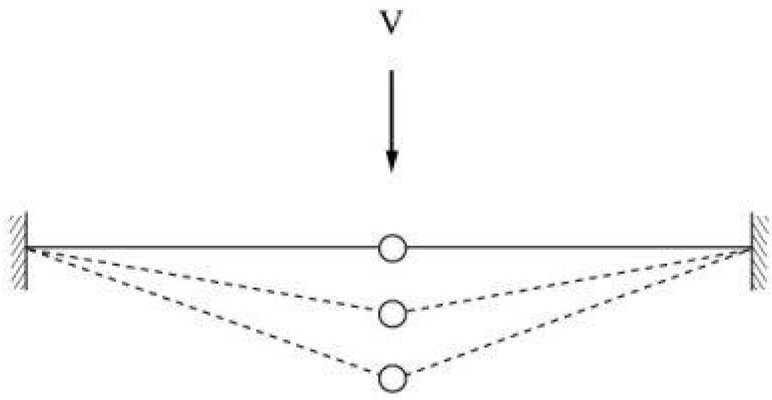


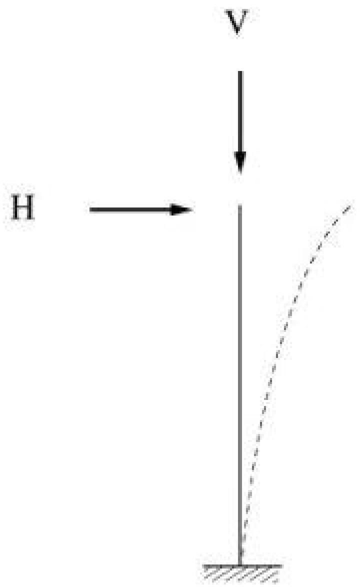
FIG. 2.6 STRESS-STRAIN RELATIONSHIPS USED FOR MATERIAL NONLINEARITY

Second, a geometric nonlinear analysis is carried out when a structure undergoes large displacements and the change of its geometric shape renders a nonlinear displacement-strain relationship. The geometric nonlinearity may exist even in the state of linear material behaviours. Cable structures such as suspension bridges are analyzed for geometric nonlinearity.

A geometric nonlinear analysis must be carried out if a structure exhibits significant change of its shape under applied loads such that the resulting large displacements change the coordinates of the structure or additional loads like moments are induced (See Fig. 2.7).



(a) Change in structural stiffness due to large displacement.



(b) Additional load induced due to displacement

FIG. 2.7 STRUCTURAL SYSTEMS REQUIRING GEOMETRIC NONLINEAR ANALYSES

Third, boundary nonlinearity of a load-displacement relationship can occur in a structure where boundary conditions change with its structural deformations due to external loads. An example of boundary nonlinearity would be compression-only boundary conditions of a structure in contact with soil foundation.

2.6 Seismic Inputs

Seismic inputs are the earthquake data that are necessary to perform different types of seismic analysis. In the context of seismic analysis and design of structures, various earthquake data may be required depending upon the nature of analysis being carried out. These data are presented in two different ways, namely, in deterministic and probabilistic forms. Seismic inputs in deterministic form are used for deterministic analysis and design of structures, while those in probabilistic form are used for random vibration analysis of structures for earthquake forces, seismic risk analysis of structures, and damage estimation of structures for future earthquakes. Seismic inputs for structural analysis are provided either in the time domain or in the frequency domain, or in both time and frequency domains. In addition, a number of earthquake parameters are also used as seismic inputs for completeness of the information that is required to perform different types of analysis. They include magnitude, intensity, peak ground acceleration/velocity/displacement, duration, predominant ground frequency, and so on. Further, certain types of analysis, such as, seismic risk analysis, damage estimation of structures, and probabilistic seismic analysis, the prediction of seismic input parameters for future earthquakes are essential.

2.6.1 Time History Records

The most common way to describe a ground motion is with a time history record. The motion parameters may be acceleration, velocity, or displacement, or all the three combined together. Generally, the directly measured quantity is the acceleration and the other parameters are the derived quantities. However, displacement and velocity can also be measured directly. Time histories of ground motions are used directly for the time domain analysis of structures subjected to deterministic seismic inputs.

Three components of the ground motion for the El Centro earthquake are shown in Fig. 2.8. The digitized versions of time histories of many earthquakes are available in web sites such as www.peer.berkeley.edu/sncat.

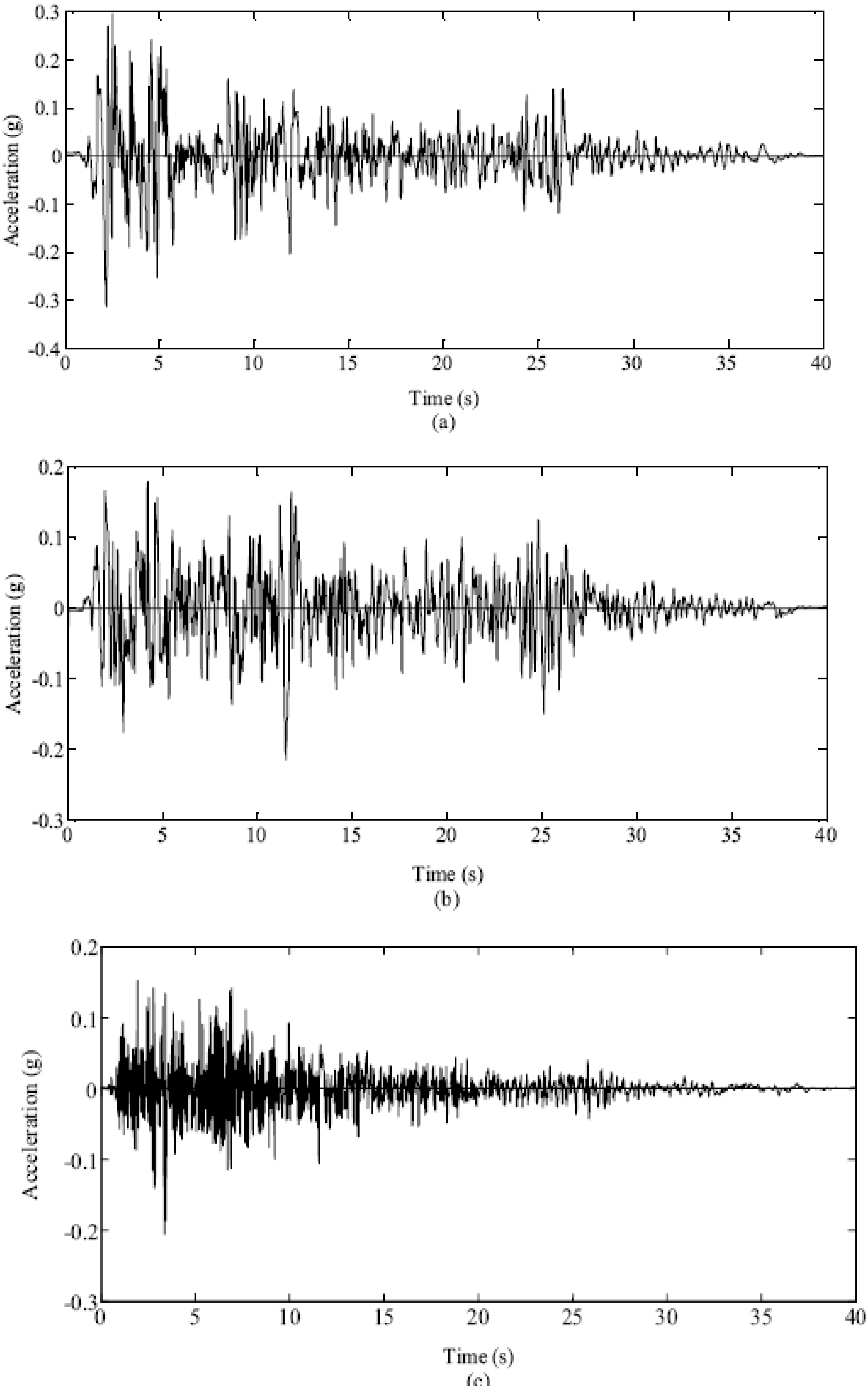


FIG. 2.8 THREE COMPONENTS OF EL CENTRO EARTHQUAKE: (A) MAJOR (HORIZONTAL); (B) MINOR (HORIZONTAL); AND (C) MINOR (VERTICAL)

CHAPTER – 3

PERFORMANCE LEVEL AND CAPACITY CURVE

CHAPTER - 3

3.0 PERFORMANCE LEVEL AND CAPACITY CURVE

3.1 Performance of Structure

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post earthquake serviceability of the building.

A seismic objective specifies the desired seismic performance of the building. Seismic performance is described by designing the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). A performance objective may include consideration of damage states for several levels of ground motion and would then be termed a dual or multiple level performance objectives. Once the building owner selects a performance objective, the engineer can identify the seismic demand to be used in the analysis and the acceptability criteria to be used for evaluation and design of the building's structural and non structural systems.

3.2 FEMA Performance Levels

Structure performance levels are given names and number designations, while non-structural performance levels are given names and letter designations. Building performance levels are a combination of a structural performance level and non-structural performance level and are designated by the applicable number. Typical performance curve for a structure is shown in fig.2

FEMA 356 defines the performance levels of a structure at several stages:

1. Linear limit: The structure response restricted to linear limit;

2. Immediate occupancy structural performance level (SP 1): The structure will be safe to occupy after the earthquake;

3. Damage control structural performance range (SP 2): A damage state between life safety and immediate occupancy performance level;

4. Life safety structural performance level (SP 3): Structure is damaged but retains a margin against onset of partial or total collapse;

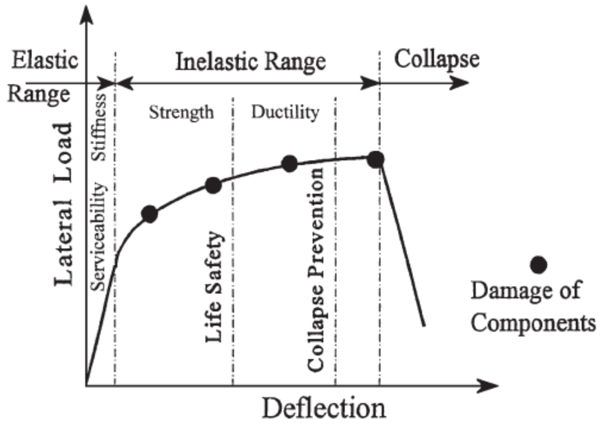


FIG. 3.1 TYPICAL PERFORMANCE CURVE FOR THE STRUCTURE.

5. Limited safety structural performance range (SP 4): A damage state between collapse prevention and life safety performance level;

6. Collapse prevention structural performance level (SP 5): The structure continues to support gravity loads but retains no margin against collapse;

7. Collapsed (SP 6).

3.3 Performance objectives

Performance objectives are statements of acceptable performance of the structure. The performance target can be specified limits on any response parameter such as stresses, strains, displacements, accelerations, etc. It is appealing to express the performance objective in terms of a specific damage state or the probability of failure against a prescribed probability demand level. Various documents promote the same concepts but differ in detail and specify different performance levels. Some of the suggested performance levels can be grouped in equivalent categories as listed in Table 3.1. It is recognized that drift levels associated with specific damage categories may vary considerably with the structural system and construction material. An attempt was made to define drift levels for different structural systems and materials. However, more research is needed, particularly in the development of realistic and quantitative estimates of drift–damage relationships. In addition, design criteria that apply to various parameters may be required by different performance objectives. To implement performance-based design, there is a need for consensus on the number and definition of performance levels, associated damage states, and design criteria.

TABLE 3.1 PERFORMANCE LEVELS, CORRESPONDING DAMAGE STATE AND DRIFT LIMITS

Performance level [3–5]	Damage state	Drift [3]
Fully operational, Immediate occupancy	No damage	<0.2%
Operational, Damage control, Moderate	Repairable	<0.5%
Life safe — Damage state	Irreparable	<1.5%
Near collapse, Limited safety, Hazard reduced	Severe	<2.5%
Collapse		>2.5%

Structural system performance can also be quantified using a reliable damage index such as that based on displacement ductility and hysteretic energy. The performance of the contents of the structure and secondary systems may be quantified using damage indices based on different parameters such as floor acceleration levels. Performance levels are associated with earthquake hazard and design levels. Some of the proposed earthquake hazard levels are listed in Table 3.2.

There are unresolved issues concerning the need to improve our quantitative understanding of site-specific ground motion characteristics, their likely effects on structures, and some aspects of near-field effects.

Simplified nonlinear analysis methods are based on pushover analysis to determine capacity and on design spectrum to represent demand. Some of the recent developments include inelastic spectra, yield point spectra and the N2 method. At each design step, design evaluations may involve response parameters such as the stresses, drift and deformation, structural accelerations, ductility demand ratios, and energy dissipation in terms of demand versus capacity. . The limiting values may be calibrated by analyzing buildings that have experienced measurable damage in seismic events for which strong motion records are available.

The static nonlinear pushover analysis may provide much of the needed information. In the pushover analysis, the structure is loaded with a predetermined or adaptive lateral load pattern and is pushed statically to target displacement at which performance of the structure is evaluated. The target displacements are estimates of global displacement expected due to the design earthquake corresponding to the selected performance level. Recent studies addressed limitations of the procedure and the selection of lateral load distribution including adaptive techniques to account for the contribution of higher modes in long period structures.

TABLE 3.2: PROPOSED EARTHQUAKE HAZARD LEVELS

Earthquake frequency	Return period in years	Probability of Exceedance
Frequent	43	50% in 30 years
Occasional	72	50% in 50 years
Rare	475	10% in 50 years
Very rare	970	5% in 50 years or 10% in 100 years
Extremely rare	2475 2% in 50	2% in 50 years

3.4 Design criteria

A fundamental question in performance-based design is to validate the appropriateness of the selected performance levels, the specific parameters used to define their minimum performance, and the seismic hazard definitions. For the case of three performance levels (serviceability, damage control and life safety or collapse prevention), three corresponding structural characteristics (stiffness, strength and deformation capacity) dominate the performance as illustrated in Fig. 1. If more intermediate performance levels are selected, then it becomes difficult to define which structural characteristics dominate the performance. It can be argued that different performance objectives may impose conflicting demands on strength and stiffness. Much research is needed to associate the displacement or drift limits with the damage states and the stated general performance objectives.

The displacements or drift limits are also functions of the structural system and its ability to deform (ductility). Design criteria may be established on the basis of observation and experimental data of deformation capacity. For example, near the collapse point, the drift limits of structural walls are different from a moment-resisting frame, which suggest that different structural systems will undergo unequal displacements. Other issues related to the damage evaluation are the quantification of the relationship between building restoration time/costs and earthquake hazard level. It is of interest to identify the damage level at which building restoration becomes impractical, which represents the state of irreparable damage.

3.5 Deformation-Controlled Design

The most suitable approach to achieve the objectives of performance-based seismic design with displacement based performance objectives appears to be the deformation-controlled design Approach. It is anticipated that deformation-controlled design will be implemented in future codes, both by enhancing force-based design through verification of deformation targets and by the development of direct deformation-based design procedures .Computer tools are needed to predict the inelastic dynamic response of complex structures. Extensive efforts are believed to be necessary to develop versatile and robust, yet efficient, numerical standard programs to simulate seismic response of three-dimensional structures taking into account various nonlinearities. It is necessary that these tools be design-oriented rather than research-oriented. The general design methodology may have to go beyond the methods that assume a single-degree-of-freedom representation of the structure. This assumption results in severe restrictions on the reliability of the estimated performance. At the risk of sacrificing simplicity, it is important to obtain a good estimate of the local displacements within the structure, take higher-mode effects into consideration, and account for the sequence.

3.6 Non-Structural Performance Levels

Non-structural components addressed in this standard include architectural components such as partitions, exterior cladding, and ceilings; and mechanical and electrical components, including HVAC systems, plumbing, fire suppression systems, and lighting. Occupant contents and furnishings (such as inventory and computers) are included in these tables for some levels but are generally not covered with specific requirements

NPA, OPERATIONAL;

NPB, IMMEDIATE OCCUPANCY

NPC, LIFE SAFETY

HAZARDS REDUCED

NOT CONSIDERED (N-E)

ACCEPTANCE CRITERIA

Three discrete levels structural performances are defined in the guidelines. These are respectively terms the immediate occupancy level, the life safety and the collapse prevention level. The immediate occupancy level is damage state in which the building has experienced only very limited damage. The life safety level is a damage state in which the building has experienced significant damage to its structural components including yielding, cracking, spelling and buckling and perhaps limited fracturing

The collapse prevention level is a state in which extreme damage, short of collapse has occurred to the structure. Building damages to this extent may experience large permanent lateral drifts as well as extensive localized failures of structure strength may also be significantly reduced. Such buildings would not be safe for post earthquake re-occupancy and may not be economically practical to repair and restore the service.

Both the collapse prevention and life safety performance kevels have associated with them, a target margin against collapse. Margins may be thought of as the inherent factor of safety between the loadings that the structure is designed to resist and the loading that would produce failure. When buildings are subjected to strong earthquakes ground shaking, they generally behave in an inelastic behaviour. Although the building code provisions for earthquake design have typically been based on providing minimum specified levels force resisting capacity in a structure, force is not a particularly useful design parameter for predicting the margin of a structure that is responding within the inelastic range of behaviour

CHAPTER – 4

PUSHOVER CURVE DEVELOPMENT OF CAPACITY SPECTRUM METHOD

CHAPTER – 4

4.0 PUSHOVER CURVE DEVELOPMENT OF CAPACITY SPECTRUM METHOD

4.1 Introduction

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the analysis of existing buildings. Elastic analysis methods available include code static lateral force procedures, code dynamic lateral force procedures and elastic procedures using demand capacity ratios. The most basic inelastic analysis method is the complete nonlinear time history analysis, which at this time is considered overly complex and impractical for general use.

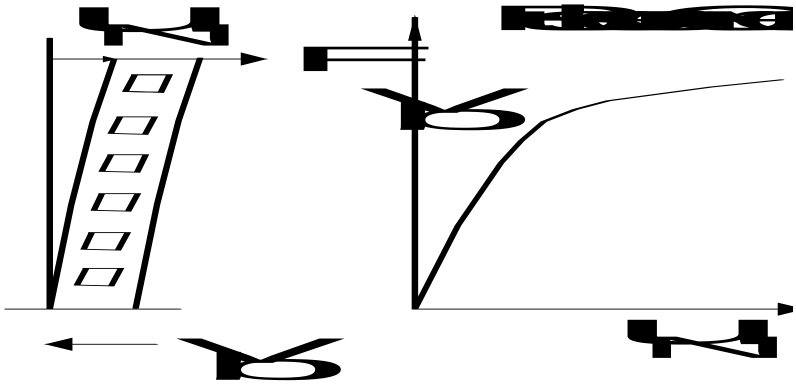
Available simplified nonlinear analysis methods, referred to as nonlinear static analysis procedures, include the capacity spectrum method (CSM) that uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement; the displacement coefficient method (e.g., FEMA-273 (ATC 1996a)that uses pushover analysis and a modified version of the equal displacement approximation to estimate maximum displacement; and the secant method that uses a substitute structure and secant stiffness.

This chapter emphasizes the use of nonlinear static procedures in general and focuses on the capacity spectrum method. This method has not been developed in detail previously. It provides a particularly rigorous treatment of the reduction of seismic demand for increasing displacement.

A major challenge to performance-based seismic design and engineering of buildings is to develop simple, yet effective, methods for designing, analyzing and checking the design of structures so that they reliably meet the selected performance objectives. Needed are analysis procedures that are capable of predicting the demands – forces and deformations – imposed by earthquakes on structures more realistically than has been done in building codes. In response to this need, simplified, nonlinear analysis procedures are there to determine the displacement demand imposed on a building expected to deform in elastically.

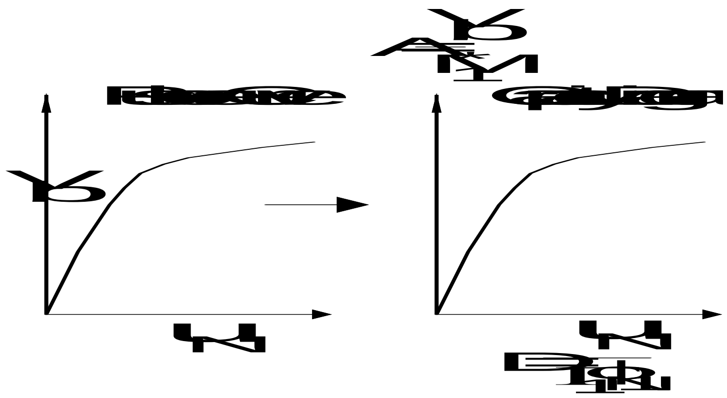
4.2 Capacity Spectrum Method

The Nonlinear Static Procedure is based on the capacity spectrum method originally developed by Freeman et al. (1975) and Freeman (1978). It consists of the following steps:

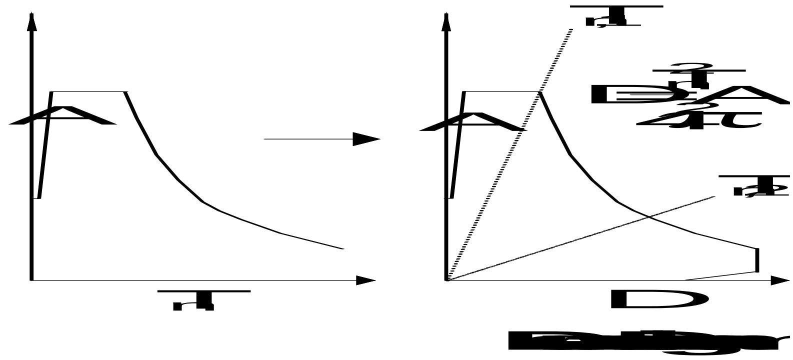


(a) Development of pushover curve

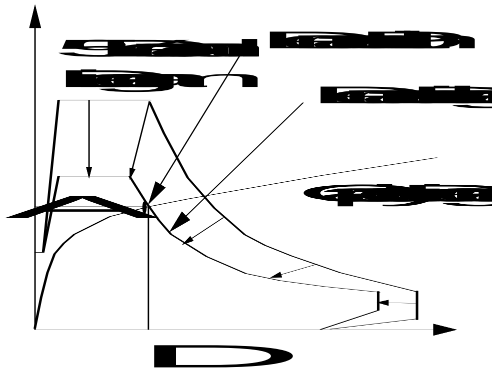
1. Develop the relationship between base shear, V_b and roof (Nth floor) displacement, u_N (Fig. 4a), commonly known as the pushover curve.
2. Convert the pushover curve to a capacity diagram, (Fig. 4b), where
3. Convert the elastic response (or design) spectrum from the standard pseudo-acceleration, A versus natural period, T_n format to the $A-D$ format, where D is the deformation spectrum ordinate (Fig. 4c).



(b) Conversion of pushover curve to capacity diagram



(c) Conversion of elastic response spectrum from standard format to A-D format



(d) determination of displacement demand

FIG.4.1 CAPACITY SPECTRUM METHOD

4. Plot the demand diagram and capacity diagram together and determine the displacement demand (Fig. 4d). Involved in this step are dynamic analyses of a sequence of equivalent linear systems with successively updated values of the natural vibration period, T_{eq} and equivalent viscous damping, $\hat{\zeta}_{eq}$ (to be defined later).

Convert the displacement demand determined in step 4 to global (roof) displacement and individual component deformation and compare them to the limiting values for the specified performance goals.

4.3 Procedure to Determine Capacity Curve

Structure capacity is represented by a pushover curve. The most convenient way to plot the force-displacement curve is by tracking the base shear and the roof displacement.

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. This is generally valid for the buildings with fundamental periods of the vibration up to about one second. For more flexible buildings with a fundamental period of vibration greater than one second, the analyst should consider addressing higher mode effects in the analysis. The following procedure can be used to construct a pushover curve:

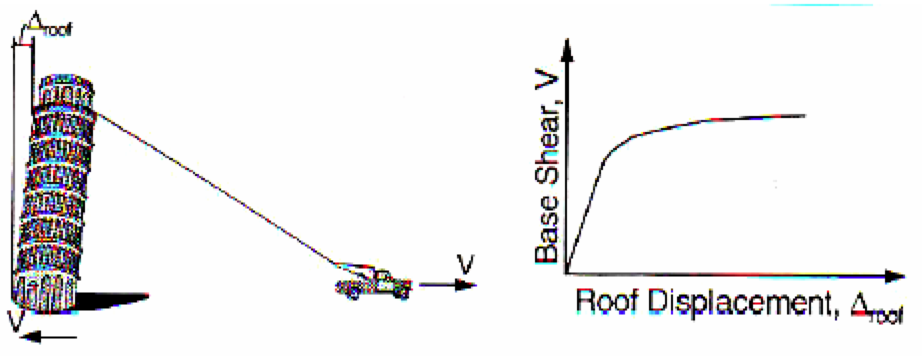


FIG. 4.2 BASE SHEAR VS ROOF DISPLACEMENT GRAPH

1. Create a mathematical model of the structure according to ATC-40.
2. Classify each element in the model as either primary or secondary.
3. Apply lateral storey forces to the structure in proportion to the product of the mass and fundamental mode shape. This analysis should also include the gravity loads.
4. Calculate the member forces for the required combinations of vertical and lateral load.
5. Adjust the lateral load so that some element (or group of elements) is stressed to within 10 percent of its member strength.
6. Record the base shear and the roof displacement. It is also important o record the member forces and rotations because they will be needed for the performance check.
7. Revise the model using zero (or very small) stiffness for the yielding elements.
8. Apply a new increment of lateral load to the revised structure such that another element (or group of elements) yields.

9. Add the increment of the lateral load and the corresponding increment of roof displacement to the previous totals to give the accumulated values of base shear and roof displacement.
10. Repeat 7, 8 and 9 until the structure reaches an ultimate limit, such as: instability from P-Δ effects; distortions considerably beyond the desired performance level; an element (or group of elements) reaching a lateral deformation level at which significant strength degradation begins or an element (or group of elements) reaching a lateral deformation level at which loss of gravity load carrying capacity occurs. Above Fig. shows the typical capacity curve.
11. Explicitly model global strength degradation. If the incremental loading was stopped in step 10 as a result of reaching a lateral deformation level at which all or a significant portion of an element's (or group of elements) load can no longer be resisted, that is, its strength has significantly degraded, then the stiffness of that element(s) is reduced, or eliminated. A new capacity curve is then created, starting with step 3 of this procedure. Create as many additional pushover curves as necessary to adequately define overall loss of the strength.

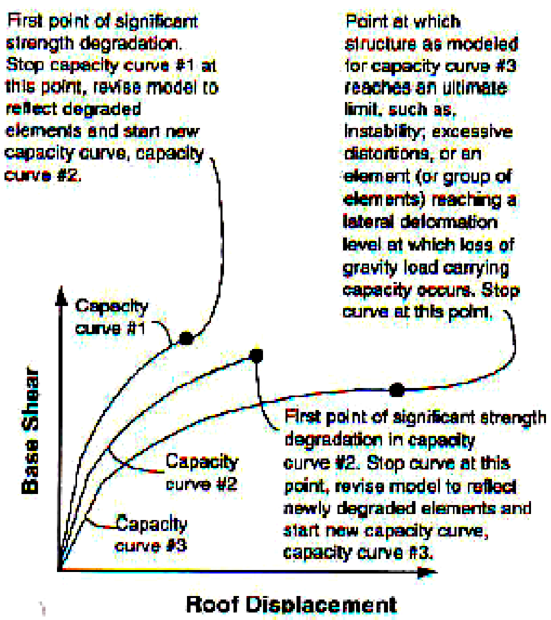


FIG 4.3 MULTIPLE CAPACITY CURVES REQUIRED TO MODEL STRENGTH DEGRADATION.

Fig. 4.3 illustrates the process, for an example where three different capacity curves are required.

Plot the final capacity curve to initially follow the first curve, then transition to the second curve at the displacement corresponding to the initial strength degradation, and so on. This curve will have a saw tooth shape as shown in Fig. 4.4.

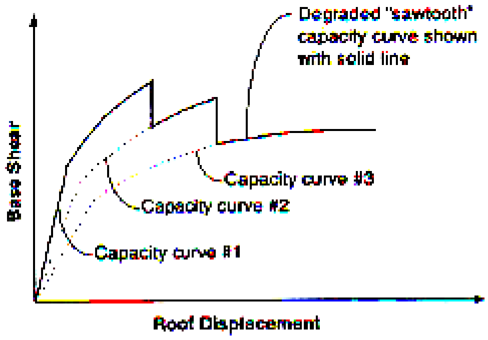


FIG. 4.4 CAPACITY CURVE WITH GLOBAL STRENGTH DEGRADATION MODEL.

4.4 Procedure to Determine Demand

Two methodologies are presented in this session.

CAPACITY SPECTRUM METHOD
DISPLACEMENT COEFFICIENT METHOD

4.4.1 Capacity spectrum method

It is based on finding a point on the capacity spectrum that also lies on the appropriate demand response spectrum, reduced for non linear effects, and is most consistent in terms of graphical representation and terminology with the balance of this document.
The demand displacement in the capacity spectrum method occurs at a point on the capacity spectrum called the performance point. This performance point represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion.
The location of the performance point must satisfy two relationships: 1) the point must lie on the capacity spectrum curve in order to represent the structure at a given displacement, and 2) the point must lie on spectral demand curve, reduced from the elastic, 5 percent damped design spectrum, that represent the non-linear demand at the same structural displacement, for this methodology, spectral reduction factors are

given in terms of effective damping. An approximate damping is calculated based on the shape of the capacity curve, the estimated displacement demand, and the resulting hysteresis loop. Probable imperfections in real building hysteresis loops, including degradation and duration effects, are accounted for by reductions in theoretically calculated equivalent viscous damping values.

In general case, determination of the performance point requires a trial and error search for satisfaction of the two criterions specified above. However, here three different procedures have been included which standardize and simplify this iterative process. These alternate procedures are all based on the same concepts and mathematical relationship but vary in their dependence on analytical versus graphical techniques.

Procedure A: This is the most direct application of the concepts and relationship described below. This procedure is truly iterative, but is formula-based and easily can be programmed into spread sheet. It is more an analytical method than graphical method. It may be the best method for beginners because it is the most direct application of the methodology, and consequently is the easiest procedure to understand.

4.4.1.1 Conceptual development of the capacity spectrum method

Conversion of the capacity curve to the capacity spectrum

To use the capacity spectrum method it is necessary to convert the capacity curve, which is in terms of base shear and roof displacement to what is called capacity spectrum, which is the representation of the capacity curve in Acceleration-Displacement Response Spectra (ADRS) format (i.e. Sa versus Sd). The required equations to make the transformations are:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_{i1}) / g}{\sum_{i=1}^{N} (w_i \phi^2{}_{i1}) / g} \right] \qquad \text{.} \qquad \text{.....................} \quad (1)$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i1}) / g \right]^2}{\left[\sum_{i=1}^{N} w_i / g \right] \left[\sum_{i=1}^{N} (w_i \phi^2{}_{i1}) / g \right]} \qquad \text{............} \qquad (2)$$

$$S_a = \frac{V/W}{\alpha_1} \qquad \text{..................} \quad (3)$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}} \qquad \text{....................} \quad (4)$$

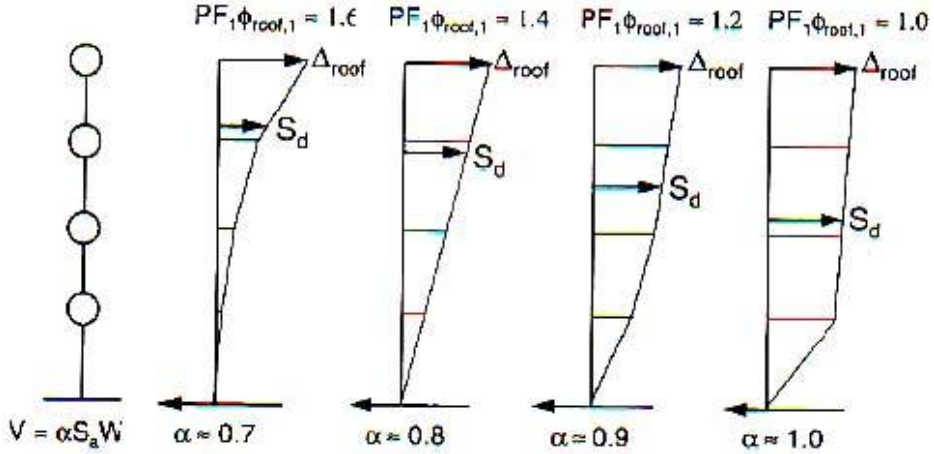


FIG 4.5 EXAMPLE MODAL PARTICIPATION FACTORS AND MODAL MASS COEFFICIENTS

Where:

PF_1 - modal participation factor for the first mode.

α_1 - Modal mass coefficient for the first natural mode.

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 superstructure.

V - Base shear.

W - Building dead weight plus likely live loads

Δ_{roof} - roof displacement (V and the associated Δ_{roof} make up points on the capacity curve).

S_a - spectral acceleration.

S_d - spectral displacement (S_a and the associated S_d make up points on the capacity spectrum).

The general process for converting the capacity spectrum, i.e. converting the capacity curve into the ADRS format, is to first calculate modal participation factor PF_1 and the modal mass coefficient α1 using the equation (1) and (2). Then for each point on the capacity curve, V, Δ_{roof} , calculate the associated point S_a, S_d on the capacity spectrum using equation (3) and (4).

Most engineers are familiar with the traditional Sa versus Sd (ADRS) representation. Fig. 4.6 shows the same spectrum in each format. In the ADRS format, lines radiating from the origin have constant period. For any point on ADRS spectrum, the period, T, can be computed using the relationship T= 2π(S_d / S_a) 1/2. Similarly, for any point on the traditional spectrum, the spectral displacement, S_d, can be computed using the relationship S_d = S_a T_2 /4π2. These two relationships are the same formula arranged in different way.

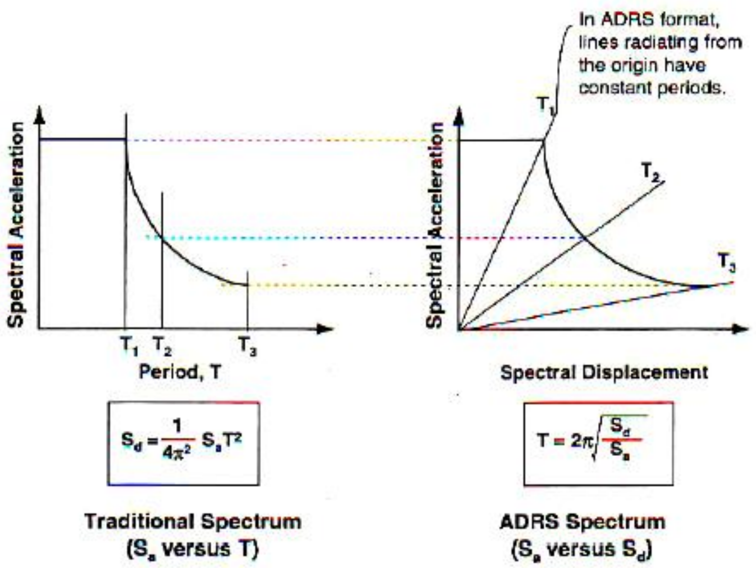


FIG 4.6 (A) RESPONSE SPECTRA IN TRADITIONAL AND ADRS FORMATS

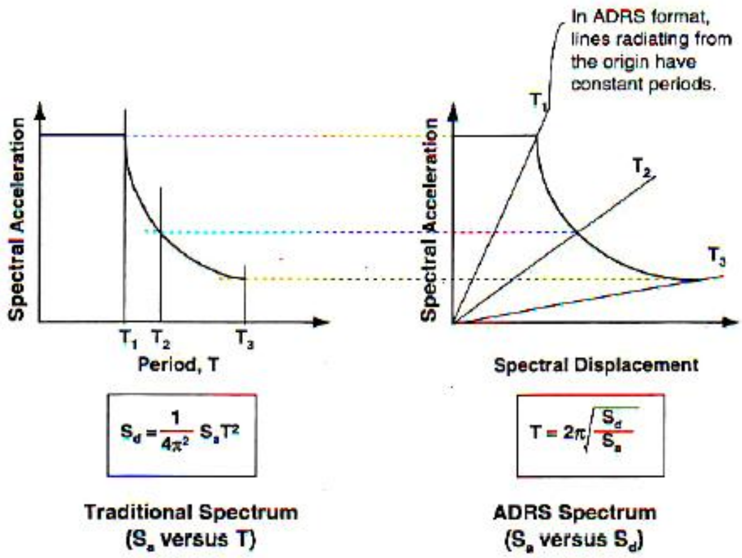


FIG 4.6(B) CAPACITY SPECTRUM SUPER IMPOSED OVER RESPONSE SPECTRA IN TRADITIONAL AND ADRS FORMATS

Fig. 4.6(b) shows the same capacity spectrum superimposed on each of the response spectra plots shown in Fig. 4 Following along the capacity spectrum, the period is constant, at T_1, up until point A. when point B is reached, the period is T_2

. This indicates that a structure undergoes inelastic displacement, the period lengthens. The lengthening period is most apparent on the traditional spectrum plot, but it is also clear on the ADRS plot, remembering that lines of constant period radiate from the origin.

Bilinear representation of the Capacity Spectrum

A bilinear representation of the capacity spectrum is needed to estimate the effective damping and appropriate reduction of spectral demand. Construction of the bilinear representation requires definition of the points a_{pi}, d_{pi}, this point is a trial performance point which is estimated by the engineer to develop a reduced demand response spectrum. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated a_{pi}, d_{pi} point. The first estimate of point api, dpi is designated ap_1, dp_1, the second ap_1, dp_2, and so on. Guidance on a first estimate of point ap_1, dp_1 is given in the step by step process for each of the three procedures. Oftentimes, the equal displacement approximation can be used as an estimate of ap_1, dp_1

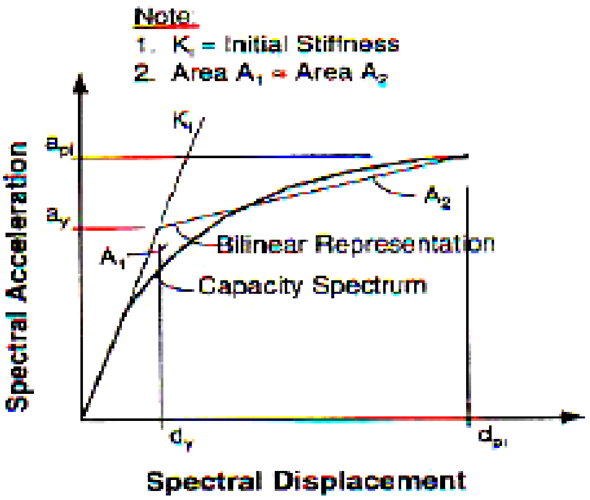


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FIG. 4.7 BILINEAR REPRESENTATION OF CAPACITY SPECTRUM FOR CAPACITY SPECTRUM METHOD.

To construct the bilinear representation draw one line up from the origin at the initial stiffness of the building using element stiffnesses. Draw a second line back from the trial performance point, a_{pi}, d_{pi}. Slope the second line such that when it intersects the first line, at point a_y, d_y, the area designated A_1 in the Fig. is approximately equal to the area designated A_2, the intent of setting area A_1 equal to A_2 is to have equal area under the capacity spectrum and its bilinear representation, that is to have equal energy associated with each curve.

Estimation of damping and reduction of 5 percent damped response spectrum

The damping that occurs when earthquake ground motion drives a structure into the inelastic range can be viewed as a combination of viscous damping that is inherent in the structure and hysteretic damping. Hysteretic damping is related to the area inside the loops that are formed when the earthquake force (base shear) is plotted against the structure displacement. Hysteretic damping can be represented as equivalent viscous damping using equations that are available in the literature. The equivalent viscous damping, βeq, associated with maximum displacement of dpi, can be estimated from the following equation:

$\beta eq = \beta o + 0.05$ .... (5a)

Where,

βo = hysteretic damping represented as equivalent viscous damping

0.05 = 5% viscous damping inherent in the structure (assumed to be constant)

The term βo can be calculated as (Chopra 1995):

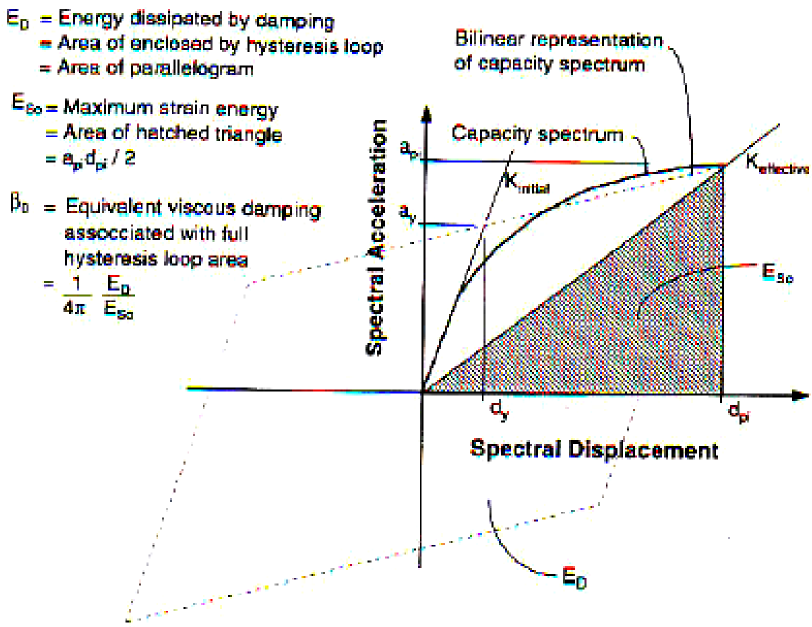


FIG. 4.8 DERIVATION OF DAMPING FOR SPECTRAL REDUCTION.

$$\beta_0 = \frac{E_D}{4\pi E_{so}} \qquad \text{..................... (5b)}$$

Where,

E_D - Energy dissipated by damping

E_{so} -Maximum strain energy

The physical significance of the terms E_D and E_{so} in equation 5a is illustrated in Fig. 4.8 E_D is the energy dissipated by the structure in a single cycle of motion, that is, the area enclosed by a single hysteresis loop. E_{so} is the maximum strain energy associated with that cycle of motion that is the area of the hatched triangle.

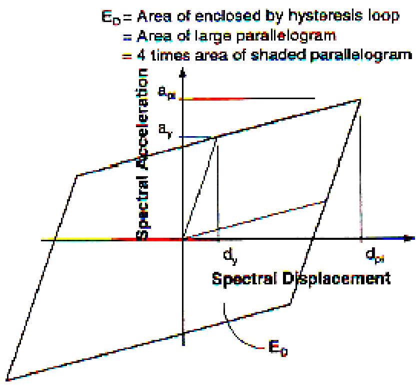


FIG. 4.9 DERIVATION OF ENERGY DISSIPATION BY DAMPING, E_D

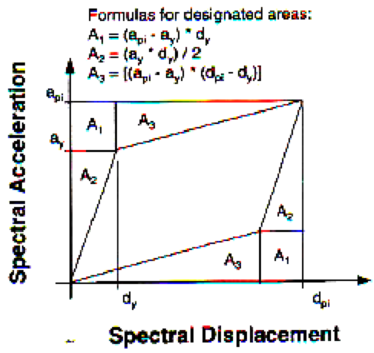


FIG. 4.10 DERIVATION OF ENERGY DISSIPATION BY DAMPING, E_D

E_D = 4(shaded area in Fig. 4.9 or 4.10)

$= 4(a_{pi}d_{pi} - 2A_1 - 2A_2 - 2A_3)$

$=4 [a_{pi}d_{pi} - a_y d_y - (dpi - d_y)(a_{pi} - ay) - 2dy(a_{pi} - a_y)]$

$= 4(a_y dpi - d_y a_{pi})$

Referring Fig. 4.8, the term E_{so} can be derived as

$E_{so} = a_{pi}d_{pi}/2$

Thus, βo can be written as:

$$\beta_0 = \frac{1}{4\pi}\frac{4(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}/2} = \frac{2}{\pi}\frac{a_y d_{pi} - d_y a_{pi}}{a_{pi} d_{pi}} = \frac{0.637(a_y d_{pi} - d_y a_{pi})}{a_{pi} - d_{pi}}$$

$$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}\% \qquad \text{....... (6)}$$

$$\beta_{eq} = \beta_0 + 5 = \frac{63.7(a_y y_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5 \qquad \text{.......... (7)}$$

The equivalent viscous damping values obtained from equation (7) can be used to estimate spectral reduction factors using relationship developed by Newmark and Hall. As shown in Fig. 11, spectral reduction factors are used to decrease the elastic (5% damped) response spectrum to reduce response spectrum with damping greater than 5% of critical damping. For damping values less than about 25%, spectral reduction factors calculated using the eqβ from equation (7) and Newmark and Hall equations are consistent with similar factors contained in base isolation code and in the FEMA guidelines.

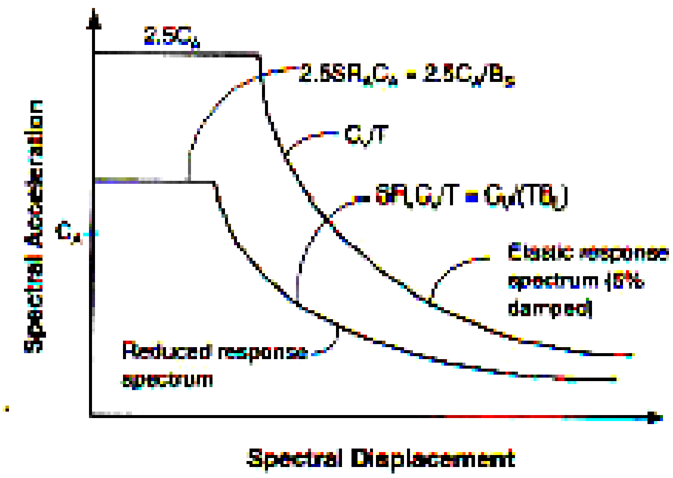


FIG. 4.11 REDUCED RESPONSE SPECTRUM

4.4.1.2 Development of demand spectrum

The 5 percent response spectrum can be developed as shown in following Fig.;

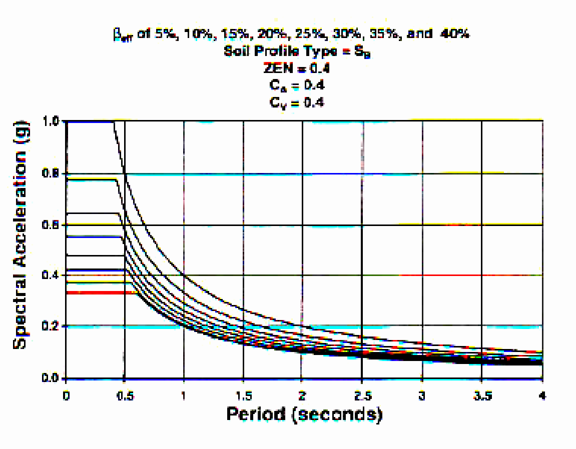


FIG. 4 12 FAMILY OF DEMAND SPECTRA IN TRADITIONAL IN SA VS T FORMAT IN ADRS FORMAT

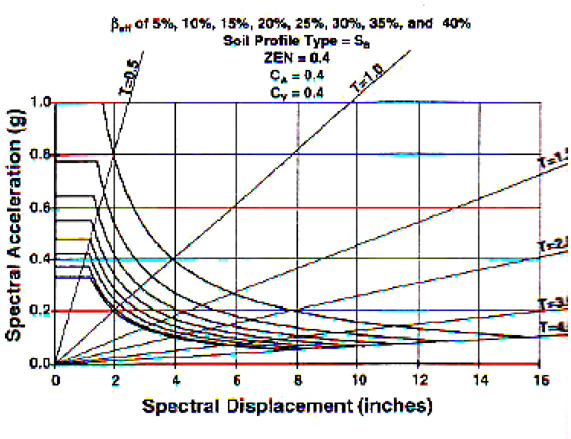


FIG. 4.13 FAMILY OF DEMAND SPECTRA

4.4.1.3 Intersection of capacity spectrum and demand spectrum

When the displacement at the intersection of the demand spectrum and the capacity spectrum, di is within 5% (0.95dpi <di < 1.05dpi) of the displacement of the trial performance point, dpi becomes the performance point. If the intersection of the demand spectrum and the capacity spectrum is not within the acceptable point is selected and tolerance, then a new api, dpi point is selected and the process is repeated. Fig. 4.14 illustrates the concept. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

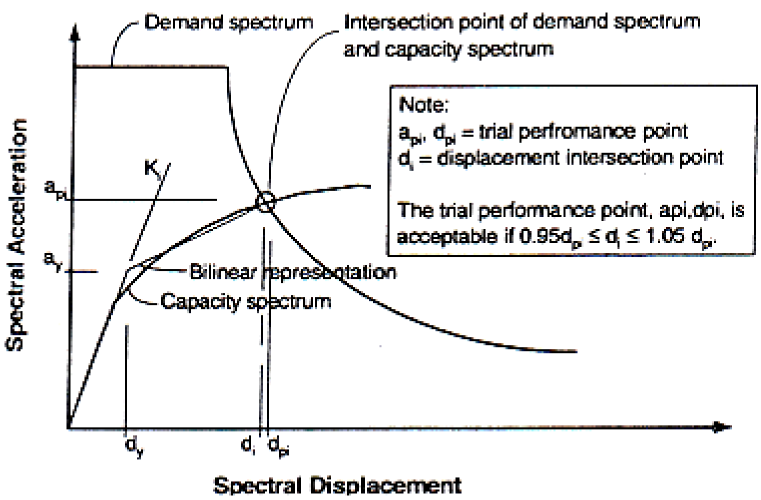


FIG. 4.14 INTERSECTION POINT OF DEMAND AND CAPACITY SPECTRUM WITHIN ACCEPTABLE TOLERANCE

When a capacity spectrum is a saw tooth curve, that is, the final composite capacity spectrum is constructed from several different capacity spectra which accounts for strength degradation of elements, special care must be taken in determining the performance point. Bilinear representation of the capacity spectrum, that is used to determine the reduction factors for the 5% damped spectrum, is constructed for a single capacity spectrum where the intersection point occurs. Fig. 15 illustrate the concept for a saw tooth capacity spectrum.

4.4.1.4 Calculating the performance point using procedure A

In this procedure, iteration is done by hand or by spreadsheet methods to converge on the performance point. This procedure is the most direct application of the principles described above. The following steps are involved:

1. Develop the 5 percent damped (elastic) response spectrum appropriate for the site.
2. Transform the capacity curve into a capacity spectrum. Plot the capacity curve on the same chart as the 5% damped response spectra as shown in Fig.4. 16.

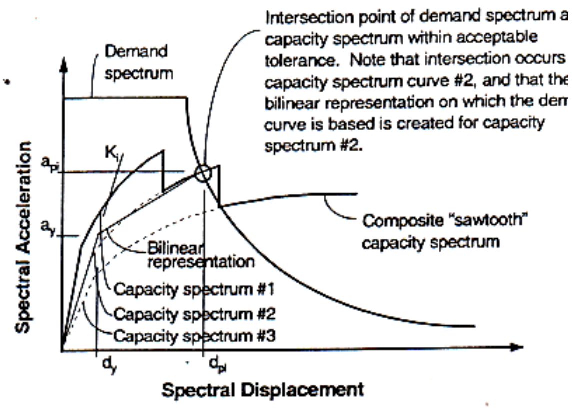


FIG. 4.15 INTERSECTION POINT OF DEMAND SPECTRUM AND SAW TOOTH CAPACITY SPECTRUM

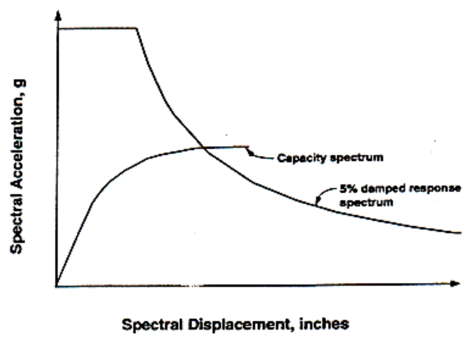


FIG. 4.16 CAPACITY SPECTRUM PROCEDURE A AFTER STEP 2.

Select a trial performance point, a_{pi}, d_{pi} as shown in Fig. 4.17.

3. Develop a bilinear representation of the capacity spectrum.

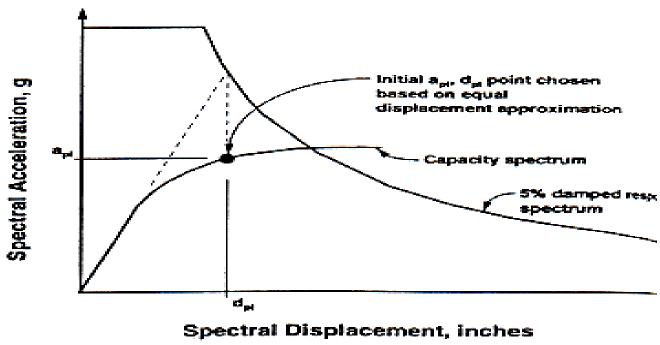


FIG. 4.17 CAPACITY SPECTRUM PROCEDURE A AFTER STEP 3

4. Calculate the spectral reduction factor as shown below :

$$SR_A = \frac{1}{B_S} \approx \frac{3.21 - 0.68\ln(\beta_{eff})}{2.12}$$

$$= \frac{3.21 - 0.68\ln\left[\frac{63.7k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}\right]}{2.12} + 5$$

$$SR_v = \frac{1}{B_L} \approx \frac{2.31 - 0.41\ln(\beta_{eff})}{1.65}$$

$$= \frac{2.31 - 0.41\ln\left[\frac{63.7k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{1.65}$$

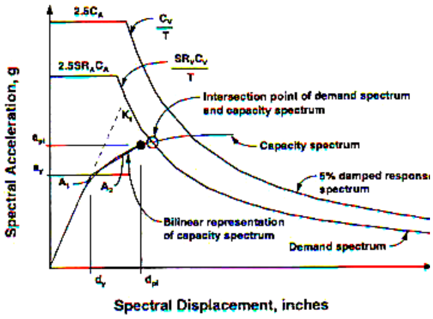
And these values should be greater than given in following table:

TABLE 4.3

Structural behaviour type	SR_A	SR_V
Type A	0.33	0.50
Type B	0.44	0.56
Type C	0.56	0.67

Develop the demand spectrum. Draw the demand spectrum on the same plot as the capacity spectrum.

Refer to Fig. 18. Determine if the demand spectrum intersects the capacity spectrum at the point, a_{pi}, d_{pi} Or if the displacement at which the demand spectrum intersects the capacity spectrum, di, is within acceptable tolerance of d_{pi}. The acceptable tolerance is illustrated in Fig. 14.



1.

FIG. 4.18 CAPACITY SPECTRUM PROCEDURE A AFTER STEP 6.

If the demand spectrum does not intersect the capacity spectrum within acceptable tolerance. Then select a new a_{pi}, d_{pi} point and return to step 4.

If the demand spectrum intersects the capacity spectrum within acceptable tolerance, then the trial performance point. a_{pi}, d_{pi}, is the performance point, ap, dp, and the displacement, dp, represents the maximum structural displacement expected for the demand earthquake.

4.4.1.5 Calculating the performance point using procedure B

This procedure makes a simplifying assumption that is not made in the other two procedures. It assumes that not the initial slope of the bilinear representation of the capacity curve remains constant, but also the point a_y, d_y and the post yield slope remains constant. This simplifying assumption allows a direct solution without drawing multiple curves because it forces the effective damping, βeff, to depend only on dpi. The following steps are involved:

Develop the 5% damped response spectrum appropriate for the site.

1. Draw the 5% damped response and spectrum and draw a family of reduced spectra on the same chart. It is convenient if the spectra plotted correspond to effective damping values (βeff) ranging from 5 % to the maximum value allowed for the building's structural behavior type. maximum βeff for type A construction is 40%, type B construction 29% and type C is 20%.

Fig. 4.19 shows an example family of demand spectra.

TABLE 4.1 STRUCTURAL BEHAVIOUR TYPE

Shaking Duration	Essentially new buildings	Average new buildings	Poor existing buildings
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

a. Site with a near source distance, N > 1.2, may be assumed to have short duration ground shaking and N < 1.2 assumed to have long duration ground shaking .
b. Building whose primary elements make up an essentially new lateral system and little strength or stiffness is contributed by noncomplying elements.
c. Building whose primary elements are combinations of existing and new elements, or better than average existing systems.
d. Building whose primary elements make up complying lateral force systems with poor or unreliable hysteretic behavior.

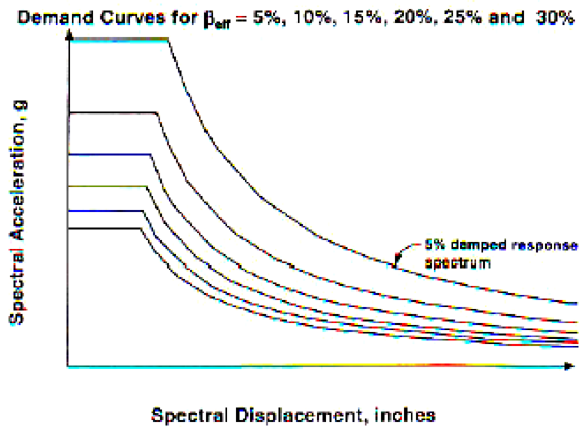


FIG. 4.19 CAPACITY SPECTRA PROCEDURE "B" AFTER STEP 2

2. Transform the capacity curve into a capacity spectrum as described earlier using equations 1, 2, 3 and 4. Plot the capacity spectrum on the same chart as the family of demand spectra, as shown in Fig. 4.20.

Develop a bilinear representation of the capacity spectrum is illustrated in figur4.e 21. The initial slope of the bilinear curve is equal to the initial stiffness of the building. The post yield segment of the bilinear representation should be run through the capacity spectrum at a displacement equal to the spectral displacement of the 5% damped spectrum at the initial pre-yield stiffness (equal displacement rule), point a*, d*. The post yield segment should be rotated about this point to balance the areas A_1 and A_2 as shown in Fig. 4.21.

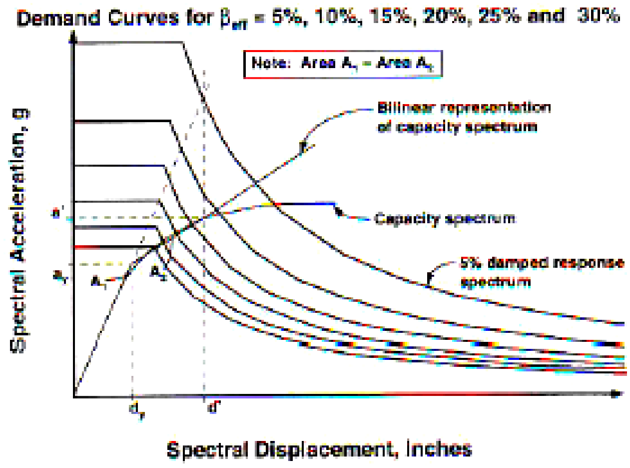


FIG. 4.20 CAPACITY SPECTRUM PROCEDURE "B" AFTER STEP 4

3. Calculate the effective damping for various displacement near the point a*, d*. The slope of the post yield segment of the bilinear representation of the capacity spectrum is given by:

Post Yield= $= \dfrac{a^* - a_y}{d^* - d_y}$ .....…………….….. (11)

For any point api, dpi, on the post yield segment of the bilinear representation, the slope is given by:

Post yield slope= $= \dfrac{a_{pi} - a_y}{d_{pi} - d_y}$ …………………….. (12)

Since the slope is constant, equation 11 and 12 can be equated:

$$\frac{a^* - a_y}{d^* - d_y} = \frac{a_{pi} - a_y}{d_{pi} - d_y}$$ .... (13)

Solving equation 13 for api we get;

$$a_{pi} = \frac{(a^* - a_y)(d_{pi} - d_y)}{d^* - d_y} + a_y$$

This value can be substituted for api in equation 7 to obtain the equation for βeff that in terms of only the unknown, dpi.

$$\beta_{eff} = \frac{63.7k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5$$

Where, k = the factor which is the measure of the extent to which the actual building hysteresis is well represented by the parallelogram of Fig. 8, either initially or after degradation. The k factor depends on the structural behaviour of the building, which in terns depends on the quality of the seismic resistance system and the duration of the ground shaking. For structure type A, B, C k values are 1.0, 2/3 and 1/3 respectively.

4. For each dpi value considered in step 5, plot the resulting dpi, βeff point in the same chart as the family of demand spectra and the capacity spectrum. Fig. 22 shows five of these points.
5. Connect the points created in step 6, to form a line. The intersection of this line with the capacity spectrum defines the performance point. This procedure provides the same results as the other procedures if the performance point is at point a*, d*. If the performance point is found to be distant from point a*, d*, then the engineer may want to verify the results using procedure A or C.

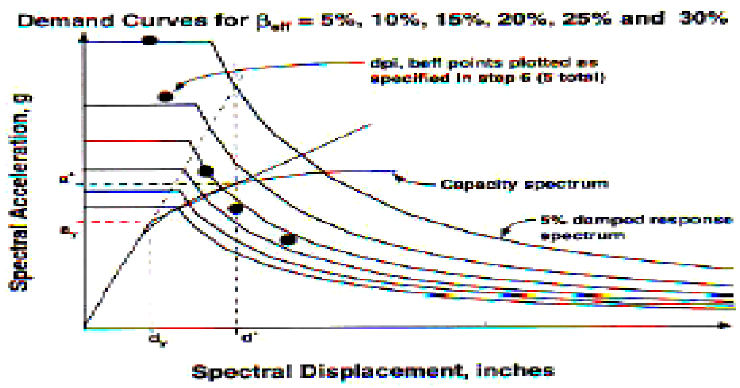


FIG. 4.21 CAPACITY SPECTRUM PROCEDURE "B" AFTER STEP 6

4.4.1.6 Calculating the performance point using procedure C

This procedure has been developed to provide a graphical solution using hand methods. It has been found to often be reasonably close to the performance point on the first try. The following steps are involved:

1. Develop the 5% damped response spectrum appropriate for the site.

2. Draw the 5% damped response spectrum and draw a family of reduced spectra on the same chart, as illustrated in Fig. 4.22. It is convenient if the spectra plotted correspond to effective damping values (βeff) ranging from 5% to the maximum value allowed for the building's structural behavior type.

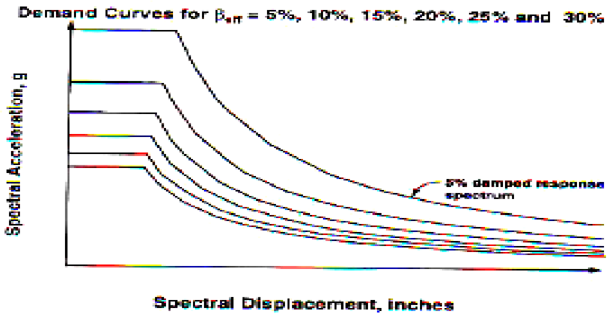


FIG. 4.22 CAPACITY SPECTRA PROCEDURE "C" AFTER STEP 2

3. Transform the capacity curve into a capacity spectrum, and plot it on the same chart as the family of the demand spectra, as illustrated in Fig. 4.23.

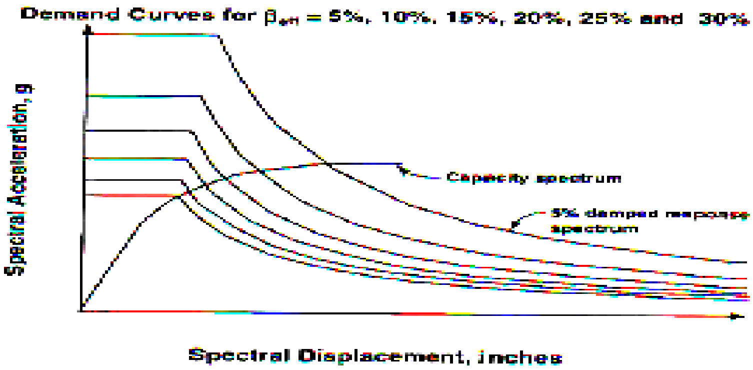


FIG. 4.23 CAPACITY SPECTRA PROCEDURE "C" AFTER STEP 3

Develop a bilinear representation of the capacity spectrum. Select the initial point ape, dpi at the furthest point out on the capacity spectrum or at the intersection with the 5% damped spectrum, whichever is less. A displacement slightly larger than that calculated using the equal displacement approximation (say 1.5 times larger) may also be reasonable estimate for the initial d_{pi}, see Fig. 4.24 for as an illustration of this step.

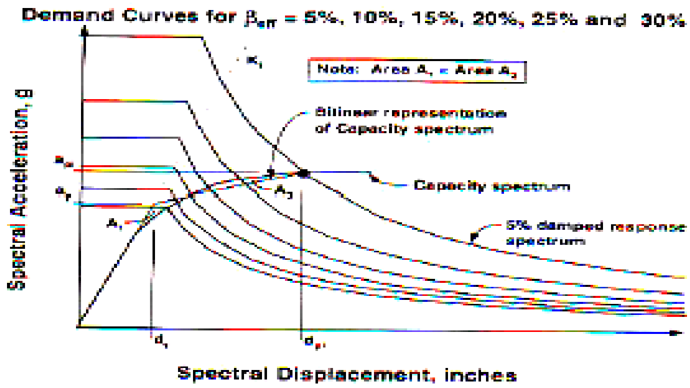


FIG. 4.24 CAPACITY SPECTRA PROCEDURE “C” AFTER STEP 4

and $[(a_{pi}/a_y)-1]/[(d_{pi}/d_y)-1]$ Determine the ratio of d_{pi}/d_y

Note that the second term is the ratio of the post yield stiffness to the initial stiffness.

4. Based on the ratios obtained in step 5, depending on the building’s structural type and find the effective damping value βeff.

Refer to Fig. 4.25. Extend the initial stiffness line, labeled line 1 in the Fig., up to intersect the 5% damped curve. Also draw a line, labeled 2 in the Fig., from origin to point a_{pi}, d_{pi}.

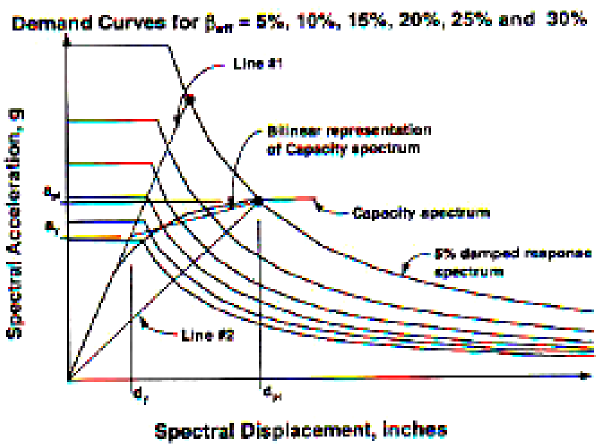


FIG. 4.25 CAPACITY SPECTRUM PROCEDURE “C” AFTER STEP 7

5. Refer to Fig. 4.26. Draw a line, labeled 3 in the Fig., from the intersection point of line 1 and the 5% damped response spectrum to the intersection point of line 2 and the reduced spectrum correspond to the βeff of approximately 24%.

Refer to Fig. 4.27. The point where line 3 intersects the capacity spectrum is taken as the estimated performance point ap_2, dp_2 point.

If the displacement is within + 5% of displacement dp1, then the point ap_2, dp_2 is the performance point or in more general terms, if displacement dp (i+1) is within + 5% of displacement dpi, then the point $a_{p\ (i+1)}$, $d_{p\ (i+1)}$ is the performance point, if the displacements are not within the specified tolerance, then proceed to step 11.

Repeat the procedure starting at step 4, incrementing i by 1. Thus in the second iteration, line 2 is drawn from the origin to point ap_2, dp_2

.

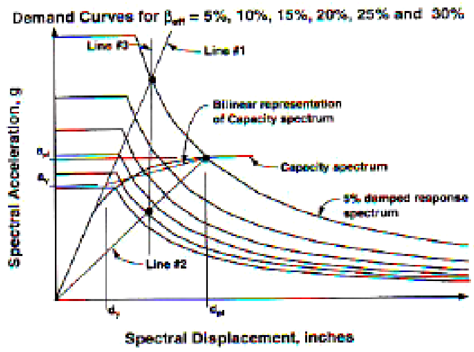


FIG. 4.26 CAPACITY SPECTRUM PROCEDURE "C" AFTER STEP 8

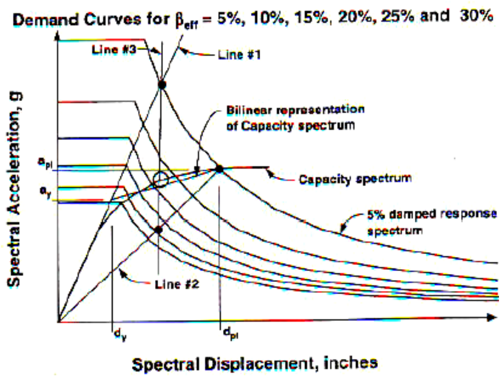


FIG. 4.27 CAPACITY SPECTRUM PROCEDURE "C" AFTER STEP 9

4.4.2 Calculating Demand Displacement Using the Displacement Coefficient

The displacement coefficient method provides a direct numerical process for calculating the displacement demand. It does not require converting the capacity curve to spectral coordinates.

1. Construct a bilinear representation of the capacity curve as follows (refer to Fig. 8-43):

a. Draw the post-elastic stiffness, Ks, by judgment to represent an average stiffness in the range in which the structure strength has leveled off.
b. Draw the effective elastic stiffness, Ks, by constructing a secant line passing through the point on the capacity curve corresponding to a base shear of o.6Vy, where Vy is defined by the intersection of the Ke, and Kc, lines.

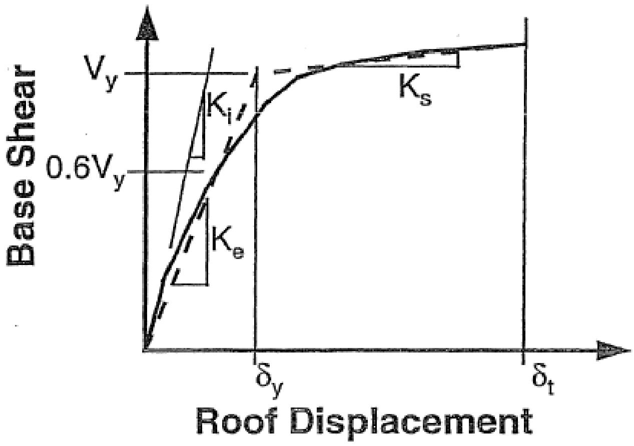


FIG. 4.28 BILINEAR REPRESENTATION OF CAPACITY CURVE FOR DISPLACEMENT COEFFICIENT METHOD

TABLE 4.2 VALUES FOR MODIFICATION FACTOR C_0

NO.OF STORIES	MODIFICATION FACTOR
1	1.1
2	1.2
3	1.3
5	1.4
10+	1.5

Calculate the effective fundamental period (Te) as:

$$Te = Ti\sqrt{Ki}/Ke$$

Where:

Tr = elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis.

Ki = elastic lateral stiffness of the building in the direction under consideration (refer to Fig. 8-43).

Ke = effective lateral stiffness of the building in the direction under consideration (refer to Fig. 8-43).

2. Calculate the target displacement, (δt) as:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^{\,2}}{4\pi^2}$$

Where:

Te =effective fundamental period as calculated in step 2 above.

C_0 = modification factor to relate spectral displacement and likely building roof displacement; estimated for C_0 can be calculating using either:

1. The first modal participation factor at the roof level.
2. The modal participation factor at the roof level calculated using the shape vector corresponding to the deflected shape of the building at the target displacement.
3. The appropriate value from table 2.
4. C_1= modification factor to relate expected maximum elastic displacements to displacements calculated for linear elastic response.

= 1.0 for Te >T0

= [1.0+ (R-1) T0/Te]/R for Te < T0 C_1need not exceed 2.0 for Te < 0.1 second

T0 = a characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R = ratio of inelastic strength demand to calculated yield strength coefficient calculated as follows:

$$R = \frac{S_a / g}{V_y / W} \cdot \frac{1}{C_0}$$

C_2 = modification factor to represent the effect of hysteresis shape on the maximum displacement response. Value of C_2 for different framing systems and performance levels are listed in table 3. Linear interpolation shall be used to estimate values of Te.

C_3 = modification factor to represent increased displacements due to second-order effects. For buildings with positive post-yield stiffness, C3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, C3 shall be calculated as:

$$C_3 = 1 + \frac{|\alpha|(R-1)^{3/2}}{T_e}$$

Where R and Te are defined above and α is the ratio of post-yield stiffness to elastic stiffness when the non-linear force-displacement relation is characterized by a bilinear relation.

Sa = response spectrum acceleration, at the effective fundamental period of the building.

Vy = yield strength calculated using the capacity curve, where the capacity curve is characterized by a bilinear relation.

4.4.2.1 Step By Step Procedure for Checking Performance at the Expected Maximum Displacement

The following steps should be followed in the performance check:

1. For global building response verify the following:
   The lateral force resistance has not degraded by more than 20% of the peak resistance. The lateral drifts satisfy the limits given in table 4.

TABLE 4.3 DEFORMATION LIMITS

	Performance level			
Inter storey drift limit	**Immediate occupancy**	**Damage control**	**Life safety**	**Structural stability**
Maximum total drift	0.01	0.01-0.02	0.02	$0.33\frac{V_i}{P_i}$
Maximum inelastic drift	0.005	0.005-0.015	No limit	No limit

2. Identify and classify the different elements in the building. Any of the following element types may be present: beam column frames, slab column frames, solid walls, punched walls, floor diaphragms and foundations.
3. Identify all primary and secondary components. This classification is needed for the deformation check in step 5.
4. For each element, use the guidelines to identify the critical components and actions to be checked.
5. The strength and deformation demands at the structure's performance point shall be equal to or less than the capacities considering all co-existing forces acting with the demand spectrum.
6. The performance of structural elements not carrying vertical load shall be reviewed for acceptability for the specified performance level.
7. Non-structural elements shall be checked for acceptability for the specified performance level.

<u>CHAPTER – 5</u>

ILLUSTRATION OF PUSHOVER ANALYSIS USING ETABS

CHAPTER – 5

5.0 ILLUSTRATION OF PUSHOVER ANALYSIS USING ETABS

5.1 General

The study in this thesis is based on nonlinear analysis of R/c moment resisting frames. This chapter presents a summary of various parameters defining the computational models, the basic assumptions and the geometry considered for this study.
The details of the building are reproduced in section 5.2.

5.2 Details of the model

Three dimensional models of R/C moment resisting frames of 6-bay in X and 4-bay in Y direction with different number of stories (5, 8 & 12) are taken for the analysis. In all the frames, the stories are 3.1 meters high, 3 meters wide and bays length is 4 meters. The modulus of elasticity and shear modulus of concrete have been
taken as E = 2.55 ×107 kN/m2 and G = 1.06 ×107 kN/m2.

TABLE 5.1 DETAILS OF THE MEMBER SIZES

Buildings	Beams (m)	Columns (m)	Storey Levels	Slab (m)
5 storey	0.45 x0. 30	0.450 x0. 450	1 - 5	0.125 thk
8 Storey	0.45 x0. 30	0.50 x 0. 50	1 - 5	0.125 thk
		0.40 x 0.40	6 - 8	
12 Storey	0.45 x 0.30	0.70 x 0.70	1 - 7	0.125 thk
		0.50 x 0.50	8 - 9	
		0.40 x 0.40	10 - 12	

Three models have been considered for the purpose of this work.

- 5 storey 3-D R/C moment resisting frames.
- 8 storey 3-D R/C moment resisting frames.
- 12 storey 3-D R/C moment resisting frames.

The plan and sectional elevation of the two buildings are as shown below.

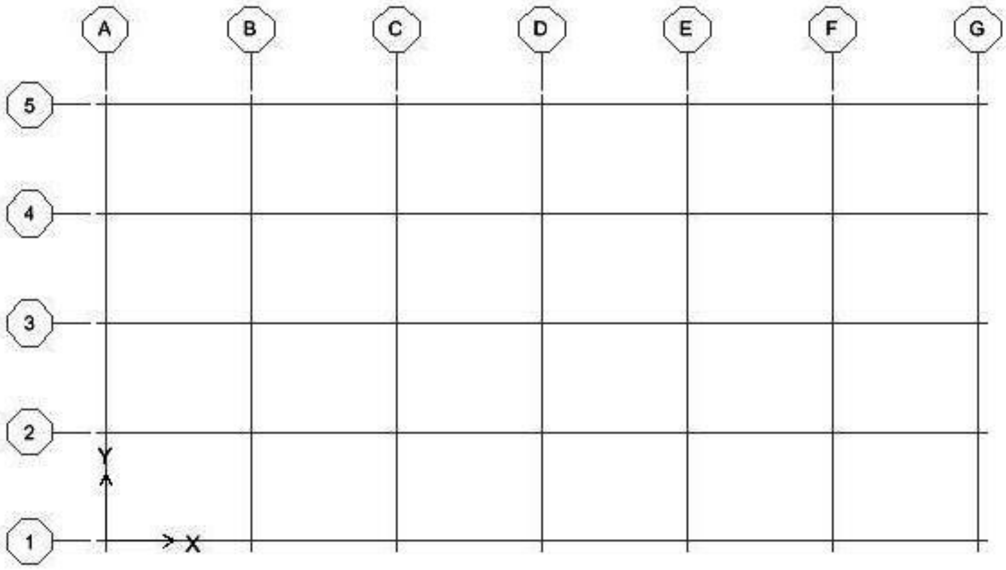


FIG.5.1 PLAN VIEW FOR 5, 8 & 12 STOREY R/C FRAME BUILDING

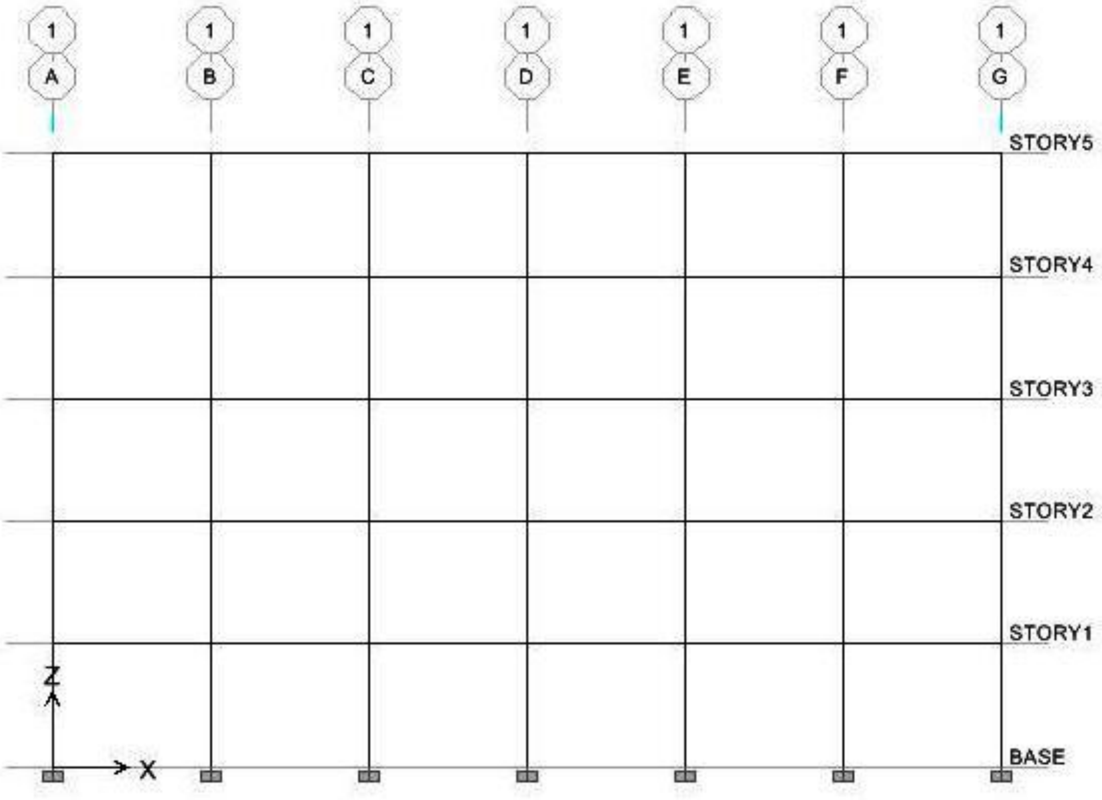


FIG.5.2 ELEVATION VIEW FOR 5 STOREY R/C FRAME BUILDING

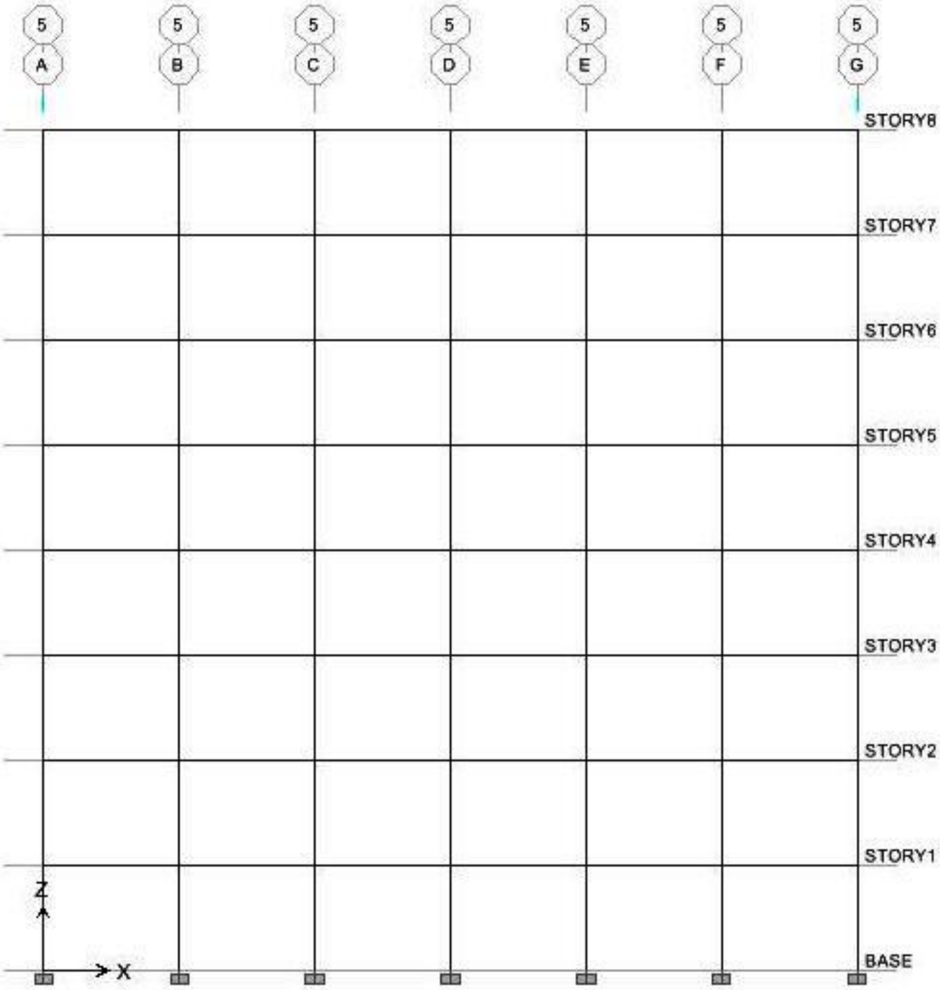


FIG.5.3 ELEVATION VIEW FOR 8 STOREY R/C FRAME BUILDING

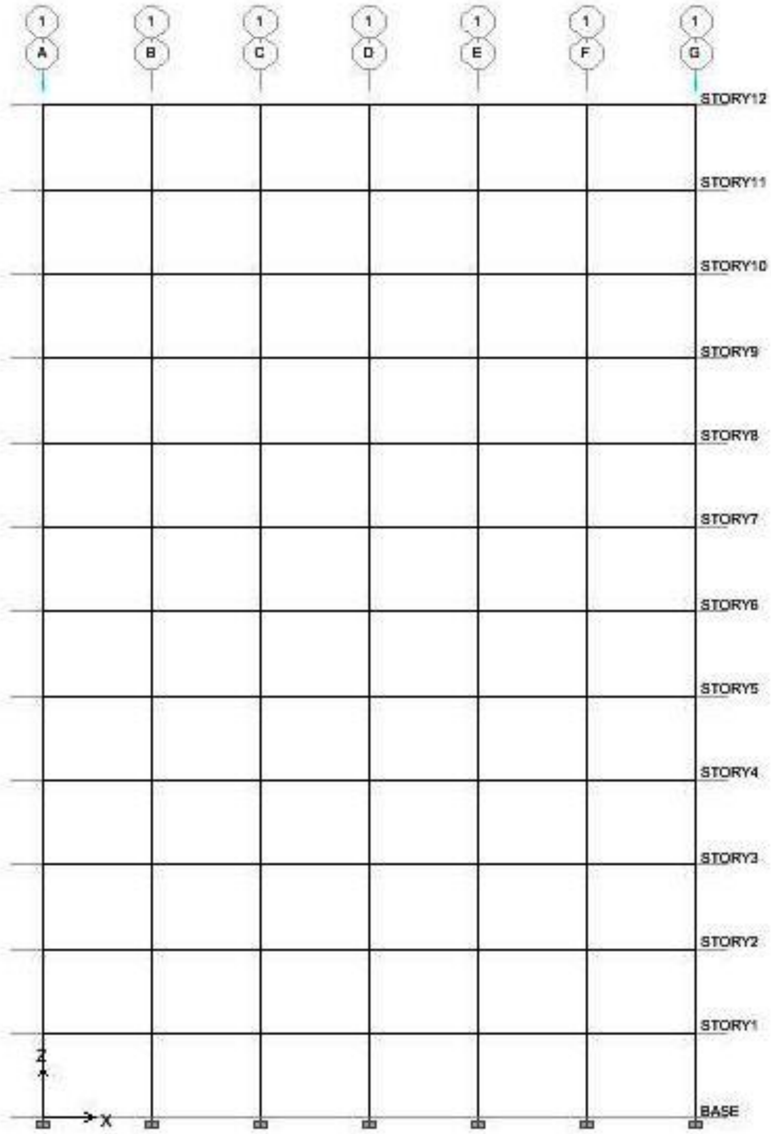


FIG.5.4 ELEVATION VIEW FOR 12 STOREY R/C FRAME BUILDING

5.2.1 Defining the material properties, structural components and modelling the structure

The required material properties like mass, weight density, modulus of elasticity, shear modulus and design values of the material used can be modified as per requirements or default values can be accepted.

Beams and column members have been defined as 'frame elements' with the appropriate dimensions and reinforcement.

Soil structure interaction has not been considered and the columns have been restrained in all six degrees of freedom at the base.

Slabs are defined as area elements having the properties of shell elements with the required thickness. Slabs have been modeled as rigid diaphragms.

5.2.2 Assigning loads

After having modeled the structural components, all possible load cases are assigned. These are as follows:

5.2.2.1 Gravity loads

Gravity loads on the structure include the self weight of beams, columns, slabs, walls and other permanent members. The self weight of beams and columns (frame members) and slabs (area sections) is automatically considered by the program itself.

The wall loads have been calculated and assigned as uniformly distributed loads on the beams.

Wall load = unit weight of brickwork x thickness of wall x height of wall.

Unit weight of brickwork = 20KN/m3

Thickness of wall = 0.23m

Wall load on roof level =20 x (3.1-0.45) x 1.5=6.9KN/m (parapet wall height = 1.5m)

Wall load on all other levels = 20 x 3 x (3.1-0.45) =12.19 KN/m

(wall height = 3m)

Live loads have been assigned as uniform area loads on the slab elements as per IS 1893(Part 1) 2002

Live load on roof 1.5 KN/m2 and Live load on all other floors 3.0 KN/m2.

As per Table 8, **Percentage of Imposed load to be considered in Seismic weight calculation**, IS 1893 (Part 1) 2002, since the live load class is up to 3 KN/m2 , 25% of the imposed load has been considered..Quake loads have been defined considering the response spectra for medium soil as per IS 1893 (Part 1) 2002.

5.2.2.1.1 Defining load combinations

According to IS 1893 (Part 1) 2002 for the limit state design of reinforced and prestressed concrete structures, the following load combinations have been defined

1.5(DL+LL)	DL- Dead Load
1.2(DL+LL+EL)	LL- Live load
1.2(DL+LL-EL)	EL- Earthquake load.
1.5(DL+EL)	
1.5(DL-EL)	
0.9DL+1.5EL	
0.9DL-1.5EL	

5.2.2.2.2 Earthquake lateral loads

The design lateral loads at different floor levels have been calculated corresponding to fundamental time period and are applied to the model. The method of application of this lateral load varies for rigid floor and flexible floor diaphragms.

In rigid floor idealization the lateral load at different floor levels are applied at centre of rigidity of that corresponding floor in the direction of push in order to neglect the effect of torsion.

While idealizing the floor diaphragms as flexible, the design lateral load at all floors is applied such that the lateral load at each floor is distributed along the length of the floor in proportion to the mass distribution.

In our case, the slabs have been modeled as rigid diaphragms and in this connection, the centre of rigidity at each floor level has been determined and the earthquake lateral loads have been applied there.

5.2.3 ANALYSIS OF THE STRUCTURE

Namely three types of analysis procedures have been carried out for determining the various structural parameters of the model. Here we are mainly concerned with the behavior of the structure under the effect of ground motion and dynamic excitations such as earthquakes and the displacement of the structure in the inelastic range.

The analyses carried out are as follows:

- Response Spectrum Analysis
- Pushover analysis.

5.2.3.1 Response Spectrum Analysis

Here we are primarily concerned with observing the deformations, forces and moments induced in the structure due to dead, live loads and earthquake loads. The load case 'Dead' takes care of the self weight of the frame members and the area sections. The wall loads have been defined under a separate load case 'Wall' and the live loads under the case 'Live'. Analysis is carried out for all three cases for obtaining the above mentioned parameters.

Modal analysis is carried out for obtaining the natural frequencies, modal mass participation ratios and other modal parameters of the structure. Response spectrum analysis of the three models are done in the zone V where

Z = 0.36 considering zone factor v

I = 1.0 considering residential building.

R = 5.0 considering special RC moment resistant frame (SMRF)

S a /g = 2.5

response spectrum analysis is carried out using the spectra for medium soil as per **IS 1893 (Part 1) 2002.**

The spectral acceleration coefficient (Sa/g) values are calculated as follows.

For medium soil sites,

Sa/g = 1 + 15T, ($0.00 \leq T \leq 0.10$), (T= time period in seconds)

= 2.50, ($0.10 \leq T \leq 0.55$)

= 1.36/T, ($0.55 \leq T \leq 4.00$)

5.2.3.1.1 Response spectrum analysis in ETABS

The step by step procedure is as follows

- Defining quake loads under the load type 'quake' and naming it appropriately.
- Defining response spectrum function as per IS 1893 (Part 1) 2002. The values of Sa/g Vs. T can be linked in the program in the form of a data file.

- Modifying the quake analysis case with the appropriate analysis case type, applied loads and scale factors.
- Running the analysis**.**

5.2.3.2 Push over analysis

Push over analysis is a static, non-linear procedure that can be used to estimate the dynamic needs imposed on a structure by earthquake ground motions**.** In this procedure a predefined lateral load pattern is distributed along the building height. The lateral forces are then monotonically increased in constant proportion with a displacement control at the control node of the building until a certain level of deformation is reached. F o r this analysis nonlinear plastic hinges have been assigned to all of the primary elements. Default moment hinges (M3-hinges) have been assigned to beam elements and default axial-moment 2-moment3 hinges (PMM-hinges) have been assigned to column elements. The floors have been assigned as rigid diaphragms by assigning diaphragm constraint.

Lateral load profiles

IS 1893 (Part 1) 2002 parabolic lateral load (PLL) at floor 'i' is given by-

$$Qpi \equiv Vb \frac{Wihi2}{\sum_{j=1}^{n} Wjhj2}$$

Triangular lateral load (TLL) at floor 'i' is given by

$$Qpi \equiv Vb \frac{Wihi}{\sum_{j=1}^{n} Wjhj}$$

Uniform lateral load (ULL) at floor 'i' is given by

$$Qpi \equiv Vb \frac{Wi}{\sum_{j=1}^{n} Wj}$$

Where,

Qpi= lateral loads as per IS: 1893-2002 and ATC-40 at each floor level

W = total seismic weight the structure

Wi= seismic weight of floor i

hi= height of floor measured from base

n = is the number of levels at which the masses are lumped.

5.2.3.3 Pushover analysis in ETABS

The step by step procedure for buildings with rigid floor diaphragm is as follows**:**

- A three dimensional computer model was generated.
- Linear static, modal and response spectrum analysis were performed for specified response spectrum.
- Centers of rigidity at various floor levels are calculated and are applied to the model.
- The calculated lateral load is distributed along the height of the building.
- The lateral loads at different floor levels are applied at centre of rigidity of the respective floor level.
- The rigid floor condition is given to the floors at different levels.
- The primary elements are identified and plastic hinges are assigned. The beam elements are assigned with plastic hinge as given in ATC-40 and FEMA – 273, 356. The beam elements are assigned with moment (M3) hinges and the column elements are assigned with axial load, moment in 2 and 3 – directions (PMM) hinges.
- Pushover analysis cases are then defined. The first case is for dead and live loads starting from zero initial conditions (unstressed state). The second case is defined for the calculated lateral loads and starts from the end conditions of the previous state. Non-linear parameters are defined as per requirements or default values are accepted.
- Analysis is then run and pushover curves are obtained.

CHAPTER – 6

RESULTS FOR PUSHOVER ANALYSIS

CHAPTER - 6

6.0 RESULTS FOR PUSHOVER ANALYSIS

The inelastic analysis of the structures under static and dynamic loading is performed by the nonlinear analysis software "ETABS".

6.1 Modal Properties

Modal properties of the structure model were obtained from the linear dynamic modal analysis which is the primary step used in pushover analysis.

TABLE 6.1- ELASTIC DYNAMIC PROPERTIES OF THE FRAMES

Modal Period	5 Storey	8 Storey	12 Storey
Mode 1	0.629	0.978	1.238
Mode 2	0.592	0.920	1.154
Mode 3	0.578	0.892	1.098

Modal period increases as the height of the building increases.

6.2 Base Shear (KN) vs Displacement (m):

This shows the variation of base shear with displacement For the 5, 8 & 12 storey.

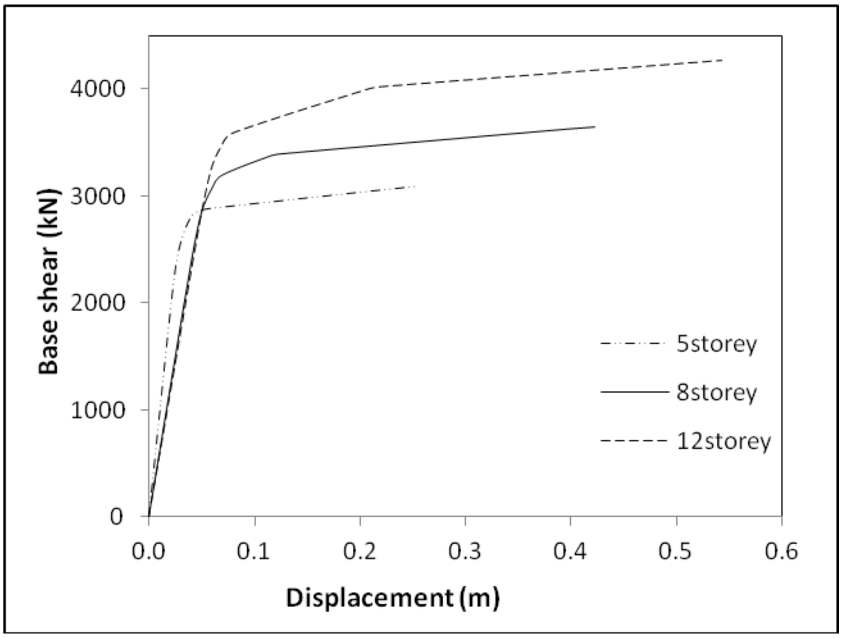


FIG.6.1 CAPACITY CURVE

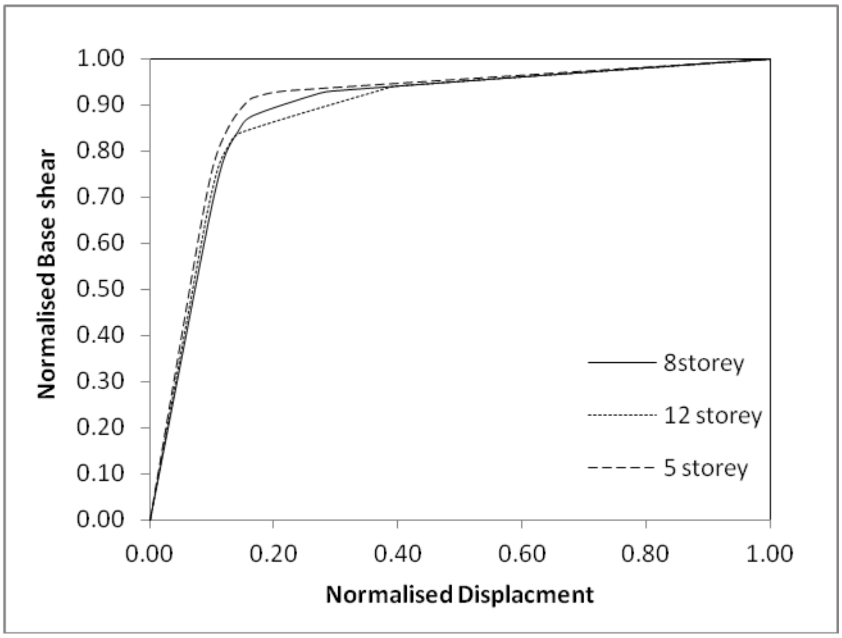


FIG 6.2. NORMALISED CAPACITY CURVE

6.3 6.3. Storey ht. vs Displacement:

Table 6.1 Storey ht. Vs Displacement curve for the 5, 8 & 12 storey R/C frame.

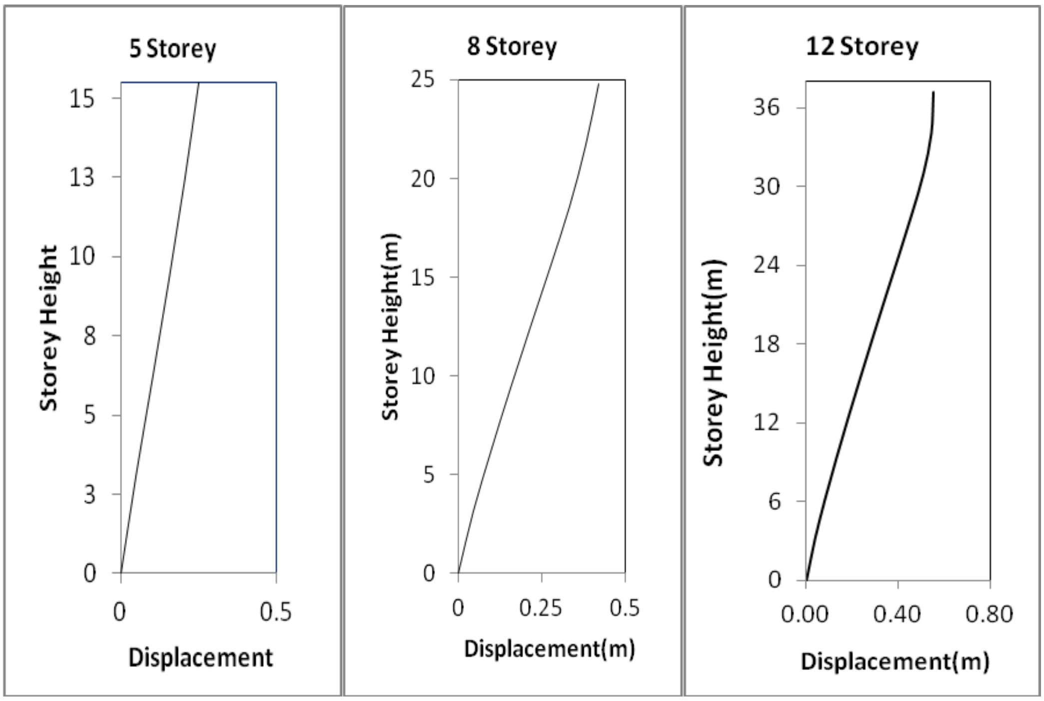


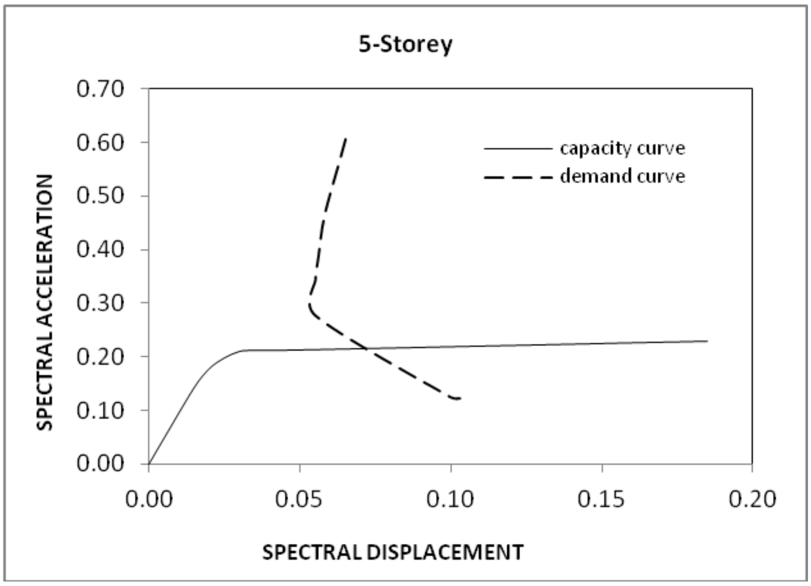
FIG.6.3 STOREY HT. VS DISPLACEMENT CURVE FOR THE 5, 8 & 12 STOREY R/C FRAME.

Story displacements were extracted from pushover database at each step of pushover analyses.

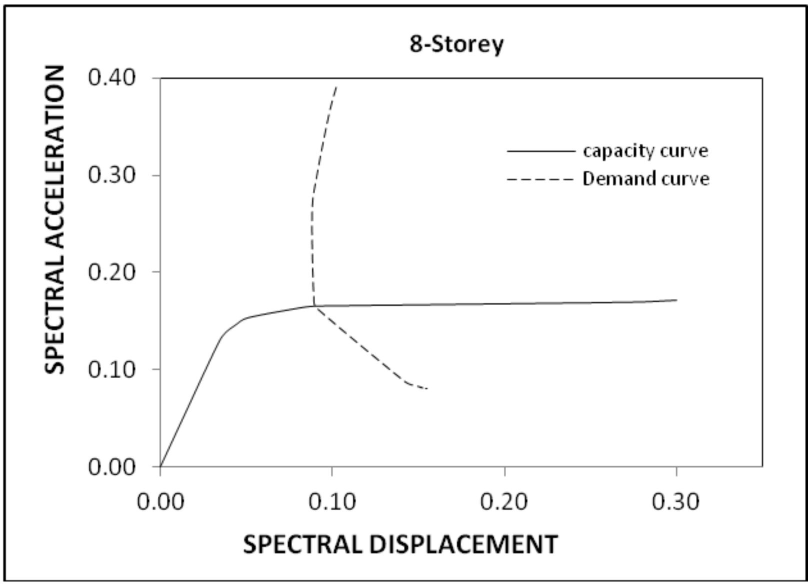
- (storey height vs story displacement) were developed to illustrate the variation in story displacement with increase in storey height.
- The structure's ability to survive an earthquake may be measured in terms of the expected state of damage of the structure after the earthquake.
- Displacement Variation in upper storey becomes nonlinear as the storey height increases.

6.4 Capacity Spectrum

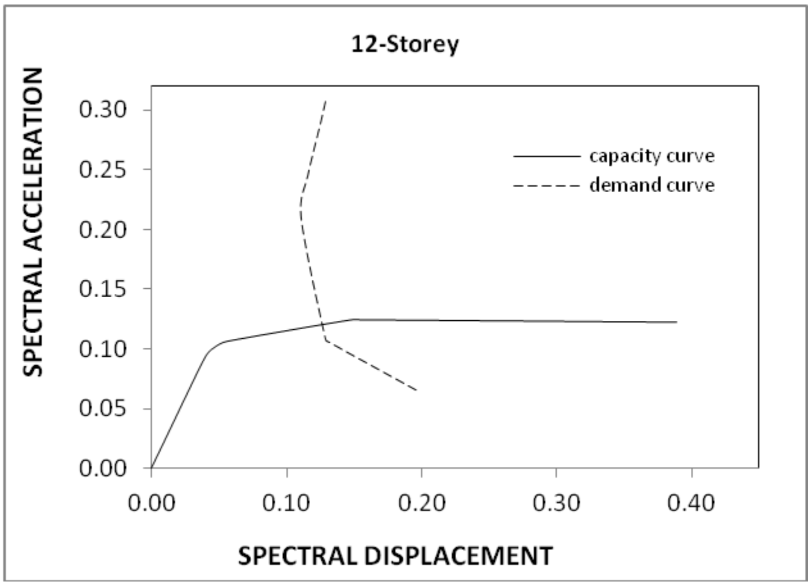
The curve shows the variation of spectral acceleration vs spectral displacement for the considered 5, 8 and 12 storey r/c frame.



(a)



(b)



(c)

FIG.6.5 . CAPACITY-DEMAND CURVES A) 5 STORY, B) 8 STORY AND C) 12 STORY.

The main output of a pushover analysis is in terms of response demand versus capacity. The structure has a good resistance because the demand curve intersects the capacity envelope near the elastic range.From the fig 6.5. It can be concluded that the structure will behave safely during the imposed seismic excitation and need not to be retrofitted.

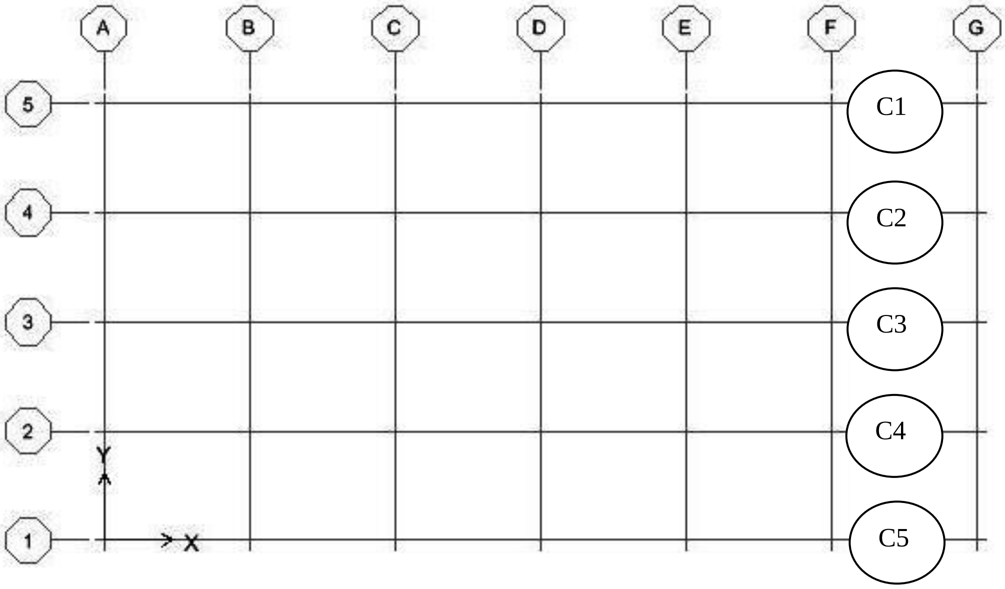


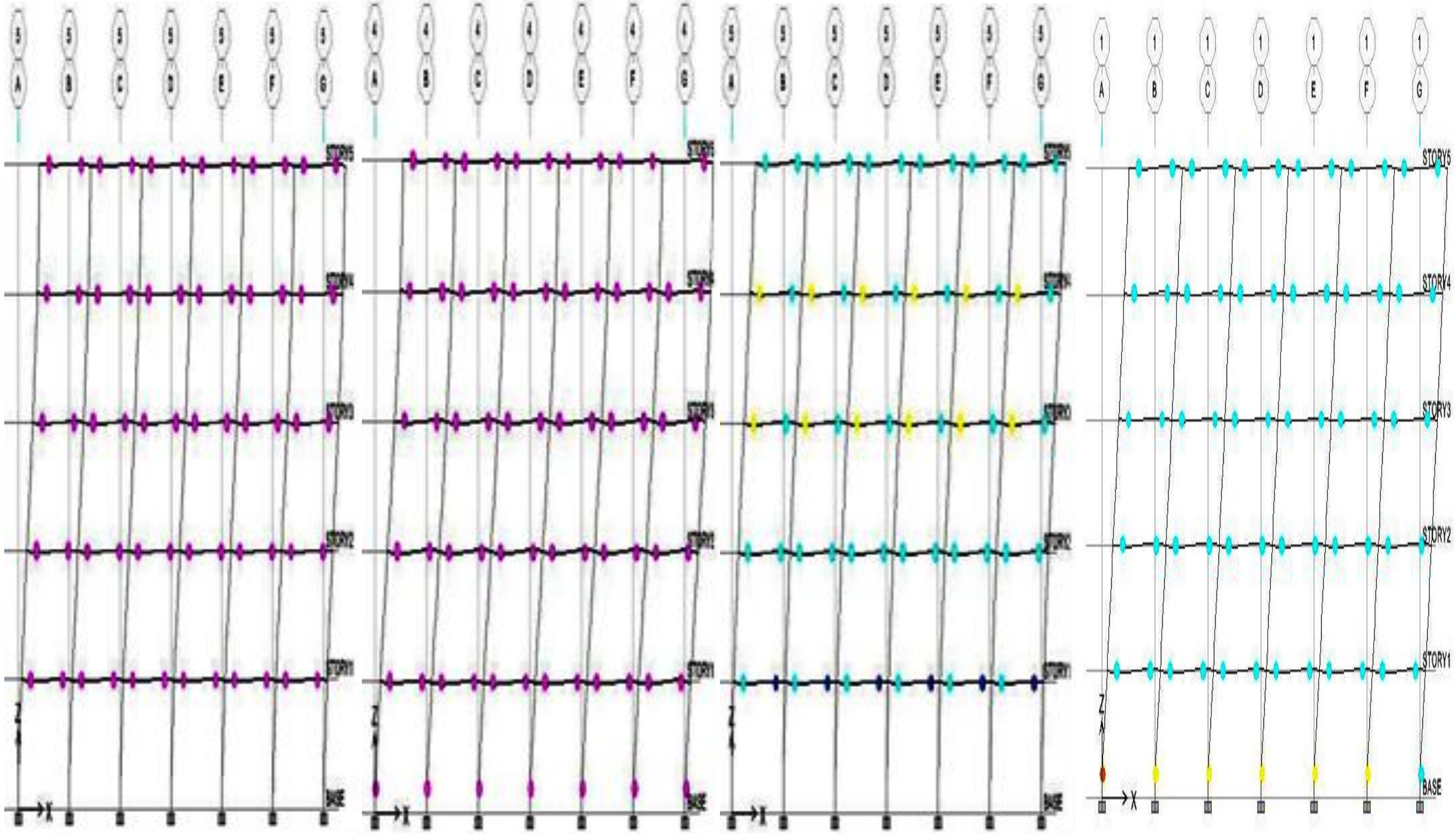
FIG. 6.6 PLAN SHOWING LOCATION OF COLUMN FOR SHEAR FORCE AND BENDING MOMENT

6.5 Comparison of Shear force And Bending Moment in linear static and nonlinear analysis

Shear force and bending moment values in column C1,C2,C3,C4 & C5(location shown in fig .6.6) for both linear static and non linear analysis.

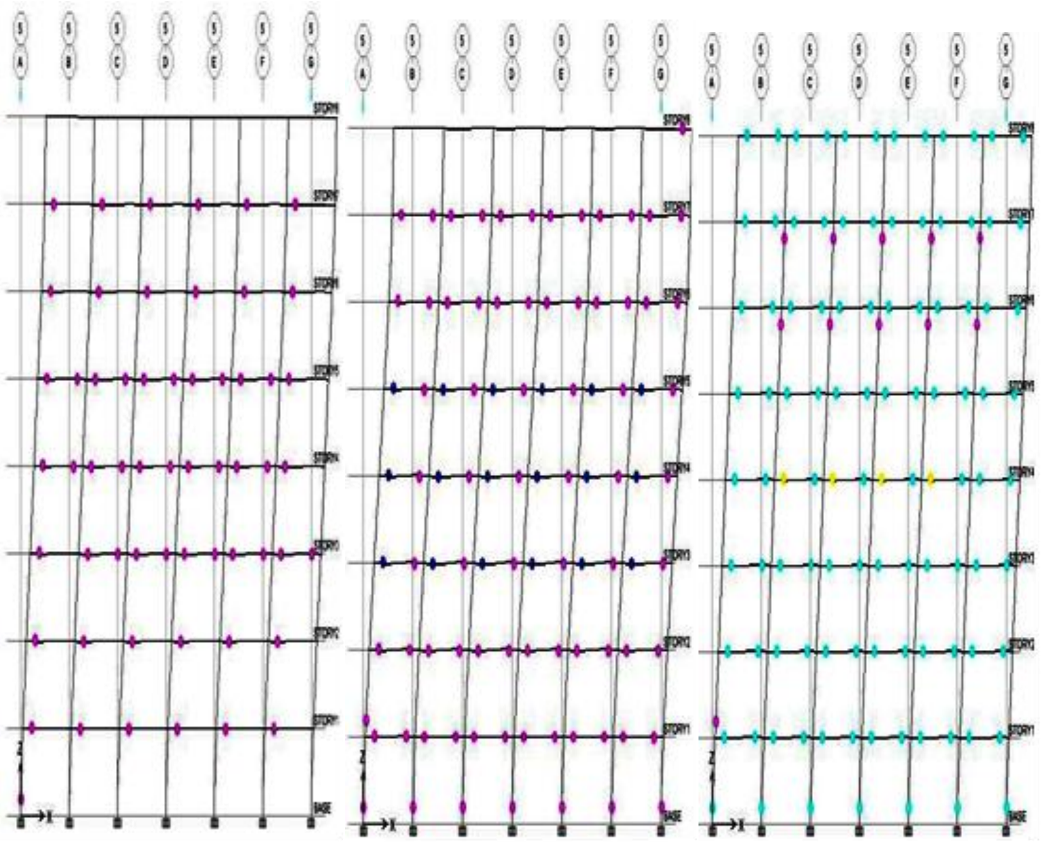
Col. No.	**SHEAR FORCE**					
	Linear Static Analysis			**Nonlinear Static Analysis**		
	5 storey	8 storey	12 storey	5 storey	8 storey	12 storey
C1	43.24	50.0	61.54	84.52	101	122.45
C2	43.24	50.0	61.54	91.15	111	147
C3	43.24	50.0	61.54	91.46	113	151
C4	43.24	50.0	61.54	91.15	111	147
C5	43.24	50.0	61.54	84.52	101	122.45
Col. No.	**BENDING MOMENT**					
	Linear Static Analysis			**Nonlinear Static Analysis**		
	5 storey	8 storey	12 storey	5 storey	8 storey	12 storey
C1	83	107	194.28	176	271	587.54
C2	83	107	194.28	194	299	651.67
C3	83	107	194.28	194	304	662.75
C4	83	107	194.28	194	299	651.67
C5	83	107	194.28	176	271	587.54

Formation of Plastic Hinges



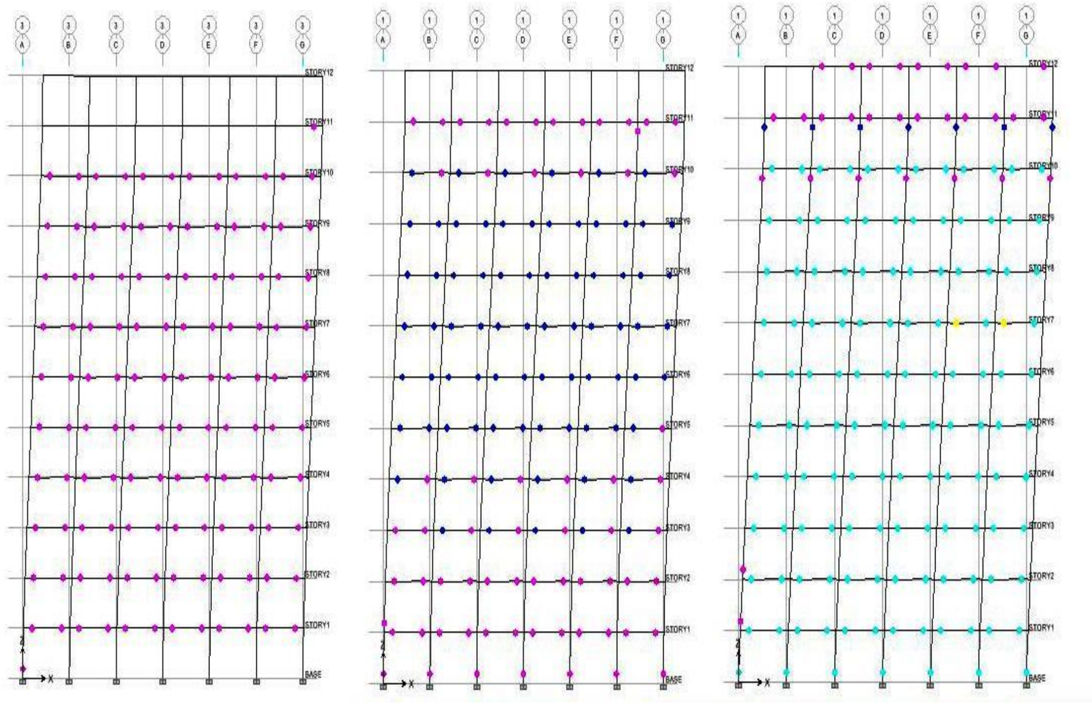
Yielding **IO** **LS** **C**

FIGURE 6.6 HINGES PATTERNS 5 STORY BUILDING FOR DIFFERENT DISPLACEMENTS LEVELS



Yielding **IO** **LS**

FIGURE 6.7 HINGES PATTERNS 8 STORY BUILDING FOR DIFFERENT DISPLACEMENTS LEVELS



Yielding **IO** **LS**

FIGURE 6.8 HINGES PATTERNS 12 STORY BUILDING FOR DIFFERENT DISPLACEMENTS LEVELS

- Location of weak points and potential failure modes that structure would experience in case of a seismic event is expected to be identified by pushover analyses.
- Sequential formation of plastic hinges gives us the failure pattern or sequence of column/beam failure. This is a valuable information in the dynamic analysis and in designing the structure. Hence we need to strengthen only selected member and not all the members of the same storey.
- The retrofit strategies can be devised from Pushover analysis, as it not only shows the point of occurrence of the first plastic hinge but also the formation of hinge in succession.
- Economical structure can be designed for the earthquake resistant structure.
- Formation of hinges starts from beam ends and then in lower stories column and then propagates to the upper stories.

6.6 Ductility Demand: The ductility demands imposed on the frames at the various performance levels are found tablulated in table 6.2

TABLE 6.2 DUCTILITY DEMAND OF 5,8,& 12 STOREY R/C FRAMES.

S.no.	Displacement level	5 storey		8 storey		12 storey	
		Roof Displacement δroof (mm)	Ductility Demand δroof/δyield	Roof Displacement δroof (mm)	Ductility Demand δroof/δyield	Roof Displacement δroof (mm)	Ductility Demand δroof/δyield
1	δ_{yield}	26.1	1	39.3	1	49.4	1
2	δ_{IO}	32.7	1.25	69.5	1.77	74.7	1.51
3	δ_{LS}	55.3	2.12	123.6	3.14	196.5	3.98
4	δ_{CP}	256.0	9.81	422.2	10.74	551.8	11.17
5	δ_{C}	∞	-	-	-	-	-

- Plastic yielding occurs at base-support sections of first-story column members and at both end-sections of beam members, which signifies strong column and weak-beam behavior typical of earthquake resistant building construction.
- The structure becomes flexible as the roof displacement increases.

6.7 Comparison of Shear force and bending moment in linear static and nonlinear static pushover analysis.

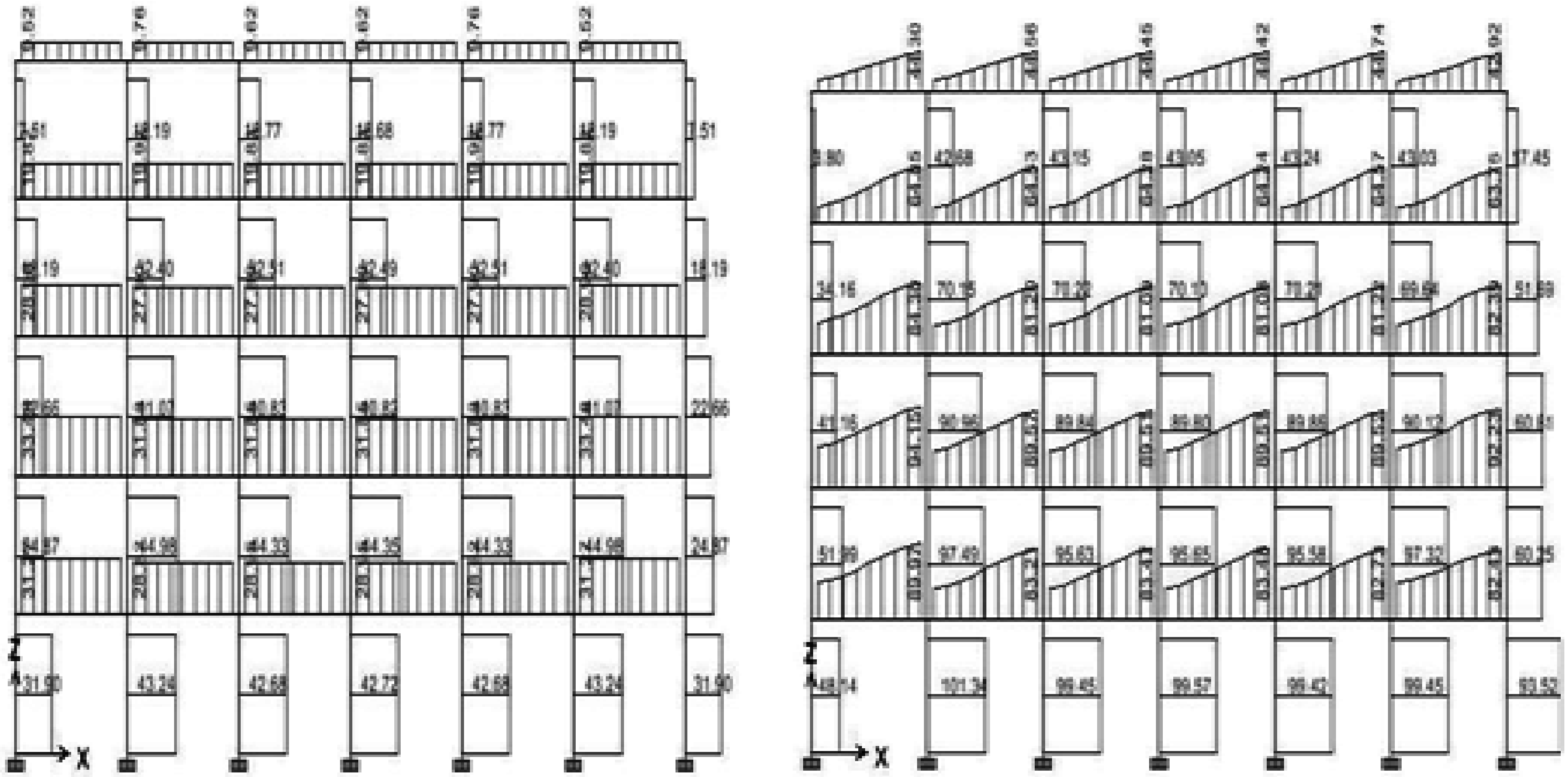


FIG. 6.8 SHEAR FORCE FOR THE 5 STOREY R/C FRAME BUILDING

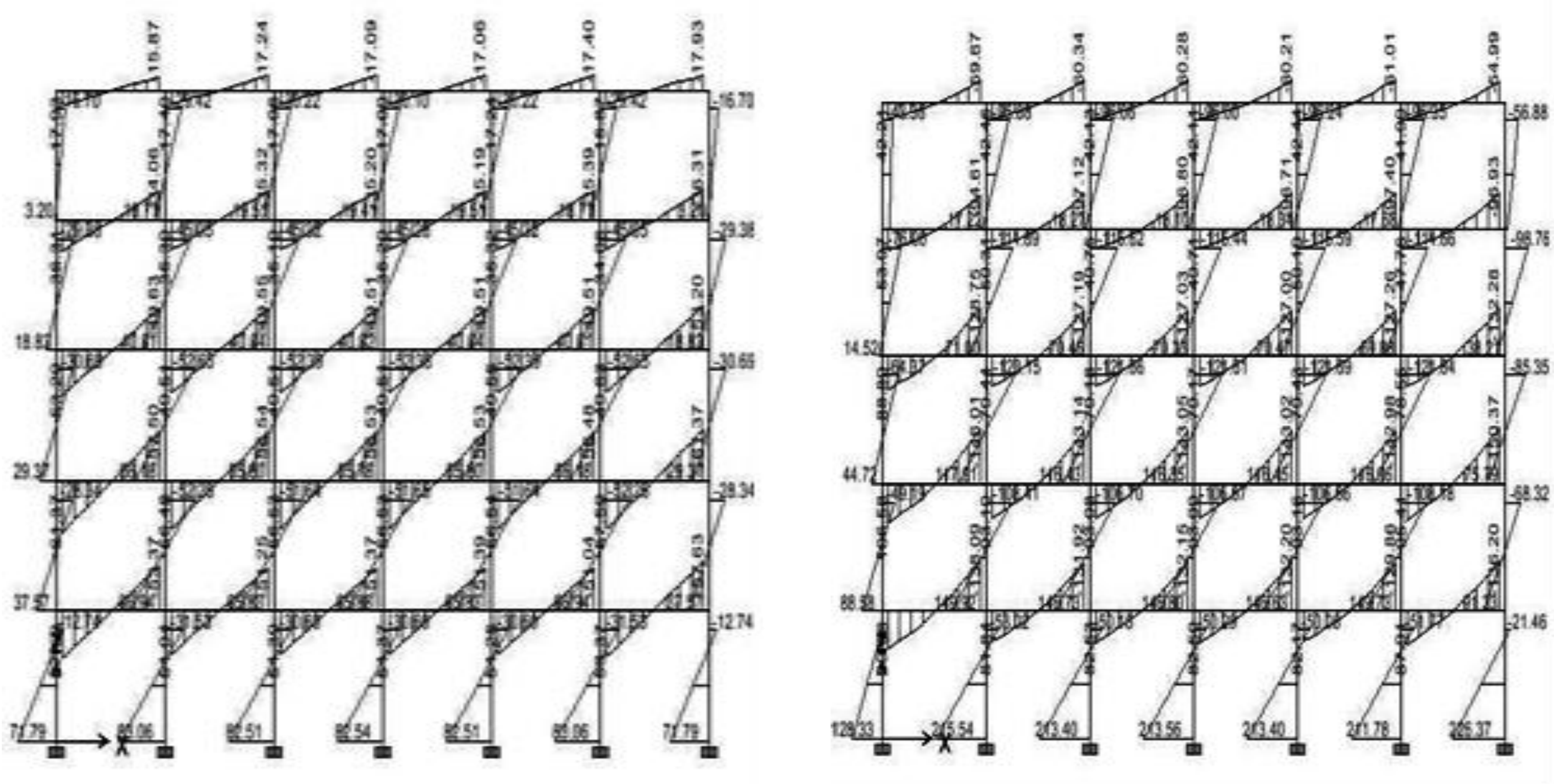


FIG. 6.9 BENDING MOMENT DIAGRAM FOR 5 STOREY R/C FRAME

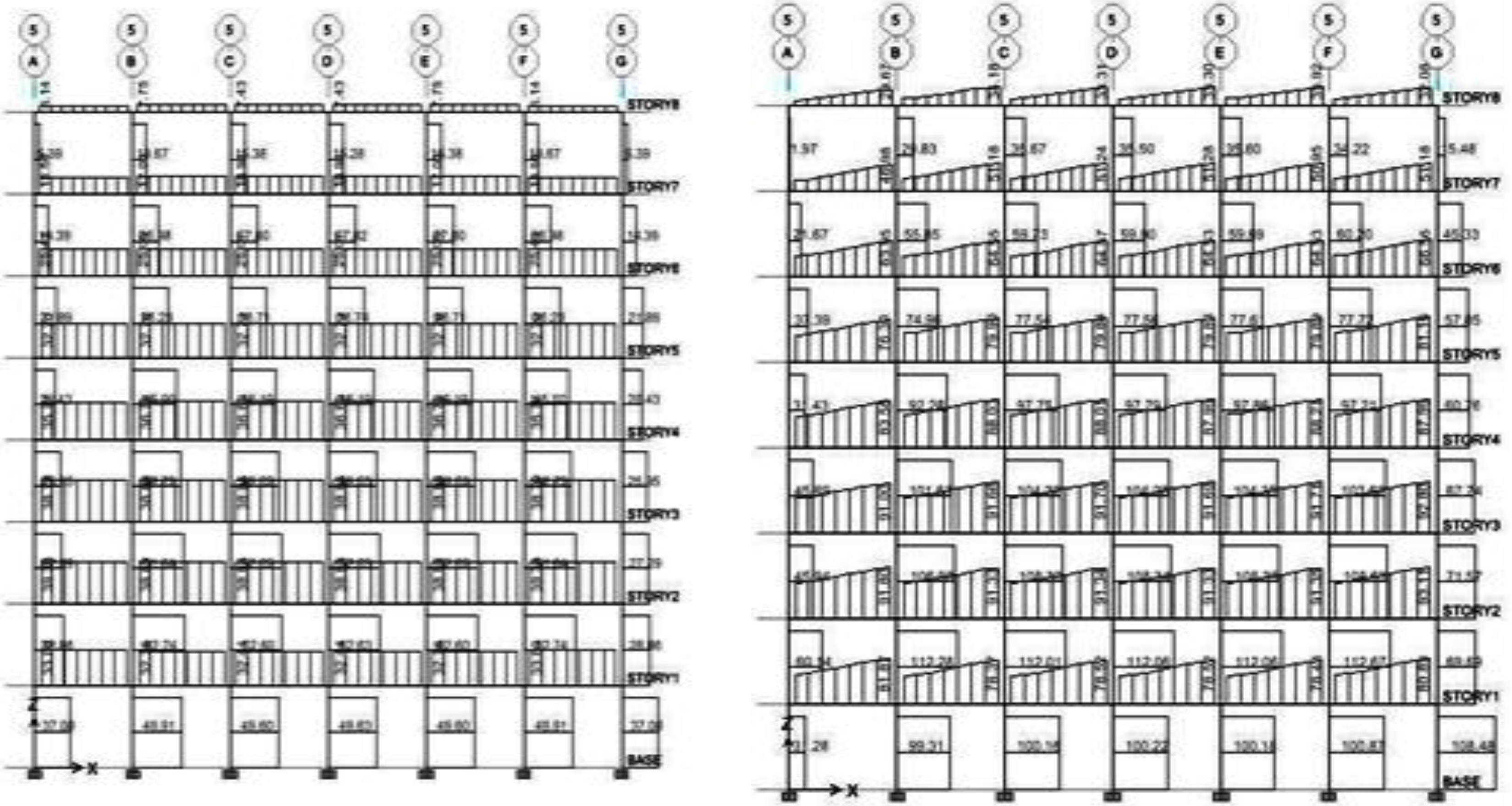


FIG. 6.10 SHEAR FORCE DIAGRAM FOR 8 STOREY R/C FRAME MOMENT DIAGRAM

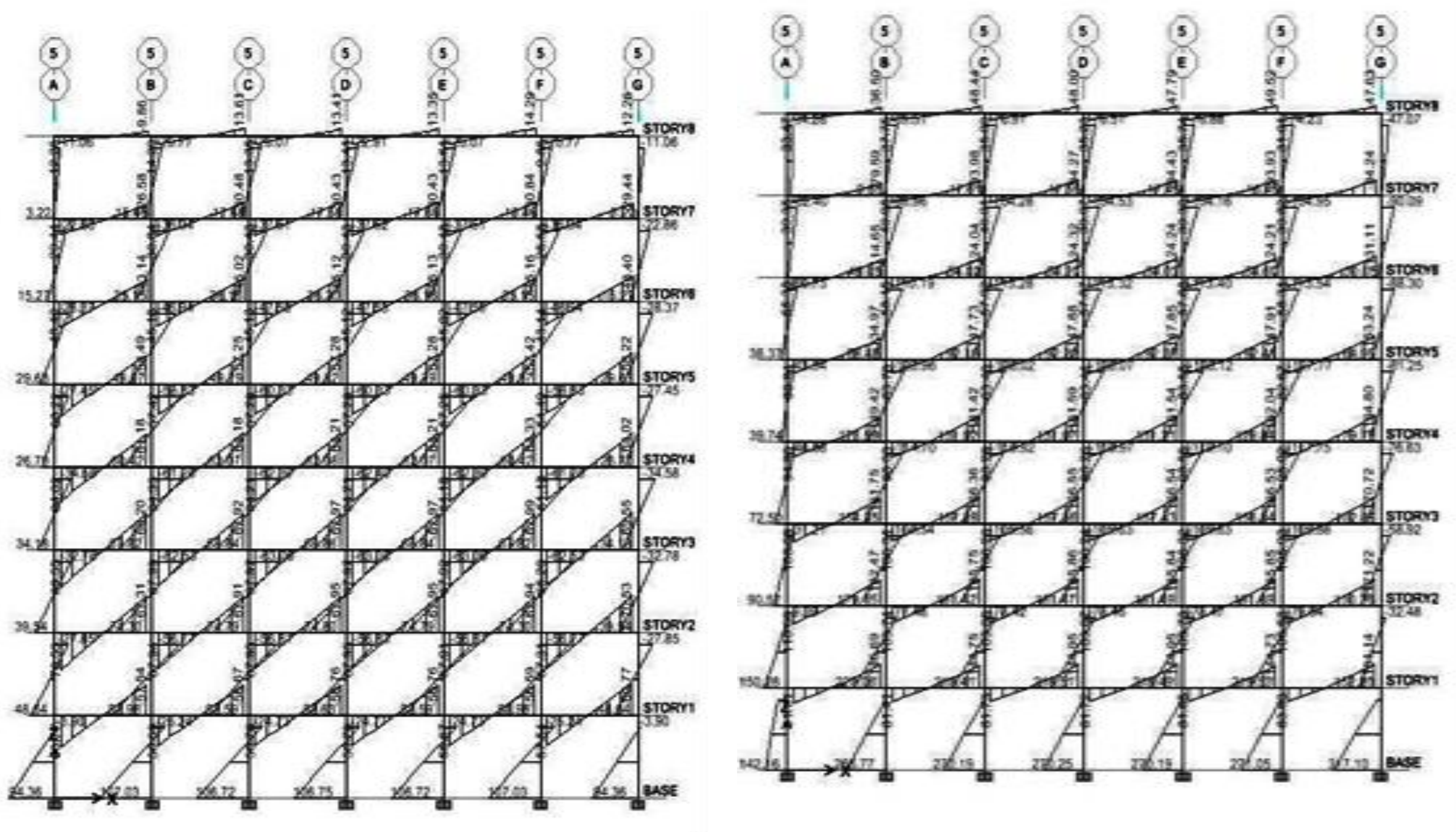


FIG. 6.11 BENDING MOMENT DIAGRAM FOR 8 STOREY R/C FRAME

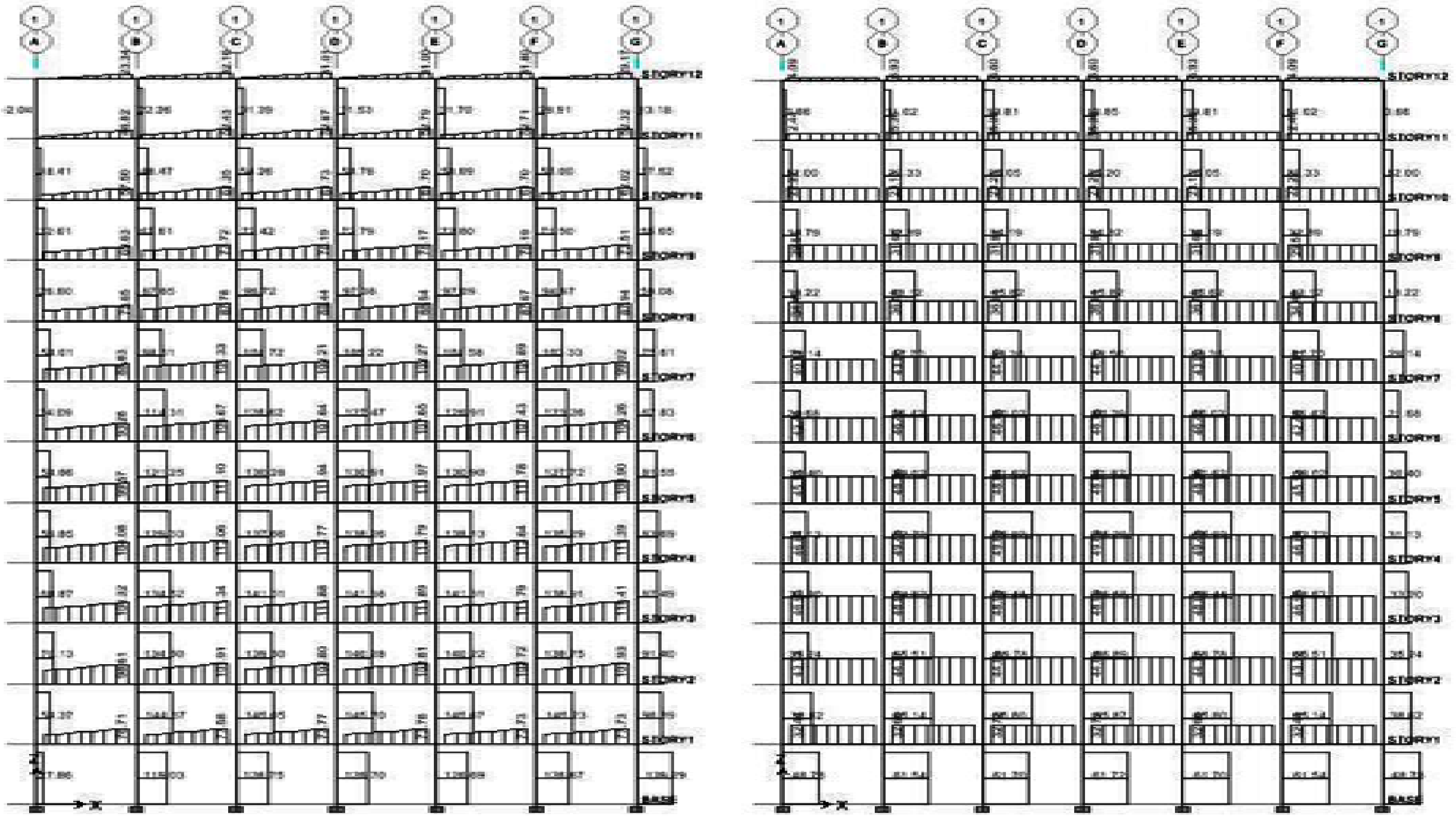


FIG.6.12 SHEAR FORCE FOR 12 STOREY R/C FRAME

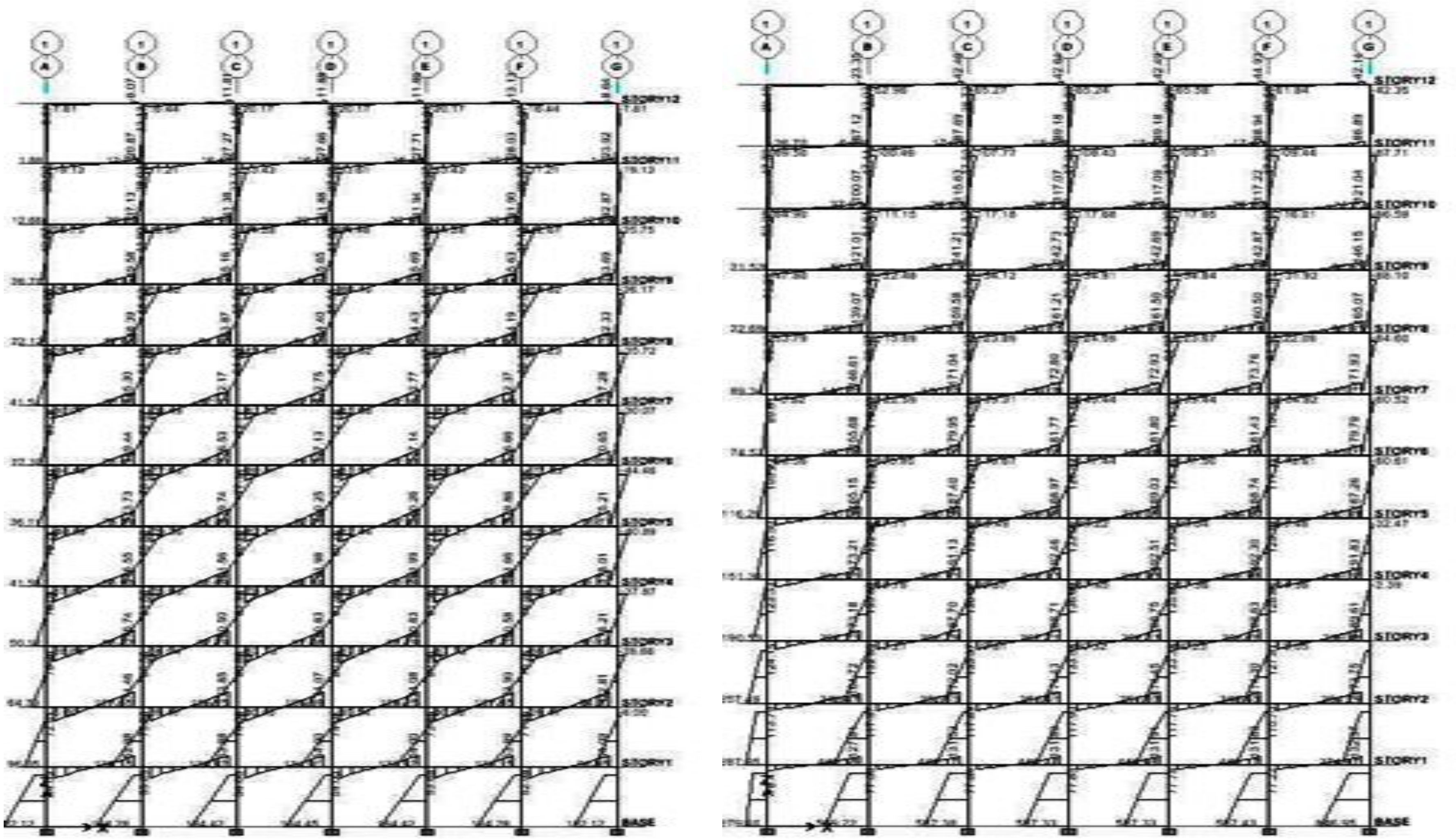


FIG.6.13 BENDING MOMENT DIAGRAM FOR 12 STOREY R/C FRAME

CHAPTER – 7

DISCUSSIONS OF RESULTS & CONCLUSION

CHAPTER - 7

7.0 DISCUSSIONS OF RESULTS & CONCLUSION

7.1 7.1 Pushover Analysis

The present work has been carried out to study pushover analysis on low, medium and high rise building using ETABS.

7.1.1 Effect on Base shear

Comparing the Base shear for the static linear and static nonlinear analysis like pushover analysis for the different cases.It is seen that
(1) in 5 storey frame the base shear for the pushover analysis is 2.23 times the elastic base shear.
(2) in 8storey frame the base shear for the pushover analysis is 2.26 times the elastic base shear.
(3) in 12 storey frame the base shear for the pushover analysis is 2.10 times the elastic base shear.

7.1.2 Effect on capacity curve

Results obtained for the present work is carried out using ETABS.
Fig.6.1 shows the combined results for the capacity curve in which the base shear increases significantly in the elastic range for the very small increase in displacement and then displacement increases significantly as compared to base shear .This shows the structure has good resistance against expected imposed seismic loads and the curve for the three different cases follow the same pattern.

7.1.3 Variation of Roof Top displacement

Fig.6.3 shows the variation of roof top displacement with story height.

- (Storey height vs Storey displacement) were developed to illustrate the variation in story displacement with increase in storey height.

- The curve showing Variation of Displacement with storey height is linear in low rise building and the curve becomes nonlinear as the storey height increases for the medium and high rise building.

TABLE.7.1

Frame	Displacement (m)	
	Base	Top
5 Storey	0	0.248
8 Storey	0	0.423
12 Storey	0	0.552

7.1.4 Effect on Demand Curve

Fig .6.4 shows the variation of capacity and demand curve the three cases.

- The fig.6.4 shows the structure has a good resistance against imposed earthquake as the demand curve intersects the capacity envelope near the elastic range which shows good structural resistance.
- From the fig 6.4. it can be concluded that the structure will behave safely during the imposed seismic excitation and need not to be retrofitted.

7.1.5 Sequential Formation of plastic Hinges

- Sequential formation of plastic hinges gives us the failure pattern or sequence of column/beam failure. This is a valuable information in the dynamic analysis and in designing the structure. Hence we need to strengthen only selected member and not all the members of the same storey.
- Formation of hinges starts from beam ends and then propagates to the upper stories beam and then in lower stories column and then propagates to the upper stories. Most of the hinges developed in the beams and few in the columns but with limited damage.

7.1.6 Effect on shear Force

Comparing the shear force for the static linear and static nonlinear analysis like pushover analysis for the different cases.

(1) in 5 storey frame there is 97.89% increase in shear force in non linear analysis as compared to linear static.

(2) in 8 storey frame there is 115.68% increase in shear force in non linear analysis as compared to linear static.

(3) in 12 storey frame there is 105.14% increase in shear force in non linear analysis as compared to linear static.

7.1.7 Effect on Bending Moment

Comparing the shear force for the static linear and static nonlinear analysis like pushover analysis for the different cases .

(1) in 5 storey frame there is 130.89% increase in bending moment in non linear analysis as compared to linear static.

(2) in 8 storey frame there is 171.10% increase in bending moment in non linear analysis as compared to linear static.

(3) in 12 storey frame there is 202.41% increase in bending moment in non linear analysis as compared to linear static.

Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be useful and effective tool for performance based design. On the basis of numerical work and discussion of results obtained after following conclusion were drawn.

- The non linear behaviour of the structure can be easily assesed by the pushover analysis.
- The base shear, shear forces, bending moment values shows that the strength is still available to take up the more forces after the elastic analysis is done.
- More economical Structure can be designed as the structure is having the capacity to take up more forces.

- The structure has good resistance against expected imposed seismic loads.It can be concluded that the structure will behave safely during the imposed seismic excitation and need not to be retrofitted.
- The retrofit strategies can be devised from Pushover analysis, as it not only shows the point of occurrence of the first plastic hinge but also the formation of hinge in succession.
- The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behaviour of structures.

CHAPTER – 8

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