DISPLACEMENT BASED DESIGN, (DBD), NONLINEAR STATIC PUSHOVER ANALYSIS TO VERIFY THE PROPER COLLAPSE MECHANISM OF STRUCTURES

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DEDICATION

TO MY MOTHER ,HUSBAND, SON, DAUGHTERS AND FRIENDS

WITH SINCERE LOVE

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a_y	Spectral acceleration at yield
a_{pi}	Spectral acceleration at point c
A	Point corresponds to unload conditions
A	Cross sectional area
A_g	Gross area
A_n	Ordinate of the pseudo-acceleration response or design spectrum
A_s	Area of tension reinforcement
A_w	Area of the web cross section,= b_w*d
B	Point has resistance equal to the nominal yield strength
b_w	Web width
C	Point corresponds to the nominal strength
C_1	Modification Factor to relate maximum inelastic displacement
C_2	Modification Factor to represent the effect of hysteresis shape
C_1	Modification Factor to consider p-Δ effect
C_c^*	Normalized base shear capacity
C_0	Modification Factor to relate spectral displacement
CQC	Complete Quadratic Combination
CSM	Capacity Spectrum Method
c	Damping of a structure
D	Point represents initial failure

DC	Damage control (sp-2)
d_y	Spectral displacement at yield
d_{pi}	Spectral displacement at point c
DCM	Displacement Coefficient Method
E	Point corresponds to the residual resistance
E_c	Modulus of elasticity of concrete
E_D	Energy dissipated by damping
ELF	Equivalent Lateral Force
E_s	Modulus of elasticity of steel
E_{so}	Maximum strain energy
F_{no}	Lateral forces
F_m	Maximum base shear for the *m*th mode
I_{eff}	Effective moment of inertia
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
IO	Immediate Occupancy (sp-1)
[K]	Lateral Stiffness Matrix
K	K factor depends on structural behavior
$K_{effective}$	Effective lateral stiffness of the building under consideration
$K_{initial}$	Initial lateral stiffness of the building under consideration
LDP	Linear Dynamic Procedure
LS	Life Safety

LSP	Linear Static Procedure
M_n	Generalized nth modal mass
MDF	Multi degree system of freedom
MPA	Modal Pushover Analysis
m	Mass of structure
$N_{A\text{-}A}$	Non- structural elements are generally in place
NDP	Nonlinear Dynamic Procedure
NEHRP	Guidelines for Seismic Rehabilitation of building
NP-B	Non- structural element immediate occupancy
NP-C	Non- structural components and system but no collapse
NP-D	Extensive damage to non-structural components
NP-E	Nonstructural elements
NSP	Nonlinear Static Procedure
P	Axial force in member
PF_1	Participation factors for the first natural mode .
p	Generalized modal coordinates p
P_{mo}	Response to the *m*th mode
Q	Generalized load
Q_{CE}	Expected strength of a component or element at the deformation level under consideration
Q_{CL}	Lower –bound estimate of the strength of a component or element at the deformation level under consideration for force-controlled actions

q	Product of the mode shape matrix and a vector of generalized modal coordinates p
q_m	Displacement at the *n*th location due to *m*th mode shape
q_{mo}	Roof displacement
R	Strength Ratio
r_{no}	Peak value of nth-mode contribution
r_n^{st}	Modal static response
$r_n(t)$	Peak value of nth-mode contribution to response
r_o	Peak value of the total response
RSA	Response Spectrum Analysis
r(t)	Total Response
r_t	Story drift
S^*	Story shears determined by response spectrum analysis
S^*j	Total mass at each floor
S^*_n	Lateral forces distribution to the structure
S_a	Spectral acceleration
SDF	Single Degree of Freedom
S_d	Spectral displacement
S_n	Spatial distribution of the effective earthquake forces
Sp-5	Structural Stability (Sp-5)
Sp-6	Non- structural element immediate occupancy
SRSS	Square-Root-of-Sum-of-Squares

$T_{effective}$	Effective fundamental period of the building in the direction under consideration
T_{eq}	Natural period
T_i	Elastic Fundamental Period
t	Vector of ones corresponding to Rotational DOFs
ULP	Uniform Load Pattern
u_n	Response Spectrum
V	Design shear force
Vs	Nominal shear strength provided by shear reinforcement
V_i	Design shear force at level I
W	Total weight considered for seismic effect
w_i	Tributary weight at the location Ii from 1 to N
α	Modal mass coefficient
β_{eff}	The effective damping
β_o	Equivalent viscous damping
Δ^*	Effective displacement
δ_t	Target displacement
ϕ_1	Fist mode shape
ϕ_n	N-th mode shape
ϕ_{nm}	The magnitude of the *m*th-mode at the *n*th location
Γ_n	Modal participation Factor
L_n	Nth modal excitation factor

ρ	Tension reinforcement ratio
ρ_t	Ratio of temperature and shrinkage reinforcement
ρ_b	Balanced steel ratio
ρ_{max}	Maximum tension reinforcement ratio
ρ_{min}	Minimum tension reinforcement ratio
ρ'	Compression reinforcement ratio
$\theta_{eff}(t)$	Total excitation
ω_n	Generalized nth modal frequency
ξ_n	Damping ratio
z	Zone factor per UBC Table(16-N)

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ABSTRACT

Under the pressure of recent developments, seismic codes have begun to explicitly require the identification of sources of inelasticity in structural response, together with the quantification of their energy absorption capacity. Ideally, such performance evaluation of structural systems subjected to earthquake loading should be based on nonlinear time history analysis. However, the intrinsic complexity and the additional computational effort required by time history analysis do not justify its use in ordinary engineering applications. As a result of the above, nonlinear static, as opposed to dynamic, pushover analysis has been gaining significance over recent years as a tool for design verification.

In the pushover procedure, a static lateral load, which is distributed approximately equivalent to seismic loads generated by an earthquake, is applied to the structure. The structure is then displaced (pushed over) incrementally to the level of deformation expected during the earthquake (target displacement) while keeping the applied load distribution pattern. Base shear and corresponding displacement at each displacement stage are used to build the pushover curve and then the seismic structural deformations and the performance level of the structure are estimated. The nonlinear load-

deformation characteristics of individual components and elements of the structure are considered in the model to account for the possibility of exceeding elastic limits.

In this study, and following a brief review of the latest developments in the field, the concept and accuracy of the displacement-based pushover method is explored through comparison with results from linear static, linear dynamic and nonlinear dynamic analyses. Therefore, an 8-story building with a total height of 30.4m was considered. The structural system of the building consists of nine reinforced concrete ordinary moment resisting frames in each direction with four shear walls in Y direction only. The building was modeled as a three dimensional system using the SAP2000 software and the design seismic parameters including the fundamental period, base shear, joint displacement and joint rotation for the assumed model were determined using the static force procedure, as recommended in the UBC-97 code, response spectrum analysis using the UBC-97 design response spectrum, time history analysis using the EL-Centro earthquake record and finally using the pushover method. Results of analysis were compared, through illustrative charts, and discussed.

The method was capable of predicting the sequence of yielding and failure of structural components and the progress of the overall capacity curve of the structure, thus verifying the adequacy of the seismic load. In addition, the pushover method can evaluate the performance level of reinforced concrete buildings subjected to seismic loading.

Introduction

1. Introduction

An earthquake, which is a sudden and rapid shaking of the earth caused primarily by plate tectonics, is one of the most devastating natural hazards that cause great loss in life and property. More than 10,000 people perish each year due to earthquakes and the economic losses estimated for the period 1929-1950 are in excess of $10 billion.(A.S.Elnashi) .

In the past few years, the earthquake engineering community has been reassessing its procedures, in the wake of two most damaging earthquakes which caused extensive damage, loss of life and property (Northridge, California, 17 January 1994; $20 billion and 34 dead; Hyogo-ken Nanbu, Japan, $150 billion and 6000 dead). Taking into account the short duration of earthquakes (averaging about 10-30 seconds), the amount of energy released per second must be very large compared to other forms of natural hazards. The recent earthquake-resistant design philosophies aim at producing structures that can withstand a certain level of ground shaking without excessive damage.

Generally, four distinct analytical procedures can be used for systematic rehabilitation of structures (FEMA-273, 1997): Linear Static, Linear Dynamic, Nonlinear Static (Pushover) and Nonlinear Dynamic Procedures (NDP). Linear- elastic procedures (linear static and linear dynamic) are the most common procedures in seismic analysis and design of structures due to their simplicity. Such procedures are efficient as long as the structure behaves within elastic limits. If the structure responds beyond the elastic limit, linear analyses may indicate the location of first yielding but cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. On the other hand, Nonlinear (static and dynamic) procedures are the solutions that can overcome this problem and show the performance level of structures at any loading level. These procedure help demonstrate how structures work by identifying modes of failure and the potential for progressive collapse. Nonlinear procedures help engineers to understand how a structure will behave when subjected to major earthquakes.The four procedures are described in more detail in the next sections.

2. Linear Static Procedure (LSP)

Under this procedure, design seismic forces, their distribution over the structure, and the corresponding internal forces and system displacements are determined using a linear-elastic static analysis. This procedure may give sufficiently accurate results when the structure is expected to respond elastically to ground shaking; in other words, when the ductility demands on the structure are suitably low. This procedure is not recommended for irregular structures. FEMA-273 (1997) listed a method to determine the applicability of this procedure using a demand capacity ratio (DCR). "If all of the computed DCRs for a component are less than or equal to 1.0, then the component is expected to respond elastically to earthquake ground shaking being evaluated" FEMA-273 . "If the DCRs computed for all of the critical actions of all components of the primary elements are less than 2.0, then the linear procedures are applicable"FEMA-273. Regarding the natural period of the structure and distribution of lateral forces, different codes and guidelines propose different methods to estimate them using empirical formulas.

3. Linear Dynamic Procedure (LDP)

Under this procedure, design seismic forces, their distribution over the structure, and the corresponding internal forces and system displacements are the same as the LSP but it has improvement in that it include effect the higher modes on the response of the structure . The main difference between this procedure and the LSP is that the response calculations are carried out using either modal spectral 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. In case of multi-storey buildings or other cases when higher modes play a significant role on the response of the structure , this procedure is the only elastic procedure allowed if the structure is believed to respond elastically to earthquake ground shaking. The requirement that all significant modes to be included in the response analysis may be satisfied by including sufficient modes to capture at least 90% of the participating mass of the structure in each of the principal horizontal directions.

4. Nonlinear Static Procedure (NSP)

This procedure, often called "Pushover analysis" implements simplified nonlinear techniques to estimate seismic structural deformations and forces . It can be used to estimate the dynamic demands imposed on structures by earthquake ground motion. A static lateral load, which is distributed approximately equivalent to the distribution of seismic loads generated by an earthquake, is applied to the structure. The structure is then displaced (pushed over) incrementally to the level of deformation expected during the earthquake (target displacement). Base shear and corresponding displacement at each displacement stage are used to build the pushover curve. The nonlinear load-deformation characteristics of individual components and elements of the structure are considered in the model to account for the possibility of exceeding elastic limits. Nonlinear procedures may be used for any structure. These procedures are especially recommended for analysis of buildings having irregularities (FEMA-373, 1997). NSP should not be used for structures in which higher mode effects are significant unless an LDP evaluation is also performed to capture the effect of higher modes. Since the main objective of this study is to apply this procedure to buildings and evaluate its validity, principles of the NSP will be discussed in some detail in the next chapters.

5. Nonlinear Dynamic Procedure (NDP)

This procedure is commonly known as nonlinear time history analysis. It is the most accurate procedure to represent earthquake effect. This procedure is suitable for any structure except for wood frame structures (FEMA-273, 1997). The main difference between this procedure and the NSP is the force input. The input in this procedure is an earthquake record in the form of time vs. ground acceleration that is applied at the base

of the structure. The response of the structure is computed (incrementally ,) and the stresses and deformations obtained in a pervious step are considered as initial conditions for the next step. Also, the design displacement is not established using a target displacement, instead displacements are determined directly through dynamic analysis using a specific ground motion time history. Because material inelastic response is considered directly in the model during analysis, the calculated internal forces will be a reasonable approximation of those expected during earthquake. The main disadvantage of this method its high cost. Due to uncertainty in the earthquake records, more than are time-history record should be used which increase the cost .According to FEMA-273(1997), at least 3 time history records should be performed to take care of the uncertainty in the time-history records. If three time histories analysis are performed, the maximum response of the parameter of interest shall be used for design or evaluation. If seven records or more are used for time history analysis, the average response of the parameter of interest may be considered FEMA-273. Also, a special computer program with nonlinear material and hysteretic models is required to perform this type of seismic analysis.

6. Literature Review

It is instructive to review recent work on pushover analysis, as applied only in earthquake analysis of structures. This is undertaken below in chronological order, followed by a more detailed discussion of some selected papers of more pertinence to the current work.

The use of inelastic static analysis in earthquake engineering dates back to the work of Gulkan and Sozen (1974) or earlier, where a single degree of freedom system is derived to represent equivalently the multi-degree of freedom structure. The load-displacement curve of this substitute to the real structure is evaluated by either finite element analysis or hand calculation to obtain the initial and post-yield stiffness, the yield strength and the ultimate strength. Simplified inelastic analysis procedures for multi-degree of freedom systems have also been proposed by Saiidi and Sozen (1981) and Fajfar and Fischinger (1988). Therefore, pushover analysis per se is not a recent development. However, this review is concerned with multi-degree of freedom inelastic analysis of complex structures, which is relatively recent.

Several publications discussed the advantages and disadvantages of pushover analysis, with varying degrees of success. Lawson, Vance and Krawinkler (1994) discussed in some detail the range of applicability, the expected realism for various structural systems and the difficulties encountered in pushover analysis .

Attempts for improving the procedure have been made, with varying degrees of rigor and success. The simplest and most pragmatic of which is the work of Sasaki et al (1996). This work can running several pushover analyses under force vectors representing the various modes deemed to be excited in the dynamic response. If the individual pushover curves, converted to spectral displacement-spectral acceleration space using the dynamic characteristics of the individual modes, are plotted alongside the composite spectra, it becomes apparent which mode would be the cause of more damage and where is the damage will likely occur. The procedure is intuitive, and does indeed identify potential problems that conventional single mode pushover analysis fails to point out. It, however, falls short of the work of Bracci et al (1997), which is the most recent in-depth study of pushover analysis, and is therefore reviewed in greater detail herein.

An adaptive procedure is described in the paper by Bracci et al (1997), and attributed to a previous publication by Reinhorn and Vladescu. This comprises starting the analysis assuming a certain force distribution, usually triangular. Loads imposed in subsequent increments are calculated from the instantaneous story resistance and the base shear in the previous step.

The aforementioned paper by Lawson et al (1994) is recalled with a view to clarifying the existing obstacles towards refinement of the performance of static inelastic analysis. The authors stated that the method '"has no theoretical background and will provide approximate information at best".' It is further explained that 'the issue of seismic design evaluation has little to do with accuracy, since no two earthquakes are alike'. This statement is accurate and revealing, since a procedure that takes the earthquake characteristics into account would clearly be very attractive. Notwithstanding the generalisations in the paper, four steel structures subjected to seven earthquakes were studied dynamically as well as analysed statically using DRAIN 2DX. The results gave very good correlation between static and dynamic response for the 2 storey structure, adequate correlation for the 5 storey case and completely unacceptable comparisons for the 10 and 15 storey building frames. It is surprising that the force distribution based on a square root of sum of squares (SRSS) including spectral ordinates resulted in exceptionally poor results. Estimates of normalised lateral displacements differed by more than 350% between static and dynamic analysis for the SRSS load distribution. The authors attributed this to the method leading to over-representation of higher modes. This is conceivably the reason for the low levels of structural strength observed when using IDARC with the fully adaptive procedure mentioned above.

An adaptive procedure also discusses the important issue of the roof displacement at which assessment of the dynamic response is mapped by the static analysis. The procedure proposed by Qi and Moehle (1991) and Miranda (1991) to construct a SDOF system that may replace the MDOF in dynamic analysis is recalled as one option to evaluate the target top displacement. A simple form is also proposed, whereby the elastic displacement of the MDOF system is calculated from its fundamental period and the spectral ordinate corresponding to it. Comparisons quoted between the two methods vary by 18%, 6%, 5% and 6% for the 2, 5, 10 and 15 storey structures, respectively. However, there was no pattern as to which is consistently more or less conservative. In

spite of the latter point, it seems that reasonable estimates of the target displacement are achievable, hence this issue is not discussed further in this paper.

Three questions were posed by the authors, these are: (i) to what extent does pushover analysis simulate dynamic analysis? (ii) how sensitive are the results to characteristics of the ground motion and the structural model? and (iii) is roof displacement an adequate control parameter for assessment and at what level should comparisons be undertaken? The response to these three questions sums the state of development of the method and its potential to augment, or even replace, inelastic dynamic analysis. The three questions were used at the end of this study to gauge the significance or otherwise of the developments presented in the current work.

. There is clearly further developments to address the following problems:

- Combining pushover, conventional or advanced, with fibre models where no prior assumptions are made on the behaviour of the member, and where the moment-curvature response is derived from the material characterisation.

- Fully adaptive pushover analysis that takes into account both the current level of local resistance and higher mode contributions.
- Inclusion in the updating process of the load vector a measure of relative spectral amplification corresponding to current periods of vibration.
- Investigating the most realistic and the most stable approaches for updating the applied actions shape vector in adaptive pushover.
- Potential for including more features in pushover analysis that renders it closer to time-domain inelastic dynamic analysis, such as earthquake duration and features peculiar to near-source earthquake records.

7. Research Significance

Under the pressure of recent developments, seismic codes have begun to explicitly require the identification of sources of inelasticity in structural response, together with the quantification of their energy absorption capacity. Ideally, such performance evaluation of structural systems subjected to earthquake loading should be based on nonlinear time history analysis. However, the intrinsic complexity and the additional computational effort required by the later do not justify its use in ordinary engineering applications. As a result of the above, nonlinear static, as opposed to dynamic, pushover analysis has been gaining significance over recent years as a tool for design verification. Indeed, and despite its simplicity and ease of use, this numerical tool can provide information on many important response characteristics that cannot be obtained from an elastic static or dynamic analysis.

8. Research Organization

This study consists of five chapters. The first chapter is an introduction to the four different methods used in seismic analysis of structures, particularly, the nonlinear static or pushover method that constitutes the body of the research. In chapter Two, a theoretical background for the nonlinear static procedure is provided and the two methods available for conducting a pushover analysis are reviewed. These are: (1) Capacity Spectrum Method (CSM) and (2) Displacement Coefficient Method (DCM). In addition, structural performance levels, modeling rules and acceptance criteria for the nonlinear static procedures are also given in chapter Two. Chapter three describes the case study of a three dimensional model for which the seismic analysis, using the pushover method and the three other methods, will be conducted as well as the research methodology. Results of analysis are given, discussed and compared in chapter four whereas a summary of the study, conclusions and recommendations are provided in chapter five.

Nonlinear Static Procedure

1. Introduction

This chapter is intended to give a background about the Nonlinear Static Procedure (NSP), which is presented by FEMA-273 (1997) as a procedure that can be used to perform systematic rehabilitation of structures. The NSP in ATC (1996) and FEMA-273 (1997) is based on the Capacity Spectrum Method which was originally developed by Freeman et al. (1975) and Freeman (1978). Simplified nonlinear analysis procedures implement the pushover analysis methods such as Capacity Spectrum Method (CSM) (ATC, 1996) and Displacement Coefficient Method (DCM) (FEMA-273, 1997); these methods will be briefly presented in this chapter. Since pushover analysis is essential for NSP, its theoretical background is presented next.

2. Theoretical Background for Pushover Analysis

The static pushover analysis has no particularly accurate theoretical background. It is based on the argument that the response of a Multi Degree of Freedom (MDF) structure is essentially governed by a single model that remains constant throughout the time history analysis (Dutta, 1999). The governing equation of motion for a linear MDF system to horizontal earthquake motion (single excitation) $\ddot{q}_g(t)$ is:

$$m\{\ddot{q}\} + c\{\dot{q}\} + k\{q\} = -m\{\iota\}\{\ddot{q}_g\}(t) \qquad (1)$$

Where m, c and k are the mass, classical damping and lateral stiffness matrices of the system, respectively, and $\{t\}$ is a vector of ones corresponding to translational DOFs in the direction under consideration and zeros corresponding to rotational DOFs. In the modal analysis approach, which is of course for linear static systems, the displacement vector relative to ground q is represented as a truncated series in the form of a coordinate transformation. Specifically, q is written as the product of the mode shape matrix ф and a vector of generalized modal coordinates p:

$$q(t) = \sum_{r=1}^{N} \phi_r p_r(t) = \phi p(t) \tag{2}$$

where:

$$\phi = \left| \ \phi_1 \phi_2 \dots\dots\dots \phi_m \dots\dots\dots \phi_N \ \right| \tag{3}$$

$$\mathrm{P} = [p_1(t)\ p_2(t) \dots p_m(t) \dots p_N(t)]^T \tag{4}$$

where: $\phi_1 \phi_2 \dots \phi_m \dots \phi_N$ are N mode shape vectors and $p_1(t)\ p_2(t) \dots p_m(t) \dots p_N(t)$ are N modal coordinates. By substituting Eq (2) into equation (1), it can be rewritten as:

$$\sum_{r=1}^{N} m\phi_r \ddot{p}_r(t) + \sum_{r=1}^{N} c\phi_r \dot{P}_r(t) + \sum_{r=1}^{N} K\phi_r p_r(t) = -m\{t\}\ddot{q}_g(t) \tag{5}$$

Premultiplying each term in this equation by ϕ_n^T gives:

$$\sum_{r=1}^{N} \phi_n^T m\phi_r \ddot{p}_r(t) + \sum_{r=1}^{N} \phi_n^T c\phi_r \dot{P}_r(t) + \sum_{r=1}^{N} \phi_n^T K\phi_r p_r(t) = -\phi_n^T m\{t\}\ddot{q}_g(t) \tag{6}$$

Because of the orthogonality relationships, all terms in each of the summations vanish, except the $r = n$ term, reducing this equation to:

$$\left(\phi_n^T m \phi_n\right)\ddot{p}_n(t) + \left(\phi_n^T c \phi_n\right)\dot{p}_n(t) + \left(\phi_n^T k \phi_n\right)p_n(t) = -\left(\phi_n^T m\{t\}\ddot{q}_g(t)\right) \qquad (7)$$

The above equation can be rewritten as:

$$\left(M_n\right)\ddot{p}_n(t) + \left(C_n\right)\dot{p}_n(t) + \left(K_n\right)p_n(t) = -\left(L_n \ddot{q}_g(t)\right)$$

The above equation is in a form similar to the equation of motion for SDF system considering the nth mode only and it can be rewritten as:

$$\ddot{p}_n(t) + 2\xi_n\omega_n \dot{p}_n(t) + \omega_n^2 p_n(t) = -\left(\Gamma_n \ddot{q}_g(t)\right) \qquad (9)$$

where: $\phi_n^T m \phi_n = M_n$ = generalized nth modal mass, ξ_n =generalized nth modal damping, ω_n = generalized nth modal frequency, L_n =nth modal excitation factor, and $\Gamma_n = \left(L_n / M_n\right)$. Eq (9) is the standard modal equation (Chopra and Goal, 2001). The right side of Eq. (1) can be interpreted as effective inertia forces resulting from earthquakes excitation:

$$Q_{eff}(t) = -m\{t\}\ddot{q}_g(t) \qquad (10)$$

The spatial distribution of these forces over the structure is defined by the vector $s = m\{t\}$ and the time variation $\ddot{q}_g(t)$. The contribution of the nth mode to s and $Q_{eff}(t)$ are:

$$s_n = \Gamma_n m \phi_n \qquad \text{and} \qquad Q_{eff,n}(t) = -s_n \ddot{q}(t) \qquad (11)$$

For linear systems, the response of the MDF to $Q_{eff,\mathrm{n}}(t)$ is entirely in the nth mode, with no contribution from other modes.

The solution to Eq. (9) can be obtained by comparing this equation to the equation of motion for an elastic SDF system possessing the following vibration properties: natural frequency ω_n and damping ratio $\xi_{n,}$ both for the nth mode of the MDF system. If the system is subjected to $\ddot{u}_g(t) = \ddot{q}_g(t)$, equation of motion will be:

$$\ddot{u}_n(t) + 2\xi_n\omega_n \dot{u}_g(t) + \omega^2{}_n u_n(t) = -\ddot{u}_g(t) \qquad (12)$$

Comparing Eqs. (9) and (12) gives:

$$p_n(t) = \Gamma_n u_n(t) \qquad (13)$$

And substituting Eq. (13) in (2) gives the floor displacements due to nth mode:

$$q_n(t) = \Gamma_n \phi_n u_n(t) \qquad (14)$$

Any response quantity r (t) such as story drift, internal element forces, etc., can be represented as (Chopra and Goel, 2001):

$$r_n(t) = r_n{}^{st} A_n(t) \qquad (15)$$

Where $= r_n{}^{st}$ denotes the modal static response, the static value of r due to the external forces s_n and $A_n(t) = (\omega_n{}^2\, u_n(t)/g)$ is the pseudo-acceleration response of the nth-mode SDF system (Chopra, 2001).

Equations (14) and (15) represent the response of the MDF system to $Q_{eff,\,\mathrm{n}}(t)$. Therefore, the response of the system to total excitation $Q_{eff}(t)$. is:

$$q(t) = \sum_{n=1}^{m} q_n(t) = \sum_{n=1}^{m} \Gamma_n \phi_n u_n(t) \qquad (16)$$

$$r(t) = \sum_{n=1}^{m} r_n(t) = \sum_{n=1}^{m} r_n{}^{st} A_n(t) \qquad (17)$$

The first m modes are considered in the above equations m << N. Equations (14) and (15) define the contribution of the nth-mode to the response, and Eqs. (16) and (17) reflect combining the response contributions of m modes. However the modal expansion of the spatial distribution of the effective earthquake forces, s_n, was used in the derivation of these standard equations, which provides a rational basis for the modal pushover analysis procedure (Chopra and Goel, 2001).

The peak value r_o of the total response r (t) can be estimated directly from the response spectrum for the ground motion. In the response spectrum analysis (RSA) the peak value r_{no} of nth-mode contribution $r_n(t)$ to response $r(t)$ is determined from:

$$r_{no} = r_n^{st} A_n \qquad (18)$$

Where A_n is the ordinate $A(T_{n,}\xi_n)$ of the pseudo-acceleration response or (design) spectrum for the nth-mode SDF system, and $T_n = 2\pi / \omega_n$ is the natural vibration period of the nth-mode of the MDF system. The modal peak responses are combined according to the Square-Root-of-Sum-of-Squares (SRSS) or the Complete Quadratic Combination (CQC) rules. The SRSS rule provides an estimate of the peak value of the total response:

$$r_0 = \left(\sum_{n=1}^{m} r_{no}^2 \right)^{\frac{1}{2}} \qquad (19)$$

It can be noticed that static analysis of the structure subjected to lateral forces

$$f_{no} = \Gamma_n m \phi_n A_n \qquad (20)$$

will provide the same value of $r_{no,}$ the peak nth-mode response as in Eq. (18) (Chopra, 2001). Alternatively, this response value can be obtained by static analysis of the structure subjected to lateral forces distributed over the structure according to:

$$s_n^* = m\phi_n \tag{21}$$

and the structure is pushed until the roof displacement reaches q_{rno}, the peak value of the roof displacement (or the control displacement in a more general term) due to the nth-mode, which from Eq. (14) is:

$$q_{rno} = \Gamma_n \phi_{rn} u_n \tag{22}$$

Where $u_n = {A_n g}/{\omega_n^2}$. Obviously, u_n and A_n are available from the response (or design) spectrum (Chopra and Goel, 2001).

The peak modal responses, r_{no}, each determined by a single pushover analysis, can be combined according to Eq. (19) to obtain an estimate of the peak value r_o of the total response. This is the basis for the modal pushover analysis (MPA), which was developed by Chopra and Goel (2001). It is obvious the (MPA) for linearly elastic systems is equivalent to the well-known response spectrum analysis (RSA).

A similar approach with few differences was used to derive the basis for pushover with capacity spectrum method (CSM) (Dutta, 1999). From Eq. (13), which is repeated here for convenience:

$$p_n(t) = \Gamma_n u_n(t) \tag{13}$$

The maximum $p_{mo,}$ which is the response due to the mth mode can be written as:

$$p_{mo} = \Gamma_m S_d(\omega_{m,} \xi_m) \tag{23}$$

Where $S_d(\omega_{m,}\xi_m)$ = spectral displacement corresponding to damping ξ_m and natural frequency $\omega_{m.}$. Multiplying both sides of the above equation by ϕ_{nm} (the magnitude of the mth –mode at the nth location) yields:

$$q_{nm} = \phi_{nm} p_{mo} = \phi_{nm} \Gamma_m S_d(\omega_{m,} \xi_m) \tag{24}$$

Where q_{nm} = displacement at the nth location due to mth mode shape. It is obvious that the above equation is similar to Eq. (22). Using Eq. (24) the spectral displacement can be solved as:

$$S_d = \frac{q_{nm}}{\phi_{nm}\Gamma_m} = \frac{q_{nm}}{PF_1\phi_{nm}} = \Delta^* \tag{25}$$

This can also be defined as the effective displacement $(\Delta^* = S_d)$ where:

$$PF_1 = \frac{\sum_{i=1}^{N} w_i \phi_i}{\sum_{i=1}^{N} w_i \phi_{im}^2} \tag{26}$$

Where w_i = tributary weight at the location i varying from 1 to N being the total number of discrete weight for pushover mode shape locations.

The maximum base shear for the mth mode can be from the force vector:

$$F_m = \Gamma_m m \phi_m S_a(\omega_m, \xi_m) \tag{27}$$

Where $S_a(\omega_m, \xi_m)$ = spectral acceleration corresponding to damping ξ_m and frequency ω_m. The base shear capacity can obtained adding all the terms of the force vector. Thus:

$$V = \sum_{i=1}^{N} F_{im} = \alpha_1 S_a W \qquad (28)$$

Where S_a in the above equation is the normalized spectral acceleration. This can be used to define normalized base shear capacity as follows (ATC, 1996; Duuta, 1999):

$$C_C^* = S_a = \frac{V/W}{\alpha_1} \qquad (29)$$

Where:

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{im})/g\right]^2}{\left[\sum_{i=1}^{N} w_i/g\right]\left[\sum_{i=1}^{N} (w_i \phi_{im}^2)/g\right]} \qquad (30)$$

And $\sum_{i=1}^{N} w_i = W =$ total weight considered for seismic effect.

From the above formulation it is clear that given the base shear vs. displacement at any location in a MDF system subjected to any arbitrary chosen distribution of lateral (demand) forces, it is possible to convert them to Δ^* and C_C^* capacity as a comparison to the S_a vs. S_d demand format by using Eqs. (25) and (29), respectively. Creating a pushover curve includes applying the push force (or lateral displacement) incrementally to build the base shear displacement curve which can be converted to Δ^* vs. C_C^* curve. Intersection of this curve with the S_a vs. S_d response spectrum (the conventional response spectrum, S_a vs. T can be converted to acceleration-displacement response spectrum, S_a vs. S_d) is the performance point which gives the demand displacement.

It also becomes clear that the pushover mode acts like a SDF system if PF and α are assumed to be equal to unity in Eqs. (25) and (29). In this case the shear force vs.

displacement curve can be used synonymously with the spectral acceleration vs. displacement (S_a vs. S_d).

3. Nonlinear Static Procedures

3.1 Capacity Spectrum Method, CSM

Applied Technology Council (ATC-1996) presented a nonlinear procedure to evaluate performance of reinforced concrete buildings subjected to seismic loading. This procedure uses the static pushover analysis to:

1. Represent the structure's lateral force resisting capacity.
2. Determine the displacement demand produced by the earthquake intensity on the structure.
3. Verify an acceptable performance level.

In general, performance of the structure is accepted when the structural capacity is larger than the demand required to satisfy a proper performance level.

ATC (1996) adopts the Capacity Spectrum Methods (CSM) to determine the demand displacement, which is the maximum expected response of a building during a ground motion. The demand displacement in the CSM occurs at the point on the capacity (pushover) curve called the performance point. This performance point represents the condition for which the seismic capacity of the structure is equal to seismic demand imposed on the structure by the specified ground motion. Determination of the performance point requires a trial and error procedure.

ATC (1996) presented the CSM in detail and explained through a step-by-step procedure how to apply this method. The main steps can be briefed as:

1. Develop the pushover (capacity) curve, which represents the relationship between the base shear V and the roof displacement(δ), Figure 1. (Roof is the control node in the case of buildings). Using nonlinear computer programs, pushover curve can be built with no iteration, when a linear computer program is used, developing the pushover curve requires iteration and many steps.

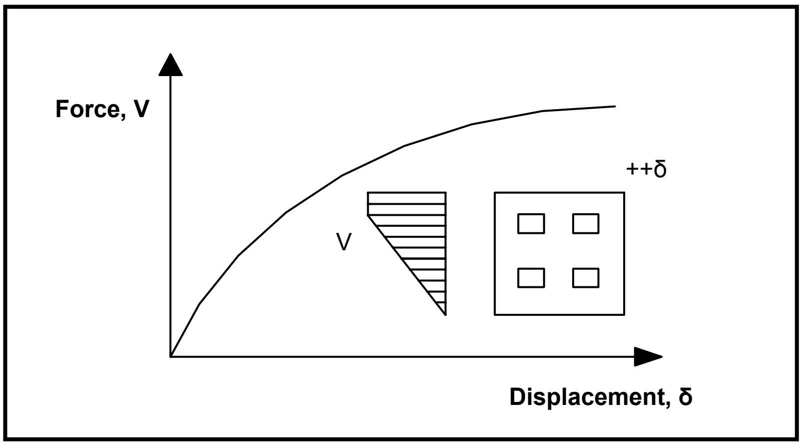


Figure 1. Pushover (capacity) curve: Base shear vs. roof displacement

2. Convert the pushover curve to the capacity spectrum curve using the equations: (ATC-96) chapter-8

$$S_{ai} = \frac{v_i / w}{\alpha_1} \qquad\qquad S_{di} = \frac{\Delta_{roof}}{\left(PF_1 * \phi_{1,roof}\right)}$$

See Figure 2.

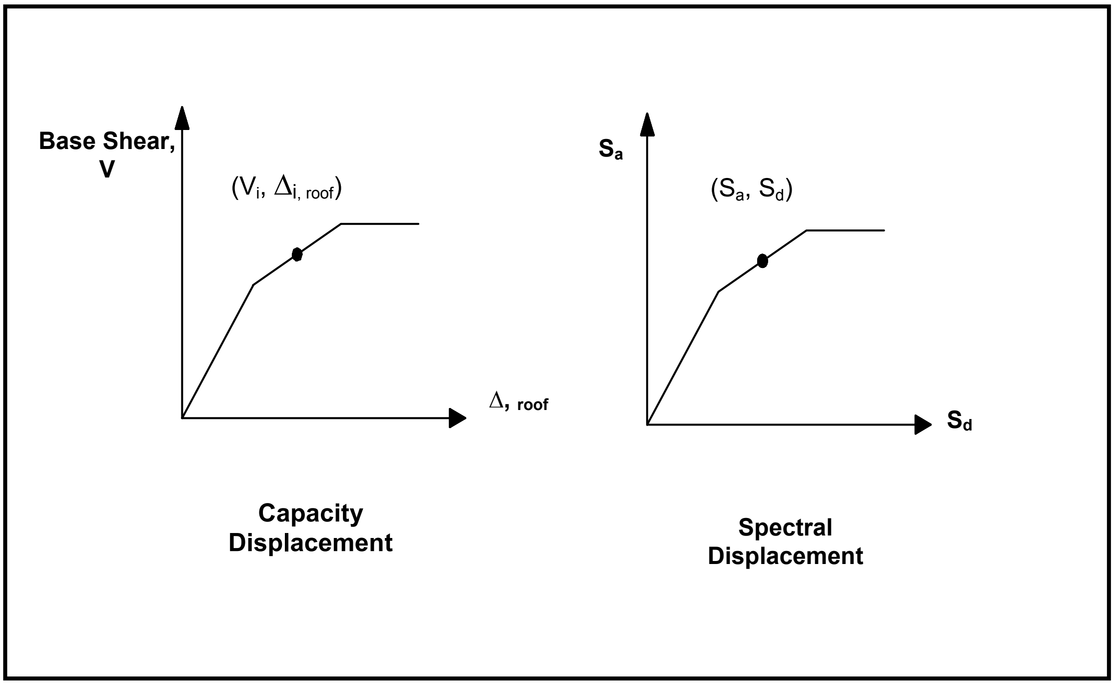


Figure 2. Conversion of pushover curve to capacity spectrum curve

3. Convert the elastic response spectrum from the standard format S_a *vs.* T to Acceleration Displacement Response Spectrum (ADRS) format S_a *vs.* S_d .

4. Determine the displacement demand as the intersection of the capacity spectrum curve and the spectral demand curve, reduced from elastic 5-percent-damped design spectrum. See Figure 3.

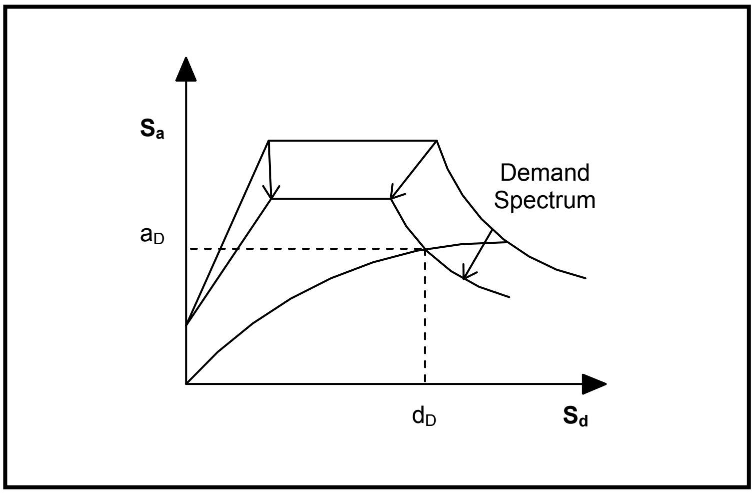


Figure 3. Intersection of the capacity spectrum curve and the spectral demand curve defines the displacement demand

The point of intersection represents the nonlinear demand at the same structural displacement. This step needs iterations. Each iteration includes calculating updates values of the natural period T_{eq} and the effective damping β_{eff}. An approximately effective damping is calculated based on the shape of the capacity curve, the estimated displacement demand and the resulting hysteretic loop. See Figure 4.

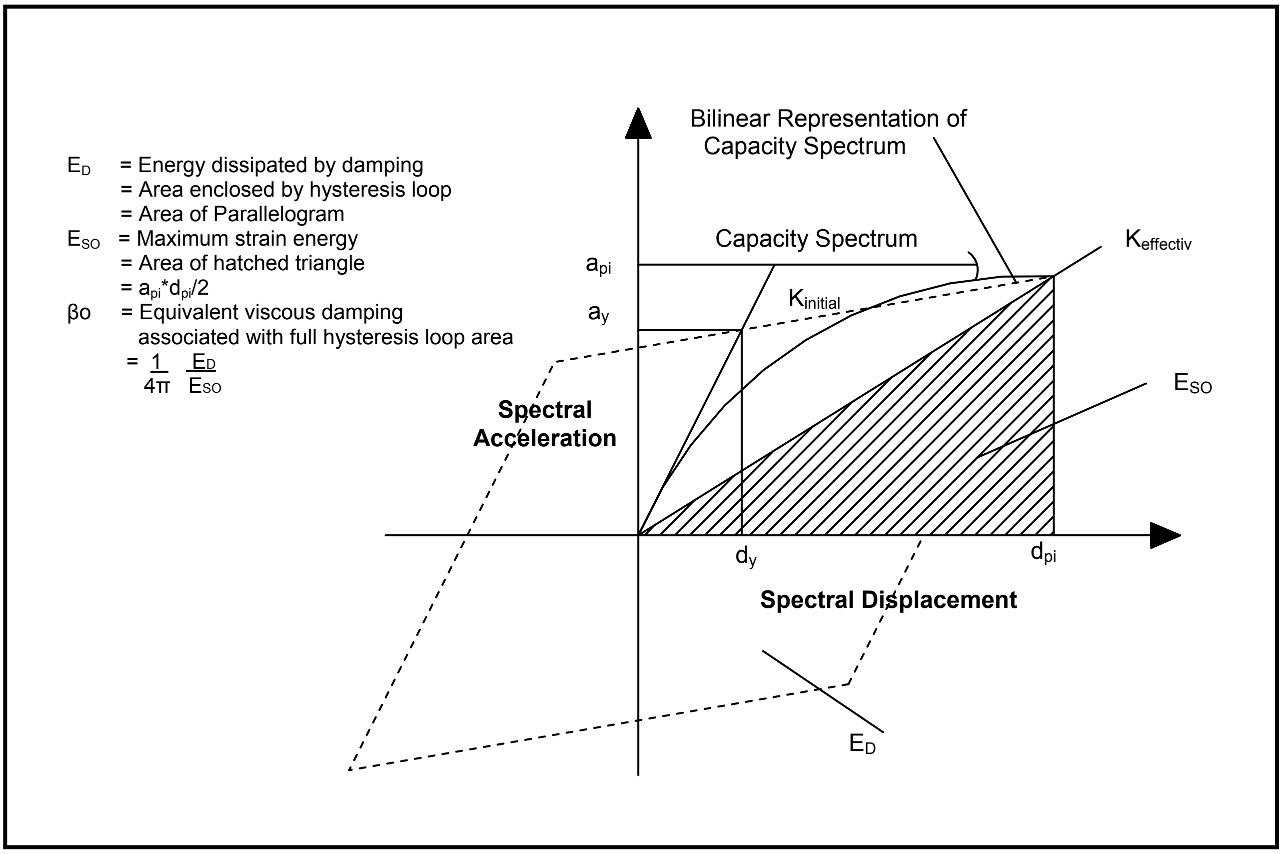


Figure 4. Estimation of the effective damping, β_{eff} .(ATC-96 ,chapter 8)

5. Convert the displacement demand determined in the previous step back to global roof displacement.

6. Evaluate the deformations of individual components corresponding to demand displacement with the capacity of that component. In general, if the deformation demand in deformation-controlled components exceeds permissible values, then the component is deemed to violate the performance criteria.

3.1.1 Procedures to Estimate Demand Displacement

ATC (1996) proposed three procedures called A, B and C to determine the demand displacement. Procedure C is a graphical one appropriate for hand analysis. Procedure B is an analytical one and it is the best for spread sheet programming. Procedure A is characterized as the clearest and most direct application of the methodology and it is convenient for spreadsheet programming.

3.1.2 Estimation of Damping

Estimation of equivalent viscous damping is performed by representing the hysteretic damping as equivalent viscous damping. For the case where the capacity curve is replaced by bilinear curve as shown in Figure 4, equivalent viscous damping β_0 can be calculated as (Priestly et al 1994…Chopra, 2001):

$$\beta_0 = \frac{1}{4\pi} \frac{E_D}{E_{SO}} \tag{31}$$

Where E_D and E_{so} are shown in Figure 4 and β_0 is the equivalent viscous damping. The effective damping β_{eff} associated with maximum displacement can be written as:

$$\beta_{eff} = k\beta_0 + 0.05 = \frac{0.637k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 0.05 \tag{32}$$

Where 0.05 is the viscous damping inherent in the structure (assumed to be constant), k factor is discussed below, and the rest of symbols are shown in Figure 4.

The k factor depends on the structural behavior of the building, which in turn depends on the quality of the seismic resisting system and the duration of ground shaking. This factor is a measure of the extent to which the actual structure hysteresis is well represented by the parallelogram of Figure 4 either initially or after degradation. ATC (1996) simulates three categories of structural behavior. Structural type A represents stable, reasonably full hysteretic loops most similar to Figure 4. A k factor of 1 is assigned for behavior type A, except at higher damping values. Type B is assigned a k of 0.67 (except for higher damping values). It represents a moderate reduction area. Type C represents poor hysteretic behavior with a substantial reduction of loop area (severely pinched) and assigned a k of 0.33. Table 1 presents values for damping Modification Factor, k .

Table 1. Values for damping modification factor, k .(ATC_96)

Structural Behavior Type	β_0 (percent)	k
Type A	≤ 16.25 > 16.25	1.0
Type B	≤ 25 > 25	0.67
Type C	> 25	0.33

3.2 Displacement Coefficient Method, DCM

FEMA-273 (1997) and FEMA--356 (1997) presented the FEMA-273 Guidelines for Seismic Rehabilitation of buildings. In these guidelines, a nonlinear static procedure is presented as a simplified and efficient procedure to evaluate seismic nonlinear response of buildings. DCM uses pushover analysis and modified version of the equal displacement approximation to estimate maximum displacement demand since it implements some coefficients to modify the elastic displacement. Under the pushover analysis, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformation and forces are determined.

The nonlinear load-deformation characteristics of individual components and elements of the structure are modeled directly. The mathematical model of the structure is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced when the structure is subjected to the considered earthquake intensity. This target displacement may be calculated by any procedure that accounts for the effects of the nonlinear response on displacement amplitude. FEMA-356 (1997) represented a procedure that can be used to calculate the target displacement for buildings. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable of those expected during the design earthquake.

For structures that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions; then the maximum forces and deformations should be used for design. The analysis model shall be discretized in sufficient detail to represent adequately the load deformation response of each component along its length. Particular attention shall be paid to identify locations of inelastic action along the length of the components, as well as its ends.

The NSP requires defining the control point in the structure. FEMA -273 (1997) considers the control node to be the center of mass of the building. The displacement of the control node is compared with the target displacement, which characterizes the effects of earthquake ground shaking.

3.2.1 Lateral Load Patterns

Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. At least two vertical distributions of lateral load along with building height shall be considered according to the FEMA-273 (1997) procedure. The first vertical distribution should be the uniform pattern and the second one should be selected from one the other two patterns:

1. Uniform pattern: This load pattern is based on lateral forces that are proportional to the total mass at each floor level. $S_j^* = m_j$ (where the floor number J=1,2,……)
2. Equivalent lateral force (ELF) patterns: $S_j = m_j h_j^k$ where h_j is the height of the j th floor above the base, and the exponent $k = 1$ for fundamental period $T_i \le 0.5\,\text{sec}$, $k = 2$ for $T_1 \ge 2.5\,\text{sec}$; and varies linearly in between. This pattern may be used if more than 75% of the total mass participates in the fundamental mode in the direction under consideration.
3. SRSS distribution: S^* is defined by the lateral force back-calculated from the story shears determined by response spectrum analysis of the structure (including a sufficient number of modes to capture 90% of the total mass), assumed to be linearly elastic. The appropriate ground motion spectrum should be used for the response spectrum analysis.

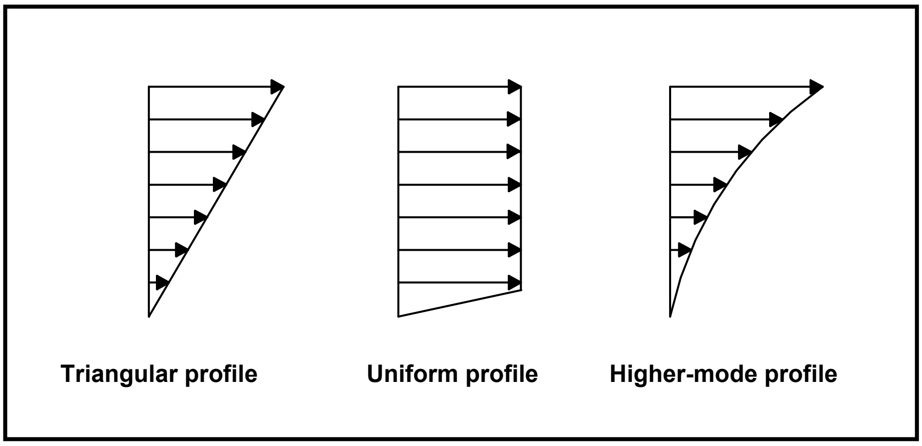


Figure 5. Uniform pattern ,Equivalent pattern, SRSS pattern,

3.2.2 Estimation of Target Displacement

The effective fundamental period T_e shall be calculated in the direction under consideration using the force–displacement relationship (pushover curve).The nonlinear relation between the base shear and displacement of the control point shall be replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building. The effective lateral stiffness shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the yield strength(ATC-96). The effective fundamental period T_e shall be calculated as:

$$T_e = T_i \sqrt{\frac{k_i}{k_e}} \tag{33}$$

$T_i =$ Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

$K_i =$ Elastic lateral stiffness of the building in the direction under consideration

$K_e =$ Effective lateral stiffness of the building in the direction under consideration

See Figure 6.

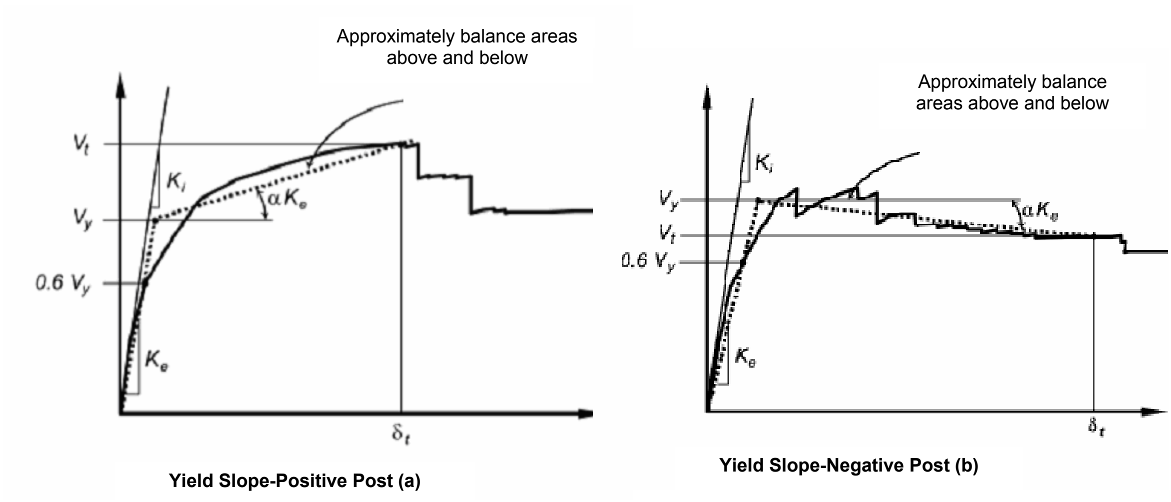


Figure 6. Effective lateral stiffness (FEMA-273)

The relation between base shear force and lateral displacement ranges between zero and 150% of the target displacement δ_t. As mentioned before, δ_t may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude. One rational procedure is presented by FEMA-273 (1997) for buildings as:

$$\delta_t = C_0 C_1 C_2 C_3 S_4 \frac{T_e^2}{4\pi^2} g \tag{34}$$

where,

T_e = Effective fundamental period of the building in the direction under consideration as given by Eq. 33.

C_0 = Modification Factor to relate spectral displacement and likely building roof displacement. C_0 can be calculated using one of the following approaches:

1. The first modal participation factor at the level of the control node.
2. The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement.
3. The appropriate value from Table 2.

Table 2. Values of modification factor, C_o^{+}(ATC-96)

	Shear Buildings *		Other Buildings
Number of Stories	Triangular Load Pattern	Uniform Load Pattern	Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

* Buildings in which, for all stories, interstorey drift decreases with increasing height.

+ Linear interpolation shall be used to calculate intermediate values.

C_1 = Modification factor to relate maximum inelastic displacement to displacement calculated for linear elastic response.

T_0 = The period on the response spectrum associated with the transition from the constant acceleration segment to the constant velocity segment.

R = Strength ratio that should be calculated as:

$$R = \frac{S_a}{\frac{v_y}{w}} \frac{1}{c_2} \tag{35}$$

where,

S_a = Response spectrum acceleration, g, at the effective fundamental period and damping ratio of the building in the direction under consideration.

C_2 = Modification factor to represent the effect of the hysteresis shape on the maximum displacement response. Values for C_2 are listed in Table 3-3 (FEMA-356, 1997) which are repeated here in Table 3 for convenience.

Table 3. Values for modification factor, C_2★.(ATC-96)

Structural Performance Level	T ≤ 0.1 seconds Framing Type 1*	T ≤ 0.1 seconds Framing Type 2+	T ≥ 0.1 seconds Framing Type 1*	T ≥ 0.1 seconds Framing Type 2+
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

* Structures to which more than 30 % of the storey shear at any level is resisted by any combination of the following components, elements or frames: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, unreinforced masonry walls,

+ All frames not assigned to framing Type 1.

★ Linear interpolation shall be used to calculate intermediate values.

C_3 = Modification factor to consider the $p-\Delta$ effect. For buildings with positive post yield stiffness, C_3 shall be calculated using Equation (2-38). Values for C_3 shall be set equal to 1.0 for buildings with negative post-yield stiffness, values of C_3 have an upper limit set in section 3.3.1.3 of Fema-273 (1997).

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{\frac{3}{2}}}{T_e} \qquad (36)$$

Where R and T_e as defined above and α is the ratio of post yield stiffness to effective elastic stiffness, as shown in Figure 6.

3.2.3 Procedures to Perform Pushover Analysis using DCM

To perform Pushover analysis using the DCM (FEMA-273, 1997), the following is a step-by-step procedure:

1. Compute the natural period of the structure for the direction under consideration using elastic dynamic analysis.
2. Define lateral load pattern from the specified three load patterns mentioned before; two patterns should be used and the following steps should be repeated for each pattern.

3. Using nonlinear analysis model, the intensity of lateral load is increased incrementally and the control node displacement corresponding to each load increment is determined to plot the pushover curve (control node displacement vs. base shear). Pushover curve shall be established for control node displacement ranging between zero and 150% of the target displacement, δ_t .
4. Idealize the pushover curve as a bilinear curve as shown in Figure 4.
5. Calculate effective period (T_e)using Eq. 33.
6. Pushover curve is used to estimate the target displacement by means of Eq. 34. This step may require iteration if the yield strength and stiffness of the simplified bilinear relation are sensitive to the target displacement.
7. Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node should be used to evaluate the performance of components and elements.
8. For deformation controlled is flexure deformation controlled actions, the deformation demands are compared with the maximum values for the component.
9. For force-controlled actions (e.g. shear in beams), the strength capacity is compared with the force demand.
10. If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components or elements, exceeds permissible values, then the action, component, or element is deemed to violate the performance criteria.

4. General Procedure to Perform Pushover Analysis

1. An elastic structural model is developed that includes all new and old components that have significant contributions to the weight, strength, stiffness, and/or stability of the structure and whose behavior is important in satisfying the desired level of seismic performance. The structure is loaded with gravity loads in the same load combination(s) as used in the linear procedures before proceeding with the application of lateral loads.

2. The structure is subjected to a set of lateral loads, using one of the load patterns (distributions) described in the (ATC-96). At least two analyses with different load patterns should be performed in each principal direction (ATC-96).

3. The intensity of the lateral load is increased until the weakest component reaches a deformation at which its stiffness changes significantly (usually the yield load or member strength). The stiffness properties of this "yielded" component in the structural model are modified to reflect post-yield behavior, and the modified structure is subjected to an increase in lateral loads (load control) or displacements (displacement control), using the same shape of the lateral load distribution or an updated shape as permitted in theATC. Modification of component behavior may be in one of the following forms:

a. Placing a hinge where a flexural element has reached its bending strength; this may be at the end of a beam, column, or base of a shear wall.

b. Eliminating the shear stiffness of a shear wall that has reached its shear strength in a particular story.

c. Eliminating a bracing element that has buckled and whose post-buckling strength decreases at a rapid rate.

d. Modifying stiffness properties if an element is capable of carrying more loads with a reduced stiffness

4. Step 3 is repeated as more and more components reach their strength. Note that although the intensity of loading is gradually increasing, the load pattern usually remains the same for all stages of the "yielded" structure, unless the user decides on the application of an adaptive load pattern (Bracci et al., 1995). At each stage, internal forces and elastic and plastic deformations of all components are calculated.

5. The forces and deformations from all previous loading stages are accumulated to obtain the total forces and deformations (elastic and plastic) of all components at all loading stages.

6. The loading process is continued until unacceptable performance is detected or a roof displacement is obtained that is larger than the maximum displacement expected in the design earthquake at the control node.

Note: Steps 3 through 6 can be performed systematically with a nonlinear computer analysis program using an event-by-event strategy or an incremental analysis with predetermined displacement increments in which iterations are performed to balance internal forces.

7. The displacement of the control node versus base shear at various loading stages is plotted as a representative nonlinear response diagram of the structure. The changes in slope of this curve are indicative of the yielding of various components.

8. The control node displacement versus base shear curve is used to estimate the target displacement. Note that this step may require iteration if the yield strength and stiffness of the simplified bilinear relation are sensitive to the target displacement.

9. Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node are used to evaluate the performance of components and elements of the structure .

a. For deformation-controlled actions (e.g., flexure in beams), the deformation demands are compared with the maximum permissible values.

b. For force-controlled actions (e.g., shear in beams), the strength capacity is compared with the force demand.

10. If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components, or elements, exceeds permissible values, then the action, component, or element is deemed to violate the performance criterion. Asymmetry of a building in the direction of lateral loading will affect the force and deformation demands in individual components. Asymmetric elements and components in a building, such as reinforced concrete shear walls with T- or L-shaped cross section, have force and deformation capacities that may vary substantially for loading in opposite directions. Accordingly, it is necessary to perform two nonlinear analyses along each axis of the building with loads applied in the positive and negative directions, unless the building is symmetric in the direction of lateral loads or the effects of asymmetry can be evaluated with confidence through judgment or auxiliary calculations.

As noted in Step 1 of the NSP, gravity loads need to be applied as initial conditions to the nonlinear procedure, and need to be maintained throughout the analysis. This is because superposition rules applicable to linear procedures do not, in general, apply to nonlinear procedures, and because the gravity loads may importantly influence the development of nonlinear response. The gravity-load combinations are the same as in the linear procedures. As noted previously, the use of more than one gravity-load combination will greatly increase the analysis effort in the NSP. It may be possible by inspection to determine that one of the two specified combinations will not be critical.

The mathematical model should be developed to be capable of identifying nonlinear action that may occur either at the component ends or along the length of the component. For example, a beam may develop a flexural plastic hinge along the span (rather than at the ends only), especially if the spans are long or the gravity loads are relatively high. In such cases, nodes should be inserted in the span of the beam to capture possible flexural yielding between the ends of the beam.

The general concepts of the Nonlinear Static Procedure method are summarized in Figure 7.

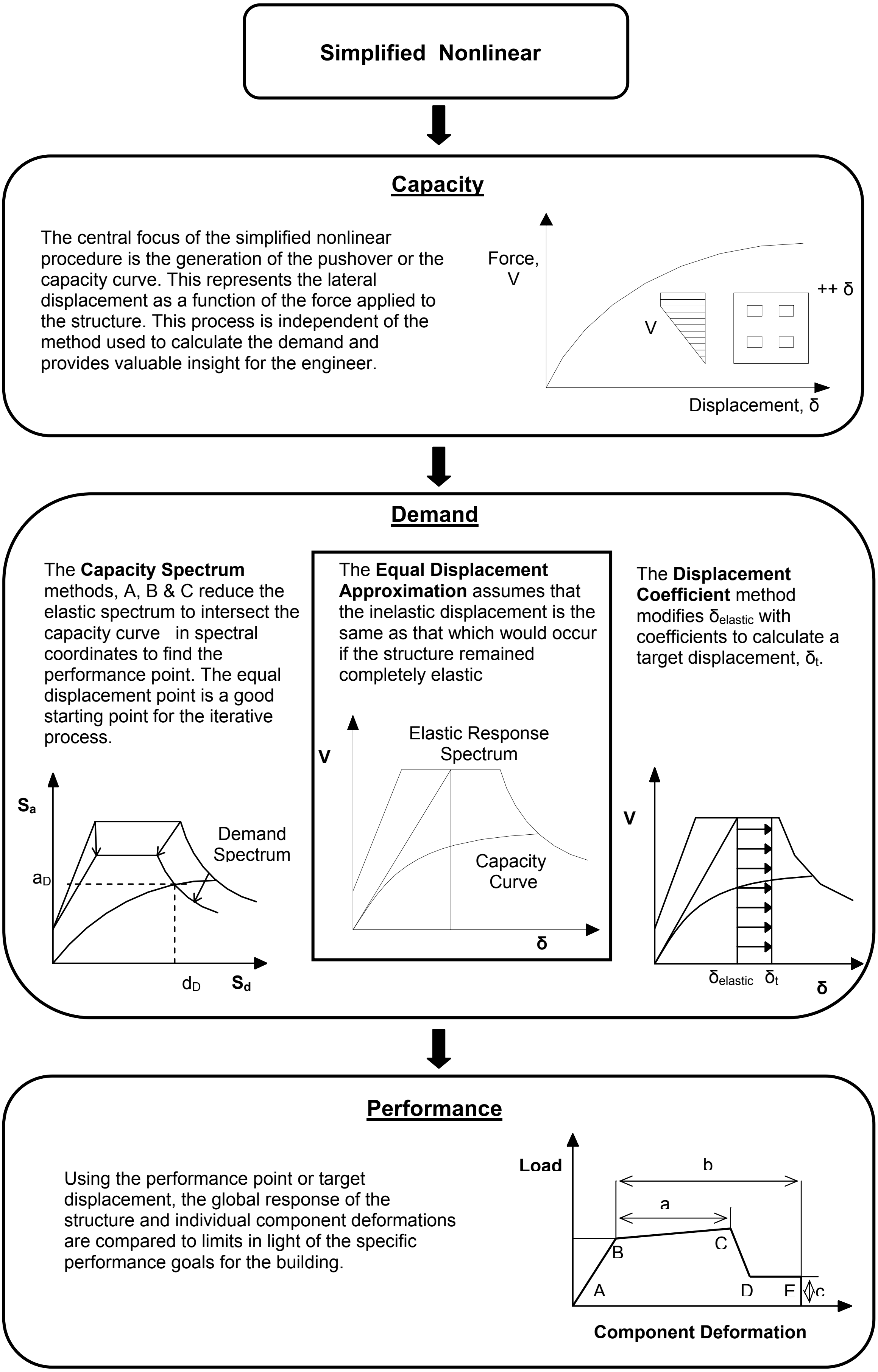


Figure 7. Nonlinear Static Procedure (ATC-96)

5. Structural Performance Levels

Performance objectives have two essential parts – a damage state and a level of seismic hazard. 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 state for several levels of ground motion and would then be termed a dual or multiple-level performance objective.

The target performance objective is split into Structural Performance Level (SP-n where n is the designated letter). These may be specified independently, however, the combination of the two determines the overall Building Performance level.

Structural Performance Levels are defined as (FEMA-273):

- **Immediate Occupancy (SP-1)**: Limited structural damage with the basic vertical and lateral force resisting system retaining most of their pre-earthquake characteristics and capacities.
- **Damage Control (SP**-2): A placeholder for a state of damage somewhere between Immediate Occupancy and Life Safety.
- **Life Safety (SP**-3): Significant damage with some margin against total or partial collapse. Injuries may occur with the risk of life-threatening injury being low. Repair may not be economically feasible.
- **Limited Safety (SP-4)**: a placeholder for a state of damage somewhere between Life Safety and Structural Stability.
- **Structural Stability (SP-5)**: Substantial Structural damage in which the structural system is on the verge of experiencing partial or total collapse.

Significant risk of injury exists. Repair may not be technically or economically feasible.

- **Non Considered (SP-6)**: Placeholder for situations where only non-structural seismic evaluation or retrofit is performed.

Non Structural Performance Levels are defined as:

- **Operational (NA-A):** Non-structural elements are generally in place and functional. Back-up systems for failure of external utilities, communications and transportation have been provided.
- **Immediate Occupancy (NP-**B): Non-structural elements are generally in place but may not be functional. No back-up systems for failure of external utilities are provided.
- **Life Safety (NP-**C): Considerable damage to non-structural components and systems but no collapse of heavy items. Secondary hazards such as breaks in high-pressure, toxic or fire suppression piping should not be present.
- **Reduced Hazards (NP-**D): Extensive damage to non-structural components but should not include collapse of large and heavy items that can cause significant injury to groups of people.
- **Not Considered (NP-**E): Non-structural elements, other than those that have an effect on structural response, are not evaluated.

Combinations of Structural and Non-structural Performance Levels to obtain Building Performance Level are shown in Table 4.

Table 4. Buildings Performance Levels(FEMA-273)

Building Performance Levels						
Non-structural Performance Levels	Structural Performance Levels					
	SP-1 Immediate Occupancy	SP-2 Damage Control (Range)	SP-3 Life Safety	SP-4 Limited Safety (Range)	SP-5 Structural Stability	SP-6 Not Considered
NP-A Operational	1-A Operational	2-A	NR	NR	NR	NR
NP-B Immediate Occupancy	1-B Immediate Occupancy	2-B	3-B	NR	NR	NR
NP-C Life Safety	1-C	2-C	3-C Life Safety	4-C	5-C	6-C
NP-D Reduced Hazards	NR	2-D	3-D	4-D	5-D	6-D
NP-E Not Considered	NR	NR	3-E	4-E	5-E Structural Stability	Not Applicable

Legend

	Commonly referenced Building Performance Levels (SP-NP)
	Other possible combinations of SP-NP
NR	Not recommended combinations of SP-NP

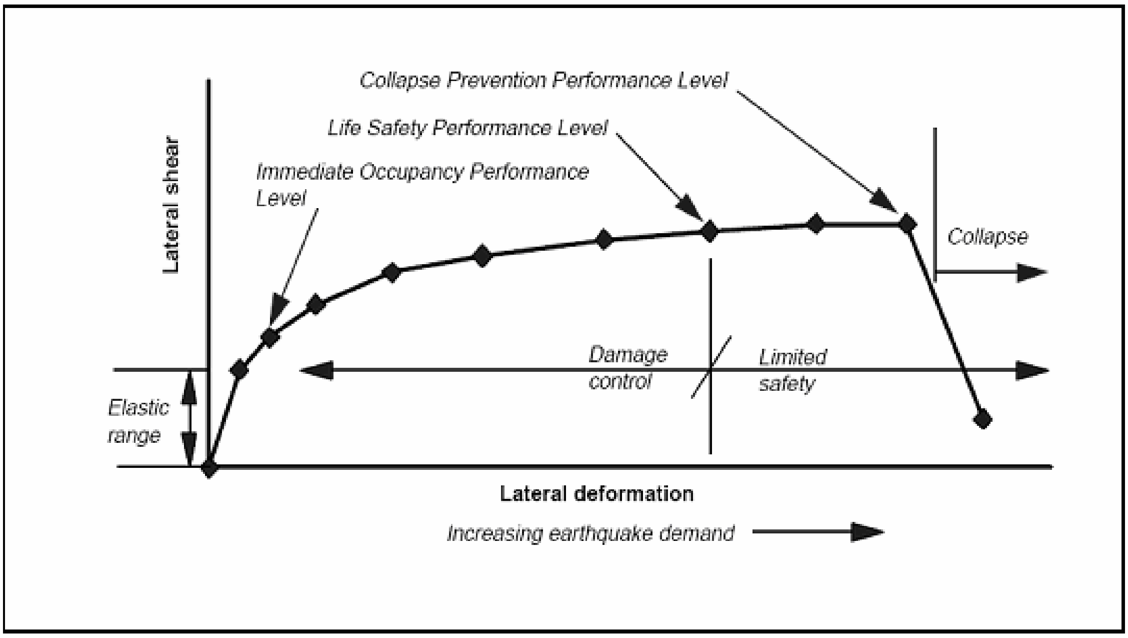


Figure 8. Performance and structural deformation demand for ductile structures

(FEMA-274)

6. Modeling Rules

This section presents rules for developing analytical models of existing concrete buildings. The rules are intended for use with a nonlinear static procedure, the rules are based on principles of mechanics, observed earthquake performance, a broad range of experimental results, and engineering judgment.

6.1 Loads

The nonlinear analysis of a structure should include the simultaneous effects of gravity and lateral loads. Gravity loads should include dead loads and likely live load. Dead load can be taken as the calculated structure self-weight without load factors, plus realistic estimates of flooring, ceiling, partition, and either nonstructural elements. In general, because of the nonlinear nature of the interactions, it is not appropriate to carry out the gravity load analysis and lateral load analysis separately and then superimpose their results.

Lateral loads should be applied in predetermined patterns that represent predominant distribution of lateral inertia loads during critical earthquake response. Lateral loads may be lumped at the floor levels. Lateral loads should be applied in increments that allow the engineer to track the development of the inelastic mechanism. Gravity loads should be inplaced during lateral loading. The effect of gravity loads acting through lateral displacement, the so-called $P-\Delta$ effect.

6.2 Global Building Considerations

Analytical models for evaluation must represent complete three-dimensional characteristics of building behavior, including mass distribution, strength, stiffness, and deformability, through a full range of global and local displacements. Full three dimensional static inelastic analysis often requires significant effort. Therefore static methods and dynamic elastic methods are not able to adequately represent the full effect of torsion response. Any structural, nonstructural, and soil elements that can affect the building assessment must be modeled. In addition, every component carrying gravity loads must be checked. The main impacts of soil –structure interaction are to modify the target lateral displacement and to provide additional flexibility at the base level that may relieve inelastic deformation demands in the superstructure. Because the net effect is not readily known it is recommended that foundation flexibility be included routinely in the analysis level.

6.3 Element Models

For models including concrete frames with shear walls, (combined frame wall elements), horizontal elements are reinforced concrete diaphragms. The analytical model should represent the strength, stiffness, and deformation capacity of beams, columns, beam columns, and shear walls. All components shall be modeled by considering flexural and shear rigidities.

Modeling Local Response: The analytical models for beams, columns, and joints should be capable of representing the controlling deformation and failure mode.

- Beams may develop inelastic response associated with flexure, shear, development, splices, and slip of bars embedded in joints.
- Columns may develop inelastic response associated with flexure, axial load, shear and splice failure.

- Beam-column joint strength may limit the forces that can develop in the adjacent framing members. The primary failure mode concern is joint shear failure. The analytical model should represent these potential modes where they may occur. Beam plastic hinging may be represented directly in computer programs in SAP 2000 ver 7.4 that models inelastic response.

- Concrete shear walls: shear walls that are continuous and solid, elements should represent the strength, stiffness, and deformation capacity of the wall in–plane loading. The response of walls with intermediated aspect ratio is usually influenced by both flexure and shear.

- Concrete floor diaphragms: the analytical model for a floor diaphragm should represent the strength, stiffness, and deformation capacity for in-plane loading. Diaphragm axial, shear, and flexural deformations should be modeled unless the diaphragms can be considered rigid and are strong enough to remain essentially elastic under the applicable earthquake loads. The model should allow assessment of diaphragm shear, flexure, anchorage, splicing, and connections to vertical components. In general the evaluation or retrofit design must consider how the diaphragm connects vertical and lateral force resisting elements and how it braces elements subject to out-of-plane loads or deformations. This methodology considers only cast–in–place concrete diaphragms. Slabs commonly serve multiple purposes; they are part of the floor or roof system to support gravity loads, they function as tension and compression flanges for floors beams, and they act as a part of the horizontal diaphragm. The floor slab may develop shear, flexural, and axial forces associated with the transmission of forces from one vertical lateral force resisting element to another, or with the slab action as a bracing element for portions of the building that are loaded out of the plane. The diaphragms can be rigid or flexible.

- Foundations: The analytical model should allow assessment of soil and structural foundation components and should represent the nonlinear response of the foundation system. For simplicity foundations may be represented as rigid footings, flexible strip footing, or pile foundations. Appropriate models for equivalent linear stiffness and strength should be employed depending on the foundation type.

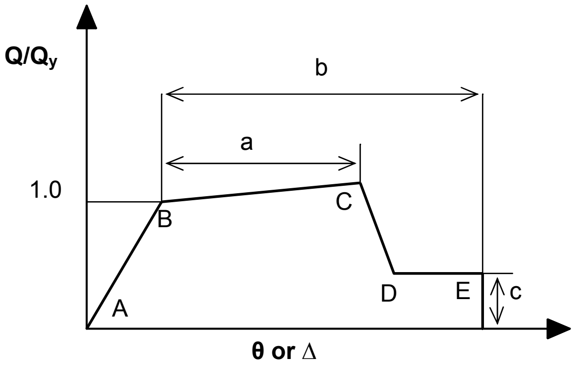
6.4 Component Models

In general, the model must represent the stiffness, strengths, and deformability of structural components. Two approaches are presented. One approach is to calculate relevant properties directly by using basic principles of mechanics. The second approach is to use present modeling rules described in detail in this section. These rules were derived by the project team on the basis of available test data, analytical methods, and engineering judgment. Some combination of the two approaches is permissible and is likely to be used in a typical building analysis.

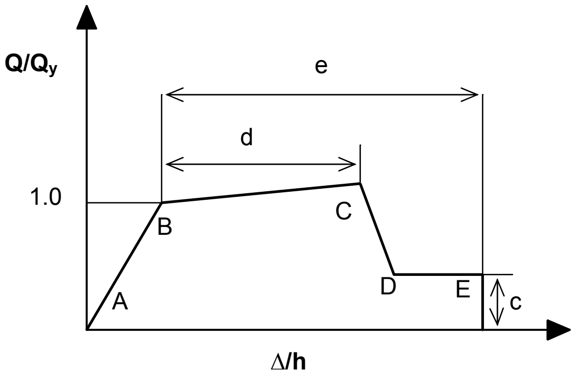
Component behavior generally will be modeled using nonlinear load-deformation relation defined by a series of straight–line segments. Q_y refers to the strength of the component and Q refers to the demand imposed by the earthquake. As shown in Figure 9, the response is linear to an effective yield point, B, followed by yielding (possibly with strain hardening) to point C followed by strength degradation to point D, followed by final collapse and loss of gravity load capacity at point E .

- Point A corresponds to the unloaded condition. The analysis must recognize that gravity loads may induce initial forces and deformation that should be accounted for in the model.
- Point B has resistance equal to the nominal yield strength.
- The slope between point B to C, ignoring the effects of gravity loads acting through lateral displacement, is usually taken as between 5% to 10% of the initial slope. This strain hardening, which is observed for most reinforced concrete components, may have an important effect on the redistribution of internal forces among adjacent components.
- The ordinate at C corresponds to the nominal strength.
- The abscissa at C corresponds to the deformation at which significant strength degradation begins. Beyond this deformation, continued resistance to reversed cyclic lateral forces can no longer be guaranteed. For brittle components this deformation is the same as the deformation at which yield strength is reached. For ductile components, this deformation is larger than the yield deformation. Gravity load resistance may or may not continue to deform larger than the abscissa at C.
- The drop in resistance from C to D represents initial failure of the component. It may be associated with phenomena's fracture of longitudinal reinforcement, spalling of concrete or sudden shear failure following initial yield. Resistance to lateral loads beyond point C usually is unreliable. Therefore, primary components of the lateral force resisting system should not be permitted to deform beyond this point.
- The residual resistance from D to E may be non-zero in some cases and may be effectively zero in others. The residual resistance usually may be assumed to be

equal to 20 % of the nominal strength. The purpose of this segment is to allow modeling of components that have lost most of their lateral force resistance but that are still capable of sustaining gravity loads.



(a) Deformation



(b) Deformation Ratio

Figure 9. Generalized force-deformation relations for concrete elements or components

6.5 Materials Models

The material models should consider all available information, including building plans, original calculations and design criteria, site observations, testing, and records of typical materials and construction practices prevalent to the time of construction. Successful application of the methodology requires good information about the building. In general, material properties should be established by inspection and testing.

6.5.1 Concrete

Evaluation of concrete material properties should involve determination of compressive strength, modulus of elasticity.

6.5.2 Reinforcement

Evaluation of reinforcement should consider grade, surface deformations, surface conditions (including corrosion), and bar placement and detailing.

6.6 Component Initial Stiffness

Reinforced concrete component stiffness may be represented by a secant value defined by the effective yield point of the component. For flexure–dominated components, this stiffness corresponds approximately to the fully-cracked stiffness. For shear–dominated components, this stiffness corresponds approximately to uncracked stiffness. The stiffness value may be determined as a function of material properties (considering current condition), component dimensions, reinforcement quantities, boundary conditions, and stress and deformation levels. In many cases, it will be impractical to calculate effective stiffness directly from basic mechanics principles. Instead, effective initial stiffness may be based on the approximate values of Table 5.

Table 5. Effective Stiffness Values (ATC-

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns with compression due to design gravity loads $\geq 0.5\ A_gf'_c$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $< 0.3\ A_gf'_c$ or with tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: It shall be permitted to take I_g for T-beams as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in Section 6.4.1.3. For columns with axial compression falling between the limits provided, linear interpolation shall be permitted. Alternatively, the more conservative effective stiffness shall be used.

6.7 Component Strength

Actions in a structure are classified as either deformation-controlled or force-controlled.

- Deformation-controlled actions are permitted to exceed elastic limits under applicable earthquake loads. Strengths for deformation-controlled actions should be taken equal to expected strengths obtained experimentally or calculated by using accepted mechanics principles. The tensile stress in yielding longitudinal reinforcement should be assumed to be at least 1.25 times the nominal yield strength. Procedures specified in ACI 318 may be used to calculate strengths, except that strength reduction factors, ϕ, should be taken equal to 1.0, and other procedures specified in this document should govern where applicable.

  For the structure covered by this methodology, deformation-controlled actions are limited to the following:

a) Flexure (in beams, slabs, columns, and walls)

b) Shear distortion in walls and wall segments

c) Connection rotation at slab-column connections

Pushover method is a displacement-based procedure, that is, its basis lies in estimating the expected lateral displacements and the resulting local deformations and internal force demands. For ductile components subject to deformation controlled actions, performance is measured by the relation of deformation demand to deformation capacity. Component ductility demand is classified into three levels, as listed in Table 6.

Table 6. Component ductility demand classification(ATC-96)

Maximum Value of Displacement Ductility	Classification
< 2	Low ductility demand
2 to 4	Moderate ductility demand
>4	High ductility demand

- Force-controlled actions are not permitted to exceed elastic limits under applicable earthquake loads. Strengths for force-controlled components should be taken equal to lower bound strengths obtained experimentally or calculated using established mechanics principles.

7. Response Limits

To determine whether a building meets a specified performance objective, response quantities from a nonlinear static analysis are compared with limits for appropriate performance levels. This section presents those structural response limits, which constitute acceptance criteria for the building structure. The response limits fall into two categories:

- Global building acceptability limits. These response limits include requirements for vertical load capacity, lateral load resistance and lateral drift which will be discussed in the next section.
- Element and component acceptability limits. Each element (frame, wall, diaphragm, or foundation) must be checked to determine if its components respond within acceptable limits. This part won't be considered in this study.

7.1 Global building Acceptability Limits

- Gravity Loads: The gravity load capacity of the building structure must remain intact for acceptable performance at any level. Where an element or component loses capacity to support gravity loads, the structure must be capable of redistributing its load to other elements or components of the existing or retrofit system.

- Lateral Loads: As discussed before, some component types are subject to degrading over multiple load cycles. If a significant number of components degrades, the overall lateral force resistance of the building may be affected. The lateral load resistance of the building system, including, resistance to the effects of gravity loads acting through lateral displacements, should not degrade by more than 20% percent of the maximum resistance of the structure. Where greater degradation occurs, either the structure should be redesigned or alternative methodologies should be employed to refine the estimates of expected response.

- Lateral Deformations: Maximum total drift is defined as the interstory drift at the performance point displacement. Maximum inelastic drift is defined as the portion of the maximum total drift beyond the effective yield point. Table 7 presents deformation limits for various performance levels.

Table 7. Deformation Limits(ATC-96)

Interstory Drift Limit	Performance level			
	Immediate Occupancy	**Damage Control**	**Life Safety**	**Structural Stability**
Maximum Total drift $\frac{\Delta}{h}$	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

Where V_i is the total calculated lateral shear force in story I and Pi is the total gravity load (i.e. dead plus likely live load) at story i.

8. Acceptance Criteria for Nonlinear Procedures

Table 8. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Beam-Column Joints(FEMA-273)

Conditions			Modeling Parameters [4]: Plastic Shear Angle, radians a	Modeling Parameters [4]: Plastic Shear Angle, radians b	Modeling Parameters [4]: Residual Strength Ratio c	Acceptance Criteria [4]: Plastic Rotation Angle, radians — Performance Level: IO	Component Type: Primary LS	Primary CP	Secondary LS	Secondary CP
i. Interior joints [2, 3]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
ii. Other joints [2, 3]										
$\frac{P}{A_g f'_c}$	Trans. Reinf. [1]	$\frac{V}{V_n}$								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.0075	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.0075	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	–	0.0	0.0	0.0	0.005	0.0075
≥ 0.4	NC	≥ 1.5	0.0	0.0	–	0.0	0.0	0.0	0.005	0.0075

1. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A joint is conforming if hoops are spaced at □ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming.
2. P is the design axial force on the column above the joint and A_g is the gross cross-sectional area of the joint.
3. V is the design shear force and V_n is the shear strength for the joint.
4. Linear interpolation between values listed in the table shall be permitted.

Table 9. **Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Beams**(FEMA-273)

Conditions			Modeling Parameters [3]			Acceptance Criteria [3]				
			Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure [1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear [1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span [1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at □ *d*/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_S) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.

Table 10. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns(FEMA-273)

Conditions			Modeling Parameters [4]			Acceptance Criteria [4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure [1]										
$\frac{P}{A_g f_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear [1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height [1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$ [1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at □ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.

Case Study and Research Methodology

1. Case Study

An eight-story building with a story height equals to 3.8m, total height of 30.4m, area of each floor = 364m^2, thickness of slabs =120mm , thickness of shear wall for the first and second floors = 250mm, and in the other floors 20 cm are considered in this study. The building lies in zone z = 0.2 on soil type S_D. The structural system of the building consists of nine reinforced concrete ordinary moment resisting frames in each direction spaced as shown in plan of Figure 10-a, with four shear walls in the Y direction. The materials used in this model are concrete with f_c' = 25MPa and steel reinforcement with f_y = 420MPa. The dimensions for all beams = 0.5*0.5m and the dimensions of columns are as given in the following table.

Table 11. Dimensions of columns in case study model, mm

Dimensions of Columns, cm			
Floor #	**Column C_1**	**Columns C_2**	**Columns C_3**
1	400 * 700	500 * 700	500 * 850
2	400 * 600	400 * 700	500 * 750
3	400 * 550	400 * 600	400 * 750
4	400 * 500	400 * 600	400 * 650
5	300 * 500	350 * 600	400 * 550
6	300* 400	350 * 500	300 * 550
7	300 * 350	300 * 400	300 * 450
8	300 * 300	300 * 300	300 * 350

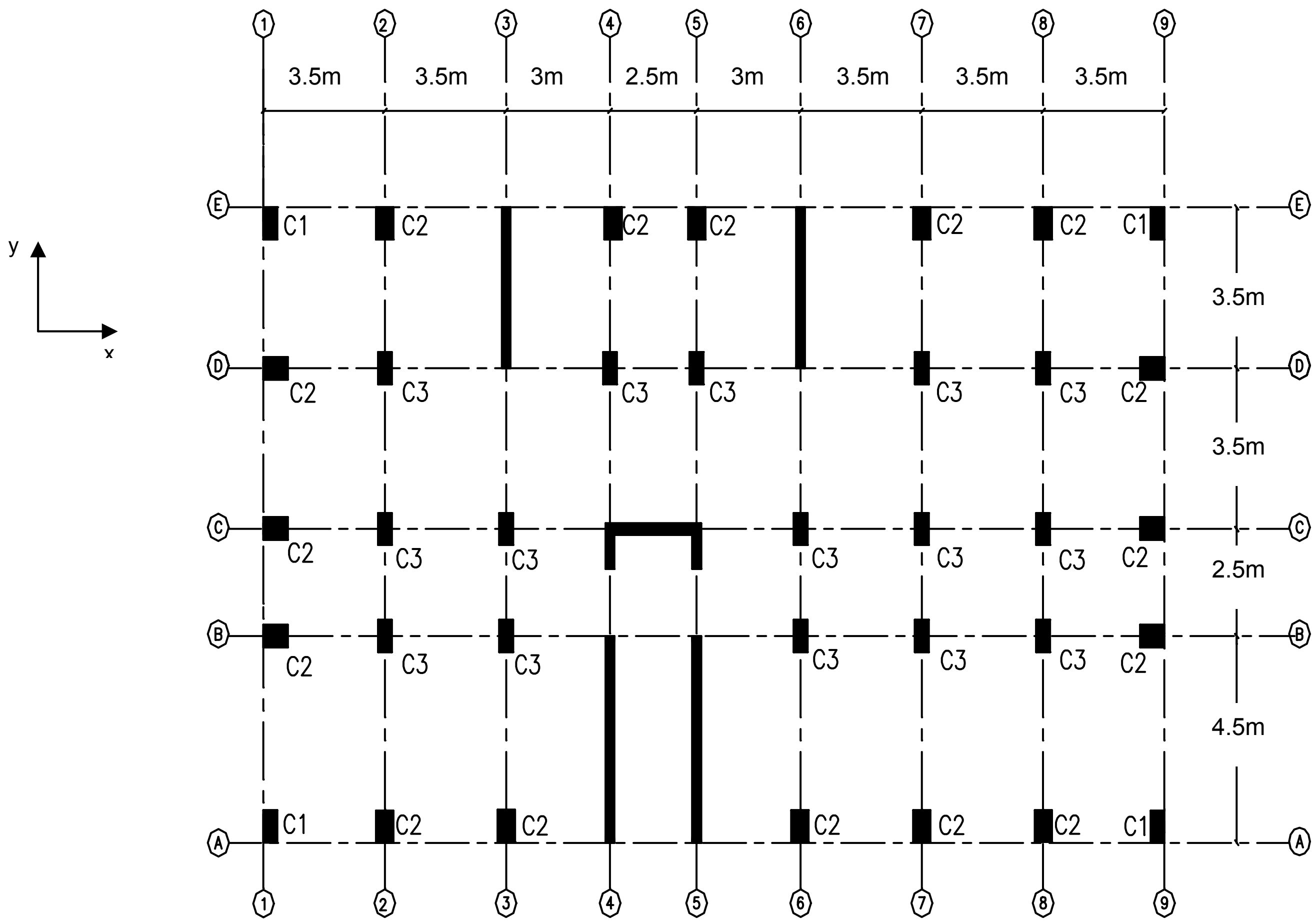


Figure 10 – a. Typical Floor P

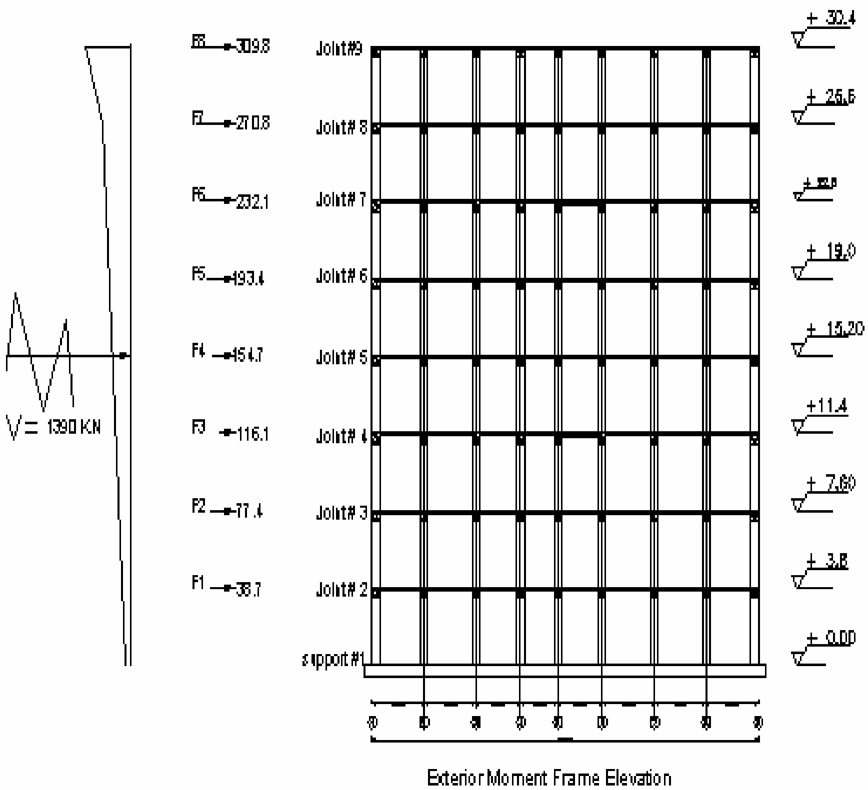


Figure 10 - b. Moment Frame in x direction

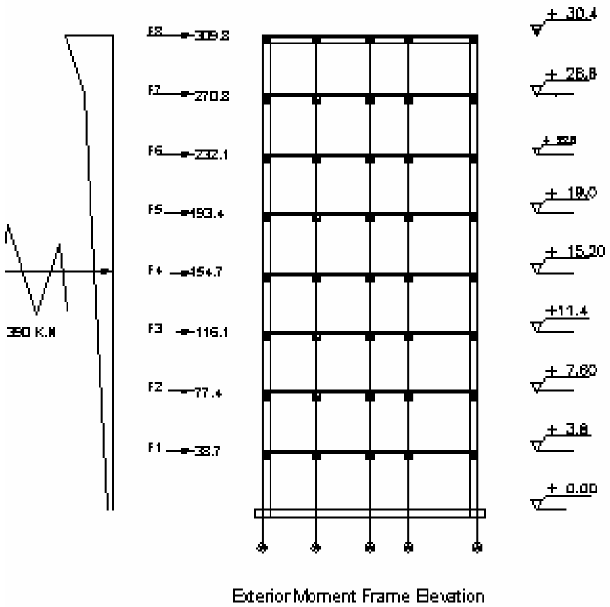


Figure 10 – c Moment Frame in y direction

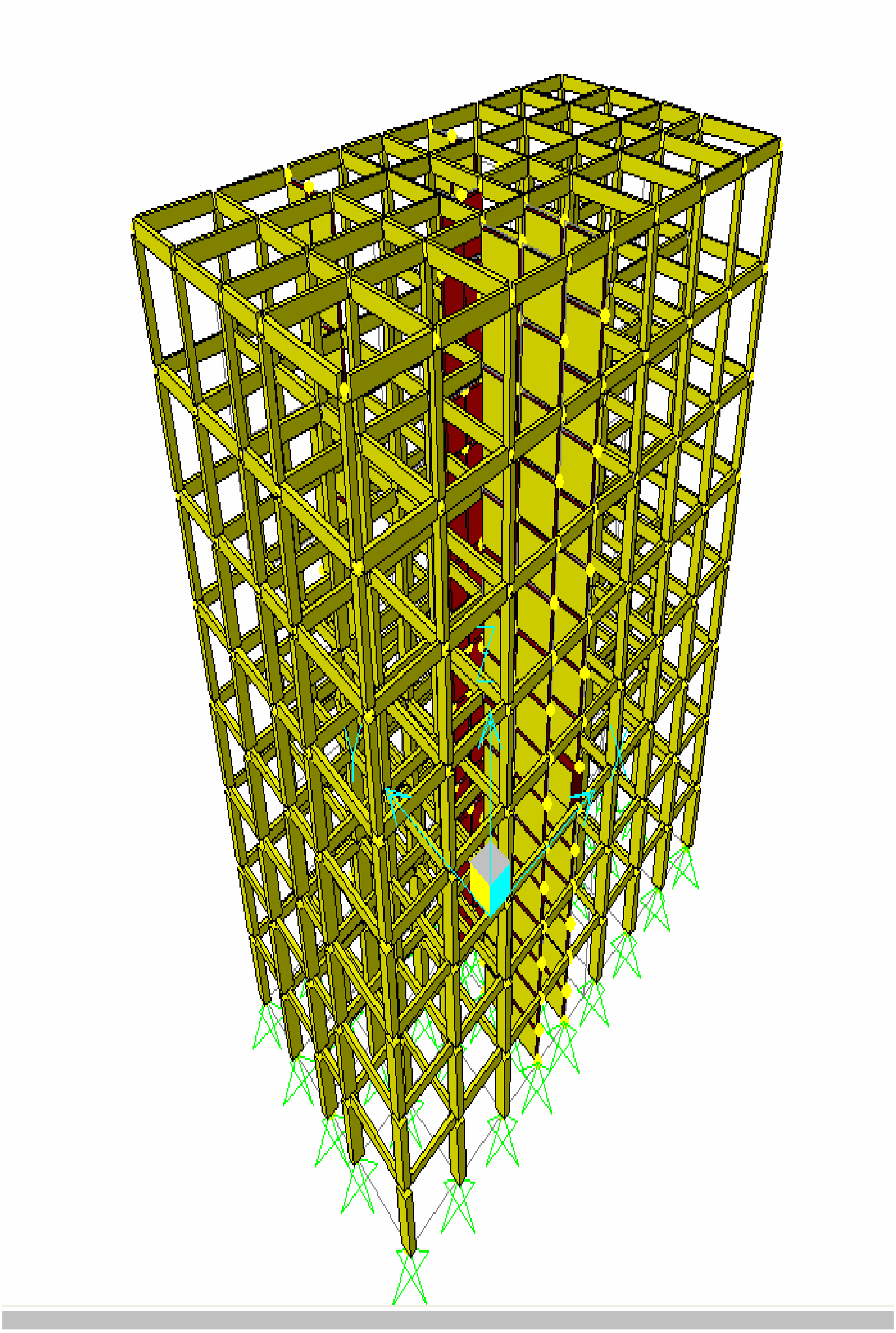


Figure 10 - d. Three dimensional model

2. Research Methodology

1. A three dimensional eight-story building with a total height of 30.4m was modeled. As mentioned in the previous section, the structural system of the building consists of reinforced concrete ordinary moment resisting frames in both direction and shear walls only in Y direction. See Figure 10-a through 10-d.
2. The service dead and live loads on the slabs in kN/m^2 were assumed as follows: service dead load = $10kN/m^2$ and service live load = $3kN/m^2$.
3. The seismic analysis for the assumed 3D model was constructed using four different approaches as follows:

(a) Static Force Procedure: as recommended in the UBC-97 Code.

(b) Response Spectrum Method using the UBC-97 design response spectrum.

(c) Time History Analysis using the El-Centro earthquake record.

(d) Pushover Method.

The analysis was performed using the SAP2000 software for the four methods.

4. Analysis results including fundamental period, base shear, displacement and rotation for the assumed building were compared using the four methods.

Calculations, Results and Discussion

1. General

In this chapter, the four methods of analysis considered in this research, i.e. static force method, response spectrum method, time history analysis and pushover method, are described in more detail and the results of each method concerning the fundamental period, base shear force, displacement and rotation of joints are discussed and compared.

2. Static Force Procedure

The static force procedure is a method that replaces the seismic lateral force by an equivalent static lateral force for simplicity in computations. The method is based on the concept of seismic base shear, whereby the structure is designed to resist a force applied at the ground equals to a constant times the total weight of the structure and is then transmitted to each story of the structure. This constant depends on regional and geological conditions, importance, natural period, ductility and stiffness distribution of the structure, and some other factors.

This method may be applied in the following cases according to reference UBC-97 :

- Regular and irregular structures in zone 1.
- Regular structures with height equal to or less than 73m.
- Irregular structures with 5 stories or less and with total height less than or equal to 20m.

According to the static force procedure, the total base shear, V, is obtained from the response spectra and may be expressed as follows:

$$V = \frac{C_v I}{RT} W = \begin{array}{l} \le \dfrac{2.5\, C_a\, I}{R} W \\ \ge 0.11\, C_a\, I\, W \\ \ge \dfrac{0.8\, Z\, N_v\, I}{R} W \;\; for zone\ 4 \end{array}$$

(37)

where:

W = mass weight

C_a = Acceleration coefficient as per UBC Table (16-Q)

C_v = Velocity coefficient as per UBC Table (16-R)

I = Importance factor equal to 1.25 for essential and hazardous Facilities

R = response modification factor as per UBC (Table 16-N)

Z = zone factor as per UBC Table (16-1)

N_v = Near source factor as per UBC Table (16-T)

T = The fundamental period of the structure

The fundamental period of the structure may be calculated using the following formula:

$$T = C_t (h_n)^{\frac{3}{4}} \qquad (38)$$

where:

h_n = Total height of the building in meters

C_t = Coefficient taken as follows:

= 0.0853 for steel moment resisting frames.

= 0.0731 for concrete and steel moment resisting frames.

= 0.0488 for all other buildings.

2.1 Vertical Distribution of Base Shear Force

For the static force procedure, the UBC-97 code assumes a triangular distribution of forces. In addition, for building with period T > 0.7 sec, the code raises the force share of the top story from the total base shear. Such increase is intended to include the effect of higher modes in tall buildings. Consequently, the distribution of base shear may be expressed as follows:

- For $T \leq 0.7$ sec , the story force, F_x, as fraction of the base shear equals:

- $$F_x = \frac{w_x h_x}{\sum w_x h_x} V \qquad (39)$$

where,

w_x = weight of story under consideration

h_x = height of story under consideration

w_i = weight of all stories including the story under consideration

h_i = height of all stories including the story under consideration

- For T > 0.7sec, the top force F_t is given:

$$F_t = 0.07\ T\ V \leq 0.25\ V \qquad (40)$$

and the rest of the bas shear shall be distributed as given previously by Eq. 39.

2.2 Results of the Static Force Procedure

The principles of the static force procedure described above can be applied to the structure under study shown in Figure 10-b since it is a regular building with total height less than 73m. Accordingly, using Eqs. 37 and 38 the base shear force and the fundamental period of the structure may be calculated as:

$$V = \frac{0.4 * 1}{8.5 * 0.95}(28030) = 1388.48 \cong 1390\,KN$$

$T = 0.071*(30.4)^{0.75} = 0.95$ seconds

Since the fundamental period of the structure is more than 0.7 sec, then the top force F_t should be included equals to:

$F_t = 0.07*0.95*1390 = 92.4$ kN

Using Eq. 39, the total base shear force is distributed horizontally for each floor. Floor forces are given in Table 12 and are shown graphically in Figure 11.

Table 12. Horizontal distribution of base shear force

Floor #	W_i (kN)	h_i (m)	W_ih_i (kN.m)	F_x (kN)	F_x+ F_t (kN)
8	2550	30.4	77520	216.8	309.8
7	3640	26.6	96824	270.8	270.8
6	3640	22.8	82992	232.1	232.1
5	3640	19	69160	193.4	193.4
4	3640	15.2	55328	154.7	154.7
3	3640	11.4	41496	116.1	116.1
2	3640	7.6	27664	77.4	77.4
1	3640	3.8	13832	38.7	38.7
Total	**28030**		**464816**		**1390**

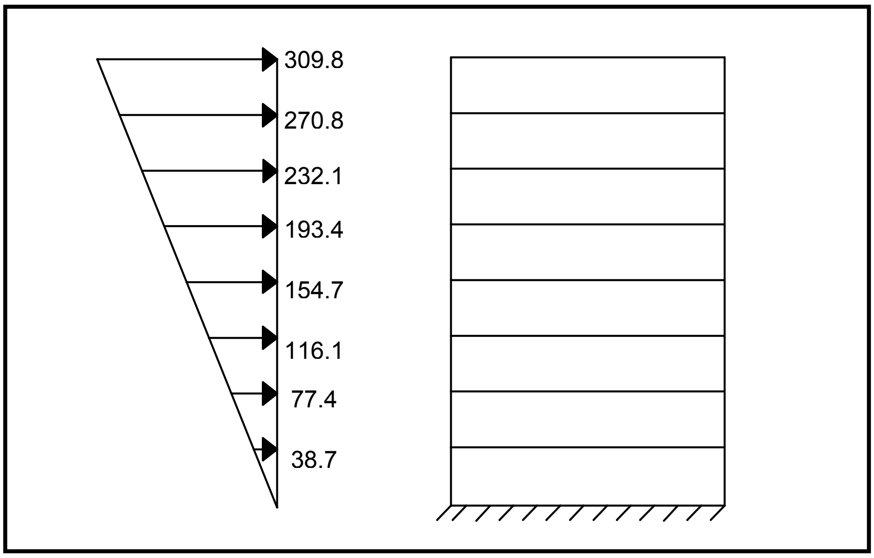


Figure 11. Horizontal distribution of base shear force

After the design seismic forces have been determined for the structure in the form of lateral horizontal force applied at each story level, the structure is modeled as a three dimensional system and the equivalent lateral static forces given in Table 12 are applied to each floor. SAP2000 program is then used to determine the resulting seismic forces and displacements. Results in the form of translational and rotational displacements of joints 1 through 9 of the exterior column (at support # 1) in three directions are listed in Table 13.

Table 13. Displacement and rotation of joints 1 through 9 of the exterior column at support # 1, static force method

	Displacement			Rotation		
Joint #	U_x (cm)	U_y (cm)	U_z (cm)	R_x (rad)	R_y (rad)	R_z (rad)
1	0	0	0	0	0	0
2	0.367	0.019	-0.024	$-1.86*10^{-4}$	$1.39*10^{-3}$	$-1.72*10^{-5}$
3	1.111	0.058	-0.049	$-2.37*10^{-4}$	$1.91*10^{-3}$	$-5.30*10^{-5}$
4	2.044	0.106	-0.073	$-2.94*10^{-4}$	$2.17*10^{-3}$	$-9.83*10^{-5}$
5	3.013	0.158	-0.096	$-3.41*10^{-4}$	$2.01*10^{-3}$	$-1.47*10^{-4}$
6	3.932	0.207	-0.122	$-4.47*10^{-4}$	$1.74*10^{-3}$	$-1.94*10^{-4}$
7	4.849	0.254	-0.147	$-6.51*10^{-4}$	$1.77*10^{-3}$	$-2.41*10^{-4}$
8	5.643	0.294	-0.167	$-7.80*10^{-4}$	$1.57*10^{-3}$	$-2.82*10^{-4}$
9	6.288	0.324	-0.178	$-1.55*10^{-3}$	$1.51*10^{-3}$	$-3.15*10^{-4}$

Results show that the displacement of the exterior column joints increases as we move upward, i.e. larger displacements are expected for taller structures. Also note that since the equivalent static forces calculated using the static force method are applied in the x direction, larger displacements in that direction are generated. Displacement and rotation of the joints are plotted in the next figures.

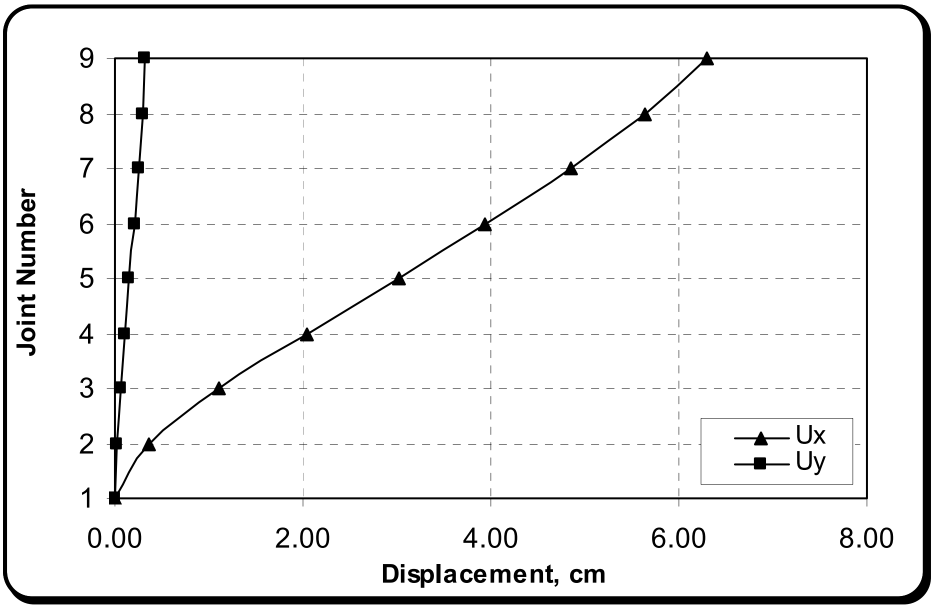


Figure 12. Displacement of joints 1-9 at support # 1, static force method

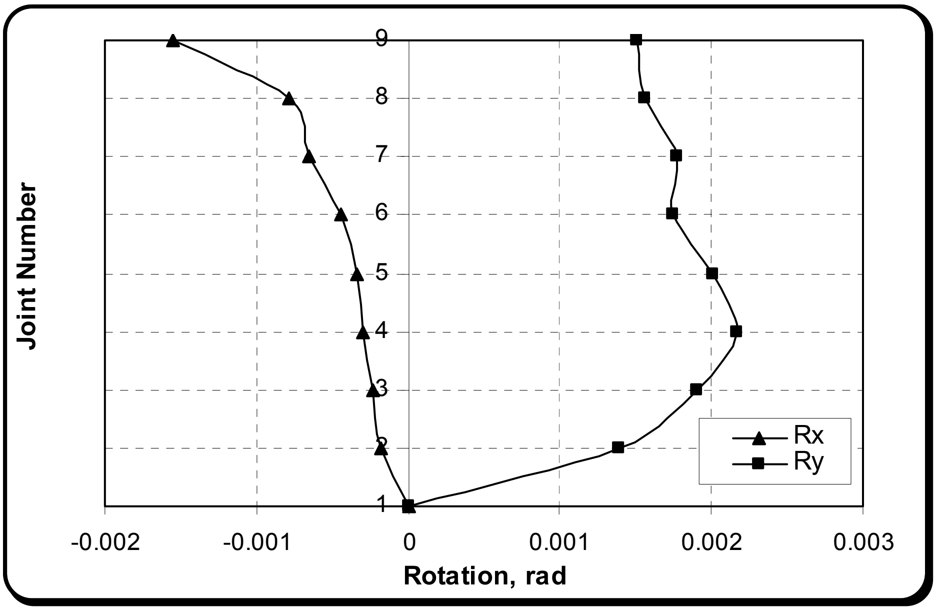


Figure 13. Rotation of joints 1-9 at support # 1, static force method

3. Response Spectrum Analysis

The response spectrum method is used for structures that do not conform to the requirements of the static force procedure. In this method structures are modeled as multiple degree of freedom systems where the resulting frequencies and mode shapes are extracted by modal analysis. For each mode, the spectral forces and displacement are found and then combined as needed, e.g by, SRSS, CQC. The combined forces are then divided by the response modification factor, R, to obtain the design forces. Similarly, elastic displacements are multiplied by a ductility demand factor which is explicitly given by UBC-code as 0.7R. The combined number of modes is taken to include mass participation of at least 90% of the total mass.

The UBC-97 code response spectrum as given in Figure 14 is used with to the given soil profile and the proper zone factor. Alternatively, a site–specific response spectrum is permitted if it takes all site characteristics into consideration including seismic hazard analysis, in addition, it has to be based on 90% probability that the event will not be exceeded in 50 years reference . A damping ratio $\xi = 5\%$,

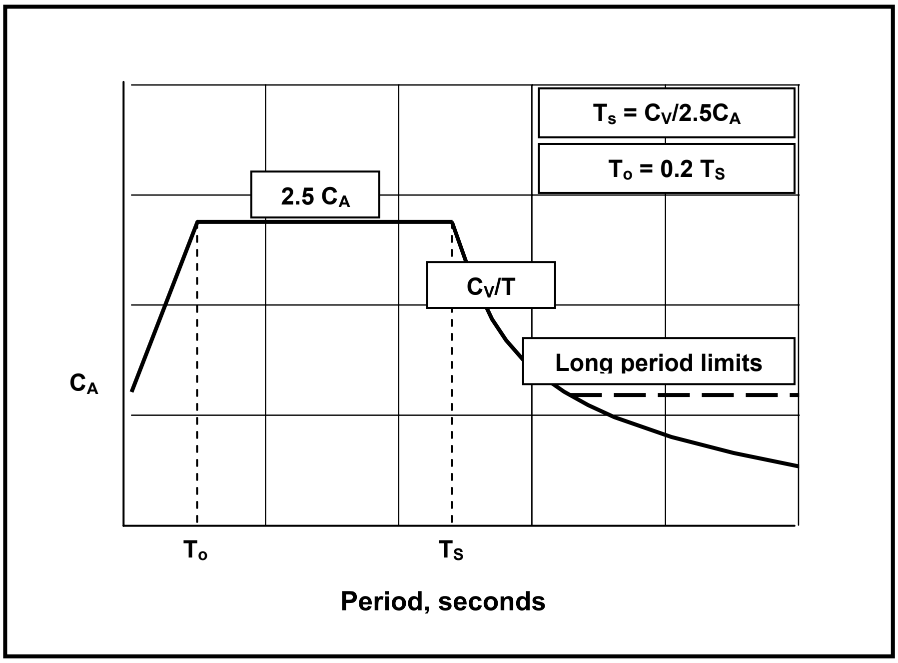


Figure 14. UBC-97 design response spectrum

According to UBC code, the base shear obtained using response spectrum procedures can be reduced to a fraction of the base shear obtained by static force procedures, but shall not be less than that. This reduction in the response spectrum results is expressed in the following form:

- Regular structures $V_{RS} \geq 90\% \; V_{SF}$
- Irregular structures $V_{RS} \geq 100\% \; V_{SF}$

where;

V_{RS} = Base shear obtained from response spectrum method

V_{SF} = Base shear obtained from static force method

For the structure considered in this study, the seismic coefficients C_a and C_v are obtained from the UBC tables 16-Q and 16-N, respectively. Therefore, and refering to Figure 14, the design response spectrum can be built from the UBC spectrum which is a function of the seismic coefficients, C_a and C_v. Pairs of period and acceleration are listed below in Table 14 and the final design response spectrum is shown in Figure 15.

Table 14. Design response spectrum pairs

Period (sec)	Acceleration (g)	Period (sec)	Acceleration (g)
0	0.44	1	0.64
0.1	0.99	1.25	0.51
0.12	1.10	1.5	0.43
0.2	1.10	1.75	0.37
0.3	1.10	2	0.32
0.4	1.10	2.5	0.26
0.5	1.10	3	0.21
0.58	1.10	6	0.11
0.6	1.07	10	0.06

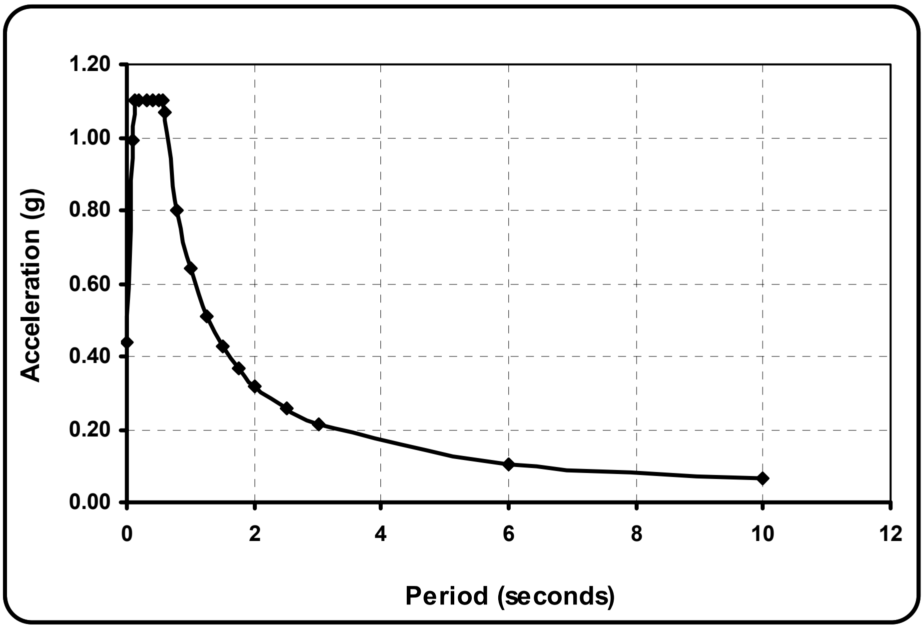


Figure 15. Response Spectrum

3.1 Results of the Response Spectrum Analysis

Using the software SAP2000 software, the design response spectrum constructed in the previous section is applied to the 8-story three-dimensional model considered in the study. After completion of input and computer run of input information, all periods and their mode shapes, nodal forces, displacements, internal moments, shears and normal forces will be available in the output file. Note that the summation of total reactions in any direction is the base shear in that direction. Table 15 gives the fundamental period of the first 24 modes and the mass participation ratio in percent for the displacement in x, y and z directions. It can be seen that the cumulative sum of the mass participation ratio is 99.7, 95.3 and 89.3% for displacements in x, y and z directions, respectively.

Table 15. Modal patricipation mass ratios

Mode Shape #	Period (sec)	Individual Mode (%)			Cumulative Sum (%)		
		U_x	U_y	U_z	U_x	U_y	U_z
1	1.322	72.281	0.000	0.381	72.281	0.000	0.381
2	0.888	0.001	66.732	0.028	72.282	66.732	0.410
3	0.451	12.668	0.003	1.645	84.949	66.736	2.055
4	0.253	1.602	0.284	51.974	86.552	67.019	54.029
5	0.240	0.061	0.558	0.730	86.613	67.578	54.759
6	0.228	4.075	0.243	18.556	90.688	67.821	73.314
7	0.213	0.023	17.039	1.032	90.711	84.860	74.347
8	0.147	2.855	0.000	0.927	93.566	84.860	75.274
9	0.109	1.294	0.004	4.564	94.860	84.864	79.837
10	0.097	0.637	0.020	4.026	95.497	84.884	83.863
11	0.090	0.000	6.612	0.011	95.497	91.496	83.874
12	0.082	1.144	0.000	1.116	96.641	91.496	84.991
13	0.079	0.002	0.231	0.000	96.643	91.726	84.991
14	0.070	0.185	0.001	1.589	96.828	91.728	86.580
15	0.067	0.702	0.000	1.055	97.529	91.728	87.634
16	0.061	0.232	0.000	0.069	97.762	91.728	87.703
17	0.058	0.052	0.000	1.046	97.814	91.728	88.749
18	0.054	0.204	0.000	0.562	98.018	91.728	89.311
19	0.052	0.000	3.427	0.001	98.018	95.156	89.311
20	0.052	0.497	0.001	0.245	98.515	95.157	89.556
21	0.049	0.516	0.000	0.003	99.030	95.157	89.559
22	0.045	0.678	0.000	0.132	99.708	95.157	89.691
23	0.043	0.000	0.097	0.000	99.708	95.254	89.691
24	0.039	0.000	0.000	0.000	99.708	95.254	89.691

For each mode, displacement and rotation of each node of the structure can be found and then combined using the SRSS method. Displacement and rotation of joints 1 through 9 at support # 1, in x, y and z directions, are summarized in Table 16. Also see Figures 16 and 17 for the displacement and rotation of the joints, respectively. In addition, combining the summation of total reactions in x direction using the SRSS method yielded a base shear in the x direction equals 2082kN.

Table 16. Displacement and rotation of joints 1 through 9 of the exterior column at support # 1, response spectrum method

	Displacement			Rotation		
Joint #	**U_x (cm)**	**U_y (cm)**	**U_z (cm)**	**R_x (rad)**	**R_y (rad)**	**R_z (rad)**
1	0	0	0	0	0	0
2	0.394	0.013	0.012	$5.20*10^{-5}$	$1.37*10^{-3}$	$1.13*10^{-5}$
3	1.212	0.042	0.025	$8.16*10^{-5}$	$1.95*10^{-3}$	$3.74*10^{-5}$
4	2.236	0.080	0.036	$9.67*10^{-5}$	$2.24*10^{-3}$	$7.12*10^{-5}$
5	3.300	0.124	0.045	$1.03*10^{-4}$	$2.00*10^{-3}$	$1.09*10^{-4}$
6	4.317	0.169	0.054	$9.69*10^{-5}$	$1.54*10^{-3}$	$1.45*10^{-4}$
7	5.314	0.213	0.060	$8.08*10^{-5}$	$1.55*10^{-3}$	$1.78*10^{-4}$
8	6.210	0.253	0.064	$6.70*10^{-5}$	$1.37*10^{-3}$	$2.06*10^{-4}$
9	6.998	0.290	0.066	$3.53*10^{-5}$	$8.59*10^{-4}$	$2.30*10^{-4}$

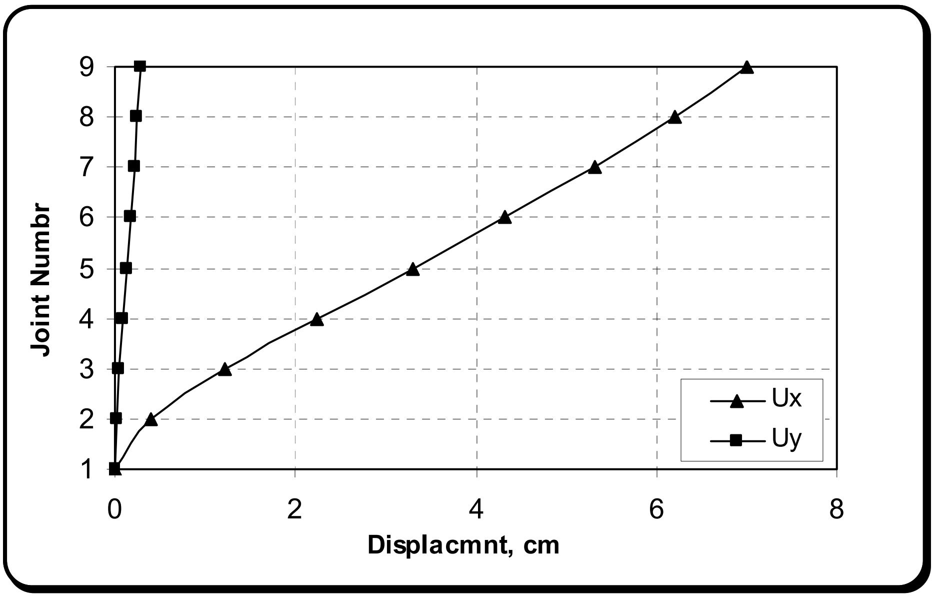


Figure 16. Displacement of joints 1-9 at support # 1, response spectrum method

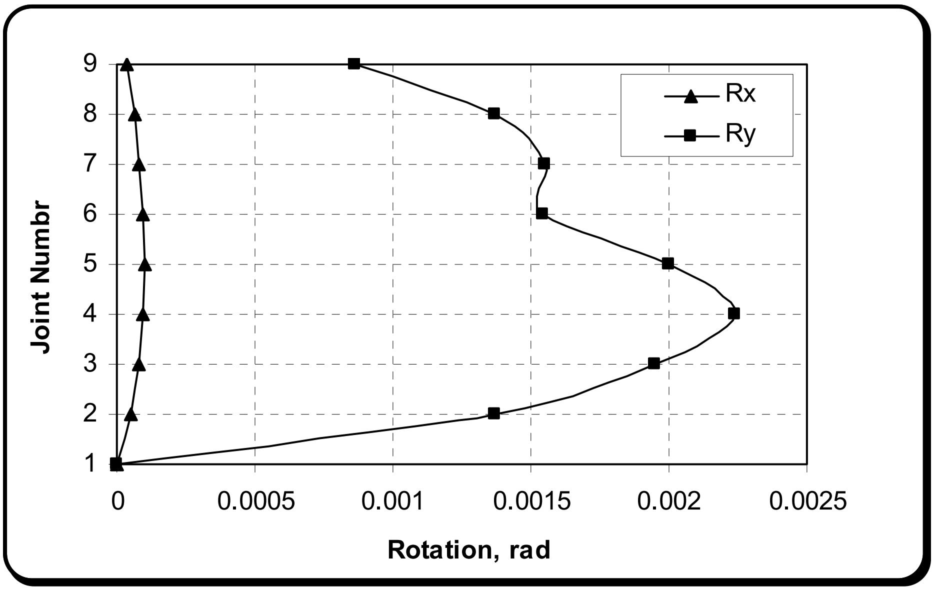


Figure 17. Rotation of joints 1-9 at support # 1, response spectrum method

4. Time History Analysis

Time history analysis is the general method used for large and complex structures, and is conducted by using numerical methods. Since ground motion records are needed for this type of analysis, the code requires that at least three pairs of records be used. These records shall reflect site characteristics and seismic hazard. These records can either be scaled from actual records, or, artificially generated (synthetic records).

The code also specifies that if only three records are used in the analysis, the maximum response quantities must be taken as the maximum of the three. However, if seven records are used in the analysis, the response values may be averaged over the seven records.

Time history analysis may be performed using elastic and inelastic structural properties. In the elastic analysis procedures, the design forces and displacement are obtained by modifying the results of the computer output by the factors R and μ . In inelastic procedures, forces and displacement are directly obtained from the analysis. It should be noted that for inelastic analysis, an approved hystersis model is needed which must be based on experimental and analytical results.

In this study only one earthquake record will be used for demonstration purposes, that is, the El-Centro earthquake, which is portrayed in Figure 18.

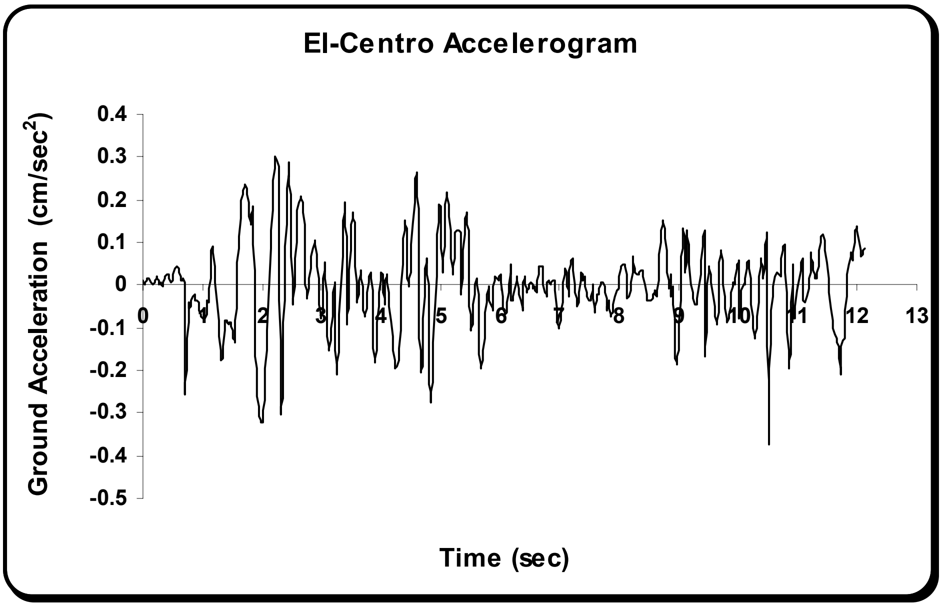


Figure 18. El-Centro earthquake accelerogram

4.1 Results of the Time History Analysis

After completion of the input and selection of the analysis procedure, the computer runs the input information and the history of all elastic nodal forces, history of elastic displacements, history of internal moments, history of shears and history of normal forces may be obtained from the output file. In addition, the base shear history may be traced from the computer output to find its maximum value. Time history analysis of the considered structure gave a fundamental period of 1.32 seconds and a base shear force of 1384 kN. Furthermore, displacement history of joints 1 through 9 at support # 1 and the maximum absolute displacement of these joints in the x direction are given in Figures 19 and 20, respectively.

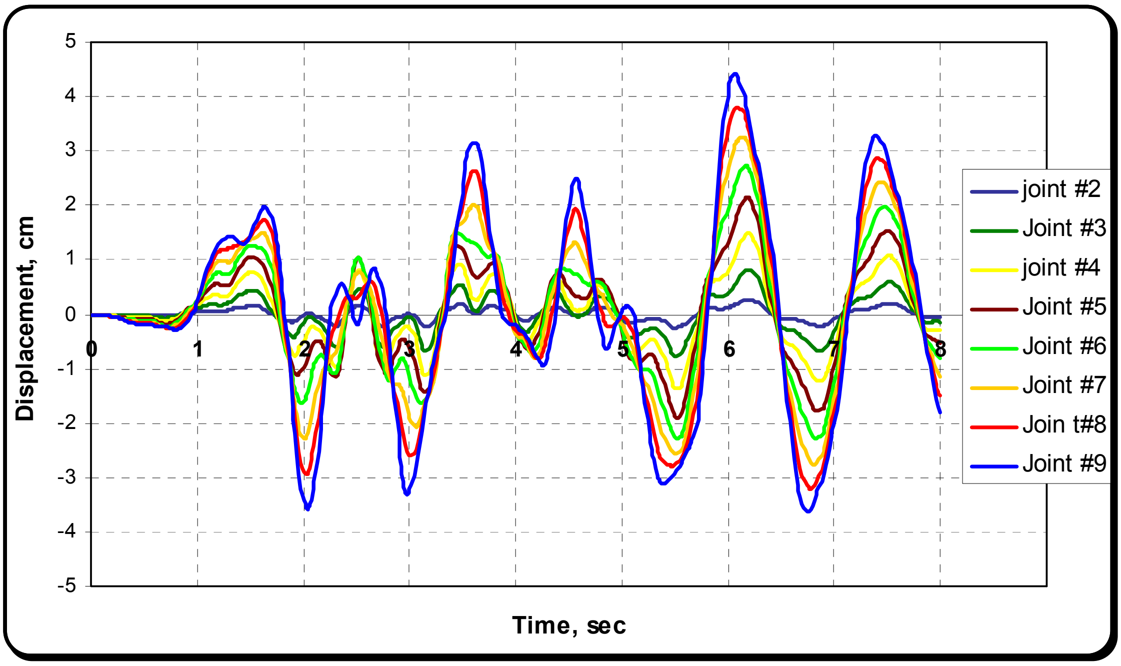


Figure 19. History of displacements of joints 1-9 at support # 1

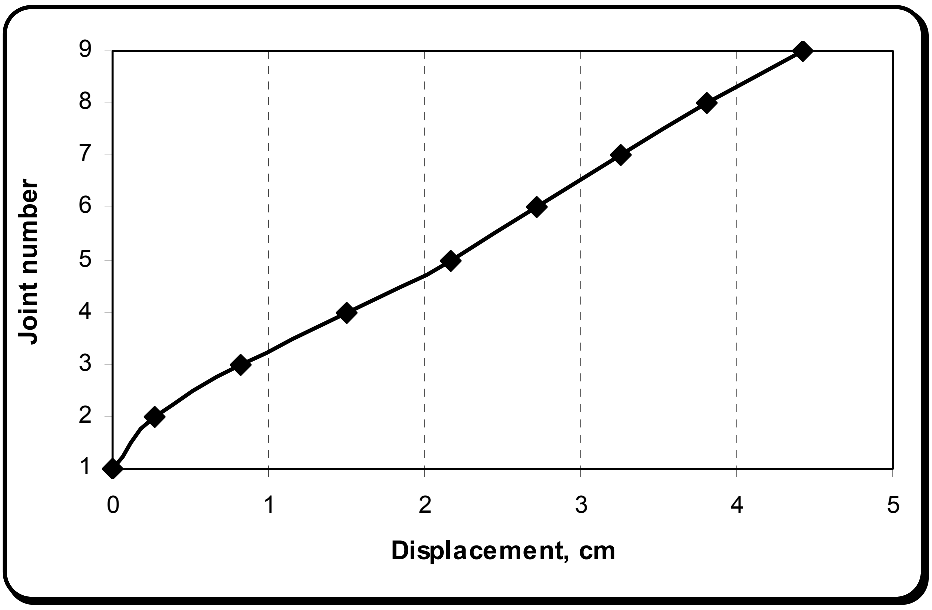


Figure 20. Maximum absolute displacement of joints 1-9 at support # 1 in x direction

5. Nonlinear Static Pushover Analysis

The ATC-40 and FEMA-273 documents have developed modeling procedures, acceptance criteria and analysis procedures for the pushover analysis. These documents define a force-deformation criteria for plastic hinges used in pushover analysis. As shown in Figure 21, five points labeled, A, B, C, D and E are used to define the force-deflection behavior of a hinge and three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge. IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention, respectively. The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-273 documents.

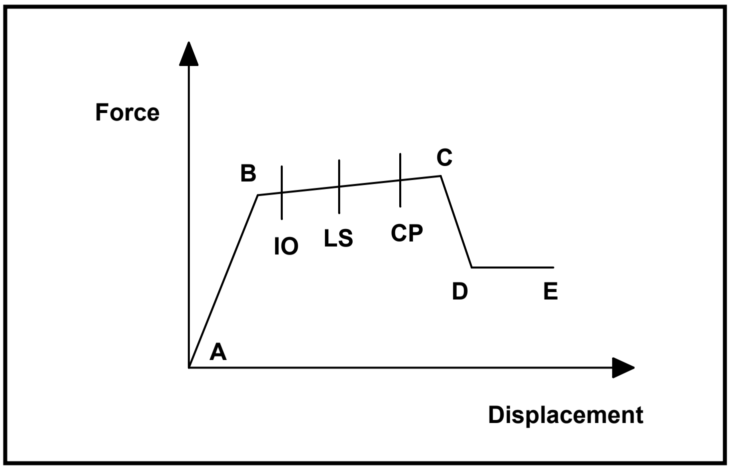


Figure 21. Generalized force-displacement relation

This section presents the steps used in performing a pushover analysis of a simple three-dimensional building using SAP2000 ver 7.4 program. Steps 1 through 6 review the pushover analysis method.

1. Create the basic computer model, same model which is used for static and dynamic analyses. Note that for each section, material properties and section

dimensions shall be defined and assigned in addition to supports and end conditions.

2. Define properties and acceptance criteria for the plastic hinges. Alternatively, the program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members.

3. Assign the location of the plastic hinges in the model by selecting the frame members shown in Figures 22 and 23 for beams and columns, respectively.

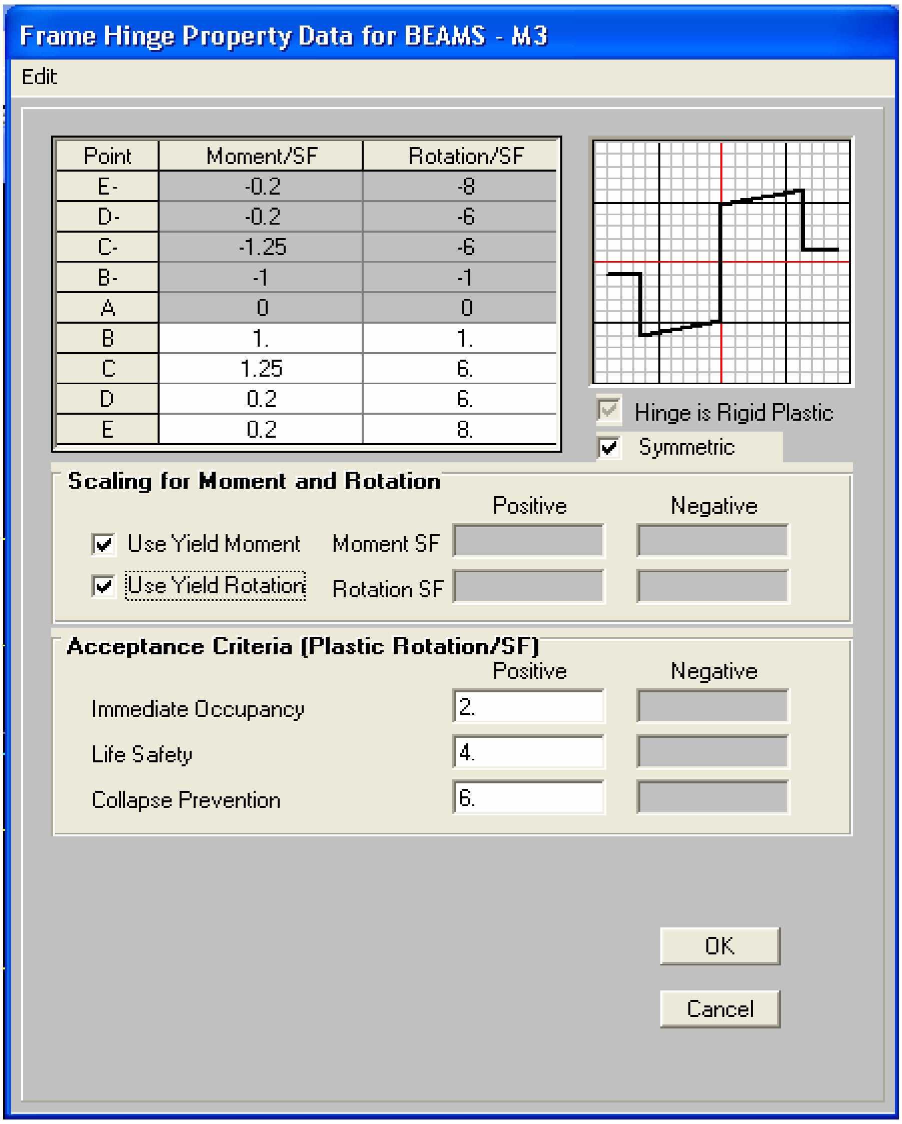


Figure 22. Assigning hinge properties to Beams

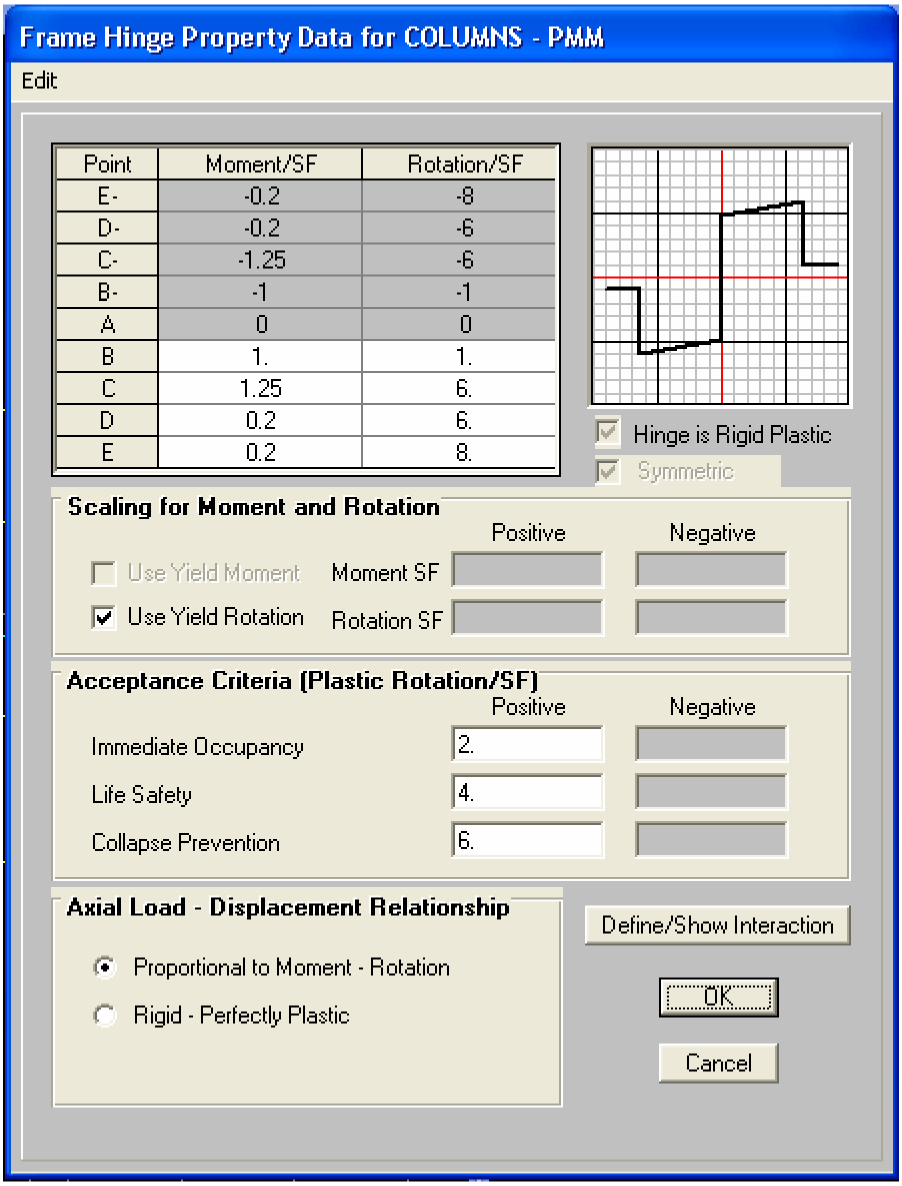


Figure 23. Assigning hinge properties to columns

4. Define the pushover load cases. In SAP2000 more than one pushover load case can be defined for same analysis. Also a pushover load case can start from the final conditions of another load case that was previously run in the same analysis. Recording to ATC-96 and SAP2000 the first pushover load case is_used to apply gravity load and the subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain

5. defined force level, or they can be displacement controlled, that is, pushed to a specified displacement level .

Typically, a gravity load pushover is force controlled and lateral pushovers are displacement controlled. SAP2000 allows the distribution of lateral force used in the pushover analysis to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user–defined static load case. The dialog box shown in Figure 24 shows how the displacement controlled lateral pushover case that is based on a user-defined static lateral load pattern (named) push x is defined in this example. It should be mentioned that the target displacement to which the structure is pushed using the pushover analysis method can be taken as 4% of the total height of the structure according to SAP2000-Manuall

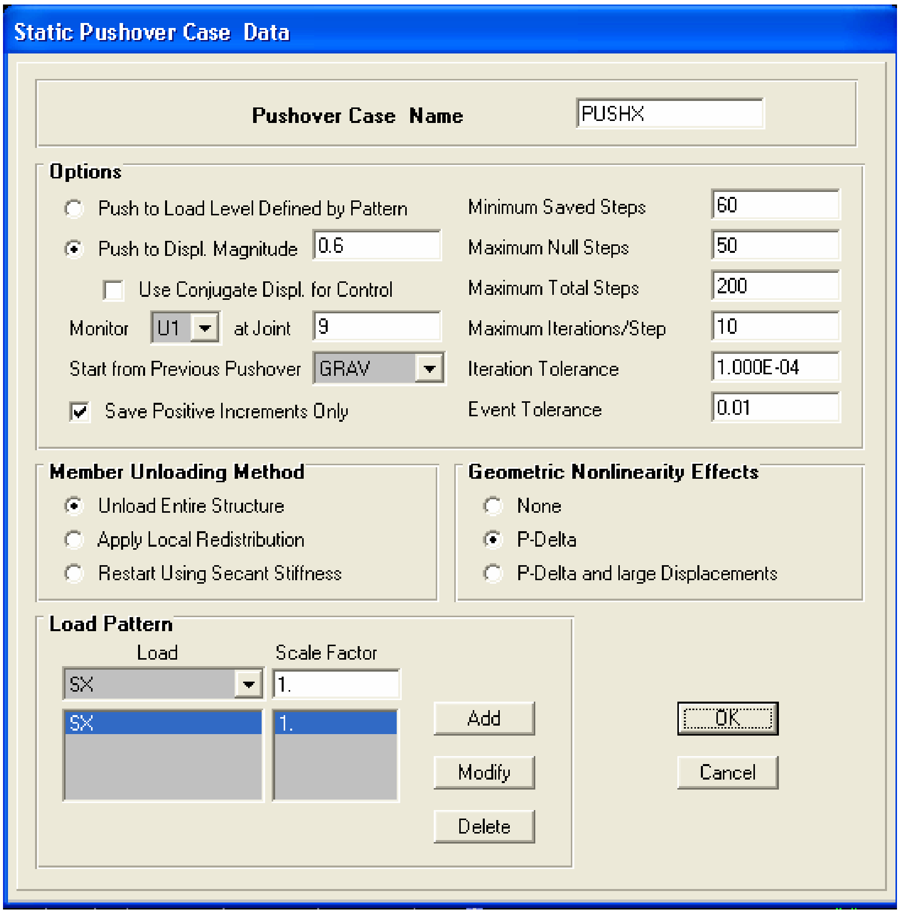


Figure 24. Defining the pushover load case

6. Run the basic static analysis and, after static analysis, run the static nonlinear pushover analysis.
7. Display the pushover curve as shown in Figure 25. The program plots the capacity spectrum curve (green one) and the demand spectra (Orange one curve), in which the intersection of the two curves defines the performance point.

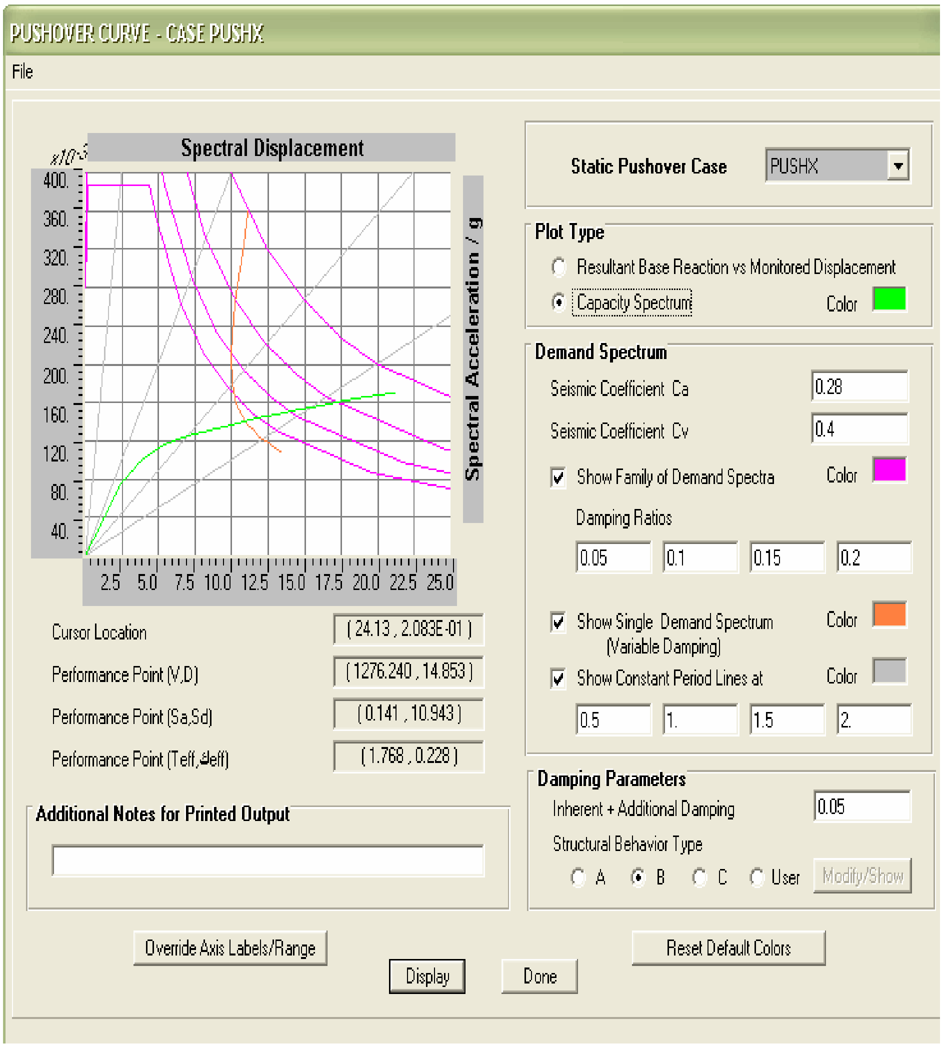


Figure 25. Pushover curve in the x- direction

5.1 Defining the Pushover Hinge Properties

As mentioned earlier the SAP2000 program includes several built-in default hinge properties for beams and columns such as concrete axial hinge, concrete shear hinge, concrete moment hinge and concrete P-M-M hinge properties. This section explains how the concrete moment hinge properties can be generated.

First of all, the concrete moment hinge property is provided in the form of a moment-rotation curve for tension and compression as shown in Figure 26.

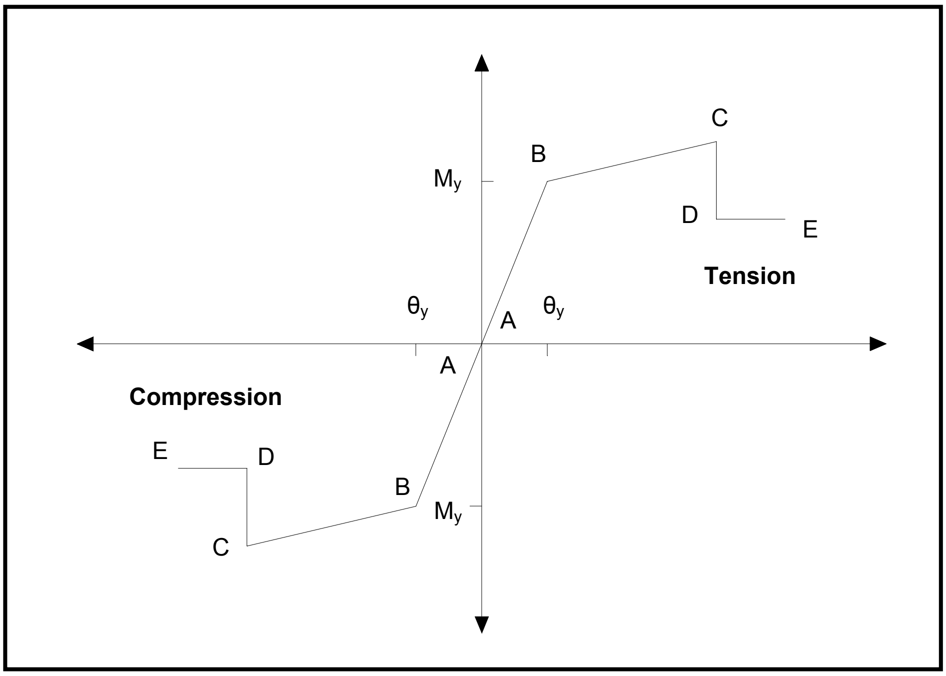


Figure 26. General hinge moment-rotation curveAs an example, consider the beam cross section shown in Figure 27. For this beam, f_c' and f_y are assumed to be equal to 25 MPa and 420 MPa, respectively. The stress-strain curve of steel is assumed bilinear whereas the stress-strain curve of the concrete is assumed as that proposed by Kent and Park for unconfined concrete. See Figures 28 and 29.

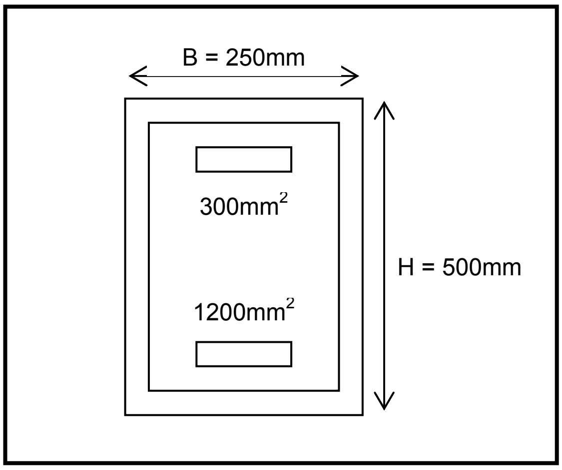


Figure 27. Beam cross section

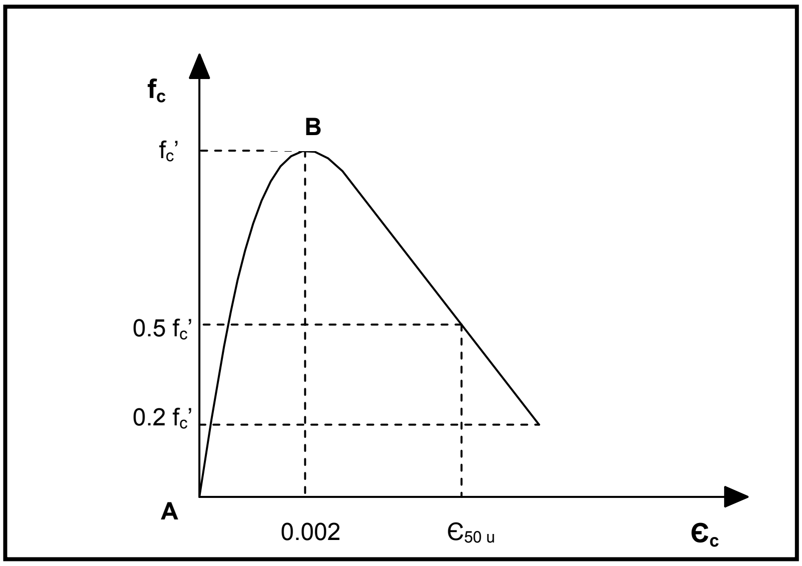


Figure 28. Stress-strain curve of unconfined concrete, Kent and Park(1975)

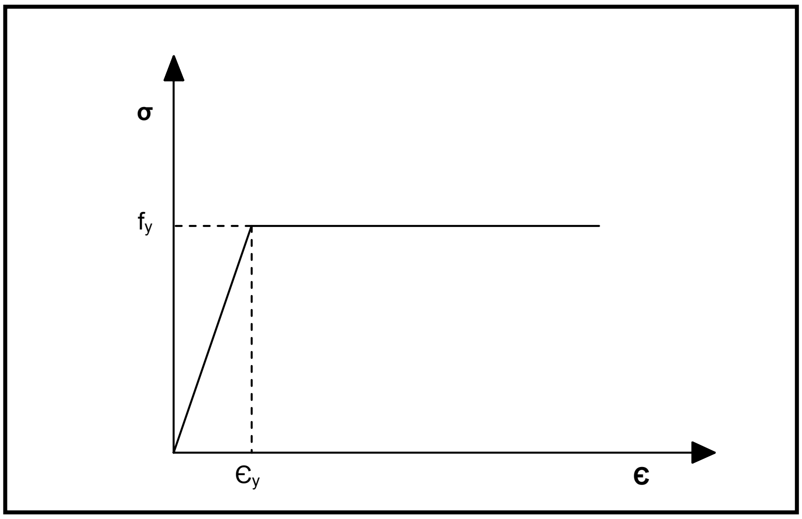


Figure 29. Stress-strain curve of steel, bi-linear

Referring to Figure 28, the suggested stress strain curve of concrete in region AB can be given as, Kent and Park, (1975):

$$f_c = f_c'\left[\frac{2\varepsilon_c}{0.002} - \left(\frac{\varepsilon_c}{0.002}\right)^2\right] \tag{41}$$

and,

$$\varepsilon_{50u} = \frac{3 + 0.002 * f_c'}{f_c' - 1000} \tag{42}$$

where in Eq. 42 f_c' is the concrete compressive strength in psi (1 psi = 0.00689 N/mm^2

In order to generate the beam moment hinge property, a moment rotation curve for the beam shall be first developed. The general procedure can be described as follows: in order to draw the moment-curvature relationship for the given beam section, the strain of concrete shall be incremented, and for each strain value a corresponding average rectangular stress is found by equating areas (forces) under the idealized stress-strain curve. Then, the location of the concrete compressive force is determined by taking first moment of areas about the corresponding strain level. For the assumed stress-strain curve of concrete, area under curve for each strain increment can only be determined by numerical integration methods. The Simpson's 3/8 rule has been used to carry out the integration procedure.

Once the average concrete stress and the location of the concrete compressive force are determined, force equilibrium is used to find the neutral axis depth whereas moment equilibrium and strain compatibility are applied to determine the moment and curvature of the section, respectively. See Figure 30.

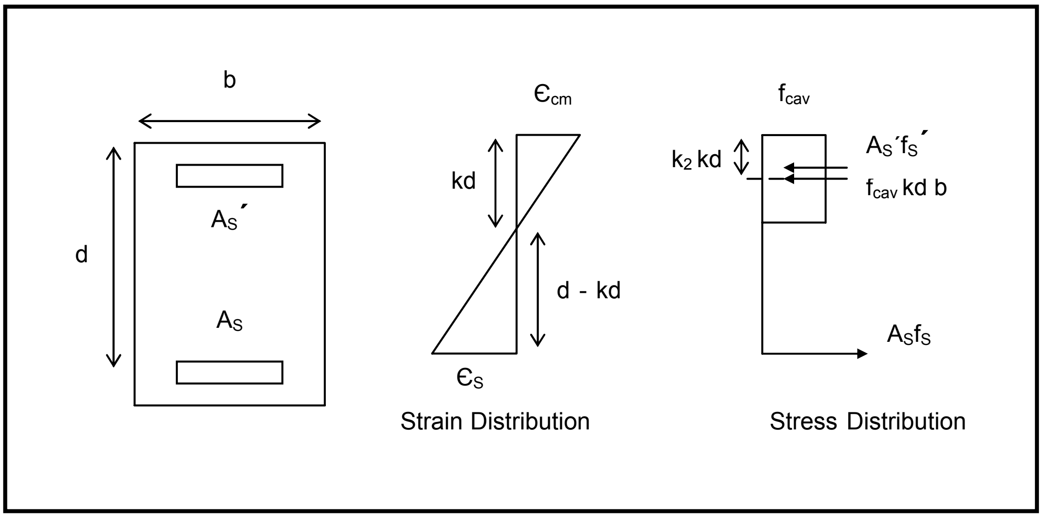


Figure 30. Strain and stress distributions of a beam section

Note that since the beam section contains compression steel, the depth of neutral axis, kd, shall be determined using a trail and error procedure that eventually satisfies force equilibrium. The M-φ curve of the considered beam is shown in Figure 31.

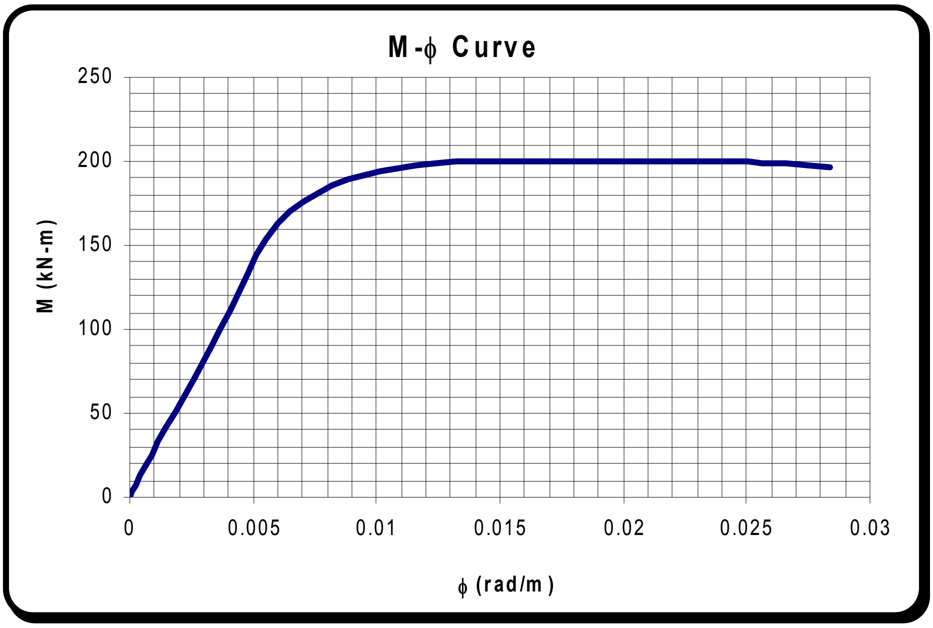


Figure 31. Moment-curvature curve, $P_u = 0$

From the moment–curvature curve shown above, values of the yielding moment, M_y, curvature at yield, φ_y, and the ultimate moment, M_u, are extracted, these are

(112kn-m 0.011 rad/m), and (118-0.05 kn-m, rad/m) , respectively. Assuming that the length of plastic hinge, L_p is taken as half the total depth of member, the hinge moment rotation curve can be developed using the following formulas:

$$\theta_y = \varphi_y * L_P$$

$$K_e = \frac{M_y}{\theta_y} \qquad (43)$$

$$\alpha K_e = \frac{M_u - M_y}{\theta_u - \theta_y} \qquad (44)$$

where α is the slope of the between points B and C in Figure 26 and according to SAP2000 program manual, α might be taken as 10% total strain hardening of steel. Points A, B and C on the hinge moment-rotation curve are then determined using Table 9 and knowing the tension, compression and balanced steel ratios, concrete strength, section dimensions and the level of concrete confinement by transverse reinforcement. The hinge moment-rotation curve of the considered beam section is given in Figure 32.

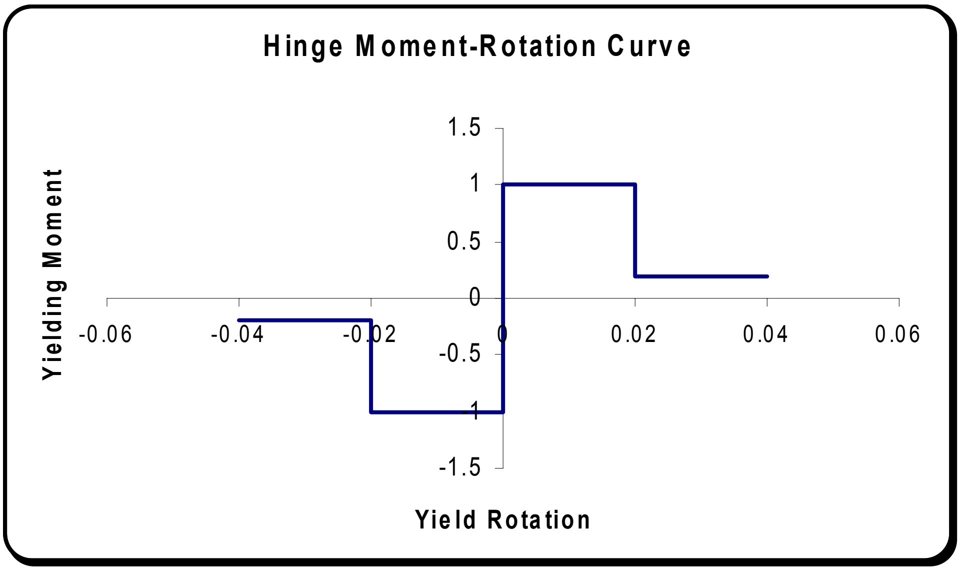


Figure 32. Hinge moment-rotation curve

5.2 Results of the Pushover Method

Using the SAP2000 software, the pushover method was carried out for the 8-story three dimensional model assumed in the study. The program displays the pushover and capacity spectrum curves where their intersection defines the performance point. Table 17 gives the results of the pushover method where for each step a point on the pushover curve of base shear vs. displacement is defined and the total number of plastic hinges in each step and the distribution of this number between performance levels are also listed.

Table 17. Results of pushover method

			Number of Plastic Hinges								
Step #	Displacement (cm)	Base Shear (kN)	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
0	-0.005	0	1696	0	0	0	0	0	0	0	1696
1	1.28	269.13	1695	1	0	0	0	0	0	0	1696
2	3.33	669.91	1597	99	0	0	0	0	0	0	1696
3	5.39	931.14	1449	247	0	0	0	0	0	0	1696
4	7.39	1064.1	1304	392	0	0	0	0	0	0	1696
5	9.50	1143.9	1241	452	3	0	0	0	0	0	1696
6	11.51	1200.3	1202	448	46	0	0	0	0	0	1696
7	13.53	1248.9	1177	403	116	0	0	0	0	0	1696
8	15.87	1297.3	1155	325	214	2	0	0	0	0	1696
9	17.90	1334.8	1140	270	282	4	0	0	0	0	1696
10	20.13	1372.4	1120	247	300	29	0	0	0	0	1696
11	22.28	1403.4	1096	228	304	68	0	0	0	0	1696
12	24.64	1431.9	1081	203	269	143	0	0	0	0	1696
13	26.87	1456.2	1064	178	249	205	0	0	0	0	1696
14	29.45	1481.3	1045	160	226	265	0	0	0	0	1696
15	29.95	1485.7	1041	158	222	274	0	1	0	0	1696
16	26.69	803.84	1041	158	222	274	0	0	1	0	1696

The capacity curve is shown in Figure 33. Note that the performance point had occurred at step number 7 at a displacement of 13.53 cm and a base shear of 1249 kN.

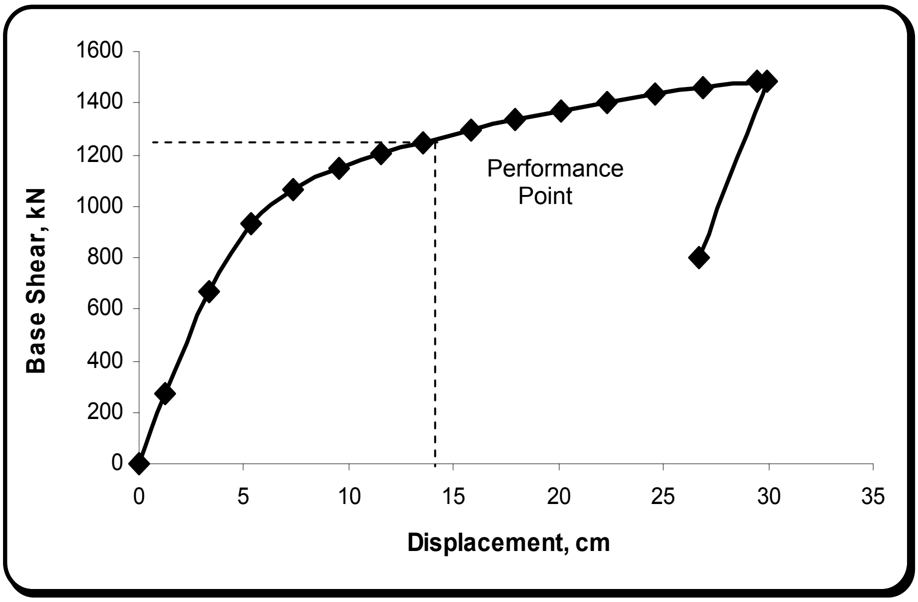


Figure 33. Capacity (Pushover) curve

Referring to Table 17, the table shows that for each step of pushing the number of plastic hinges that occurred in members increases for each performance level till total collapse of structure. This can also be observed visually in the deformed shapes of Figures 34 (a) to (q)

In addition, deformation limits can be checked at the performance point level as follows:

- Total displacement at the performance point = 135.3 mm
- Total height of structure = 30.4 m = 30400 mm
- Ratio of performance point displacement / total height = 1353/30400 = 0.005

Referring back to Table 7 in chapter two, drift limitations are met for the immediate occupancy performance level.

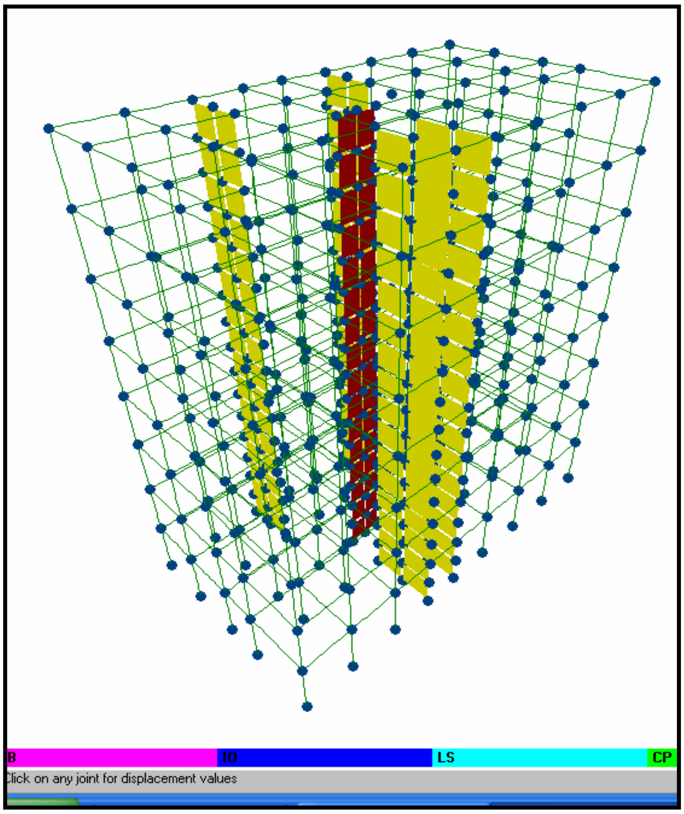


Figure 34-a. Deformed shape at step 0
No. of hinges 0
Displacements = -0.005 (mm)

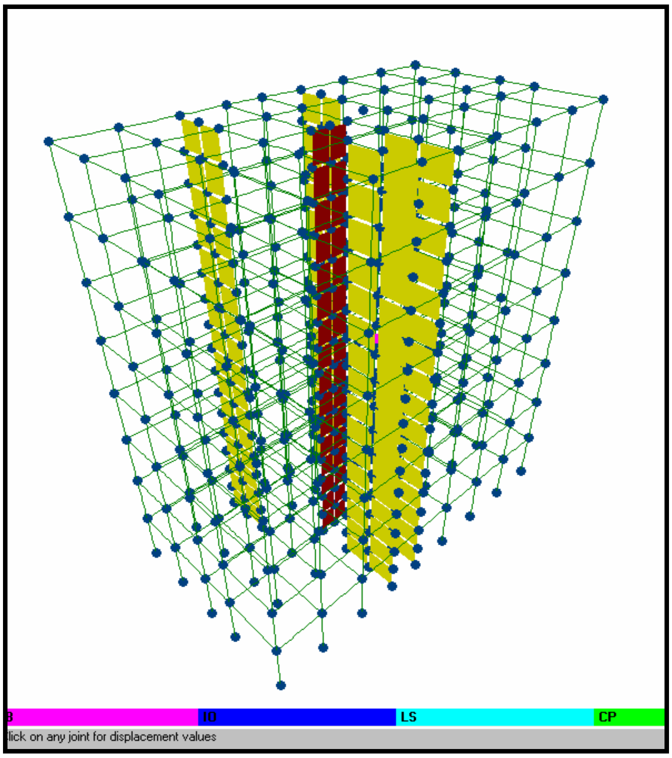


Figure 34-b. Deformed shape at step # 1
No. of hinges 1
Displacements = 12.82 mm

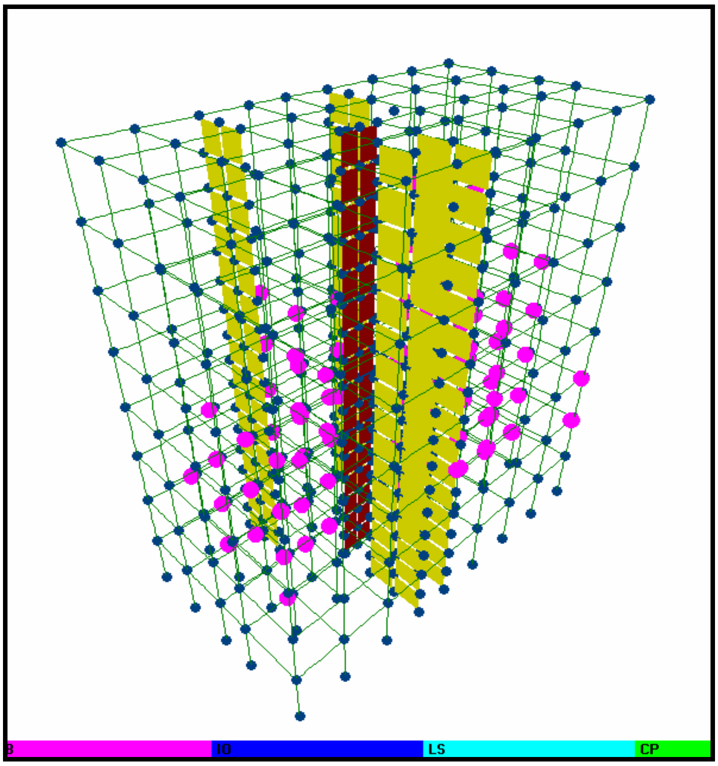


Figure 34-c. Deformed shape at step # 2
No. of hinges 99
Displacement = 33.3 mm

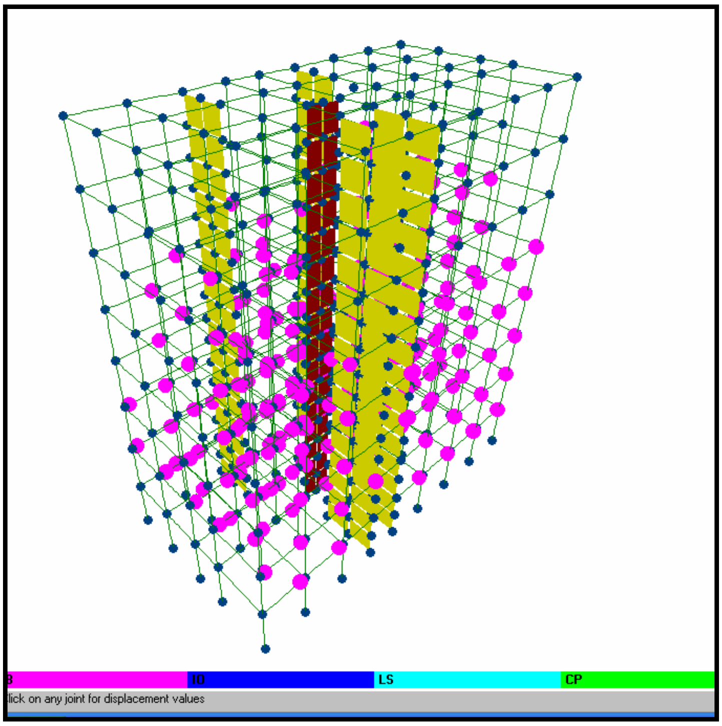


Figure 34-d. Deformed shape at step # 3
No. of hinges 247
Displacements = 53.9 mm

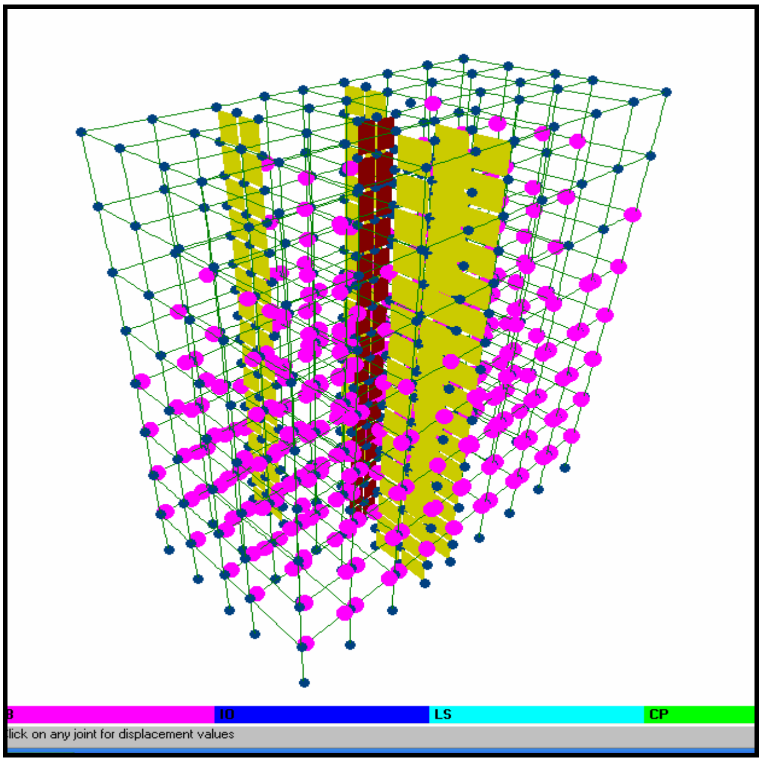


Figure34-e. Deformed shape at step # 4
No. of hinges 392
Displacement (73.9 mm)

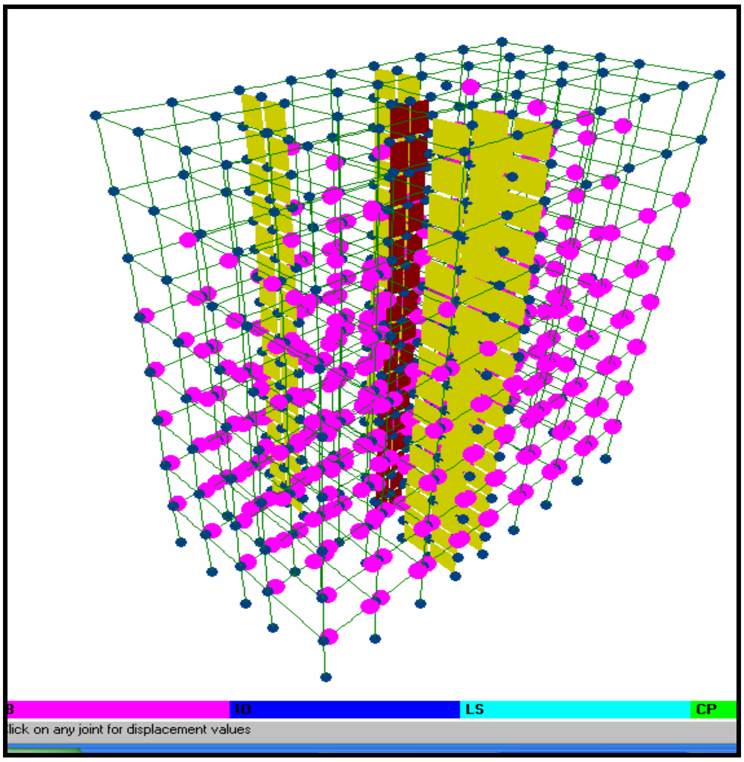


Figure 34-f. Deformed shape at step # 5
No. of hinges 452 B, 3 lS
Displacements(95 mm)

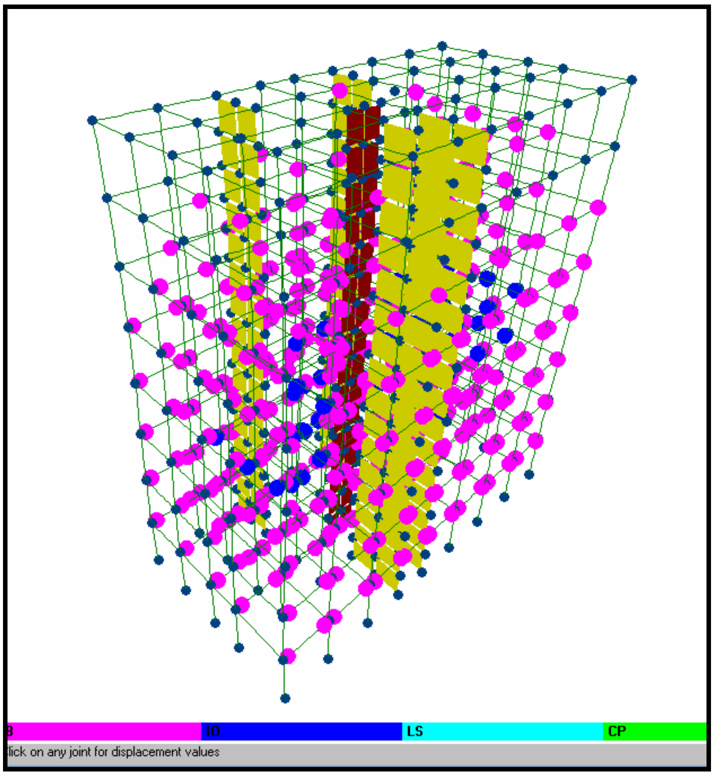


Figure 34-g. Deformed shape at step # 6
No. of hinges 448 B, 46 LS
Displacements = 115.1 mm

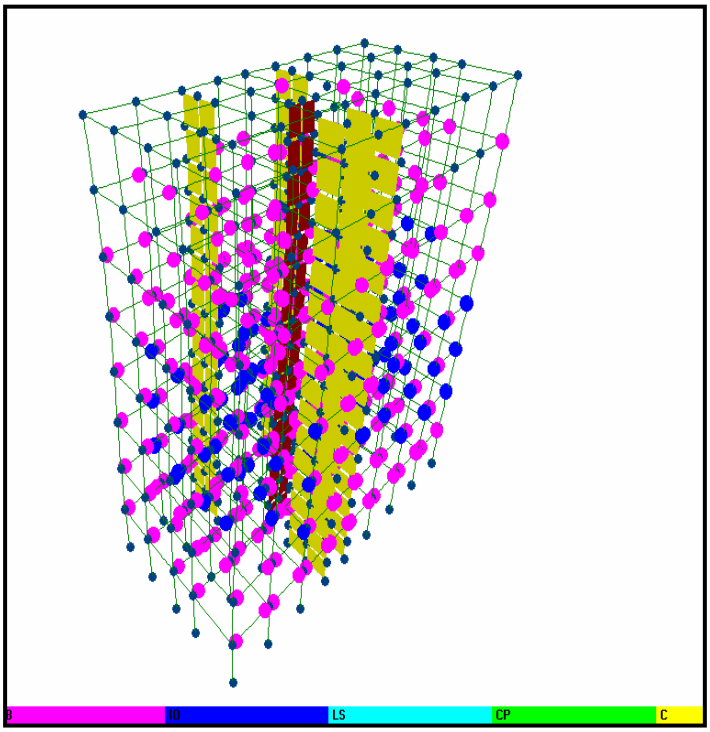


Figure 34-h. Deformed shape at step # 7
No. of hinges = 403 B, 116 LS
Displacements = 135.3 mm

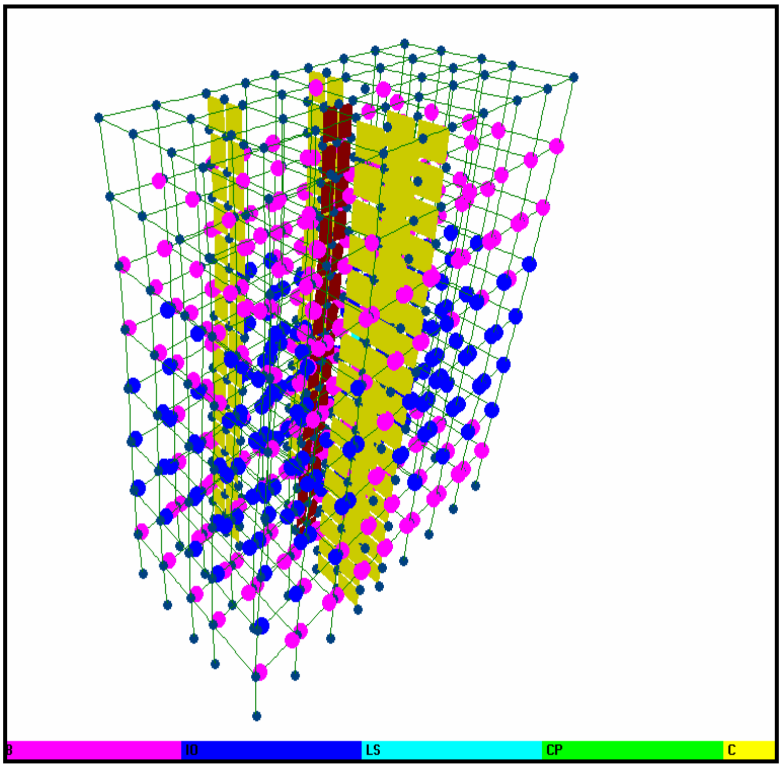


Figure 34-i. Deformed shape at step # 8
No. of hinges = 325 B, 214 LS, 2 CP
Displacements = 158.7 mm

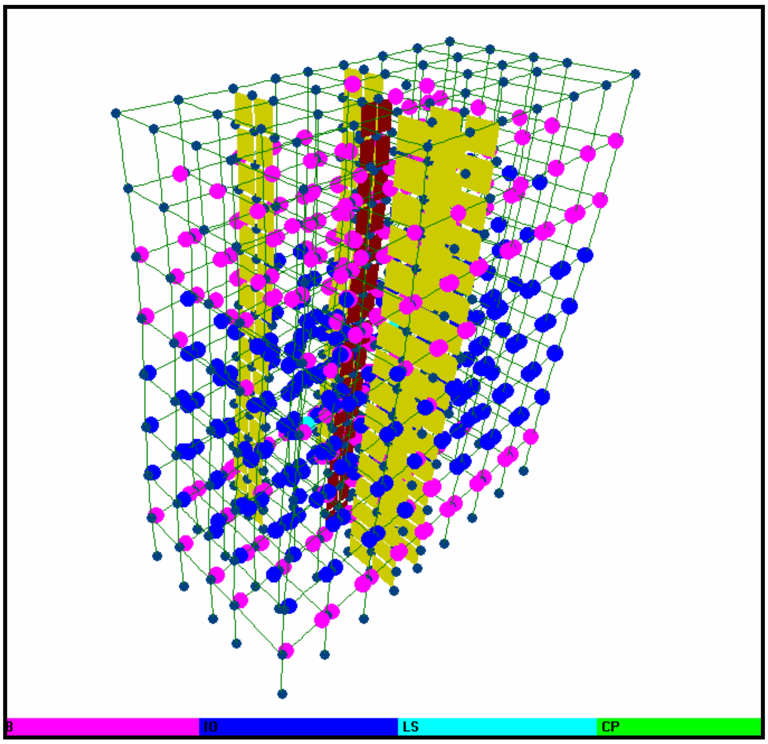


Figure 34-j. Deformed shape at step # 9
No. of hinges = 270 B, 282 LS, 4 CP
Displacement = 179 mm

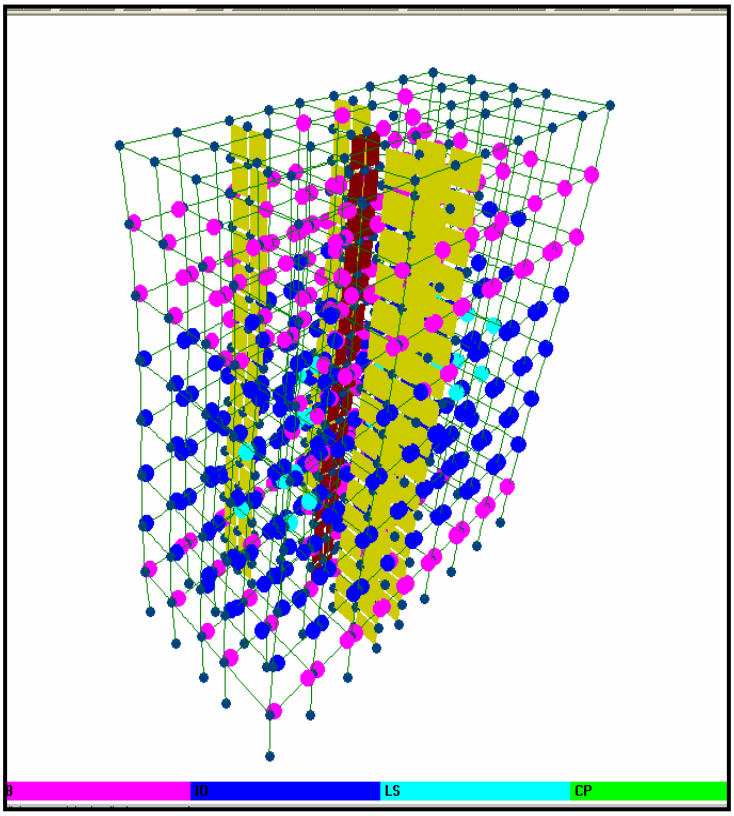


Figure 34-k. Deformed shape at step # 10
No. of hinges = 247 B, 300 LS, 29 CP
Displacements = 201.3 cm

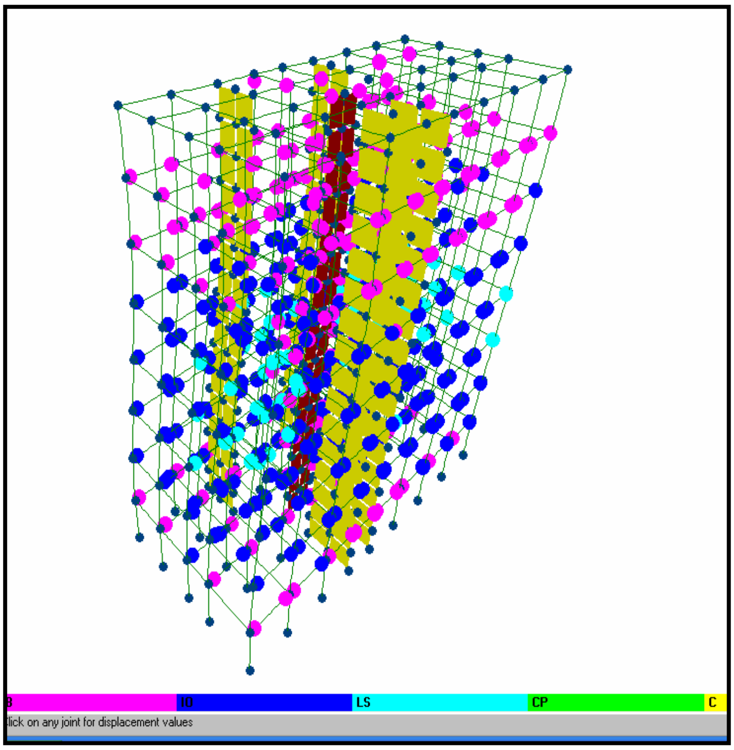


Figure 34-L. Deformed shape at step # 11
No. of hinges = 228 B, 304 LS, 68 CP
Displacements = 222.8 mm

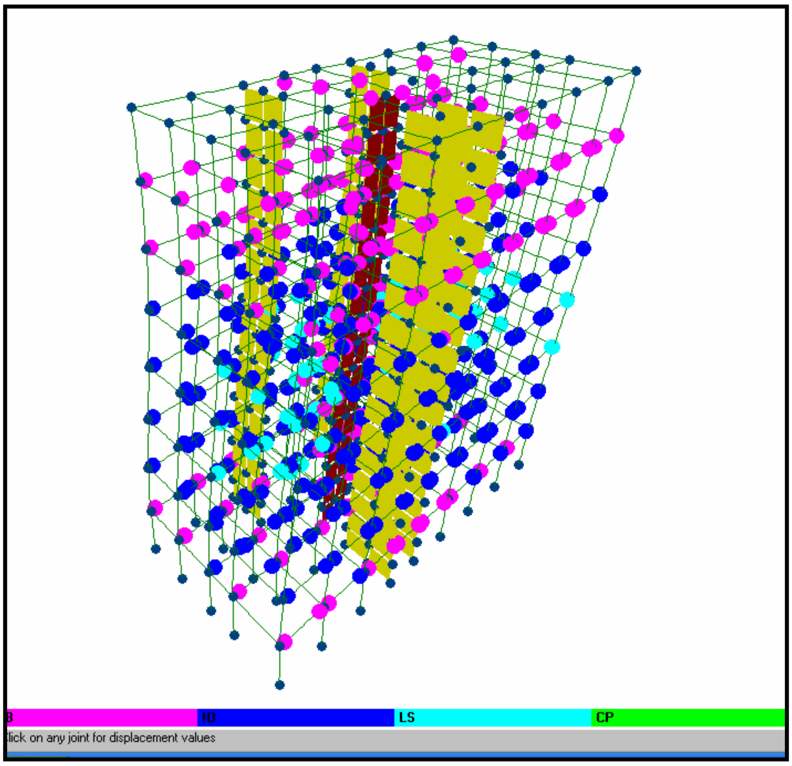


Figure 34-m. Deformed shape at step # 12
No. of hinges 203 B, 269 LS, 143 CP
Displacements (246.4 mm)

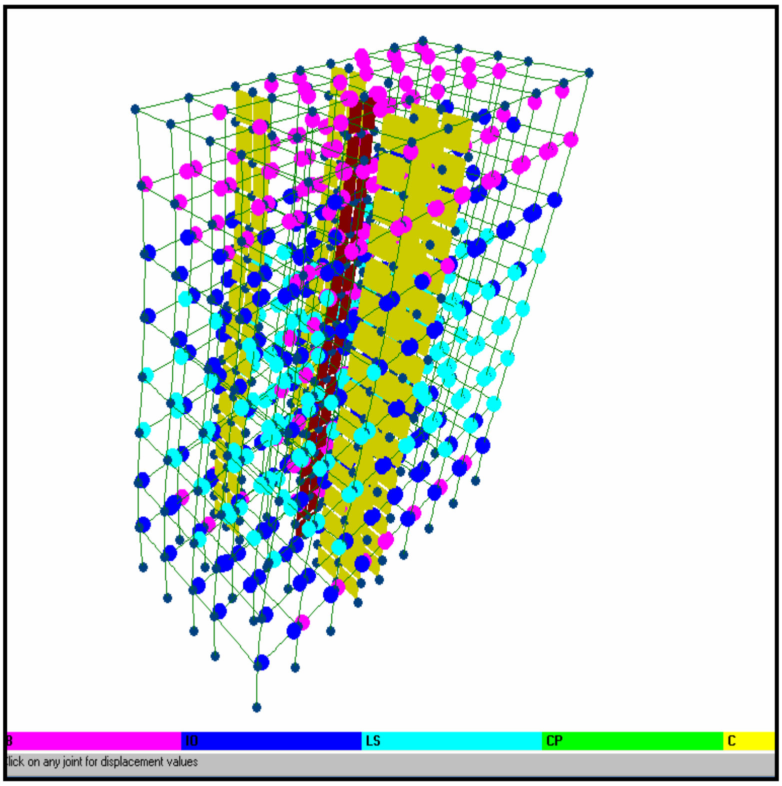


Figure 34-n. Deformed shape at step # 13
No. of hinges =178 B, 249 LS, 205 CP
Displacements (268.7 mm)

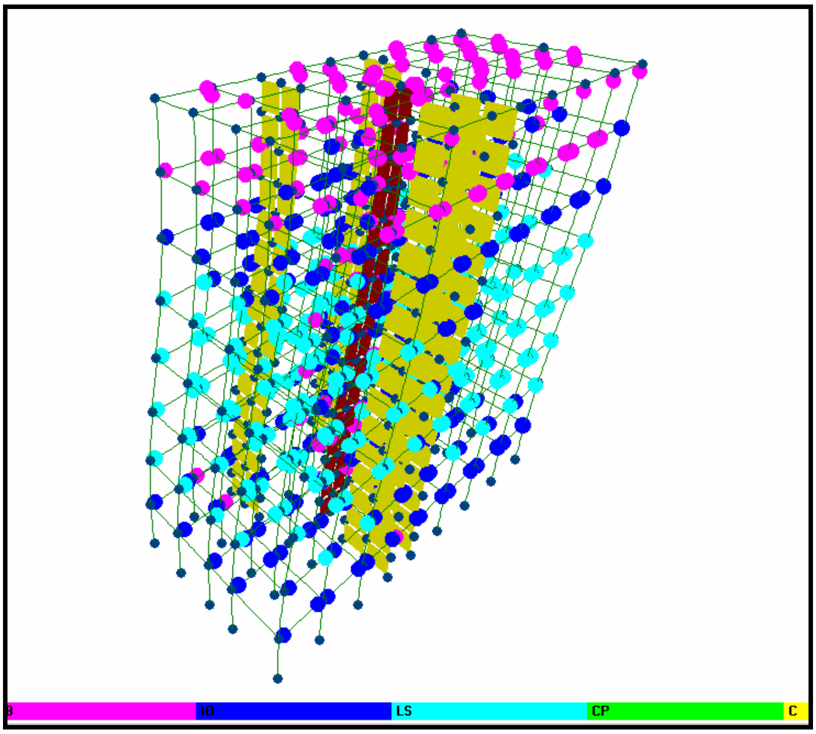


Figure 34-o. Deformed shape at step # 14
No. of hinges =178 B, 249 LS, 205 CP
Displacements(294.5 mm)

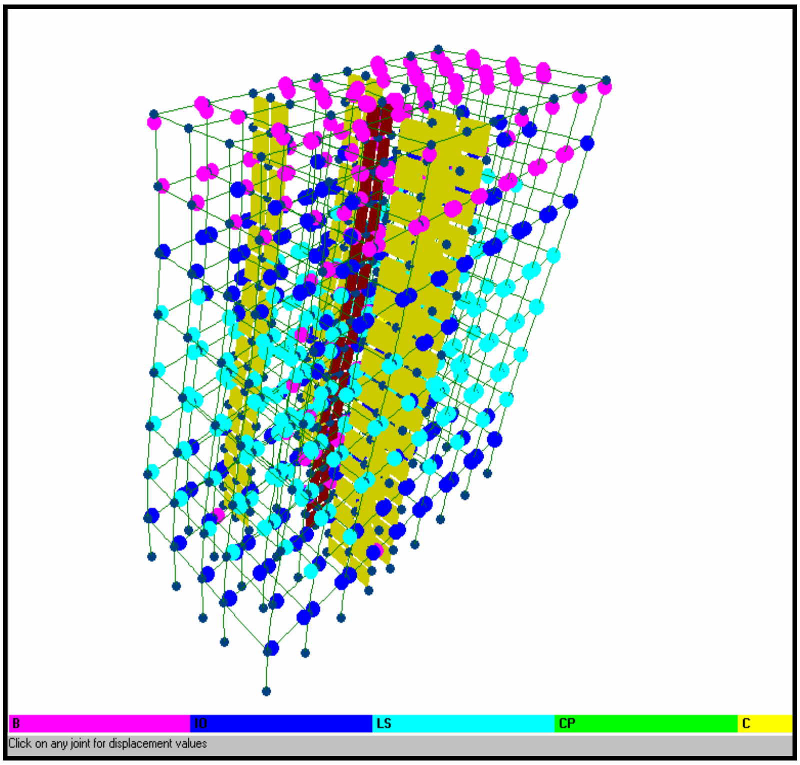


Figure 34-p. Deformed shape at step # 15
No. of hinges = 158 B, 222 LS, 0 CP, 1 D
Displacements (299.5 mm)

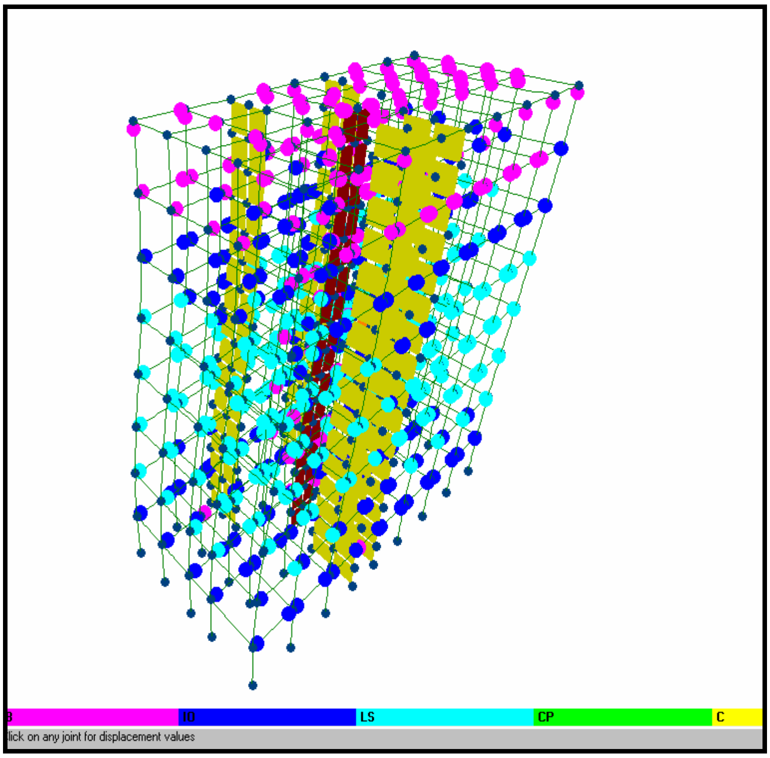


Figure 34-q. Deformed shape at step # 16
No. of hinges =158 B, 222 LS, 0 CP, 1 D, 1 E
Displacements (266.9 mm)

6. Discussion of Results

The nonlinear static procedure is intended to provide a simplified approach for directly determining the nonlinear response behavior of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism and initiation of collapse. Response behavior is gauged by a measurement of the strength of the structure at various increments of lateral displacement.

Generally, if a structure is subjected to lateral loads larger those that represented by the elastic strength, a number of elements will yield,eventually forming a mechanism. Standard methods of plastic or limit analysis can be used to determine the strength corresponding to such mechanism. If after the structure develops a mechanism, it deforms an additional substantial amount, elements within the structure may fail and thus cease to contribute strength to the structural system. In such cases, the strength of the structure will diminish with increasing deformation. Figure 35, which is a plot of the lateral structural strength vs. deformation (or pushover curve) for a hypothetical structure, illustrates theses concepts.

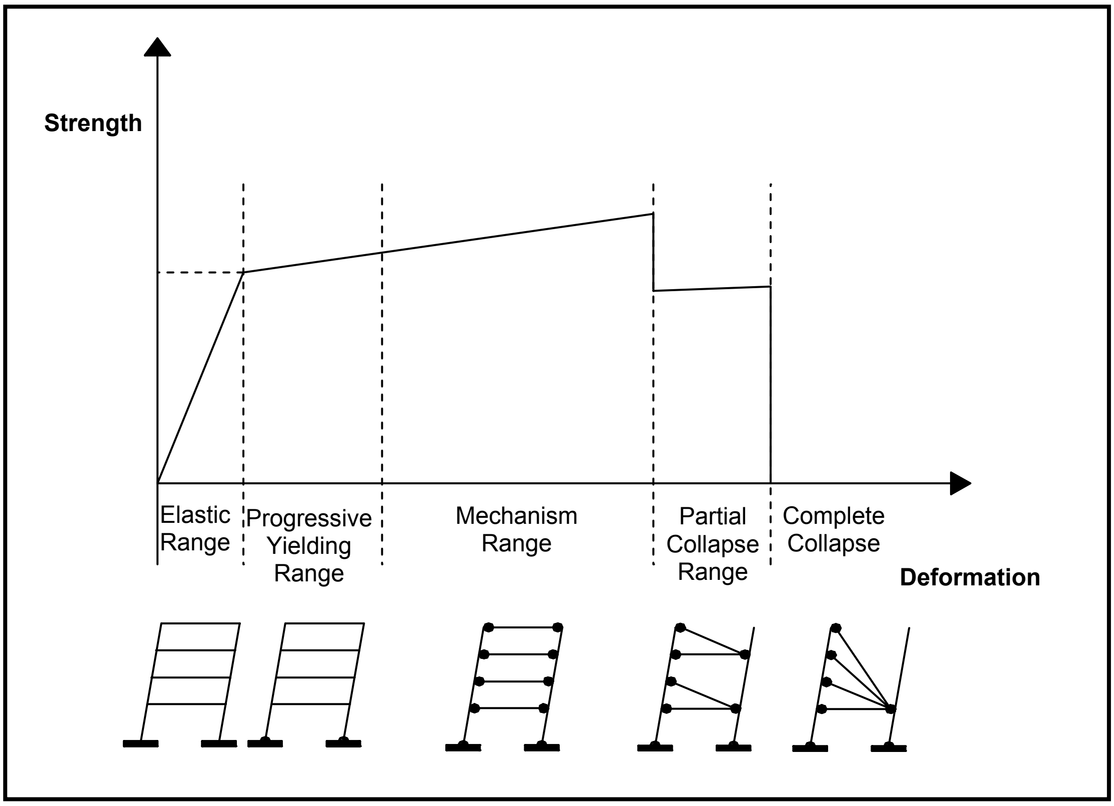


Figure 35. Strength–deformation relation for a frame structure(www.bsscoline.org)

As shown in the figure, many structures exhibit a range of behavior between the development of first yielding and development of a mechanism. When the structure deforms while elements are yielding (shown as progressive yielding), the relation between external forces and deformations cannot be determined by simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static procedure is to provide a simplified method of determining the structural response behavior at deformation levels between those that cannot be conveniently analyzed using limit state methods.

Let's go back to the three dimensional model assumed in the study, analysis results including fundamental period, base shear, rotation and displacement in x-direction, story drift and story shear are plotted in the figures below for the four methods used in the analysis: static force, response spectrum, time history and pushover methods. Figures show that, up to a certain level of accuracy, nonlinear static pushover analysis is capable of predicting the structure's seismic response and its performance level, especially when compared to results of the dynamic time history analysis.

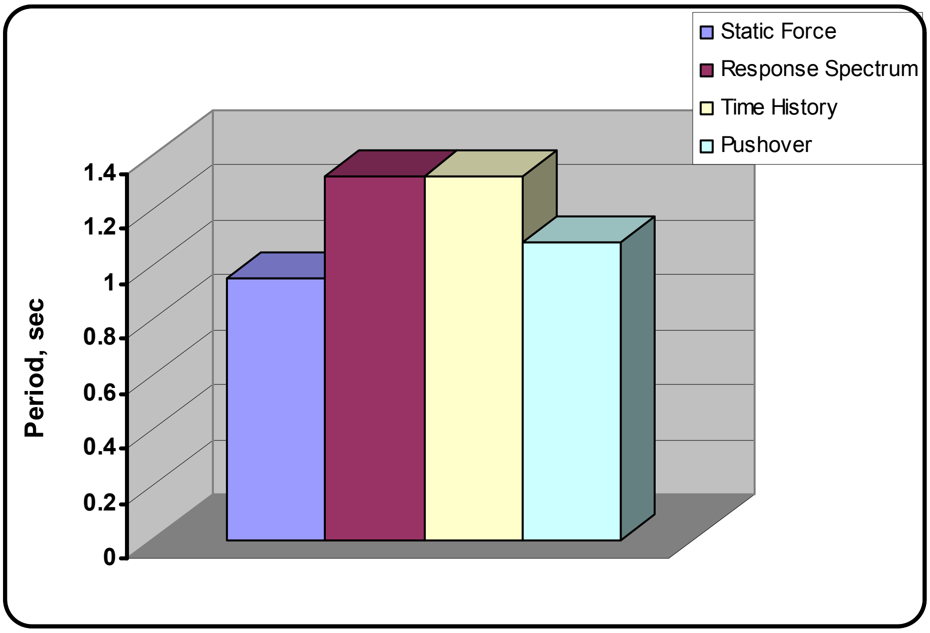


Figure 36. Fundamental Period, in seconds

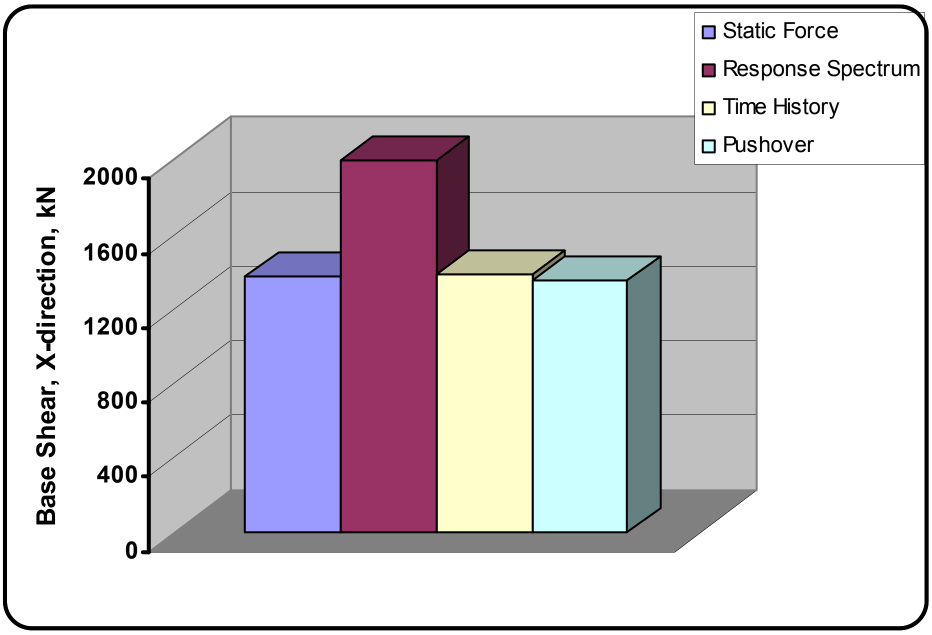


Figure 37. Base shear in x-direction

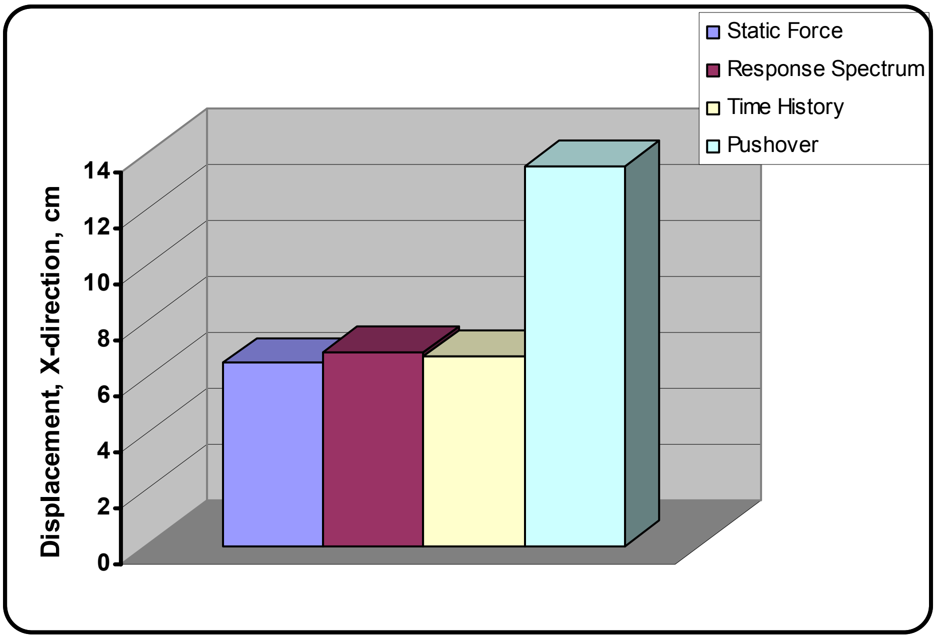


Figure 38. Maximum Displacement in x-direction

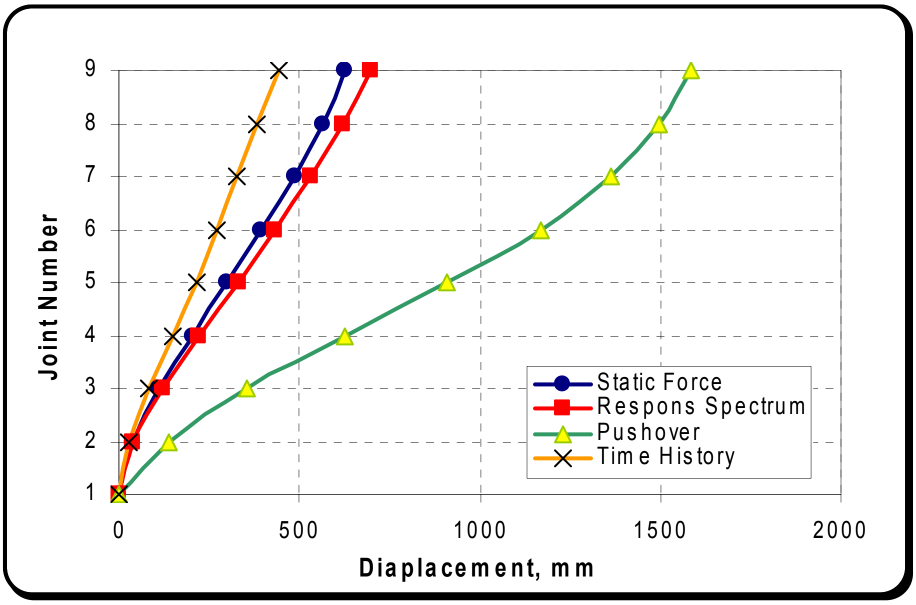


Figure 39. Displacement of joints 1-9

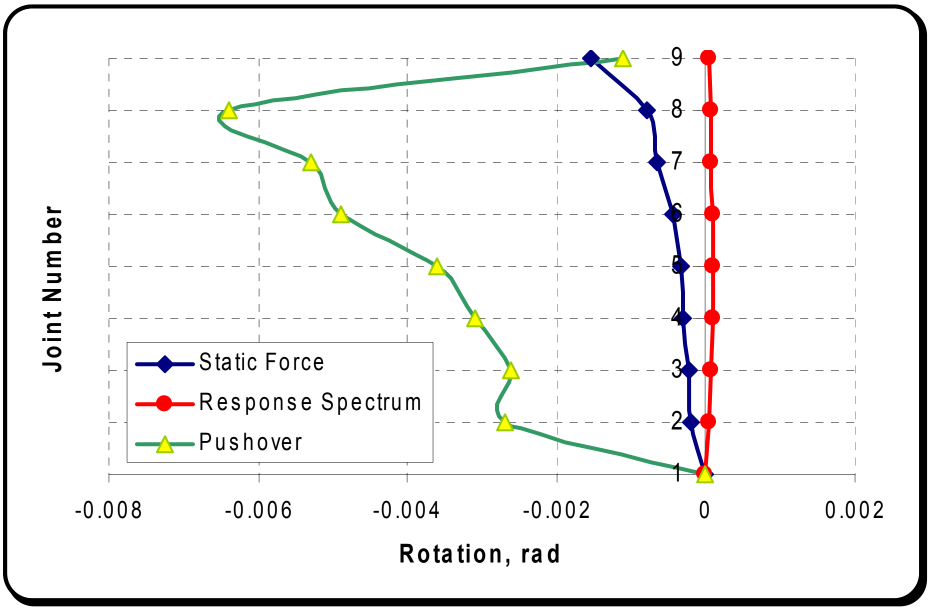


Figure 40. Rotation of joints 1-9

Rotation of joints for time history analysis = 0

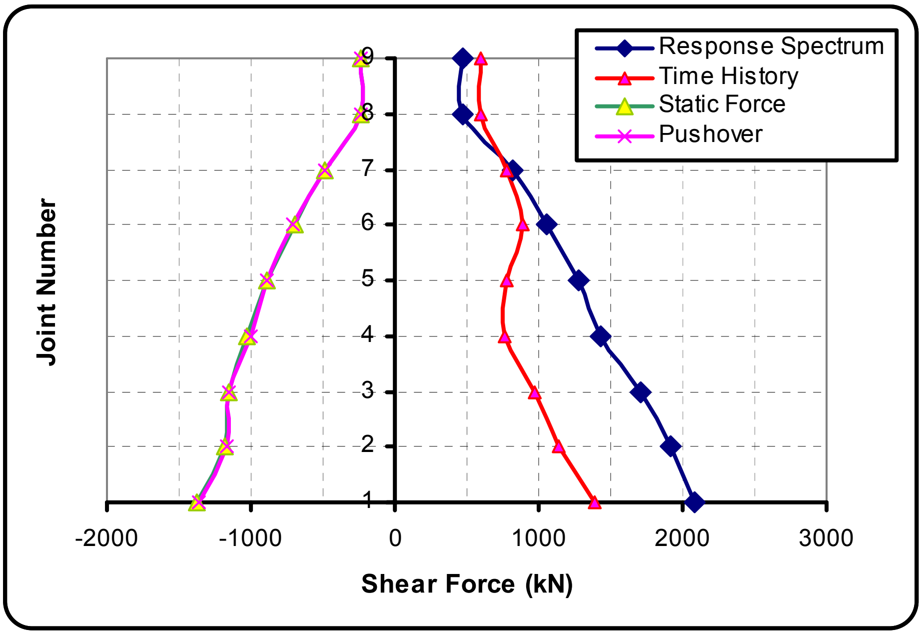


Figure 41. Story shear

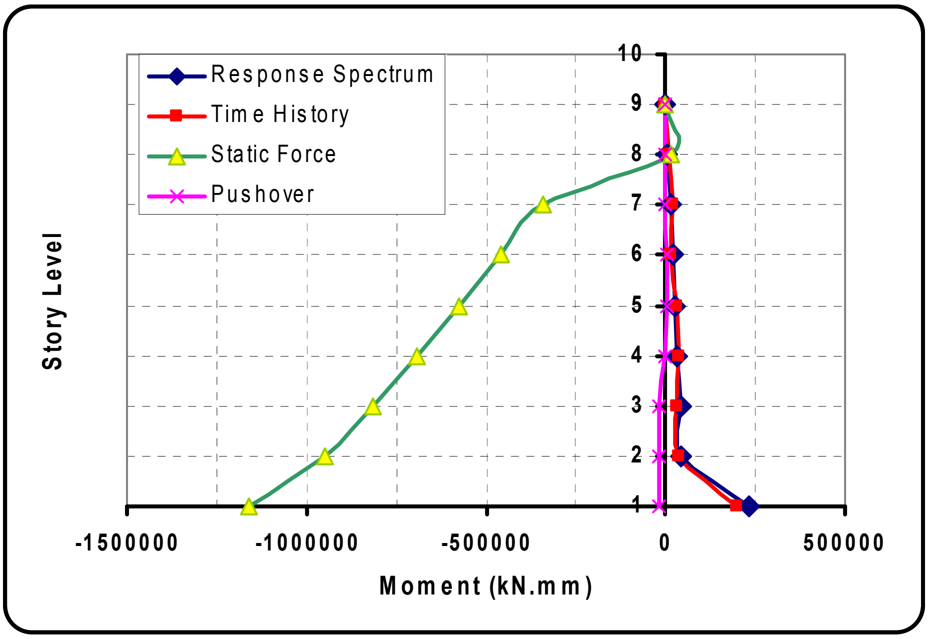


Figure 42. Overturning moment

Summary, Conclusions and Recommendations

1. Summary

Under the pressure of recent developments, seismic codes have begun to explicitly require the identification of sources of inelasticity in structural response, together with the quantification of their energy absorption capacity. Ideally, such performance evaluation of structural systems subjected to earthquake loading should be based on nonlinear time history analysis. However, the intrinsic complexity and the additional computational effort required by the latter do not justify its use in ordinary engineering applications. As a result of the above, nonlinear static, as opposed to dynamic, pushover analysis has been gaining significance over recent years as a tool for design verification. Indeed, and despite its simplicity and ease of use, this numerical tool can provide information on many important response characteristics that cannot be obtained from an elastic static or dynamic analysis.

In this study, and following a brief review of the latest developments in the field, the concept and accuracy of the pushover method is explored through comparison with results from linear static, linear dynamic and nonlinear dynamic analyses. Therefore, an 8-story building with a total height of 30.4m was considered. The structural system of the building consists of nine reinforced concrete ordinary moment resisting frames in each direction with four shear walls in Y direction only. The building was modeled as a three dimensional system using the software SAP2000 software . The design seismic parameters including the fundamental period, base shear, joint displacement and joint rotation for the assumed model were determined using the static force procedure, as recommended in the UBC-97 code, response spectrum analysis using the UBC-97 design response spectrum, time history analysis using the EL-Centro earthquake record and finally using the pushover method. Results of analysis were compared, through illustrative charts, and discussed.

2. Conclusions

1. Nonlinear static pushover analysis has served well as an efficient and easy-to-use alternative to dynamic time-history analysis, since, despite its simplicity, it is capable of providing important structural response information. Indeed, pushover can be employed to identify critical regions, where inelastic deformations are expected to be high, and strength irregularities in plan or elevation that might cause important changes in the inelastic dynamic response characteristics.

2. This type of analysis is also capable of predicting the sequence of yielding and/or failure of structural components and the progress of the overall capacity curve of the structure, thus verifying the adequacy of the seismic load.

3. When a structure deforms while elements are yielding (known as progressive yielding), the relation between external forces and deformations cannot be determined by a simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static procedure is to provide a simplified method of determining the structural response behavior at deformation levels between those that cannot be conveniently analyzed using limit state methods.

4. Nonlinear static procedure can be used efficiently to evaluate the performance level of reinforced concrete buildings subjected to seismic loading.

3. Recommendations

1. Nonlinear static pushover analysis is recommended as an efficient and easy-to-use alternative to dynamic time-history analysis due to its simplicity and capability of predicting the sequence of yielding and/or failure of structural components and evaluating the performance of reinforced concrete buildings.
2. Nonlinear static procedures are especially recommended for analysis of buildings with irregularities.
3. Pushover method should not be used for structures in which higher mode effects are significant unless a LDP evaluation is also performed to capture the effect of higher modes.

Summary, Conclusions and Recommendations

1. Summary

Under the pressure of recent developments, seismic codes have begun to explicitly require the identification of sources of inelasticity in structural response, together with the quantification of their energy absorption capacity. Ideally, such performance evaluation of structural systems subjected to earthquake loading should be based on nonlinear time history analysis. However, the intrinsic complexity and the additional computational effort required by the latter do not justify its use in ordinary engineering applications. As a result of the above, nonlinear static, as opposed to dynamic, pushover analysis has been gaining significance over recent years as a tool for design verification. Indeed, and despite its simplicity and ease of use, this numerical tool can provide information on many important response characteristics that cannot be obtained from an elastic static or dynamic analysis.

In this study, and following a brief review of the latest developments in the field, the concept and accuracy of the pushover method is explored through comparison with results from linear static, linear dynamic and nonlinear dynamic analyses. Therefore, an 8-story building with a total height of 30.4m was considered. The structural system of the building consists of nine reinforced concrete ordinary moment resisting frames in each direction with four shear walls in Y direction only. The building was modeled as a three dimensional system using the software SAP2000 software . The design seismic parameters including the fundamental period, base shear, joint displacement and joint rotation for the assumed model were determined using the static force procedure, as recommended in the UBC-97 code, response spectrum analysis using the UBC-97 design response spectrum, time history analysis using the EL-Centro earthquake record and finally using the pushover method. Results of analysis were compared, through illustrative charts, and discussed.

2. Conclusions

5. Nonlinear static pushover analysis has served well as an efficient and easy-to-use alternative to dynamic time-history analysis, since, despite its simplicity, it is capable of providing important structural response information. Indeed, pushover can be employed to identify critical regions, where inelastic deformations are expected to be high, and strength irregularities in plan or elevation that might cause important changes in the inelastic dynamic response characteristics.

6. This type of analysis is also capable of predicting the sequence of yielding and/or failure of structural components and the progress of the overall capacity curve of the structure, thus verifying the adequacy of the seismic load.

7. When a structure deforms while elements are yielding (known as progressive yielding), the relation between external forces and deformations cannot be determined by a simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static procedure is to provide a simplified method of determining the structural response behavior at deformation levels between those that cannot be conveniently analyzed using limit state methods.

8. Nonlinear static procedure can be used efficiently to evaluate the performance level of reinforced concrete buildings subjected to seismic loading.

3. Recommendations

4. Nonlinear static pushover analysis is recommended as an efficient and easy-to-use alternative to dynamic time-history analysis due to its simplicity and capability of predicting the sequence of yielding and/or failure of structural components and evaluating the performance of reinforced concrete buildings.
5. Nonlinear static procedures are especially recommended for analysis of buildings with irregularities.
6. Pushover method should not be used for structures in which higher mode effects are significant unless a LDP evaluation is also performed to capture the effect of higher modes.

طريقة الازاحه في تصميم المنشأت ، التحليل اللاخطي لتحديد ألية الانهيار في المنشات الخرسانيه

إعداد
مجد نافذ العطار

المشرف
أ. د سميح قاقيش

الملخص

تتبنى كودات التحليل الزلزالي وعلى نطاق واسع عالمياً طريقة التحليل الستاتيكي الساكن والتي تعتمد على تحويل القوى الزلزالية الديناميكية إلى قوة أفقية ستاتيكية مكافئة تؤثر على المبنى بالاتجاه الأفقي وفق المحاور الأساسية للمبنى كما تعتمد على أسس التحليل المرن في المنشآت .وتطبق هذه الطرق على تحليل المنشآت البسيطة والتي تحقق اشتراطات محدودة, كما وتتطلب الكودات السابقة تحقيق تفصيلات تنفيذية في توزيع حديد التسليح بهدف ضمان السلوك اللاخطي للمنشآت تحت تأثير الأحمال الزلزالية .

كما أن البحوث القائمة و التوجهات الحالية : تعتمد على دراسة سلوك المنشآت اللاخطية و السماح بتشكل مفاصل لدنة في المنشأت لإمتصاص الطاقة الناجمة عن الزلازل الأمر الذي يؤدي لزيادة تحمل المنشأ على الإنتقال و بالتالي زيادة اقتصادية الحل ووثوقيته من خلال تحديد ألية ميكانيكيات الإنهيار المتوقعة للمنشأ. . وقد اعتمد الباحث على الكودات التاليه

(FEMA-273،ATC-96) ويجب التنويه هنا بأن طرق القوى الستاتيكية الخطية السابقة تتصف بعدم الدقة في تحديد مراحل تشكل المفاصل اللدنة في المنشآت وبالتالي خروج المنشأ عن الاستثمار عند تشكل ميكانيكيات انهيار جزئية أو كلية ضمن أي سوية طابقية من سوياته .

يهدف هذه البحث إلى دراسة واستقصاء طريقة التحليل اللاخطي الساكن (الطريقة الدفعية) في التحليل الزلزالي للمباني .

قام الباحث بدراسة ونمذجة مبنى مكون من جملةانشائيه مختلطه (اطارات خرسانيه+ جدران قص)بالاتجاه العرضي ، وتمت دراسة النموذج الرياضي ثلاثي، الأبعاد للمبنى على الطريقه الساكنه ، والطريقه الديناميكيه (طريقة تحليل الاطوار باستخدام مخططات طيف الاستجابه ، وطريقة التحليل الزمني اللاخطي) باستخدام البرمجيه المعروفه (SAP2000)، والطريقه الدفعيه الساكنه اللاخطيه وتمت المقارنه بين النتائج بين الطرق الأربعه تحت تأثير الأحمال الزلزاليه

وتمت المقارنه بين قوى القص الناتجه في المبنى في الاتجاهين ، وبين دور الاهتزاز الرئيسي ، وبين العزوم الناتجه من تأثير القوى الزلزاليه ، وقيم الازاحه الطابقيه الناتجه من تأثير الحمولات الساكنه والحمولات المتغيره مع الزمن ، كما تم دراسة سلوك المنشأ اللاخطي ، من خلال تعريف المفاصل اللدنه وخصائصها الى النموذج الرياضي في الطريقه اللاخطيه عند نقطةالانجاز (وهي النقطه الناتجه من تقاطع طيف الاستجابه للموقع مع مخطط التحمل الأقصى للمبنى) .

. يعتمد إجراء التحليل الستاتيكي اللاخطي (الطريقة الدفعية) على ثلاث مبادئ أساسية وهي:

1- توليد مخطط الزلزالي للموقع.

2- توليد مخطط طيف التحمل أو المخطط الدفعي للبناء: باستخدام طريقة لاخطية مبسطة والذي يوضح الانتقال في أعلى سطح البناء كتابع للقوى الأفقية المطبقة على المنشأة .

3- تحديد نقطة الإنجاز أو التصميم .

4- عملية التحقق من القوى والعزوم المتولدة في عناصر المنشأة .

<u>بعد مقارنة النتائج ، توصل الباحث الى النتلئج التاليه :</u>

1- الطريقة الدفعية طريقة ذات وثوقية أكبر من حيث إظهارها لتتابع ميكانيكية الانهيار الجزئية وصولاً إلى ميكانيكية الانهيار الكلية للمبنى .

2-كما تبين بأن الطريقه الدفعيه هي قريبه من نتائج الطريقه الديناميكيه ،

3- نظراً لما تمثله مخططات طيف الاستجابة من أهمية في الطريقة الدفعية نأمل بإنشاء مخططات أطياف الاستجابة للأردن تتناسب مع نوعية التربة وأهمية المنشأ .

4- مستوى القوى الزلزالية التي تتحملها الأبنية المصممة وفق الكود الأردني أعلى من مستوى القوى الزلزالية التي يمكن أن تتعرض لها .

5- تمثل الطريقة الدفعية أهمية كبيرة في دراسة وتحقيق الأبنية على الزلازل لذلك نأمل في وضع اشتراطات خاصة لهذه الطريقة بالكودالأردني .

6 - بعد اجراء التحليل الدفعي على المنشأ وتحديد نقطة الانجاز (التصميم) للمنشأ يتم حساب القوى والتشوهات لعناصر المنشأ عند هذه النقطة والتحقق من قدرة تحمل تلك العناصر ومقارنة قيم تشوهاتها بالقيم المسموحة .

إن القيم المسموحة لتشوهات العناصر (دورانات , انتقالات) تتعلق بمستوى التحقق من الخطر الزلزالي للبناء إذ يتم تقسيم مستويات التحقق للبناء المعرض للزلزال إلى مستويات عدة ويحدد لكل مستوى قيم أعظمية مسموح بها لتشوهات عناصر المنشأ .

7:إن القيم المسموح بها لتشوهات العناصر في كل مستوى تحقق تعتمد على طبيعة الأضرار التي يمكن أن تصيب عناصر المنشأ بعد انتهاء الزلزال .

تم تقسيم الخطر الزلزالي إلى مستويات عدة اعتماداً على طبيعة الأضرار التي تصيب عناصر المنشأ وهذه المستويات هي :

7-1- المستوى الأول :(Immediate Occupancy) :

الأضرار التي تصيب عناصر المنشأ بعد انتهاء الزلزال أضرار بسيطة يمكن إهمالها.

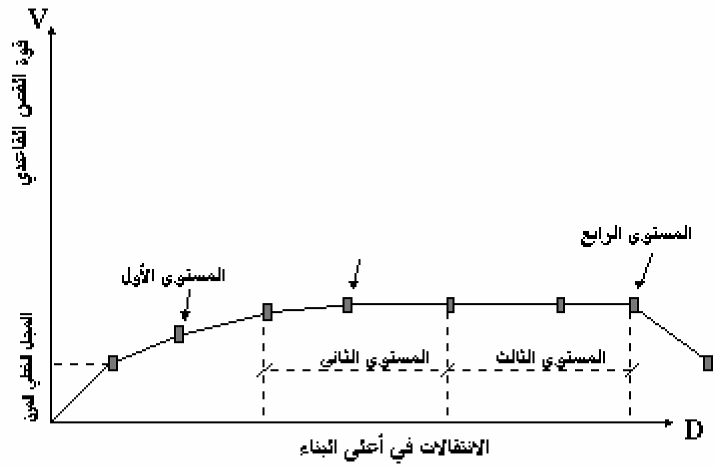
7-2- المستوي الثاني (Damage Control):

يقع بين المستوى الأول والثالث .

7-3- المستوي الثالث (Life Safety) :

الأضرار تصيب بعض عناصر المنشأ غير الأساسية والتي يمكن إصلاحها بعد انتهاء الزلزال.

7-4- المستوي الرابع (Structural Stability):
يصيب المبنى في هذا المستوى أضرار فادحة لا يمكن إصلاحها وقد تؤدي هذه الأضرار لحدوث انهيارات جزئية أو كلية للمنشأ .
إن عملية تحقق المنشأ على الخطر الزلزالي تعتمد على درجة أهمية المنشأ ويمكن توضيح مستويات التحقق للبناء من خلال الشكل التالي :



بنفس الأسلوب يتم تحديد مستوى التحقق لكل عنصر من عناصر المنشأ على حدة ويمكن توضيح ذلك من خلال الشكل التالي .