SEISMIC PERFORMANCE EVALUATION OF FRAMED BUILDINGS DESIGNED AS PER BANGLADESH NATIONAL BUILDING CODE

BY

SABBIR SIDDIQUE

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Approved as to style and content by:

(Dr. Ishtiaque Ahmed) Professor Dept. of Civil Engineering, BUET, Dhaka	Chairman (Supervisor)
(Dr. Md. Mazharul Hoque) Professor and Head Department of Civil Engineering, BUET, Dhaka	Member (ex-officio)
(Dr. Md. Abdur Rouf) Professor Department of Civil Engineering, BUET, Dhaka	Member
(Dr. Raquib Ahsan) Associate Professor Department of Civil Engineering, BUET, Dhaka	Member
(Dr. Md. Jahangir Alam) Professor Department of Civil Engineering, CUET, Chittagong	Member (External)

FEBRUARY 2006

TO

MY PARENTS

AND

Supervisor

Declaration

I hereby certify that the research work reported in this thesis has been performed by me and that this work has not been submitted elsewhere for any other purpose, except for publication.

February 2006

(Sabbir Siddique)

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NOTATIONS

$\ddot{u}^t$	Total displacement at time instant t
$\ddot{u}_g(t)$	Total displacement at time instant t due to ground motion
ω_n	Natural frequency at n^{th} mode
ζ	Critical damping
$P_{eff}(t)$	Effective earthquake force at time instant t
ω'	Radial frequency of the effective first mode
a	Acceleration due to gravity
A(t)	Pseudo acceleration
A'_s	Compression Steel area
A_g	Gross concrete area
A_s	Tensile Steel area
b_w	Width of beam stem
c	Damping coefficient
C_A	Seismic coefficient
Ct	Numerical coefficient
C_V	Seismic coefficient
d	Lateral displacement
f'_c	28 days cylinder strength of concrete
f_D	Force due to damping
f_I	Force due inertia
F_n	Lateral force at level n
f_S	Inertia force
$f_{s(t)}$	Force at time instant t
F_t	Concentrated force on roof top for accommodating higher mode
F_x	Lateral force at level x
f_y	Yield strength of steel
g	Acceleration due to gravity
h_n	Height at level n
h_x	Height at level x
Hz	Unit for frequency

I	Second moment of Inertia
k	Stiffness of a system
m	Mass of a system
M_3	Moment about major axis
$M_{b(t)}$	Moment at base at time instant t
N_A	Near source coefficient for seismic source
N_V	Near source coefficient for seismic source
P	Axial force
$P_{(t)}$	Force at time instant t
P_c	Axial force contributed by concrete
PF_1	Modal participation factor for the first mode
P_i	Total gravity load at level i
Py	Axial force up to yield
Q	Lateral load
Q_v	Lateral load up to yield level
R	Response modification factor
S_a	Spectral acceleration
S_{ai}	Spectral acceleration at time instant i
S_d	Spectral displacement
S_{di}	Spectral displacement at time instant i
T	Time period
T'	Effective time period
T_A	Coefficient
T_S	Coefficient
u	Displacement
$u(t)$	Displacement at time instant t
u_g	Displacement due to ground acceleration
u_t	Total displacement
V	Base shear
$V_{b(t)}$	Shear force at base at time instant t
V_i	Total calculated share force at level i
W	Seismic dead weight

Z	Zone coefficient
$\acute{\upsilon}$	Velocity
ΔT	Time increment
Δ_y	Yield displacement
$\Phi_{1, Roof}$	Roof level amplitude for the first mode
α_1	Modal mass coefficient for the first mode
ϕ_{i1}	Amplitude of mode 1 at level i
ρ	Steel ratio
ρ'	Compression steel ratio
ρ_{bal}	Balanced steel ratio

ABBREVIATIONS

ACI	American Concrete Institute
ADRS	Acceleration Displacement Response Spectrum
ATC	Applied Technology Council
BNBC	Bangladesh National Building Code
C	Conforming transverse reinforcement
CP	Collapse prevention performance
EPA	Effective peak acceleration
FEMA	Federal Emergency Management Agency
IO	Immediate occupancy performance
IS	Indian Standard
LS	Life safety performance
NC	Non-conforming transverse reinforcement
RSA	Response Spectrum Analysis
SAP	Structural Analysis Program of CSI
UBC	Uniform Building Code

Chapter 1

INTRODUCTION

1.1 Preamble

Response of a building during earthquake is a complicated issue. Till now, no mathematical tool is available to predict the behavior of a structure during earthquake accurately. Basically, this is because of the unpredictable nature of the earthquake excitation that might occur at a specific time and site and then resulting complicated response of a building itself.

Now-days, due to advancement of computer technology, different tools are being developed to capture and predict the response of a building due to specified earthquake excitation.

The seismic forces specified in the code are quite small relative to the actual force expected at least once in the life of the structure. Acceleration derived from actual earthquakes is surprisingly high when compared to the code forces used in design.

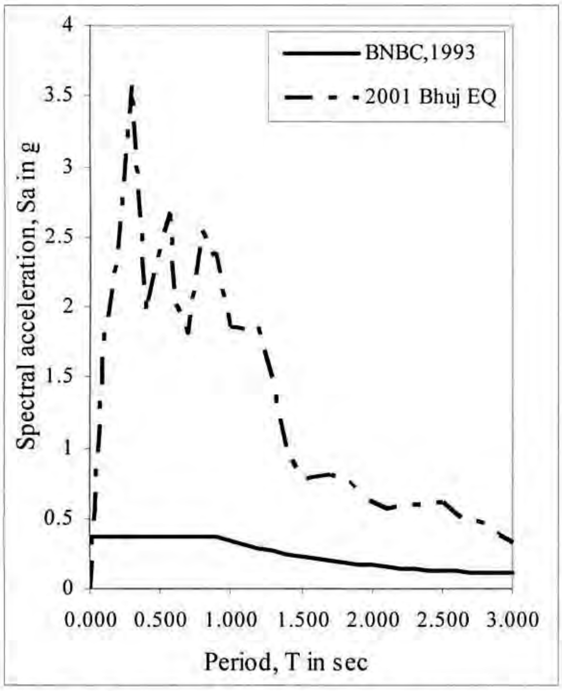


Fig. 1.1 Comparison of BNBC spectral acceleration to the 2001 Bhuj acceleration.

The Fig.1.1 shows a comparison of Bangladesh National Building Code (BNBC) response spectra for Zone-II (Seismic coefficient = 0.15) and Soil type – III superimposed with the corresponding data of 2001 Bhuj earthquakes. It can be seen that the spectral acceleration of *Bhuj* Earthquake (N78E component) in the predominant range is 4-9 times higher than the spectral acceleration of BNBC. So, the resulting stress in a member of a structure shall be much higher if analyzed with *Bhuj* response spectrum than response spectrum of BNBC, 1993.

Earthquake design philosophy lies in the uncertainty of the event. It is possible to design a structure to withstand a large earthquake and it is usually done for structure like Nuclear Reactor etc. For other structures like buildings, bridges the uncertainties of the event combined with economic consideration make the huge investment unfeasible.

There are three distinct approaches to earthquake resistant design. These are (ATC-40):

i. Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however building parts that do not carry load may sustain repairable damage

ii. Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake

iii. Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse

When earthquake is concerned, structurally main focus is given to the reserve strength beyond the elastic limit in the elasto-plastic state of a structure to remain workable during an earthquake and increase damping (thus reducing seismic demand) due to inelastic deformation. This is shown pictorially in the Fig. 1.2.

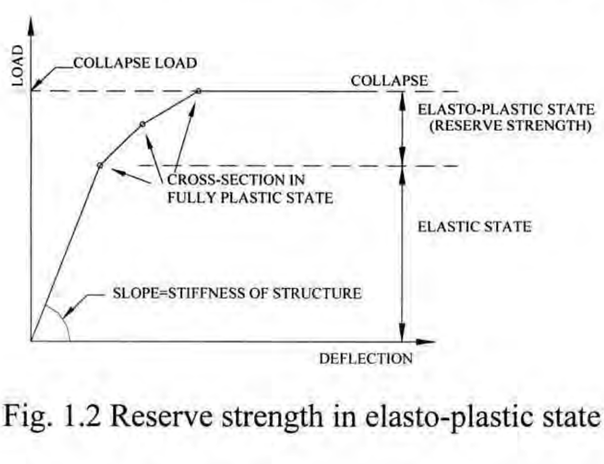


Fig. 1.2 Reserve strength in elasto-plastic state

— Each black dot, termed plastic hinge, corresponds to the fully plastic state of certain cross-section in the structure;

— Elastic state of a structure corresponds to a load level below the first plastic hinge; structures are generally designed for this load level;

— Elastic-plastic state of a structure corresponds to a load level between the first and the last hinges; this reserve strength is supposed to be used during an earthquake.

It is assumed implicitly that during the earthquake the structure will move to the inelastic range of deformation and use its reserve strength to resist the earthquake force. In the building codes like BNBC and of the similar, it is not explicitly explain how the deformation capabilities of a structure would be developed. Building codes, however, limits the total and inter-story deformation under certain level. But, essentially structures are likely to experience more deformation in its life time during a major earthquake.

Under the present provisions of BNBC and other codes, it is not possible to determine the state of a structure after a design earthquake or maximum creditable earthquake.

1.2 Background and present state of the problem

Bangladesh has a long history of good engineering practices. But unfortunately until 1993, there was no written code of standard civil engineering practice. In 1993, Bangladesh National Building Code was published by Building and Housing Research Institute which is commonly known as BNBC. The seismic design provisions of BNBC were based on the Uniform Building Code-1991 of United States of America. Especially the wide availability of computer technology has made it possible a more realistic simulation of structural behavior. The focus of seismic design in current building codes is one of life safety level. Economic losses due to recent earthquakes are estimated to be billions of Taka and the numbers will be higher if the indirect losses are included. This fact lets codes committees and decision makers think beyond life safety, which is essential in design, to alleviate economic losses. This trend creates an increased interest in performance-based design for structure. As a result of which a number of tools are now available for a more rational analysis of the structures under seismic loading (ATC-40, 1996; FEMA-356, 2000). One of the main advantages of performance-based designs its ability to show the performance situation of the structure and its components under different load intensities. The performance situation means that the damage level, if any, can be assessed and a judgment can be made as to which degree this structure can continue to service. At this backdrop, the seismic design provisions of the BNBC need to be evaluated.

Bangladesh, situated in between 20^0-34' and 26^0-38' north latitude and between 88^0-01' and 92^0-41' east longitude, is located on an active seismic zone. Part of the country extended from Sylhet to Chittagong is in the high seismic zone. BNBC-1993 divides the country into three zones of different seismic intensities. Zone –I, the least seismic zone where the expected ground acceleration is 0.075g, Zone-II- the moderate seismic zone where the expected ground acceleration is 0.15g and Zone-III- the high seismic zone where the expected ground acceleration is 0.25g. These probable ground accelerations are based on a standard probability of occurrence over a return period. From the history, it is seen that the last major earthquakes that hit the country and the adjoining area are the great Indian earthquake of 1897 and the 1918 Srimangal earthquake. Recently two major earthquakes in the Sub-continent namely in Bhuj, India and in Kandahar, Afghanistan costs thousands of lives. In recent times several small to moderate Earthquake hit Bangladesh and there are growing concerns that earthquakes of higher magnitude are likely to happen in this region in near future.

In order be able to cope with the seismic hazard, building code forms a strong basis. It is important that code provisions are scrutinized and updated following the current body of knowledge.

BNBC has different provisions for earthquake load calculation and analysis procedure. For the regular structures, Code defines a simple method to represent earthquake induced inertia forces by Equivalent Static Force for static analysis. For very tall structure or for irregular structure, Code provisions require more rigorous analysis, namely;

i. Response Spectrum Analysis and

ii. Time History Analysis

All these methods detailed in BNBC, 1993 are force based methods. As in many other codes, the level of forces prescribed by BNBC for a structure rather arbitrarily set and aimed at damage control performance objective. The Code approach is to design seismic load resisting system on the basis of a pseudo seismic load - obtained by dividing the actual load by response modification factor, R. The R value is specified by the Code for each structural system without explicitly defining the level of element (i.e. beam, column, connection etc.) ductility required for each system. Rather most codes implicitly assume that the enhanced ductile detailing would result in seismic energy dissipation and hence a reduced demand would result. The BNBC, however, even do not specify the specific level of detailing required for each category of the system. As a result, the Code forces are no better than a vague estimate of the actual force.

1.3 Objective of the Thesis

Main objectives of the present research can be summarized as below:

- To identify the short comings of the seismic design provision in BNBC
- To conduct numerical study to evaluate performance of typical frame structure buildings designed under BNBC to ascertain the level of safety against design earthquake.
- To study performance of selected frame structure buildings designed as per present code provisions with a view to assess the level of seismic performance.
- To develop recommendations for establishment of multilevel design objectives for future code provision to ensure desired performance of the building under seismic action.

1.4 Methodology

In order to achieve the above selected objectives, the research work has been initiated by studying seismic provisions of Bangladesh National Building Code (BNBC, 1993), and available literatures on Performance Based Design of building. A selection of medium rise frame structures (6 to 9 story) have been analyzed and designed using BNBC. These structures were then subjected to pushover analysis using nonlinear static procedure. The analysis included progressive damage of elements by inserting appropriate hinges as the structure is laterally pushed through. Geometric non-linearity (P-Δ effect) was included in the analysis. The resulting capacity curves (Base shear vs. roof deflection) represented structure's performance showing progressive yielding of members and ductility demand of a structure.

Earthquake demand for the structures was established as per site condition and the BNBC specified seismic coefficient. Performance of the building was evaluated by superimposing earthquake demand for the building on the developed capacity curve of the structure. Depending upon the capacity of the building under elastic demand, an inelastic seismic demand was developed incorporating inherent as well as inelastic hysteretic damping and the displacement demand (i.e. performance) of the building was then established. Capacity Spectrum method (ATC-40, 1996) was employed for finding performance situation of the buildings. Available member ductility was then compared with the acceptable limits of ATC - 40, as to what level of safety they conform to.

The general purpose finite element program SAP 2000 Nonlinear (CSI, 2000) was the tool for modeling the structures and study its behavior in terms of capacity and performance. Non-linear Static Pushover analysis using frame element of SAP 2000 was used. The program, SAP 2000, has a high reliability and is widely used as a research tool all over the world.

Typical six to nine storied residential buildings representative of medium rise buildings which are very common in Dhaka City are modeled and designed as per force level detailed in BNBC. Demand earthquake level is the design earthquake as per the specification of ATC-40. Study area is considered as Dhaka City with Soft to Medium stiff soil profile (Soil type III as per BNBC, 1993).

1.5 Layout of the Thesis

The general background, objectives of the study and methodology of the work have been presented in Chapter 1 to give basic idea of the work being done under the research. In Chapter 2, response of a building during an earthquake has been described along with the seismic load provision of BNBC, it's contents and limitations have been discussed. In Chapter 3, concept of seismic performance evaluation of structure and non-linear pushover analysis procedure have been discussed. Chapter 4 described the basic modeling and analysis parameters used in the study. Chapter 5 represented the finding of the research work. Conclusions derived from the present study and recommendation for future work has been presented in Chapter 6.

Chapter 2

BUILDING RESPONSE DURING EARTHQUAKE AND PROVISION FOR EARTHQUAKE LOAD ANALYSIS IN BNBC

2.1 Introduction

The dynamic response of the building to earthquake ground motion is the most important cause of earthquake-induced damage to buildings. Failure of the ground and soil beneath buildings is also a major cause of damage.

Most earthquakes result from rapid movement along the plane of faults within the earth's crust. This sudden movement of the fault releases a great deal of energy, which then travels through the earth in the form of seismic waves. The seismic waves travel for great distances before finally losing most of their energy.

At some time after their generation, these seismic waves reach the earth's surface, and set it in motion, which we refer to as earthquake ground motion. When this earthquake ground motion occurs beneath a building and when it is strong enough, it sets the building in motion, starting with the building's foundation, and finally transfers the motion throughout the rest of the building in a very complex way. These motions in turn induce forces which can produce damage.

2.2 Complexity of Earthquake Ground Motion

Real earthquake ground motion at a particular building site is vastly more complicated than the simple wave form of motion.

The complexity of earthquake ground motion is due to three factors: 1) The seismic waves generated at the time of earthquake fault movement were not all of a uniform character; 2) As these waves pass through the earth on their way from the fault to the building site, they are modified by the soil and rock media through which they pass; 3) Once the seismic waves reach the building site they undergo further modifications, which are dependent upon the characteristics of the ground and soil beneath the building. These three factors are referred as:

- source effects
- path effects
- local site effects

2.3 Ground Motion and Building Frequencies

The characteristics of earthquake ground motions which have the greatest importance for buildings are the duration, amplitude (of displacement, velocity and acceleration) and frequency of the ground motion. Surface ground motion at the building site, then, is actually a complex superposition of vibrations of different frequencies. At any given site, some frequencies usually predominate. The distribution of frequencies in a ground motion is referred to as its frequency content.

The response of the building to ground motion is as complex as the ground motion itself, yet typically quite different. However, the building's vibrations tend to center around one particular frequency, which is known as its natural or fundamental frequency. Natural frequency is a function of mass and stiffness of the system. In general, the shorter a building is, the higher its natural frequency and taller the building is, the lower its natural frequency.

When the frequency contents of the ground motion are centered around the building's natural frequency, the building and the ground motion are said to be in resonance with one another. Resonance tends to increase or amplify the building's response. Because of this, buildings suffer the greatest damage from ground motion at a frequency close or equal to their own natural frequency.

2.4 Response Spectra

Different buildings can respond in widely differing manners to the same earthquake ground motion. Conversely, any given building will act differently during different earthquakes, which gives rise to the need of concisely representing the building's range of responses to ground motion of different frequency contents. Such a representation is known as a response spectrum. A response spectrum is a kind of graph which plots the maximum response values of acceleration, velocity and displacement against period or frequency. Response spectra are very important "tools" in earthquake engineering.

Fig. 2.1 shows a highly simplified version of a response spectrum. Even though highly simplified, it does show how building response characteristics vary with building frequency or period: as building period lengthens, accelerations decrease and displacement increases. On the other hand, buildings with shorter periods (but higher natural frequencies), undergo higher accelerations but smaller displacements.

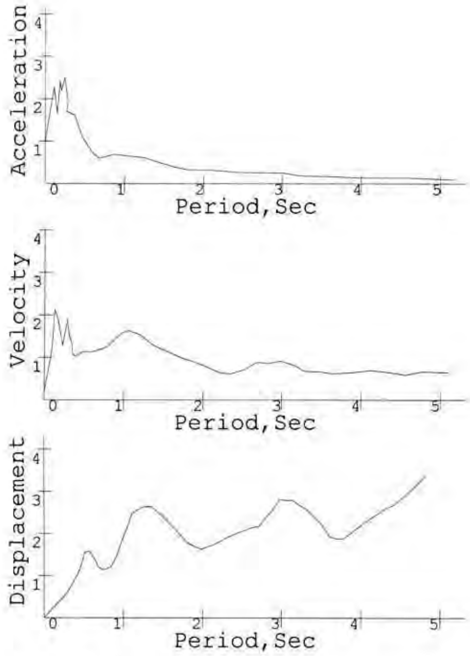


Fig. 2.1 Response spectrum

In the subsequent chapters, it will be described in more detail, the amount of acceleration which a building undergoes during an earthquake is a critical factor in determining how much damage it will suffer. The spectrum described in Fig. 2.1 provides some indication of how accelerations are related to frequency characteristics which shows one way in which response spectra can be useful, since identifying the resonant frequencies at which a building will undergo peak accelerations is one very important step in designing the building to resist earthquakes.

2.5 Analysis of Structures for Earthquake Forces

2.5.1 Equation of Motion: Earthquake Excitation

In earthquake-prone regions, the principal problem of structural dynamics that concern the structural engineers is the behavior of structures subjected to earthquake-induced motion at the base of the structure. If the displacement of the ground is denoted by u_g, the total displacement of the mass by u_t, and the relative displacement between the mass and ground by u then at each instant of time 't' these displacements are related by

$$u_t(t)=u(t)+u_g(t) \qquad 2.1$$

Both u_t and u_g refer to the same inertial frame of reference and their positive directions coincide.

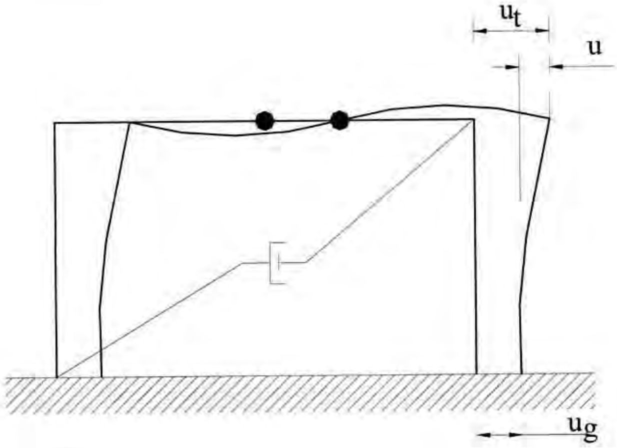


Fig. 2.2 Idealized one-story system subjected to ground acceleration.

Equation of motion for the idealized one-story system of Fig. 2.2 subjected to earthquake excitation can be written as

$f_I + f_D + f_S = 0$ *2.2*

Only the relative motion 'u' between the mass and the base due to structural deformation produces elastic and damping forces. Thus for a linear system linear elastic force, $f_S=ku$, damping force, $f_D=c\dot{u}$ and inertia force f_I is related to the acceleration $\ddot{u}^t$ of the mass by $f_I = m\ddot{u}^t$. Substituting these values to Equation 2.2,

$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g(t)$ *2.3*

This is the equation of motion governing the relative displacement or deformation *u(t)* of the linear structure of Fig. 2.3 subjected to a ground acceleration $\ddot{u}_g(t)$.

Dividing Eq. 2.3 by m gives

$\ddot{u} + 2\zeta\omega_n\dot{u} + \omega_n^{\,2}u = -\ddot{u}_g(t)$ 2.4

Where, ω_n is the natural circular frequency $= \sqrt{\dfrac{k}{m}}$

ζ is the critical damping coefficient $= \dfrac{c}{2m\omega_n}$

This is the basic equation of motion for a single degree of freedom system.

2.5.2 Code Specified Equivalent Static Load Method

Eq. 2.4 is identical to the idealized one-storied frame same as Fig. 2.2 subjected to external dynamic force, P(t) which is

$m\ddot{u} + c\dot{u} + ku = P(t)$ 2.5

Comparing Eq. 2.3 and Eq. 2.5 it is seen that the equation of motion for the structure subjected to two separate excitations – ground acceleration $\ddot{u}_g(t)$ and external force $-m\ddot{u}_g(t)$ are one and the same. Thus the relative displacement or deformation u(t) of the structure due to ground acceleration $\ddot{u}_g(t)$ will be identical to the displacement u(t) of the structure if its base were stationary and if it were subjected to an external force $= -m\ddot{u}_g(t)$. Thus the ground motion can therefore be replaced by the effective earthquake force:

$P_{eff}(t) = -m\ddot{u}_g(t)$ 2.6

In this method the dynamic earthquake effect is represented by equivalent static load at different levels. Earthquake load is a dynamic load. Due to earthquake load, a structure vibrates in different mode shapes and the load on the structure, its intensities and direction are dependent on the mode shapes.

For example, the Fig. 2.3 below shows first three fundamental modes of a shear type building.

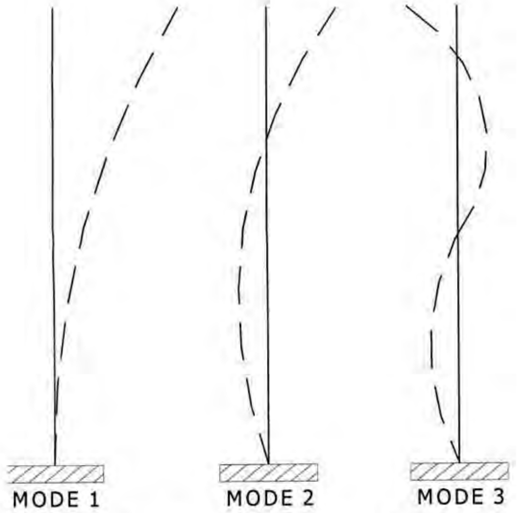


Fig. 2.3 Fundamental mode of a shear type structure

From Fig. 2.3 it is seen that different mode shape of the structure causes different load intensities and direction to the structure. If only the first mode is considered and assumed linear mode shape then the structure experiences a triangular shaped lateral load. Equivalent Static Load method as adopted in building codes is simple approximation of first mode of vibration with the mode shape considered as linear.

So, for a building of homogeneous mass, the lateral forces are likely to be as shown in Fig. 2.4.

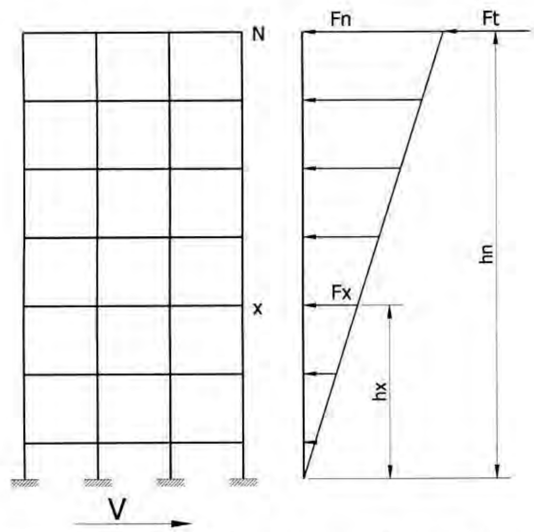


Fig. 2.4 Distribution of lateral forces in multistory building

However, for building with higher time period (flexible ones), the effect of higher modes become important. This is accounted by considering an extra concentrated force 'F_t' at the top of building. For regular shaped and non slender buildings the equivalent static method gives an approximate estimation of seismic force demand on the structure.

2.5.3 Response Spectrum Analysis

The seismic force generated in structures varies according to their dynamic properties even though they stand on the same ground and are subjected to the same seismic motion.

The response spectrum is schematically depicted in the Fig. 2.5. Three types of single-degree-of-freedom systems with the same damping constants *h1* but different natural periods Fig. 2.5(b)] are subjected in the same manner to the earthquake motion shown in the Fig. 2.5(a). However, each point mass shows a different response according to the relation between properties of earthquake motion and its natural period under the single-degree-of-freedom system.

Mass having a comparatively shorter natural period *T1* vibrates rapidly while mass having a longer natural period *T3* vibrates slowly. This situation is illustrated in the Fig. 2.5(c). The bold line plot in the Fig. 2.5(d) shows the maximum response value for a given time interval and the natural period. If the vibration characteristics continuously varied for the systems corresponding to extremely rigid structures with a very short natural period to flexible structures having long natural periods, then plot the maximum response values, a response spectrum for damping constant *h1* is obtained.

So, knowing the period of the structure, peak spectral acceleration of the structure can be estimated.

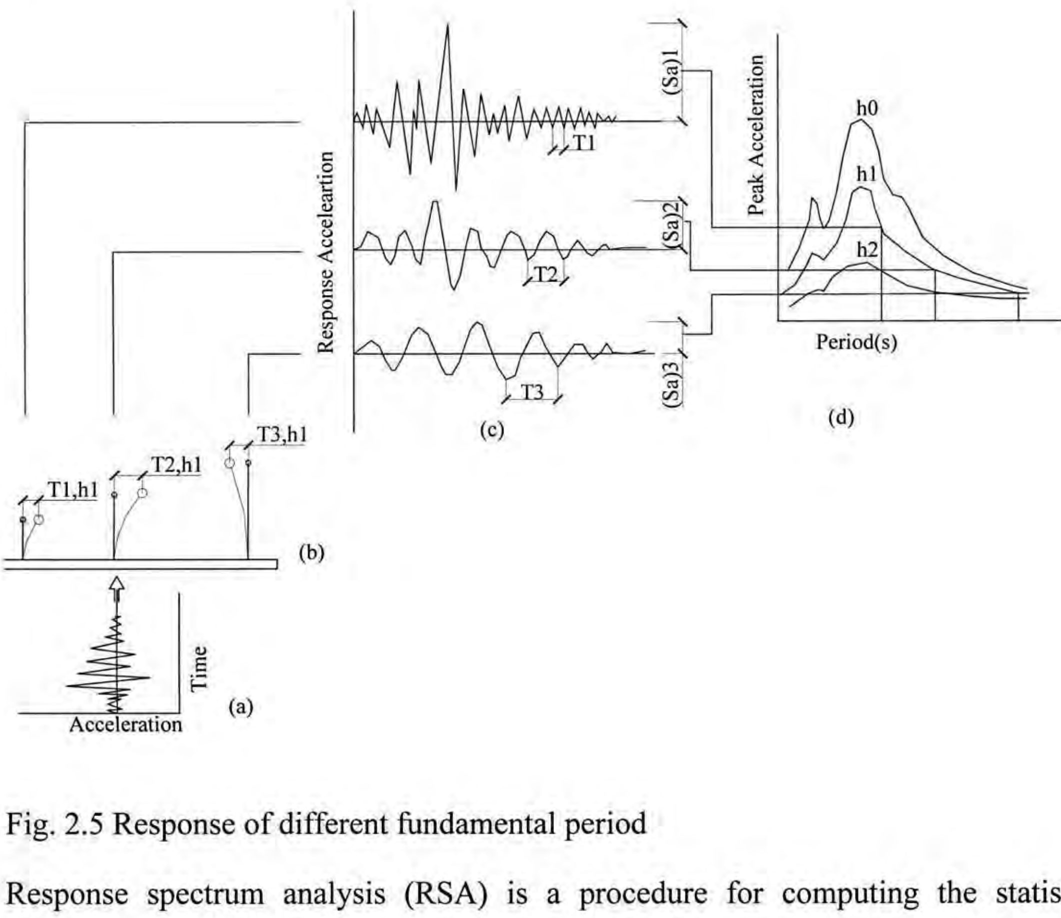


Fig. 2.5 Response of different fundamental period

Response spectrum analysis (RSA) is a procedure for computing the statistical maximum response of a structure to a base excitation (or earthquake). Each of the vibration modes that are considered may be assumed to respond independently as a single-degree-of-freedom system. Various design codes specify response spectra which determine the base acceleration applied to each mode according to its period. Having determined the response of each vibration mode to the excitation, it is necessary to obtain the response of the structure by combining the effects of each vibration mode. Because the maximum response of each mode will not necessarily occur at the same instant, the statistical maximum response is taken as the square root of the sum of the squares of the individual response.

It is clear from Eq. 2.4 that for a given $\ddot{u}_g(t)$, the deformation response u(t) of the system depends only on the natural frequency ω_n or natural period T_n of the system and its damping ration, ζ. Thus any two systems having the same values of T_n and ζ will have the same deformation response u(t) even though one system may be massive and with higher stiffness than the other.

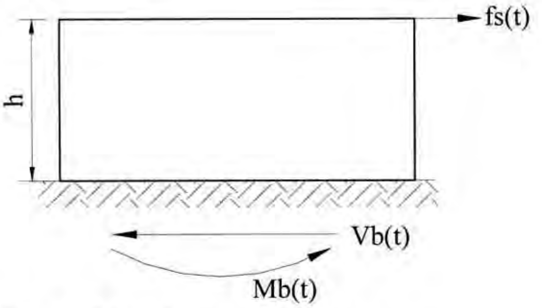


Fig. 2.6 Equivalent static force

Once the deformation response history u(t) has been evaluated by dynamic analysis of the structure, the internal forces can be determined by static analysis of the structure at each time instant. Preferred approach in earthquake engineering is based on the concept of the equivalent static force f_S. Equivalent static force, f_S at any time instant t, may be defined as

$f_S(t) = ku(t)$ 2.12

where k is the lateral stiffness of the frame. Expressing k in terms of mass gives

$f_S(t) = m\omega^2{}_n u(t) = mA(t)$... ... . 2.13

where $A(t) = \omega^2{}_n u(t)$

The equivalent static force is m times A(t), the pseudo-acceleration. The pseudo-acceleration response A(t) of a system can readily be computed from the deformation response u(t).

For the one-story frame as shown the Fig. 2.6, the internal forces like the shears and moments in the columns an beams or stress at any location can be determined at a selected instant of the time by static analysis of the structure subjected to the equivalent static lateral forces $f_s(t)$ at the same time instant. Thus a static analysis of the structure would be necessary at each time instant when the responses are desired. In particular, the base shear $V_b(t)$ and the base over-turning moment $M_b(t)$ are:

$V_b(t) = f_S(t)$ and $M_b(t) = hf_s(t)$ where h is the height of the mass above the base.

Putting the value of $f_S(t)$, one may get, $V_b(t) = mA(t)$ and $M_b(t) = hV_b(t)$

2.5.4 Time History Analysis

Earthquake excitation is time dependent, highly irregular and arbitrary in nature. Usually earthquake excitation in the form of acceleration or displacement or velocity is recorded for a time interval of 0.02 to 0.005 seconds. In this dynamic analysis procedure the response of a structure at every time interval is recorded for the whole earthquake period and the statistical average is represented. Because of its inherent complexities of the procedure and nondeterministic nature of the input ground motion, the analysis procedure has not become popular in the design houses for designing of the structures.

2.6 Provision for Earthquake Load Analysis in BNBC

Bangladesh National Building Code was published in 1993 by Housing and Building Research Institute. Like other building codes BNBC has different provisions for calculation of earthquake load and analysis procedures for structures subjected to earthquake.

The Code defines a simple method to represent earthquake induced inertia forces by Equivalent Static Force for static analysis. Two more rigorous analysis methods are also defined in the BNBC. They are:

i. Response Spectrum Analysis and

ii. Time History Analysis

2.6.1 Equivalent Static Load Method

In this method the dynamic earthquake effect is represented by an equivalent static load at different levels in proportion to mass at that level. As described earlier, this method estimates an equivalent static base shear considering seismicity of the region as well first mode behavior of the structure.

2.6.1.1 Calculation of base shear

Total design base share, denoted by V, in a given direction is determined from the following relation. $V = \frac{ZIC}{R}W$ 2.14

2.6.1.2 Response Spectrum Method

BNBC recommends that response spectrum to be used in dynamic analysis shall be either of the following:

i. Site Specific Design Spectra: A site specific response spectra shall be developed base on the geologic, tectonic, seismologic, and soil characteristics associated with the specific site.

ii. Normalized Response Spectra: In absence of a site-specific response spectrum, the normalized response spectra shall be used.

The normalized response spectrum curves provided in the code are shown for three different soil types with 5% of the critical damping in Fig. 2.7.

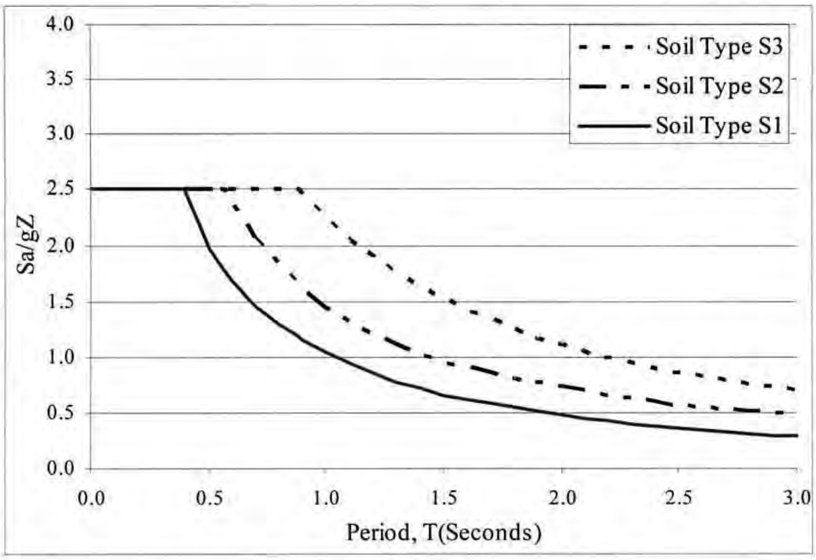


Fig. 2.7 Normalized response spectrum curves for 5% damping of BNBC, 1993

2.6.1.3 Time History Analysis

Ground motion time history developed for the specific site shall be representative of actual earthquake motions for an earthquake.

2.7 Deficiencies in the Earthquake Design Provisions of BNBC, 1993

The earthquake design provision included in the BNBC, 1993 follows from the provisions of UBC, 1991. Though the code is quite comprehensive, certain areas need to be improved.

2.7.1 The Zoning map

The code divides the country into three zones of different seismic vulnerability. In the context of recent seismic activities in the region and also considering the zoning of the recently updated codes in the neighboring country (see Fig.2.8 and Table 2.1), a review of the seismic zoning is required to be made in the near future.

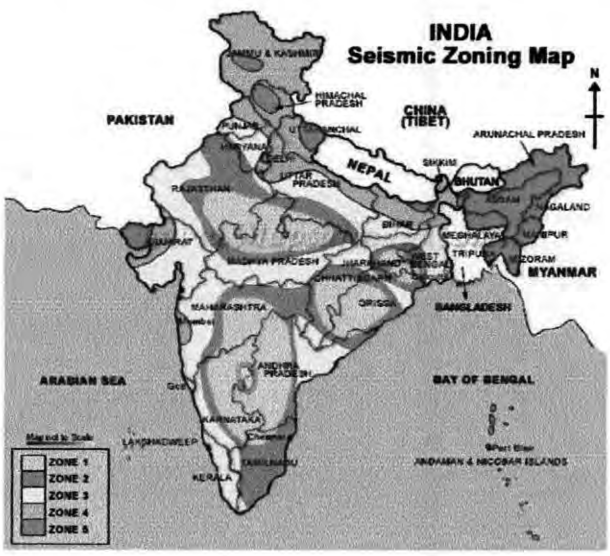


Fig. 2.8 Earthquake zoning map of India

Table 2.1 Zone Factors, Z

S.Z.	II	III	IV	V
S.I.	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

2.7.2 Structure Period

In Equivalent Static Load Method, structure period is calculated as a function of structure height. As a result, for a given structure same base shear in all direction of the structure is produced. Practically, structure period is a function of structural mass and stiffness. Unless the structure is perfectly symmetric in both axes, considerable change in structure period may be found in other direction resulting different base shear acting in that direction. Moreover, presence of infill is also likely to affect the period of the structure. The present code sheds no light in this respect.

2.7.3 Distribution of Base Shear

Unless the story mass varies, the base shear distribution along the height of the structure, as per the current provision of the code, is linear. Practically none of the fundamental mode shapes are linear. Many codes, like current Uniform Building Code(UBC, 1997) or later, Indian Code(IS) etc. recognizes non-linear distribution of base shear matching the first fundamental mode shape.

2.7.4 Response Modification Factor(R)

Most recent seismic codes include response modification factors in the definition of the equivalent lateral forces that are used for the design of earthquake resistant buildings. This factor, R is the system performance coefficient, used to reduce the elastic earthquake force considering the beneficial effects of ductility and energy-dissipating capacity of a structure. R varies from 5 for structures with poor ductility to 12 for structures with good ductility and good energy- dissipating capacity. The more ductile a system, the more energy it absorbs during an earthquake shaking and thus places lesser demand on the inelastic deformation, consequently the code specifies a corresponding lesser amount of base shear for which the structure is to be designed. Code defines that the value of 'R' will be dependent on the ductility of the structure. A general seismic reinforcement requirement is provided in the code but structural detailing required ensuring the ductility is not defined. As result, the choice of 'R' becomes very arbitrary and is not fully consistent with detailing provided in the field.

2.7.5 Provision of Seismic Detailing

Bangladesh National Building Code (BNBC, 1993) has a section in Chapter 8 (Section 8.3) on 'Special Provision for Seismic Design'. This section deals with the basic requirements of material strength, geometric configuration of the elements of the lateral load resisting system, anchorage requirement of the longitudinal reinforcement, transverse reinforcement provision to prevent buckling of the longitudinal reinforcement and other provisions to ensure ductility at the vulnerable plastic hinge zone with ensuring strong column-weak beam approach to the structural system.

The requirement of seismic provision given in the BNBC is comprehensive and well elaborated for different types of elements. But, it does not indicate quantitatively how these detailing ensure ductility to absorb seismic force.

Chapter 3

NONLINEAR STATIC (PUSHOVER) ANALYSIS AND CONCEPT OF PERFORMANCE EVALUATION OF STRUCTURES

3.1 Introduction

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

Various analysis methods are available, both linear and non-linear for performance evaluation of concrete building. The best basic inelastic method is nonlinear time history analysis method. This method is too complicated and considered impractical for general use. The central focus of this thesis is to introduce the simplified non-linear procedure for the generation of the "pushover" or capacity curve of a structure. This represents the plot of progressive lateral displacement as a function of the increasing level of force applied to the structure. Pushover analysis is a simplified static non-linear analysis method which use capacity curve and reduced response spectrum to estimate maximum displacement of a building under a given level of earthquake.

3.2 Methods to Perform Simplified Nonlinear Analysis

Two key elements of a performance-based design procedure are demand and capacity. Demand is the representation of the earthquake ground motion. Capacity is the representation of the structure's ability to resist the seismic lateral force. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design.

Simplified nonlinear analysis procedures using pushover methods, such as the capacity spectrum method and the displacement coefficient method, require determination of three primary elements: capacity, demand (displacement) and performance. Each of these elements is briefly discussed below.

3.2.1 Capacity

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required. This procedure uses a series of sequential elastic limits, some form of nonlinear analysis, superimposed to approximate a force-displacement capacity diagram of the overall structure. The mathematical model of the structure is modified to account for reduced resistance of yielding components. A lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit is reached. The capacity curve approximates how structures behave after exceeding their elastic limit.

3.2.2 Demand (displacement)

Ground motions during an earthquake produce complex horizontal displacement pattern in structures that may vary with time. Tracking this motion at every time-step to determine structural design requirements is judged impractical. Traditional linear analysis methods use lateral forces to represent a design condition. For nonlinear methods it is easier and more direct to use a set of lateral displacements as a design condition. For a given structure and ground motion, the displacement demand is the estimate of the maximum expected response of the building during the ground motion.

3.2.3 Performance

Once a capacity curve and demand displacement is defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacement imposed by the displacement demand.

3.3 Static Non-linear (Pushover) Analysis

As a building responds to earthquake ground motion, it experiences lateral displacements and, in turn, deformations of its individual elements. At low levels or response, the element deformations will be within their elastic (linear) range and no damage will occur. At higher levels of response, element deformations will exceed their linear elastic capacities and the building will experience damage. In order to provide reliable seismic performance, a building must have a complete lateral force resisting system, capable of limiting earthquake-induced lateral displacements to levels at which the damage sustained by the building's elements will be within acceptable levels for the intended performance objective. The basic factors that affect the lateral force resisting system's ability to do this include the building's mass, stiffness, damping and configuration; the deformation capacity of its elements; and the strength and character of the ground motion it must resist.

The nonlinear push-over analysis requires development of the capacity curve .The capacity curve is derived from an approximate nonlinear, incremental static analysis for the structure. In the process of performing this incremental nonlinear static analysis, a capacity curve is developed for the building. This capacity curve is simply a plot of the total lateral seismic shear demand, "V," on the structure, at various increments of loading, against the lateral deflection of the building at the roof level, under that applied lateral force. If a building had infinite linear elastic capacity, this capacity curve would be a straight line with a slope equal to the global stiffness of the structure. Since real buildings do not have infinite linear elastic capacities, the capacity curve typically consists of a series of straight-line segments with decreasing slope, representing the progressive degradation in structural stiffness that occurs as the building is subjected to increased lateral displacement, yielding, and damage. The slope of a straight line drawn from the origin of the plot for this curve to a point on the curve at any lateral displacement, "*d*," represents the secant or "effective" stiffness of the structure when pushed laterally to that displacement. A typical capacity curve of a hypothetical structure is shown in Fig. 3.1.

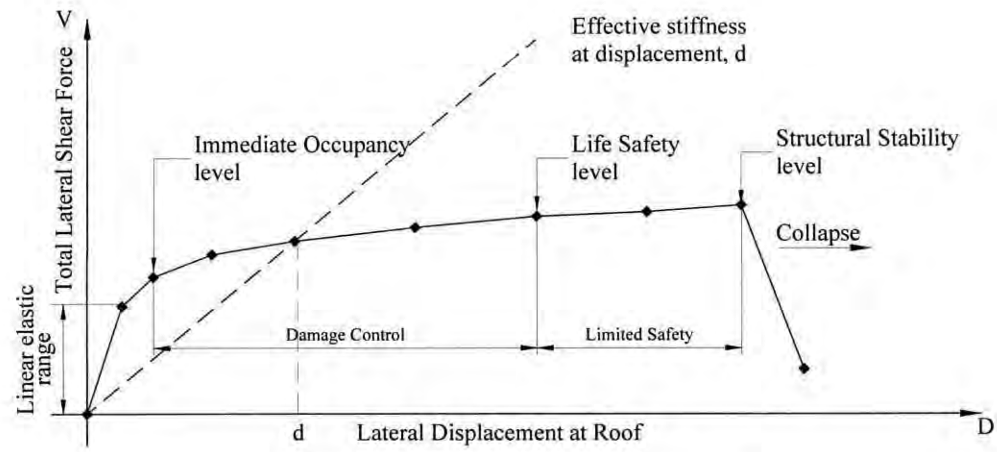


Fig. 3.1 Typical capacity curve

In Fig. 3.1, the discrete points indicated by the symbol '♦' represent the occurrence of important events in the lateral response history of the structure. Such an event may be the initiation of yield in a particulars structural element or a particular type of damage, such as spalling of cover concrete on a column or shear failure of a spandrel element. Each point is determined by a different analysis sequence. Then, by evaluating the cumulative effects of damage sustained at each of the individual events, and the overall behavior of the structure's increasing lateral displacements, it is possible to determine and indicate on the capacity curve those total structural lateral displacements that represent limits on the various structural performance levels, as has been done in Fig. 3.1.

The process of defining lateral deformation points on the capacity curve at which specific structural performance levels may be said to have occurred requires the exercise of considerable judgment on the part of the engineer. For each of the several structural performance levels and global performance levels defined in Section 3.6 to 3.8, defines global system response limits as well as acceptance criteria for the individual structural elements that make up typical buildings. These acceptance criteria generally consist of limiting values of element deformation parameters, such as the plastic chord rotation of a beam or shear angle of a wall. These limiting values have been selected as reasonable approximate estimates of the average deformations at which certain types of element behavior such as cracking, yielding, spalling, or crushing, may be expected to occur. As the incremental static nonlinear analyses are performed, the engineer must monitor the cumulative deformations of all important structural elements and evaluate them against the acceptance criteria set before.

The point on the capacity curve at which the first element exceeds the permissible deformation level for a structural performance level does not necessarily represent that the structure as a whole reaches that structural performance level. Most structures contain many elements and have considerable redundancy. Consequently, the onset of unacceptable damage to a small percentage of these elements may not represent an unacceptable condition with regard to the overall performance of the building. When determining the points along the capacity curve for the structure at which the various structural performance levels may said to be reached, the engineer must view the performance of the building as whole and consider the importance of damage predicted for the various elements on the overall behavior of the building.

The methodology described by ATC-40, incorporates the concept of "primary" and "secondary" elements to assist the engineer in making these judgments. Primary elements are those that are required as part of the lateral force resisting system for the structure. All other elements are designated as secondary elements. For a given performance level, secondary elements are generally permitted to sustain more damage than primary elements since degradation of secondary elements does not have a significant effect on the lateral load resisting capability of the building. If in the development of the capacity curve it is determined that a few elements fail to meet the acceptance criteria for a given performance level at an increment of lateral loading and displacement, the engineer has the ability to designate these "nonconforming" elements as secondary, enabling the use of more liberal acceptance criteria for these few elements. Care is exercised not to designate an excessive number of elements that are effective in resisting lateral forces as secondary.

3.3.1 Capacity Spectrum Method

The capacity spectrum method, a nonlinear static procedure, provides a graphical representation of the global force-displacement capacity curve of the structure (i.e. pushover curve) and compares it to the response spectra representations of the earthquake demands. This method is a very useful tool in the evaluation and retrofit design of existing concrete buildings. The graphical representation provides a clear picture of how a building responds to earthquake ground motion, and, as illustrated below, it provides an immediate and clear picture of how various retrofit strategies, such as adding stiffness or strength, will affect the building's response to earthquake demands.

The capacity spectrum curve for the structure is obtained by transforming the capacity curve from lateral force (V) vs. lateral displacement (d) coordinates to spectral acceleration (S_a) vs. spectral displacement (S_d) coordinates using the modal shape vectors, participation factors and modal masses obtained from a modal analysis of the structure. In order to compare the Structure's capacity to the earthquake demand, it is required to plot the response spectrum and the capacity spectrum on the same plot. The conventional response spectrum plotted in spectral acceleration vs. period coordinate has to be changed in to spectral acceleration vs. spectral displacement coordinate. This form of response spectrum is known as acceleration displacement response spectrum (ADRS).

a. ADRS Format

Capacity Spectrum method requires plotting the capacity curve in spectral acceleration and spectral displacement domain. This representation of spectral quantities is knows as Acceleration- Displacement-Response-Spectra in brief ADRS, which was introduced by Mahaney et al.,(1993). Spectral quantities like spectral acceleration, spectral displacement and spectral velocity is related to each other to a specific structural period T. Building code usually provide response spectrum in spectral acceleration *vs*. period format which is the conventional format.

Each point on the curve defined in the Fig. 3.2 is related to spectral displacement by mathematical relation, $S_d = \frac{1}{4\pi^2} S_a T^2$. Converting with this relation response spectrum in ADRS format may be obtained.

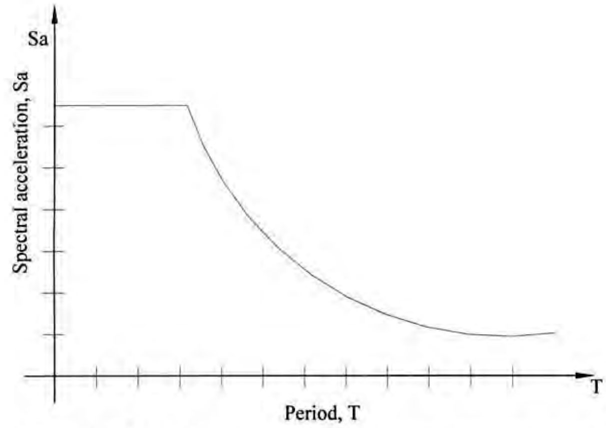


Fig. 3.2 Code specified response spectrum in Spectral acceleration vs. Period.

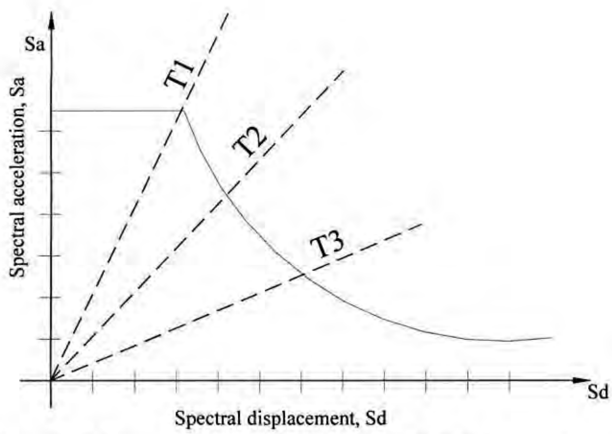


Fig. 3.3 Response spectrum in ADRS format

Any line from the origin of the ADRS format represent a constant period T_i which is related to spectral acceleration and spectral displacement by the mathematical relation,

$$T = 2\pi\sqrt{\frac{S_d}{S_a}}.$$

b. Capacity Spectrum

Capacity spectrum is a simple representation of capacity curve in ADRS domain. A capacity curve is the representation of Base Shear to Roof displacement. In order to develop the capacity spectrum from a capacity curve it is necessary to do a point by point conversion to first mode spectral coordinates.

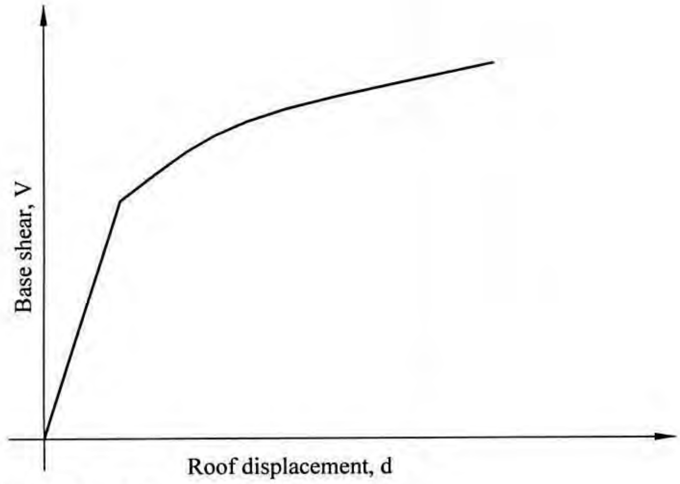


Fig. 3.4 A typical capacity curve

Any point corresponding values of base shear, V_i and roof deflection, Δ_i may be converted to the corresponding point of spectral acceleration, S_{ai} and spectral displacement, S_{di} on the capacity spectrum using relation,

$$S_{ai} = \frac{V_i / W}{\alpha_1} \text{ and}$$

$$S_{di} = \frac{\Delta_{Roof}}{PF_1 \times \Phi_{1,Roof}}$$

Modal participation factor, PF_1 is calculated using equation, $PF_1 = \left[\frac{\sum_1^N (w_i \phi_{i,1}) / g}{\sum_1^N (w_i \phi^2_{i,1}) / g} \right]$

Modal mass coefficient for the first mode, α_1 is calculated using equation,

$$\alpha_1 = \frac{\left[\sum_1^N (w_i \phi_{i,1}) / g \right]^2}{\left[\sum_1^N w_i / g \right] \left[\sum_1^N (w_i \phi^2_{i,1}) / g \right]}$$

Where:

PF_1	=	modal participation factor for the first natural mode.
α_1	=	modal mass coefficient for the first natural mode
$\Phi_{1,\,Roof}$	=	roof level amplitude of the first mode.
w_i/g	=	mass assigned to level i
ϕ_{i1}	=	amplitude of mode 1 at level i

N	=	level N, the level which is the uppermost in the main portion of the structure
V	=	base shear
W	=	building dead weight plus likely live loads
Δ_{Roof}	=	roof displacement
S_a	=	spectral acceleration
S_d	=	spectral displacement

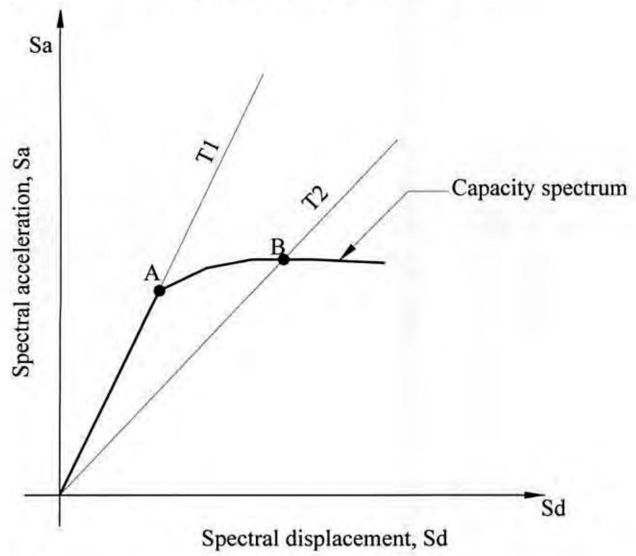


Fig. 3.5 Capacity spectrum

Fig. 3.5 shows a typical capacity spectrum converted from capacity curve of Fig. 3.4 of a hypothetical structure. It is seen in the capacity spectrum that up to some displacement corresponding to point A, the period is constant T_1. That is the structure is behaving elastically. As the structure deflects more to point B, it goes to inelastic deformation and its period lengthens to T_2.

When the capacity curve is plotted in S_a vs. S_d coordinates, radial lines drawn from the origin of the plot through the curve at various spectral displacements have a slope (ω'), where, ω' is the radial frequency of the effective (or secant) first-mode response of the structure if pushed by an earthquake to that spectral displacement.

Using the relationship $T' = \dfrac{2\pi}{\omega'}$, it is possible to calculate, for each of these radial lines, the effective period of the structure if it is pushed to a given spectral displacements.

Fig. 3-6 is a capacity spectrum plot obtained from the capacity curve of a hypothetical structure shown in Fig. 3-1 and plotted with the effective modal periods shown.

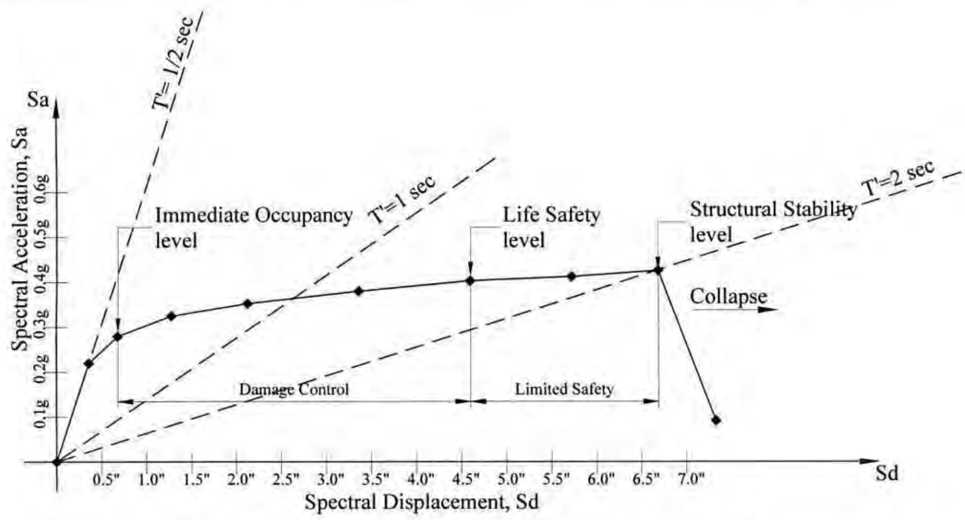


Fig. 3.6 Typical capacity spectrum of a hypothetical structure

The particular structure represented by this plot would have an elastic period of approximately ½ second. As it is pushed progressively further by stronger ground motion, this period lengthens. The building represented in Fig. 3.1 and Fig. 3.6 would experience collapse before having its stiffness degraded enough to produce an effective period of 2 seconds.

The capacity of a particular building and the demand imposed upon it by a given earthquake motion are not independent. One source of this mutual dependence is evident from the capacity curve itself. As the demand increases, the structure eventually yields and, as its stiffness decreases, its period lengthens. Conversion of the capacity curve to spectral ordinates (ADRS) makes this concept easier to visualize. Since the seismic accelerations depend on period, demand also changes as the structure yields. Another source of mutual dependence between capacity and demand is effective damping. As a building yield in response to seismic demand it dissipates energy with hysteretic damping. Buildings that have large, stable hysteresis loops during cyclic yielding dissipate more energy than those with pinched loops caused by degradation of strength and stiffness. Since the energy that is dissipated need not be stored in the structure, the effective damping diminishes displacement demand.

The capacity spectrum method initially characterizes seismic demand using an elastic response spectrum. This spectrum is plotted in spectral ordinates (ADRS) format showing the spectral acceleration as a function of spectral displacement. This format allows the demand spectrum to be "overlaid" on the capacity spectrum for the building. The intersection of the demand and capacity spectra, if located in the linear range of the capacity, would define the actual displacement for the structure; however this is not normally the case as most analyses include some inelastic nonlinear behavior. To find the point where demand and capacity are equal, a point on the capacity spectrum need to be selected as an initial estimate. Using the spectral acceleration and displacement defined by this point, reduction factors may be calculated to apply to the 5% elastic spectrum to account for the hysteretic energy dissipation, or effective damping, associated with the specific point. If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point, then it is the solution for the unique point where capacity equals demand. If the intersection is not reasonably close to the initial point, then a new point somewhere between may be assumed and repeat the process until a solution is reached. This is the performance point where the capacity of the structure matches the demand or the specific earthquake.

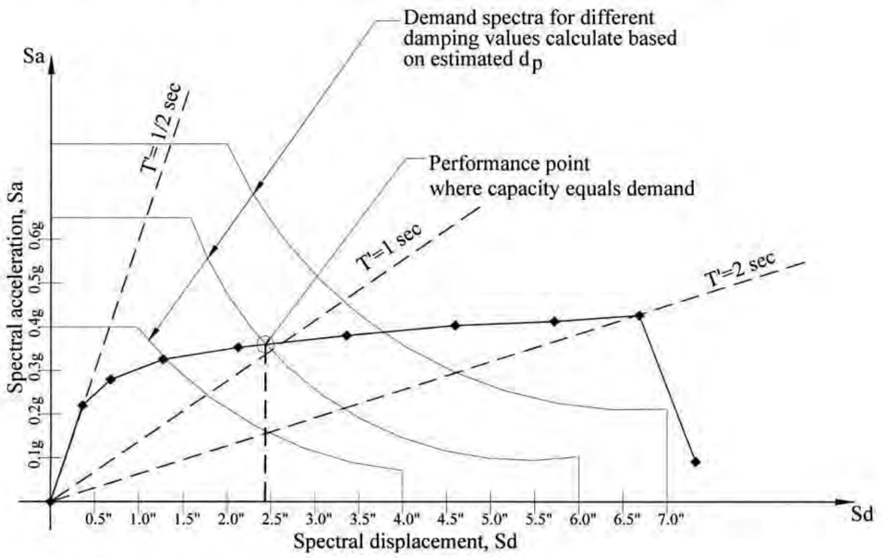


Fig. 3.7 Determination of performance point

Once the performance point has been determined, the acceptability of a rehabilitation design to meet the project performance objectives can be judged by evaluating where the performance points falls on the capacity curve. For the structure and earthquake represented by the overlay indicated in Fig. 3.7, the performance point occurs within the central portion of the damage control performance range as shown in Fig. 3-6, indicating that for this earthquake this structure would have less damage than permitted for the Life Safety level and more than would be permitted for the Immediate Occupancy level. With is information, the performance objective and/or the effectiveness of the particular rehabilitation strategy to achieve the project performance objectives can be judged.

3.3.2 Displacement Coefficient Method

Another procedure for calculating demand displacement is 'Displacement Coefficient Method' which provides a direct numerical process for calculating the displacement demand. Displacement Coefficient Method has not been explored. Performance analysis of the structures under this thesis was made using Capacity Spectrum Method.

3.4 Seismic Performance Evaluation

The essence of virtually all seismic evaluation procedures is a comparison between some measures of the "demand" that earthquake place on structure to a measure of the "capacity" of the building to resist the induced effects. Traditional design procedures characterize demand and capacity as forces. Base shear (total horizontal force at the lowest level of the building) is the normal parameter that is used for this purpose. The base shear demand that would be generated by a given earthquake, or intensity of ground motion is calculated, and compares this to the base shear capacity of the building. If the building were subjected to a force equal to its base shear capacity some deformation and yielding might occur in some structural elements, but the building would not collapse or reach an otherwise undesirable overall level of damage. If the demand generated by the earthquake is less than the capacity then the design is deemed acceptable.

The first formal seismic design procedures recognized that the earthquake accelerations would generate forces proportional to the weight of the building. Over the years, empirical knowledge about the actual behavior of real structures in earthquakes and theoretical understanding of structural dynamics advanced. The basic procedure was modified to reflect the fact that the demand generated by the earthquake accelerations was also a function of the stiffness of the structure.

The inherently better behavior of some buildings over others are also begun to recognize. Consequently, that reduced seismic demand has been assumed for some structure based on the characteristics of the basic structural material and system. The motivation to reduce seismic demand for design came because it could not be rationalized theoretically how structures resisted the forces generated by earthquakes.

This was partially the result of the fundamental assumption that structures resisted loads linearly without yielding or permanent structural deformation.

3.5 Nonlinear Static Procedure for Capacity Evaluation of Structures

Instead of comparing forces, nonlinear static procedures use displacements to compare seismic demand to the capacity of a structure. This approach included consideration of the ductility of the structure on an element by element basis. The inelastic capacity of a building is then a measure of its ability to dissipate earthquake energy. The current trend in seismic analysis is toward these simplified inelastic procedures.

The recommended central methodology is on the formulation of inelastic capacity curve for the structure. This curve is a plot of the horizontal movement of a structure as it is pushed to one side. Initially the plot is a straight line as the structure moves linearly. As the parts of the structure yield the plot begins to curve as the structure softens. This curve is generated by building a model of the entire structure from nonlinear representations of all of its elements and components. Most often this is accomplished with a computer and structural analysis software. The specifies forces and displacement characteristics are specified for each piece of the structure resisting the earthquake demand. These pieces are assembled geometrically to represent the complete lateral load resisting system. The resulting model is subjected to increasing increment of load in a pattern determined by its dynamic properties. The corresponding displacements define the inelastic capacity curve of the building. The generation of the capacity curve defines the capacity of the building uniquely and independently of any specific seismic demand. In this sense it replaces the base shear capacity of conventional procedures. When an earthquake displaces the building laterally, its response is represented by a point on this curve. A point on the curve defines a specific damage state of the building, since the deformation of its entire components can related to the global displacement of the structure.

The capacity of a particular building and the demand imposed upon it by a given earthquake motion are not independent. One source of this mutual dependence is evident from the capacity curve itself. As the demand increases the structure eventually yields and, as its stiffness decreases, its period lengthens. Since the seismic accelerations depend on period, demand also changes as the structure yields. Another source of mutual dependence between capacity and demand is effective damping. As building yields in response to seismic demand, it dissipates energy with hysteretic damping. Building that have large, stable hysteretic loops during cyclic yielding dissipate more than those with pinched loops cause by degradation of strength and stiffness. Since the energy that is dissipated need not be stored in the structure, the damping has the effect of diminishing displacement demand.

3.6 Structural Performance Levels and Ranges

The Performance of a building under any particular event is dependent on a wide range of parameters. These parameters are defined (ATC-40, 1996; FEMA 356, 2000) qualitatively in terms of the safety afforded by the building to the occupants during and after the event; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic impacts on the larger community. These performance characteristics are directly related to the extent of damage that would be sustained by the building.

The Federal Emergency Management Agency in its report 'Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA-356, 2000) defines the structural performance level of a building to be selected from four discrete structural performance levels and two intermediate structural performance ranges. The discrete Structural Performance Levels are

- Immediate Occupancy (S-1),
- Life Safety (S-3),
- Collapse Prevention (S-5), and
- Not Considered (S-6).

The intermediate Structural Performance Ranges are the

- Damage Control Range (S-2) and the
- Limited Safety Range (S-4)

The definition of these performance ranges are given by FEMA (FEMA-356, 2000).

Acceptance criteria for performance within the Damage Control Structural Performance Range may be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Structural Performance Levels. Acceptance criteria for performance within the Limited Safety Structural Performance Range may be obtained by interpolating the acceptance criteria provided for the Life Safety and Collapse Prevention Structural Performance Levels. The performance levels and ranges, as per FEMA (FEMA-356, 2000), are described is the sections that follow.

3.6.1 Immediate Occupancy Structural Performance Level (S-1)

Structural Performance Level S-1, Immediate Occupancy, may be defined as the post-earthquake damage state of a structure that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure, and is in compliance with the acceptance criteria specified in this standard for this Structural Performance Levels defined in Table 3.1 to Table 3.3

Structural Performance Level S-1, Immediate Occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

3.6.2 Damage Control Structural Performance Range (S-2)

Structural Performance Range S-2, Damage Control, may be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Immediate Occupancy Structural Performance Level (S-1) defined in Table 3.1 to Table 3.3.

Design for the Damage Control Structural Performance Range may be desirable to minimize repair time and operation interruption, as a partial means of protecting valuable equipment and contents, or to preserve important historic features when the cost of design for immediate occupancy is excessive.

3.6.3 Life Safety Structural Performance Level (S-3)

Structural Performance Level S-3, Life Safety, may be defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria *specified in FEMA (FEMA-356, 2000) for this Structural Performance Level defined in Table 3.1 to Table 3.3.*

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

3.6.4 Limited Safety Structural Performance Range (S-4)

Structural Performance Range S-4, Limited Safety, may be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5) defined in Table 3.1 to Table 3.3.

3.6.5 Collapse Prevention Structural Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, may be defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified FEMA(FEMA-356, 2000) for this Structural Performance Level defined in Table 3.1 to Table 3.3.

Structural Performance Level S-5, Collapse Prevention, means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load-carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The *structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.*

3.7 Target Building Performance Levels

Building performance is a combination of the both structural and nonstructural components. Tables 3.1, 3.2 and 3.3 (FEMA-356, 2000) describe the approximate limiting levels of structural damage that may be expected of buildings evaluated to the levels defined for a target seismic demand. These tables represent the physical states of mathematical calculation of different performance levels.

Table 3.1 Damage control and building performance levels (FEMA-356, 2000)

	Target Building Performance Levels			
	Collapse Prevention Performance Level	Life Safety Performance Level	Immediate Occupancy Performance Level	Operational Performance Level
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage	Falling hazards mitigated but many architectural, mechanical and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities as available, possibly from standby sources.
Comparison with performance intended for buildings designed under the NEHRP Provisions, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Less damage and lower risk.	Much less damage and lower risk.

Table 3.2 Structural performance levels and damage[1,2,3] – Vertical elements (FEMA-356, 2000)

		Structural Performance Levels		
		Collapse Prevention	Life Safety	Immediate Occupancy
Elements	Type	S-5	S-3	S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in non-ductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns	Minor spalling in non-ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in Joint <1/6" width.
	Drift	4% transient or permanent	2% transient; 1% permanent`	1% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact	Hinges form. Local bucking of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may intact. A few elements may experience partial fracture.	Minor or local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact	Same as primary
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail	Minor yielding or buckling of braces.
	Secondary	Same as primary	Same as primary	Same as primary
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated	Major flexural and shear cracks. Sliding at joints. Extensive crushing. failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent
Un-reinforced Masonry Infill Walls	Primary	Extensive cracking and crushing; portions of face course shed	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	Minor (<1/8" width) cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.
	Secondary	Extensive crushing and shattering; some walls dislodge.	Same as primary	Same as primary
	Drift	0.6% transient or permanent	0.5% transient; 0.3% permanent	0.1% transient; negligible permanent
Un-reinforced Masonry (Noninfill) Walls	Primary	Extensive cracking; face course and veneer may peel off. Noticeable in plane and	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets	Minor (<1/8" width) cracking of veneers. Minor spalling in veneers

		Structural Performance Levels		
		Collapse Prevention	Life Safety	Immediate Occupancy
Elements	Type	S-5	S-3	S-1
		out-of-plane offsets		at a few corner openings. No observable out-of-plane offsets.
	Secondary	Nonbearing panels dislodge	Same as primary	Same as primary
	Drift	1% transient or permanent	0.6% transient; 0.6% permanent	0.3% transient; 0.3% permanent
Reinforced Masonry Walls	Primary	Crushing; extensive cracking. Damage around openings and at corners. Some fallen units	Extensive cracking (<1/4") distributed throughout wall. Some isolated crushing	Minor (<1/8" width) cracking. No out-of-plane offsets.
	Secondary	Panels shattered and virtually disintegrated	Crushing; extensive cracking. Damage around openings and at corners. Some fallen units	Same as primary
	Drift	1.5% transient or permanent	0.6% transient; 0.6% permanent	0.2% transient; 0.2% permanent
Wood Stud Walls	Primary	Connections loose. Nails partially withdrawn. Some splitting of members and panels. Veneers dislodged	Moderate loosening of connections and minor splitting of members	Distributed minor hairline cracking of gypsum and plaster veneers.
	Secondary	Sheathing sheared off. Let in braces fractured and buckled. Framing split and fractured	Connections loose. Nails partially withdrawn. SOme splitting of members and panels.	Same as primary
	Drift	3% transient or permanent	2% transient; 1% permanent	1% transient;8 0.25% permanent
Precast Concrete Connections	Primary	Some connection failures but no elements dislodged	Local crushing and spalling at connections, but no gross failure of connections.	Minor working at connections; cracks <1/16" width at connections.
	Secondary	Same as primary	Some connection failures but no elements dislodged	Minor crushing and spalling at connections
Foundations	General	Major settlement and tilting	Total settlements <6" and differential settlements <1/2" in 30ft.	Minor settlement and negligible tilting.

1. Damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements when present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure following an earthquake.

2. Drift values, differential settlements, crack widths, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in this standard; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various Structural Performance Levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.

3. For limiting damage to frame elements of infilled frames, refer to the rows for concrete or steel frames.

Table 3.3 Structural performance levels and damage[1,2] – Horizontal elements (FEMA-356, 2000)

	Structural Performance Levels		
	Collapse Prevention	Life Safety	Immediate Occupancy
Elements	S-5	S-3	S-1
Metal Deck Diaphragms	Large distortion with buckling of some units and tearing of many welds and seam attachments	Some localized failure of welded connections of deck to framing and between panels. Minor local buckling of deck	Connections between deck units and framing intact. Minor distortions.
Wood Diaphragms	Large permanent distortion with partial withdrawal of nails and extensive splitting of elements	Some splitting at connections. Loosening of sheathing. Observable withdrawal of fasteners. Splitting of framing and sheathing.	No observable loosening or withdrawal of fasteners. No splitting of sheathing or framing.
Concrete Diaphragms	Extensive crushing and observable offset across many cracks.	Extensive cracking (<1/4" width). Local crushing and spalling	Distributed hairline cracking. Some minor cracks of larger size (<1/8" width).
Precast Diaphragms	Connections between units fail. Units shift relative to each other. Crushing and spalling at joints.	Extensive cracking (<1/4" width). Local crushing and spalling.	Some minor cracking along joints.

1. Damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements when present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure following an earthquake.

2. Drift values, differential settlements, crack widths, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in this standard; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various Structural Performance Levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.

3.8 Response Limit

To determine whether a building meets a specified performance objective, response quantities from a nonlinear analysis are compared with limits given for appropriate performance levels (ATC-40, 1996 and FEMA-356, 2000). The response limits fall into two categories:

- Global Building Acceptability Limits
- Element and Component Acceptability Limit

3.8.1 Global Building Acceptability Limits

These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift. Table 3.4 gives the limiting values as per ATC-40 for different performance levels. These are described below:

a. 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 system.

b. Lateral Loads

Some components types are subjected to degrading over multiple load cycles. If a significant number of components degrade, 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 for the extreme case.

Two effects can lead to loss of lateral load resistance with increasing displacement. The first is gravity loads acting through lateral displacements, known as the P-Δ effect. The P-Δ effect is most prominent for flexible structures with little redundancy and low lateral load strength relative to the structure's weight. The second effect is degradation in resistance of individual components of the structure under the action of reversed deformation cycles. When lateral load resistance of the building degrades with increasing displacement, there is a tendency for displacements to accumulate in one direction. This tendency is especially important for long-duration events.

Table 3.4 presents deformation limits of various performance levels. Maximum total drift is defined as the inter-story drift at the performance point displacement. Maximum inelastic drift is defined as the portion of the maximum total drift beyond the effective yield point. For Structural Stability, the maximum total drift in story *i* at the performance point should not exceed the quantity $0.33\frac{V_i}{P_i}$, where V_i is the total calculated shear force in story *i* and P_i is the total gravity load (i.e. dead plus likely live load) at story *i* *(ATC-40, 1996)*.

Table 3.4 Deformation limits (ATC-40, 1996)

	Performance Level			
Inter story Drift Limit	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Maximum total drift	0.01	0.01 ~0.02	0.02	$0.33\frac{V_i}{P_i}$
Maximum inelastic drift	0.005	0.005~0.015	No limit	No limit

3.8.2 Element and Component Acceptability Limit

a. Deformation and Force Controlled Actions

All structural actions may be classified as either deformation controlled or force-controlled using the component force versus deformation curves shown in Fig. 3.8. The Type 1 curve depicted in Fig. 3.8 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by a plastic range (points 1 to 3) with non-negligible residual strength and ability to support gravity loads at point 3. The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range (points 2 to 3). Primary component actions exhibiting this behavior shall be classified as deformation-controlled if the strain-hardening or strain-softening range is such that $e > 2g$; otherwise, they shall be classified as force controlled. Secondary component actions exhibiting Type 1 behavior shall be classified as deformation-controlled for any e/g ratio. The Type 2 curve depicted in Fig. 3.8 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) and a plastic range (points 1 to 2) followed by loss of strength and loss of ability to support gravity loads beyond point 2. Primary and secondary component actions exhibiting this type of behavior shall be classified as deformation-controlled if the plastic range is such that $e > 2g$; otherwise, they shall be classified as force controlled. The Type 3 curve depicted in Fig. 3.8 is representative of a brittle or non-ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by loss of strength and loss of ability to support gravity loads beyond point 1. Primary and secondary component actions displaying Type 3 behavior shall be classified as force-controlled (FEMA-356, 2000).

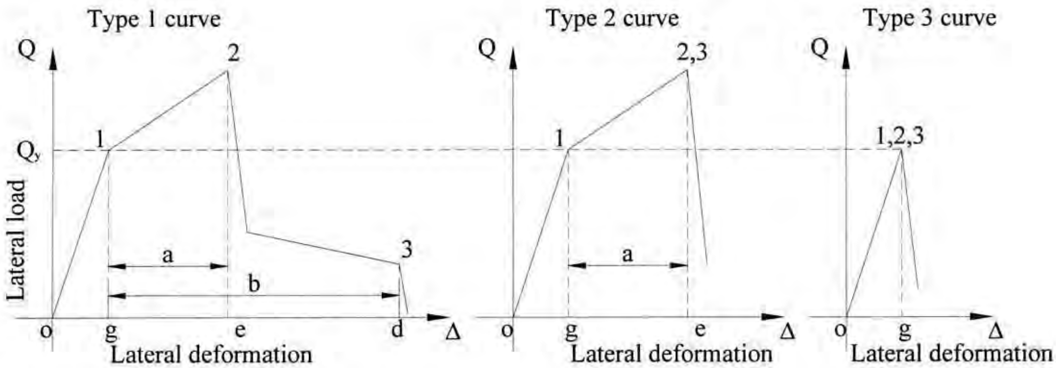


Fig. 3.8 Component force versus deformation curves(FEMA-356, 2000)

b. Deformation-Controlled and Force- Controlled Behavior

Acceptance criteria for primary components that exhibit Type 1 behavior are typically within the elastic or plastic ranges between points 0 and 2, depending on the performance level. Acceptance criteria for secondary elements that exhibit Type 1 behavior can be within any of the performance ranges. Acceptance criteria for primary and secondary components exhibiting Type 2 behavior will be within the elastic or plastic ranges, depending on the performance level. Acceptance criteria for primary and secondary components exhibiting Type 3 behavior will always be within the elastic range. Table 3-5 provides some examples of possible deformation- and force-controlled actions in common framing systems.

Table 3.5 Examples of possible deformation-controlled and force-controlled actions(FEMA-356, 2000)

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames		
Beam	Moment (M)	Shear (V)
Columns	M	Axial load (P), V
Joints	-	V^1

Component	Deformation-Controlled Action	Force-Controlled Action
Shear Walls	M, V	P
Braced Frames		
Braces	P	--
Beams	--	P
Columns	--	P
Shear Link	V	P, M
Connections	P, V, M[3]	P, V, M
Diaphragms	M, V[2]	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame connection
2. If the diaphragm carries lateral loads from vertical seismic resisting elements above the diaphragm level, then M and V shall be considered force-controlled actions.
3. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

3.9 Acceptability Limit

A given component may have a combination of both force and deformation controlled actions. Each element must be checked to determine whether its individual components satisfy acceptability requirements under performance point forces and deformations. Together with the global requirements, acceptability limits for individual components are the main criteria for assessing the calculated building response.

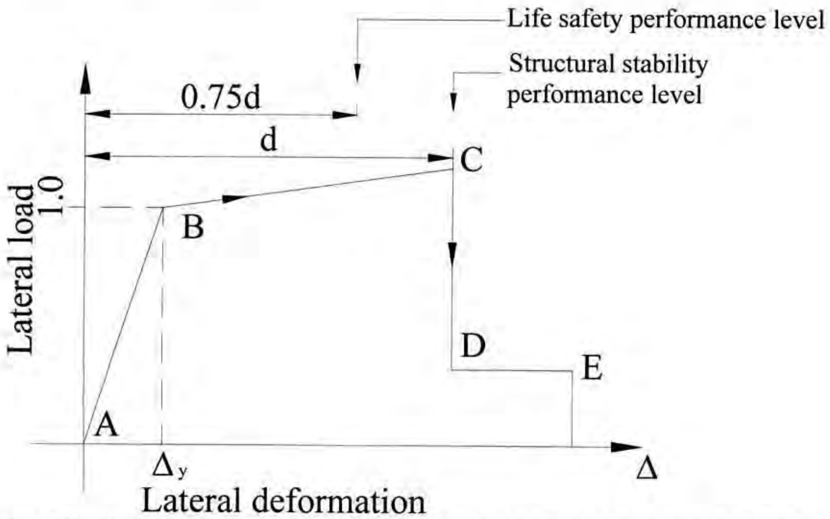


Fig. 3.9 Force-deformation action and acceptance criteria (ATC-40, 1996)

The Fig.3.9 shows a generalized load – deformation relation appropriate for most concrete components. The relation is described by linear response from A (unloaded component) to an effective yield point B, linear response at reduced stiffness from B to C, sudden reduction in lateral load resistance to D, response at reduced resistance to E, and final loss of resistance thereafter. The following main points relate to the depicted load-deformation relation:

- Point A corresponds to the unloaded condition. The analysis must recognize that gravity loads may induce initial forces and deformations that should be accounted for in the model. Therefore, lateral loading may commence at a point other than the origin of the load-deformation relation.
- Point B has resistance equal to the nominal yield strength. The slope from B to C, ignoring the effects of gravity loads acting through lateral displacements, is usually taken as between 5% and 10% of the initial slope. This strain hardening, which is observed for most reinforced concrete component, may have an important effect on the redistribution of internal forces among adjacent components.
- The abscissa at C corresponding to the deformation at which significant strength degradation begins.
- The drop in resistance from C to D represents initial failure of the component.
- The residual resistance from D to E may be non-zero in some cases and may be effectively zero in others. Where specific information is not available, the residual resistance usually may be assumed to be equal to 20% of the nominal strength.
- Point E is a point defining the maximum deformation capacity. Deformation beyond that limit is not permitted because gravity load can no longer be sustained.

Table 3.6 to Table 3.12 give the acceptance criteria (ATC-40, 1996) to be used with Nonlinear Procedures for the acceptance model of individual structural elements of a structure that has been used to evaluated for finding seismic performance of the selected buildings under this thesis.

Table 3.6 Numerical acceptance criteria for plastic hinge rotations in reinforced concrete beams, in radians (ATC-40, 1996)

			Performance Level[3]				
			Primary			Secondary	
Component Type			IO	LS	SS	LS	SS
1. Beams Controlled by Flexure[1]							
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$ [4]					
≤0.0	C	≤3	0.005	0.020	0.025	0.020	0.050
≤0.0	C	≥6	0.005	0.010	0.020	0.020	0.040
≥0.5	C	≤3	0.005	0.010	0.020	0.020	0.030
≥0.5	C	≥6	0.005	0.005	0.015	0.015	0.020
≤0.0	NC	≤3	0.005	0.010	0.020	0.020	0.030
≤0.0	NC	≥6	0.000	0.005	0.010	0.010	0.015
≥0.5	NC	≤3	0.005	0.010	0.010	0.010	0.015
≥0.5	NC	≥6	0.000	0.005	0.005	0.005	0.010
2. Beams controlled by shear[1]							
Stirrup spacing ≤ d/2			0.0	0.0	0.0	0.010	0.020
Stirrup spacing > d/2			0.0	0.0	0.0	0.005	0.010
3. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing ≤ d/2			0.0	0.0	0.0	0.010	0.020
Stirrup spacing > d/2			0.0	0.0	0.0	0.005	0.010
4. Beams controlled by inadequate embedment into beam-column joint[1]							
			0.0	0.01	0.015	0.020	0.030

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: (1) closed stirrup are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

4. V = Design Shear

Table 3.7 Numerical acceptance criteria for plastic hinge rotations in reinforced concrete columns, in radians (ATC-40, 1996)

			Performance Level[4]				
			Primary			Secondary	
Component Type			**IO**	**LS**	**SS**	**LS**	**SS**
1. Columns Controlled by Flexure[1]							
$\frac{P}{A_g f'_c}$ [5]	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$ [6]					
≤0.1	C	≤3	0.005	0.010	0.020	0.015	0.030
≤0.1	C	≥6	0.005	0.010	0.015	0.010	0.025
≥0.4	C	≤3	0.000	0.005	0.015	0.010	0.025
≥0.4	C	≥6	0.000	0.005	0.010	0.010	0.015
≤0.1	NC	≤3	0.005	0.005	0.010	0.005	0.015
≤0.1	NC	≥6	0.005	0.005	0.005	0.005	0.005
≥0.4	NC	≤3	0.000	0.000	0.005	0.000	0.005
≥0.4	NC	≥6	0.000	0.000	0.000	0.000	0.000
2. Columns controlled by shear[1,3]							
Hoop spacing ≤d/2, or $\frac{P}{A_g f'_c} \le 0.1$			0.000	0.000	0.000	0.010	0.015
Other cases			0.000	0.000	0.000	0.00	0.000
3. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤d/2			0.0	0.0	0.0	0.010	0.020
Hoop spacing >d/2			0.0	0.0	0.0	0.005	0.010
4. Columns with axial loads exceeding 0.70 [1,3]							
Conforming reinforcement over the entire length			0.0	0.0	0.005	0.005	0.010
All other cases			0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table. See Chapter 9 for symbol definitions.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: (1) closed hoops are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. To qualify, (1) hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.

4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

5. P = Design axial load

6. V = Design shear force

Table 3.8 Numerical acceptance criteria for chord rotations for reinforced concrete coupling beams (ATC-40, 1996)

		Performance Level[3]				
		Primary			Secondary	
Component Type		IO	LS	SS	LS	SS
1. Coupling beams controlled by flexure						
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f'_c}}$ [2]					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.006	0.015	0.025	0.025	0.040
Conventional longitudinal reinforcement with conforming transverse reinforcement	≥6	0.005	0.010	0.015	0.015	0.030
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤3	0.006	0.012	0.020	0.020	0.035
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≥6	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement	N/A	0.006	0.018	0.030	0.030	0.050
2. Coupling beams controlled by shear						
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f'_c}}$ [2]					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.006	0.012	0.015	0.015	0.024
Conventional longitudinal reinforcement with conforming transverse reinforcement	≥6	0.004	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤3	0.006	0.008	0.010	0.010	0.020
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≥6	0.004	0.006	0.007	0.007	0.012

1. Conventional longitudinal steel consists of top and bottom steel parallel to the longitudinal axis of the beam. The requirements for conforming transverse reinforcement are: (1) closed stirrups are to be provided over the entire length of the beam at spacing not exceeding d/3; and (2) the strength provided by the stirrups (Vs) should be at least three-fourths of the design shear.

2. V = the design shear force on the coupling beam in pounds, b_w = the web width of the beam, d = the effective depth of the beam and f'_c = concrete compressive strength in psi.

3. Linear interpolation between values listed in the table is permitted.
   IO = Immediate occupancy
   LS = Life Safety
   SS = Structural Stability

Table 3.9 Numerical acceptance criteria for reinforced concrete column axial hinge (FEMA-356, 2000)

	Plastic Deformation[1]				
	Primary			Secondary	
Component Type	IO	LS	SS	LS	SS
1. Braces in Tension (except EBF braces)	$7\Delta_T$	$9\Delta_T$	$11\Delta_T$	$11\Delta_T$	$13\Delta_T$

[1] Δ_T is the axial deformation at expected tensile yielding load.

Table 3.10 Numerical acceptance criteria for total shear angle in reinforced concrete beam-columns joints, in radians (ATC-40, 1996)

Component Type			Performance Level[4]				
			Primary[6]			Secondary	
			IO	LS	SS	LS	SS
1. Interior joints							
$\frac{P}{A_g f'_c}$ [2]	Transverse Reinforcement[1]	$\frac{V}{V_n}$ [3]					
≤0.1	C	≤1.2	0.0	0.0	0.0	0.020	0.030
≤0.1	C	≥1.5	0.0	0.0	0.0	0.015	0.020
≥0.4	C	≤1.2	0.0	0.0	0.0	0.015	0.025
≥0.4	C	≥1.5	0.0	0.0	0.0	0.015	0.020
≤0.1	NC	≤1.2	0.0	0.0	0.0	0.015	0.020
≤0.1	NC	≥1.5	0.0	0.0	0.0	0.010	0.015
≥0.4	NC	≤1.2	0.0	0.0	0.0	0.010	0.015
≥0.4	NC	≥1.5	0.0	0.0	0.0	0.010	0.015
2. Other joints							
$\frac{P}{A_g f'_c}$ [2]	Transverse Reinforcement[1]	$\frac{V}{V_n}$ [3]					
≤0.1	C	≤1.2	0.0	0.0	0.0	0.015	0.020
≤0.1	C	≥1.5	0.0	0.0	0.0	0.010	0.015
≥0.4	C	≤1.2	0.0	0.0	0.0	0.015	0.020
≥0.4	C	≥1.5	0.0	0.0	0.0	0.010	0.015
≤0.1	NC	≤1.2	0.0	0.0	0.0	0.005	0.010
≤0.1	NC	≥1.5	0.0	0.0	0.0	0.005	0.010
≥0.4	NC	≤1.2	0.0	0.0	0.0	0.000	0.000
≥0.4	NC	≥1.5	0.0	0.0	0.0	0.000	0.000

1. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A joint is conforming if closed hoops are spaced at $\leq h_c/3$ within the joint. Otherwise, the component is considered non-conforming. Also, to qualify as conforming details under condition 2, (1) closed hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.

2. The ratio $\frac{P}{A_g f'_c}$ is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional and lateral forces.

3. The ratio V/V_n is the ratio of the design shear force to the shear strength for the joint.

4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

5. No inelastic deformation is permitted since joint yielding is not allowed in a conforming building.

Table 3.11 Numerical acceptance criteria for plastic hinge rotation in reinforced concrete two-way slabs and slab-column connections, in radians (ATC-40, 1996)

		Performance Level[4]				
		Primary			Secondary	
Component Type		**IO**	**LS**	**SS**	**LS**	**SS**
1. Slabs controlled by flexure and slab column connections[1]						
$\frac{V_g}{V_0}$ [2]	Continuity Reinforcement[3]					
≤0.2	Yes	0.01	0.015	0.02	0.030	0.05
≥0.4	Yes	0.00	0.000	0.00	0.030	0.04
≤0.2	No	0.01	0.015	0.02	0.015	0.02
≥0.4	No	0.00	0.000	0.00	0.000	0.00
2. Slabs controlled by inadequate development or splicing along the span[1]						
		0.00	0.00	0.000	0.01	0.02
3. Slabs controlled by inadequate embedment into slab-column joint[1]						
		0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318, V_o = the direct punching shear strength as defined by ACI 318.

3. Under the heading "Continuity reinforcement" assume 'Yes' where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."

4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

Table 3.12 Numerical acceptance criteria for plastic hinge rotations in reinforced concrete walls and wall segments controlled by flexure, in radians (ATC-40, 1996)

			Performance Level[4]				
			Primary			Secondary	
Component Type			**IO**	**LS**	**SS**	**LS**	**SS**
1. Walls and wall segments controlled by flexure							
$\frac{(A_S - A_S')f_y + P}{t_w l_w f_c'}$ [1]	$\frac{V}{t_w l_w \sqrt{f_c'}}$ [2]	Boundary Element[3]					
≤0.1	≤3	C	0.005	0.010	0.015	0.015	0.020
≤0.1	≥6	C	0.004	0.008	0.010	0.010	0.015
≥0.25	≤3	C	0.003	0.006	0.009	0.009	0.012
≥0.25	≥6	C	0.001	0.003	0.005	0.005	0.010
≤0.1	≤3	NC	0.002	0.004	0.008	0.008	0.015
≤0.1	≥6	NC	0.002	0.004	0.006	0.006	0.010
≥0.25	≤3	NC	0.001	0.002	0.003	0.003	0.005
≥0.25	≥6	NC	0.001	0.001	0.002	0.002	0.004

1. A_s = the cross-sectional area of longitudinal reinforcement in tension, A_s' = the cross-sectional area of longitudinal reinforcement in compression, f_y = yield stress of longitudinal reinforcement, P = axial force acting on the wall considering design load combinations, t_w = wall web thickness, l_w = wall length, and f'_c = concrete compressive strength.

2. V = the design shear force acting on the wall, and other variables are as defined above.

3. The term "C" indicates the boundary reinforcement effectively satisfies requirements of ACI 318. The term "NC" indicates the boundary requirements do not satisfy requirements of ACI 318.

4. Linear interpolation between values listed in the table is permitted.

IO = Immediate Occupancy

LS = Life Safety

SS = Structural Stability

Chapter 4

SEISMIC DEMAND AND THE BASIC MODELING PARAMETERS

4.1 Introduction

In order to find a building's performance against some specified seismic demand, response quantities from a static non-linear analysis is compared with the global deformation limits for appropriate performance limits. The response limits falls into two categories:

- Global acceptability limits: These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift. They are defined in Chapter 3.

- Element and component acceptability limits: Each element (frame, wall, diaphragm, or foundation) must be checked to determine if its components respond within acceptable limits.

Building performance objectives are checked against some predefined seismic demand. Seismic demand for a structure is totally site depended. For analysis development of site dependent elastic response spectrum is needed. But unfortunately Bangladesh National Building Code (BNBC, 1993) does not have any guideline to develop such site dependent response spectrum. The Federal Emergency Management Agency (FEMA-356, 2000) and Applied Technology Council (ATC-40, 1996) has recommended standard procedure to establish seismic demand at a site. This procedure is followed in the subsequent analysis and is discussed next.

4.2 Seismic Demand

Earthquake is an uncertain phenomenon. It is not possible to predict the time and what intensity of earthquake that may hit in some specific regions. For example, large devastating earthquake that hit in the region was the Great Indian Earthquake in 12 June, 1897. Recent devastating earthquakes around the sub-continent leads to assessment that Bangladesh is very vulnerable to earthquake. It is possible to design a structure that will withstand such a major devastating earthquake but this huge investment is not always feasible economically for such an uncertain event. Thus the earthquake design philosophy adopted in building codes accepts that:

- Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however building parts that do not carry load may sustain repairable damage.
- Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake.
- Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse.

Severity of earthquakes as classified in ATC-40, 1996 is defined below.

a. The Serviceability Earthquake (SE)

The Serviceability Earthquake (SE) is defined probabilistically as the level of ground shaking that has a 50 percent chance of being exceeded in 50-year period. This level of earthquake ground shaking is typically about 0.5 times of the level of ground shaking of the Design Earthquake. The SE has a mean return period of approximately 75 years. Damage in the non structural elements is expected during Serviceability Earthquake.

b. The Design Earthquake (DE)

The Design Earthquake (DE) is defined probabilistically as the level ground shaking that has a 10 percent chance of being exceeded in a 50-year period. The DE represents an infrequent level of ground shaking that can occur during the life of the building. The DE has a mean return period of approximately 500 years. Minor repairable damage in the primary lateral load carrying system is expected during Design Earthquake. For secondary elements, the damage may be such that they require replacement.

c. The Maximum Earthquake (ME)

The Maximum Earthquake (ME) is defined deterministically as the maximum level of earthquake ground shaking which may ever be excepted at the building site within the known geologic frame work. In probabilistic terms, the ME has a return period of about 1,000 years. During Maximum Earthquake, buildings will be damaged beyond repairable limit but will not collapse.

4.2.1 Development of Elastic Site Response Spectra

Elastic response spectra for a site are based on estimate of Seismic Coefficient, C_A which represents the effective peak acceleration (EPA) of the ground and C_V which represents 5 percent-damped response of a 1-second system. These coefficients for a particular zone are dependent on the seismicity of the area, the proximity of the site to active seismic sources, and site soil profile characteristics.

4.2.1.1 Seismic zone

Bangladesh is divided into three seismic zones as per BNBC. The table below shows the values of zone coefficients of Bangladesh.

Table 4.1 Seismic zone factor Z

Zone	1	2	3
Z	0.075	0.15	0.25

4.2.1.2 Seismic Source Type

As per ATC-40 (1996), three types of seismic source may be defined as shown in Table 4.2.

Table 4.2 Seismic source type as per ATC-40, 1996

		Seismic Source Definition	
Seismic Source Type	Seismic Source Description	Maximum Moment Magnitude, M	Slip Rate, SR(mm/yr)
A	Faults that are capable to produce large magnitude events and which have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than types A and C	Not applicable	Not applicable
C	Faults that are not capable to producing large magnitude events and which have a high rate of seismic activity	$M < 6.5$	$SR < 2$

4.2.1.3 Near-Source Factor

Currently data pertaining to the active faults close to Dhaka city is not available. It is not possible to estimate the seismic source distance from a specific site being considered in this thesis. But it may be safely assumed that all the sources are located at distance more than 15 km and the Table 4.3 (ATC-40, 1996) may be used to consider the Near-Source effects for the present study.

Table 4.3 Seismic source factor

Seismic Source Type	Closed Distance to Known Seismic Source							
	$\leq$ 2km		5 km		10 km		$\geq$ 15 km	
	N_A	N_V	N_A	N_V	N_A	N_V	N_A	N_V
A	1.5	2.0	1.2	1.6	1.0	1.2	1.0	1.0
B	1.3	1.6	1.0	1.2	1.0	1.0	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

1. The near-source factor may be used on the linear interpolation of values for distance other than those shown in the table.

2. The closest distance of the seismic source shall be taken as the minimum distance between the site and the area described by the vertical projecting of source on the surface (i.e., surface projection of fault plane). The surface projecting need not include portions of the source a depths of 10km or greater. The largest value of the near-source factor considering all sources shall be use for design.

4.2.1.4 Seismic Coefficients

For each earthquake hazard level, the structure is assigned a seismic coefficient C_A in accordance Table 4.4 (ATC-40, 1996) and a seismic coefficient C_V in accordance with Table 4.5 (ATC-40, 1996). Seismic coefficient C_A represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5 times C_A represents the average value of peak response of a 5 percent-damped short-period system in the acceleration domain. The seismic coefficient C_V represents 5 percent-damped response of a 1-second system. C_V divided by period (T) defines acceleration response in the velocity domain. These coefficients are dependent on soil profile type and the product of earthquake zoning coefficient-Z, severity of earthquake-E and near source factor-N (ZEN). The soil profile types are classified in Table 4.6.

Table 4.4 Seismic coefficient C_A

Soil Profile Type	Shaking Intensity, ZEN[1,2]			
	= 0.075	**= 0.15**	**= 0.20**	**= 0.30**
S_B	0.08	0.15	0.20	0.30
S_C	0.09	0.18	0.24	0.33
S_D	0.12	0.22	0.28	0.36
S_E	0.19	0.30	0.34	0.36
S_F	Site-specific geo-technical investigation required to determine C_A			

1. The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.

2. Seismic coefficient C_A should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

Table 4.5 Seismic coefficient C_V (ATC-40, 1996)

Soil Profile Type	Shaking Intensity, ZEN			
	= 0.075	**= 0.15**	**= 0.20**	**= 0.30**
S_B	0.08	0.15	0.20	0.30
S_C	0.13	0.25	0.32	0.45
S_D	0.18	0.32	0.40	0.54
S_E	0.26	0.50	0.64	0.84
S_F	Site-specific geo-technical investigation required to determine C_V			

1. The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.

2. Seismic coefficient C_V should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

Table 4.6 Soil profile types (ATC-40, 1996)

		Average Soil Properties for Top 100 ft of Soil Profile		
Soil Profile Type	Soil Profile Name/Generic Description	Share Wave Velocity, Vs(ft/sec)	Standard Penetration Test, N or N_{CH} for cohesionless soil layers(blow/ft)	Undrained Shear Strength, S_u(psf)
S^1_A	Hard Rock	$V_S > 5{,}000$	Not Applicable	
S_B	Rock	$2{,}500 < V_S \leq 5{,}000$	Not Applicable	
S_C	Very Dense Soil and Rock	$1{,}200 < V_S \leq 2{,}500$	$N > 50$	$S_U > 2{,}000$
S_D	Stiff Soil Profile	$600 < V_S \leq 1{,}200$	$15 \leq N \leq 50$	$1{,}000 \leq S_U \leq 2{,}000$
S^2_E	Soft Soil Profile	$V_S < 600$	$N < 50$	$S_U < 1{,}000$
S_F	Soil Requiring Site-Specific Evaluation			

1. Soil profile SA is not applicable to site in Dhaka.
2. Soil profile type S_E also include any soil profile with more than 10 feet or soft clay defined as a soil with PI > 20, $W_{MC} \geq 40$ and $S_u < 500$ psf..

4.3 Establishing Demand Spectra

For the purpose of subsequent analysis to be made in this thesis, it is necessary to establish an earthquake demand spectra against which building performance will be evaluated. The following controlling parameters are considered:

Location of the site : Dhaka City

Soil profile at the site : Soil type S_E as per Table 4.6, soft soil with shear wave velocity V_S<600 ft/sec, N < 50 and S_u < 100 psf

Earthquake source type : A – considering the events similar to the great Indian Earthquake in Assam in 12 June, 1897

Near Source Factor : > 15km

Calculation of C_A			
Seismic Zone Factor, Z	=	0.15	*as per BNBC*
Earthquake Hazard Level, E	=	1	*Design Earthquake*
Near-Source Factor, N	=	1	*>15km, Table 4.3*
Shaking Intensity, ZEN	=	0.15	
For Soil Type S_E, C_A	=	0.3	*From Table 4.4*

Calculation of C_V			
Seismic Zone Factor, Z	=	0.15	*as per BNBC*
Earthquake Hazard Level, E	=	1	*Design Earthquake*
Near-Source Factor	=	1	*>15km,Table 4.3*
Shaking Intensity, ZEN	=	0.15	
For Soil Type S_E, C_V	=	0.5	*From Table 4.5*

An elastic response spectra, for each earthquake hazard level of interest at a site, is based on the site seismic coefficients C_A and C_V calculated above. The coefficient C_A represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5 times C_A represents the average value of peak response of a 5% damped short-period system in the acceleration domain. The seismic coefficient C_V represents 5% damped response of a 1-second system and when divided by period defines acceleration response in velocity domain.

The following establishes 5% damped elastic response spectra as shown in Fig. 4.1

Effective peak ground acceleration (EPA)	=	0.3	*g*	C_A
Average value of peak response	=	0.750	*g*	$2.5C_A$
Seismic coefficient, C_V	=	0.5	*g*	C_V
T_S	=	0.667	*sec*	$T_S = C_V / 2.5C_A$
T_A	=	0.133	*sec*	$T_A = 0.2T_S$

5% Elastic response spectrum has been prepared using the procedure above and shown in Fig. 4.1.

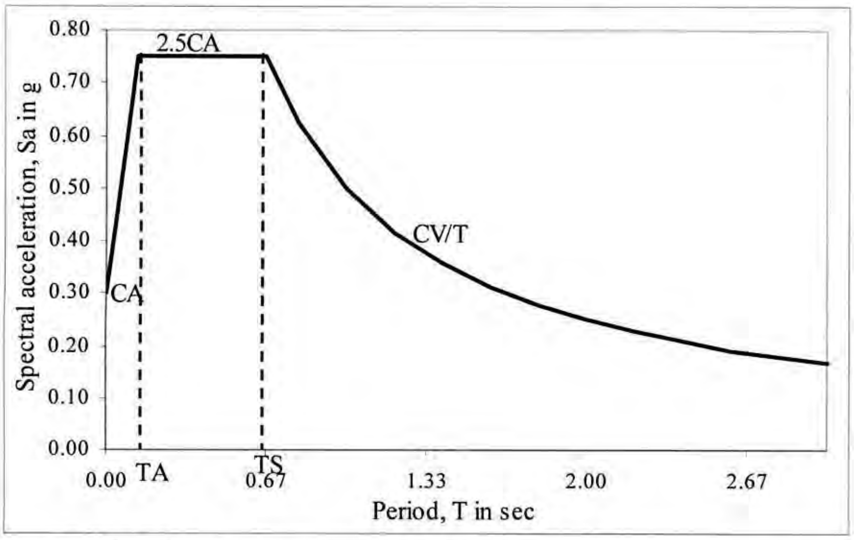


Fig. 4.1 5% Elastic response spectrum

For seismic performance evaluation purpose, this newly constructed site specific 5% elastic response spectra need to be converted in to ADRS format using relation, $S_d = \frac{T^2}{4\pi^2} S_a g$.

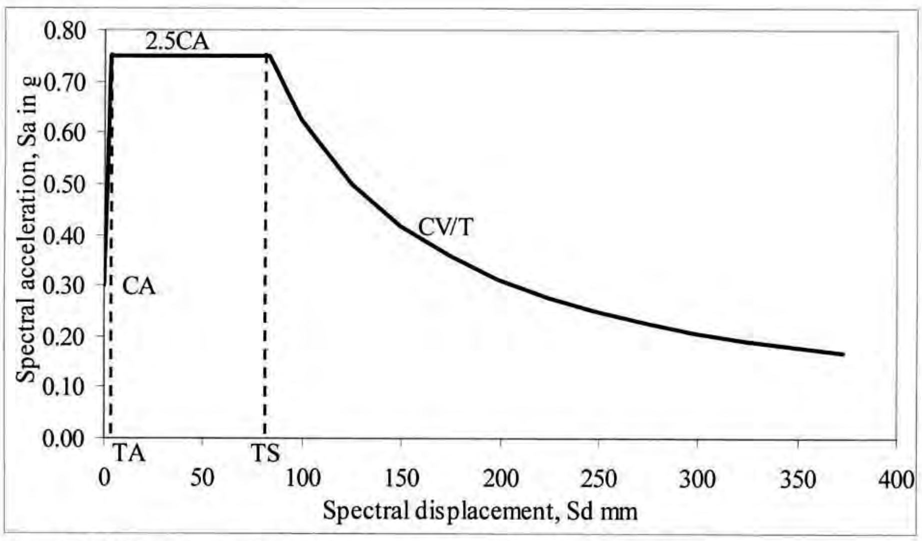


Fig. 4.2 5% Damped elastic response spectrum in ADRS format

Calculated spectral acceleration, spectral displacement with respect to the period is shown in Table 4.7. These values are used to construct the 5% Elastic response spectrum in S_a vs. Period format (Fig. 4.1) and in ADRS format (Fig. 4.2).

Table 4.7 Response quantities for 5% elastic demand

T, sec	S_a,g	S_d, m
0.000	0.300	0.00
0.133	0.750	0.00
0.667	0.750	0.08
0.800	0.625	0.10
1.000	0.500	0.12
1.200	0.417	0.15
1.400	0.357	0.17
1.600	0.313	0.20
1.800	0.278	0.22
2.000	0.250	0.25
2.200	0.227	0.27
2.400	0.208	0.30
2.600	0.192	0.32
2.800	0.179	0.35
3.000	0.167	0.37

4.4 Element Hinge Property

It is known that reinforced concrete does not respond elastically to load level about half the ultimate value. When an element is stressed beyond its elastic limit, due to inelastic deformation of the materials, the element will continue to deform disproportionate to its load, this process is called formation of plastic hinge. Hinge properties of RC members under different loading conditions are likely to be different. These are discussed in the next sections.

4.4.1 Concrete Axial Hinge

Concrete axial hinge is formed when the axial load carrying capacity of a section exceeds its elastic limit. The elastic limit for axial capacity is different for tension and compression. The limits are explained in Fig. 4.3

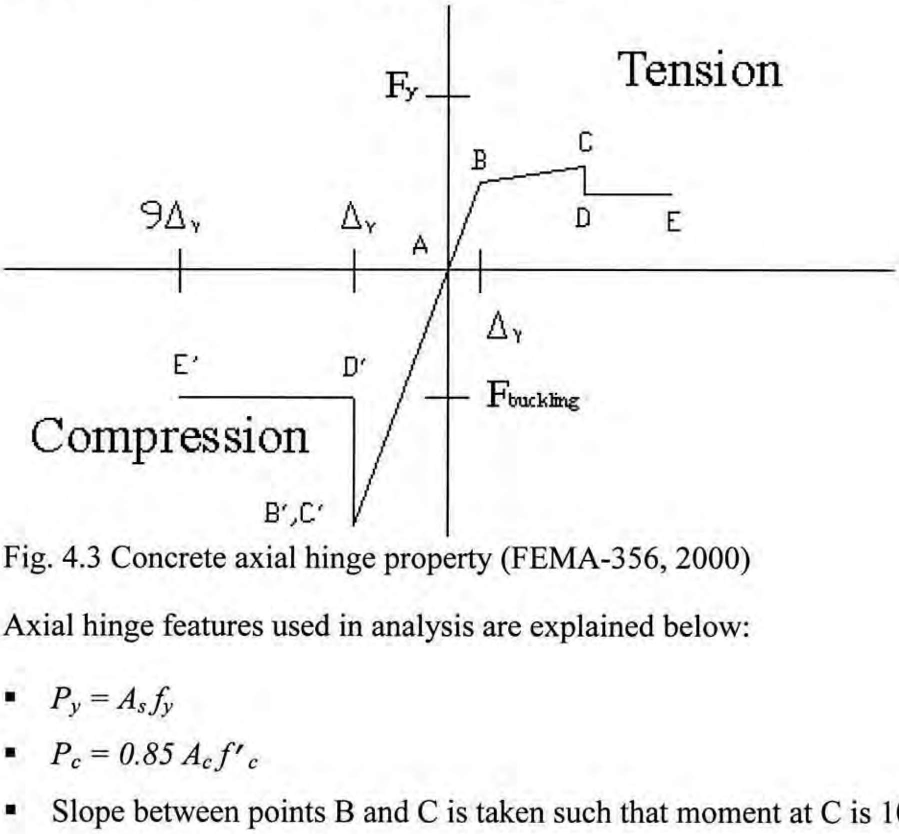


Fig. 4.3 Concrete axial hinge property (FEMA-356, 2000)

Axial hinge features used in analysis are explained below:

- $P_y = A_s f_y$
- $P_c = 0.85\ A_c f'_c$
- Slope between points B and C is taken such that moment at C is 10% higher than at B to represent strain hardening of steel
- Hinge length assumption for Δy is based on the full length
- Point B, C, D and E are based on recommendation of FEMA-356 (Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, Braces in Tension)
- Point B′ = P_c
- Point E′ taken as 9Δy

4.4.2 Concrete Moment Hinge and Concrete P-M-M Hinge

Concrete moment hinge is formed when the flexural moment carrying capacity of a section exceeds its elastic limit. The limits of flexural moment capacity and bi-axial moment with axial load are explained in the Fig. 4.4.

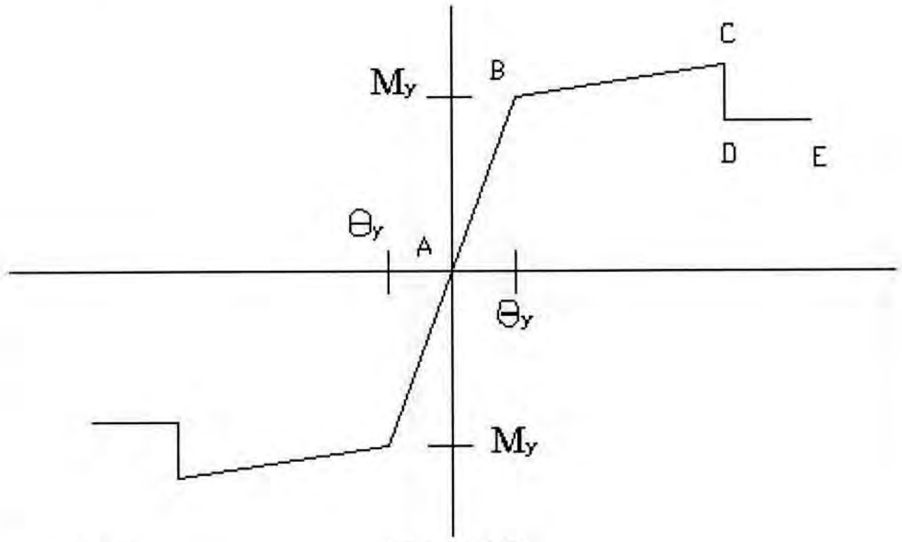


Fig. 4.4 Concrete moment and P-M-M hinge property

P-M-M hinge features used in analysis are explained below:

- Slope between points B and C is taken such that moment at C is 10% higher than at B to represent strain hardening of steel
- $\theta_y = 0$, since it is not needed
- Points C, D and E based on the recommendation of Advance Technology Council(ATC-40, 1996)
- M_y is based on reinforcement provided
- P-M-M curve is for major axis moment and is taken to be the same as the Moment curve in conjunction with the definition of Axial–Moment interaction curves.

4.4.3 Concrete Shear Hinge

Concrete shear hinge is formed when the flexural carrying capacity of a section exceeds its elastic limit. The elastic limit for flexural shear capacity for coupling beams controlled by flexure and controlled by shear is explained in Fig. 4.5 (ATC-40, 1996).

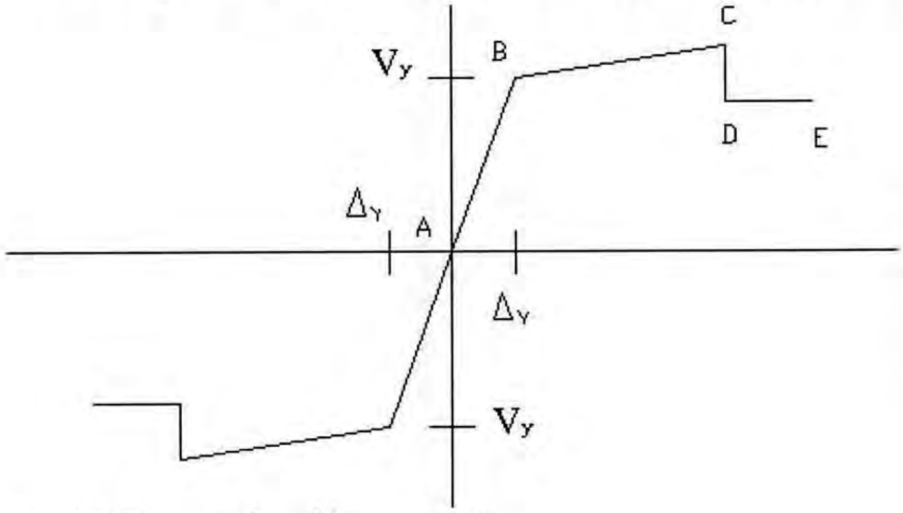


Fig. 4.5 Concrete shear hinge property

Shear hinge features used in analysis:

- Slope between points B and C is taken such that shear at C is 10% higher than at B to represent strain hardening of steel
- $V_y = 2A_s\sqrt{(f'_c)} + f_y\, A_s vd$

Points C, D and E are based on the recommendation of ATC-40, 1996.

4.5 Concrete Frame Acceptability Limits

To determine the performance objective of a structure, response quantities from a nonlinear static analysis are compared with limits for appropriate performance levels. Fig. 4.6 illustrates a generalized load-deformation relation applied in the structural components under the thesis. Curve Type I in the Fig. 4.6 has been used when the deformation is a flexural plastic hinge. Curve type II in the Fig. 4.6 has been used when the deformation is inter-story drift, shear angle, sliding shear displacement, or beam-column joint rotation.

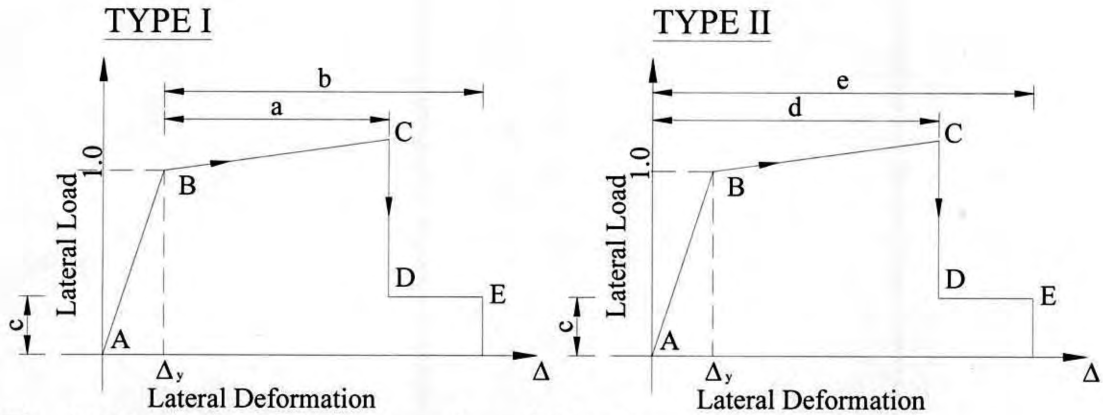


Fig. 4.6 Generalized load-deformation relations for components

Tables 4.8 to 4.11 define the modeling parameter for beam and column in terms of plastic angles within the yielding plastic hinge.

Table 4.8 Modeling parameters for nonlinear procedures – reinforced concrete beams (ATC-40, 1996)

			Modeling Parameters[3]		
			Plastic Rotation Angle, rad		Residual Strength Ratio
Component Type			a	b	c
1. Beam Controlled by Flexure[1]					
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$ [4]			
≤0.0	C	≤3	0.025	0.05	0.2
≤0.0	C	≥6	0.02	0.04	0.2
≥0.5	C	≤3	0.02	0.03	0.2
≥0.5	C	≥6	0.015	0.02	0.2
≤0.0	NC	≤3	0.02	0.03	0.2
≤0.0	NC	≥6	0.01	0.015	0.2
≥0.5	NC	≤3	0.01	0.015	0.2
≥0.5	NC	≥6	0.005	0.01	0.2
2. Beams controlled by shear[1]					
Stirrup spacing ≤d/2			0.0	0.02	0.2
Stirrup spacing > d/2			0.0	0.01	0.2
3. Beams controlled by inadequate development or splicing along the span[1]					
Stirrup spacing ≤d/2			0.0	0.02	0.0
Stirrup spacing >d/2			0.0	0.01	0.0
4. Beams controlled by inadequate embedment into beam-column joint[1]					
			0.015	0.03	0.2

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: (1) closed stirrup are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. Linear interpolation between values listed in the table is permitted

4. V = design shear force

Table 4.9 Modeling parameters for nonlinear procedures – reinforced concrete column (ATC-40, 1996)

			Modeling Parameters[4]		
			Plastic Rotation Angle, rad		Residual Strength Ratio
Component Type			a	b	c
1. Columns Controlled by Flexure[1]					
$\frac{P}{A_g f'_c}$ [5]	Transverse Reinforcement[2]	$\frac{V}{b_w d\sqrt{f'_c}}$ [6]			
≤0.1	C	≤3	0.02	0.03	0.2
≤0.1	C	≥6	0.015	0.025	0.2
≥0.4	C	≤3	0.015	0.025	0.2
≥0.4	C	≥6	0.01	0.015	0.2
≤0.1	NC	≤3	0.01	0.015	0.2
≤0.1	NC	≥6	0.005	0.005	-
≥0.4	NC	≤3	0.005	0.005	-
≥0.4	NC	≥6	0.0	0.0	-
2. Columns controlled by shear[1]					
Hoop spacing ≤ d/2 or $\frac{P}{A_g f'_c}$ [5] ≤ 0.1			0.0	0.015	0.2
Other cases			0.0	0.0	0.0
3. Columns controlled by inadequate development or splicing along the clear height[1,3]					
Hoop spacing ≤d/2			0.01	0.02	0.4
Hoop spacing >d/2			0.0	0.01	0.2
4. Column with axial loads exceeding 0.40 P_0[1,3]					
Conforming reinforcement over the entire length			0.015	0.025	0.02
All other cases			0.0	0.0	0.0

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: (1) closed hoops are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. To quality, (1) hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.

4. Linear interpolation between values listed in the table is permitted.

5. P = Design axial load

6. V = design shear force

Table 4.10 Modeling parameters for concrete axial hinge (FEMA-356, 2000)

	Modeling Parameters[1]		
	Plastic Deformation		Residual Strength Ratio
Component Type	a	b	c
1. Braces in Tension (except EBF braces)	$11\Delta_T$	$14\Delta_T$	0.8

[1] Δ_T is the axial deformation at expected tensile yielding load.

Table 4.11 Modeling parameters for nonlinear procedures-coupling beams (ATC-40, 1996)

		Modeling Parameters[3]		
		Chord Rotation, rad		Residual Strength Ratio
Component Type		d	e	c
1. Coupling beams controlled by flexure				
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f'_c}}$ [2]			
Conventional longitudinal reinforcement with	≤3	0.025	0.040	0.75
Conforming transverse reinforcement	≥6	0.015	0.030	0.50
Conventional longitudinal reinforcement with non-	≤3	0.020	0.035	0.50
Conforming transverse reinforcement	≥6	0.010	0.025	0.25
Diagonal reinforcement	N/A	0.030	0.050	0.80
2. Coupling beams controlled by shear				
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f'_c}}$ [2]			
Conventional longitudinal reinforcement with	≤3	0.018	0.030	0.60
Conforming transverse reinforcement	≥6	0.012	0.020	0.30
Conventional longitudinal reinforcement with non-	≤3	0.012	0.025	0.40
Conforming transverse reinforcement	≥6	0.008	0.014	0.20

1. Conventional longitudinal steel consists of top and bottom steel parallel to the longitudinal axis of the beam. The requirements for conforming transverse reinforcement are: (1) closed stirrups are to be provided over the entire length of the beam at spacing not exceeding d/3; and (2) the strength provided by the stirrups (Vs) should be at least three-fourths of the design shear.

2. V = the design shear force on the coupling beam in pounds, b_w = the web width of the beam, d = the effective depth of the beam and f_c' = concrete compressive strength in psi.

3. Linear interpolation between values listed in the table is permitted.

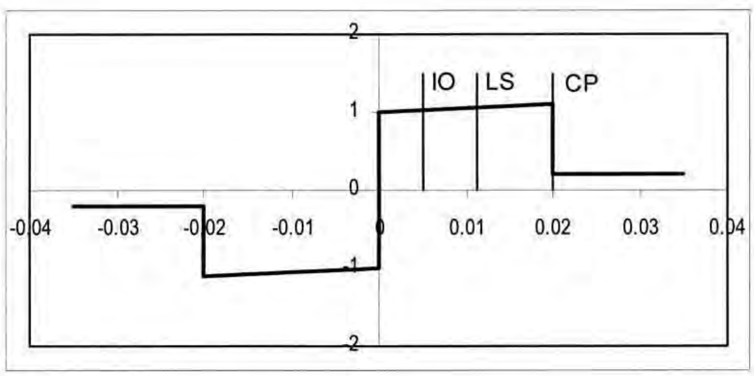
4.6 Hinge Properties for Modeling

Depending upon the longitudinal reinforcement, transverse reinforcement etc. different hinge properties may be modeled based on the modeling parameter defined through Table 4.8 to 4.11. Different points A, B, C etc. are defined in Fig. 3.9 of Chapter 3. For the purpose of the thesis, following properties of concrete hinges have been assumed.

4.6.1 Reinforced Concrete Beams - M3 Hinge

Beam model for moment and it's acceptance criteria as used in the thesis has been defined below. Considered that the beams controlled by flexure and conforming transverse reinforcement.

Point	Moment/SF	Rotation/SF[1]
E	0.2	0.035
D	0.2	0.02
C	1.1	0.02
B	1	0
A	0	0
B'	-1	0
C'	-1.1	-0.02
D'	-0.2	-0.02
E'	-0.2	-0.035



Acceptance criteria[2]

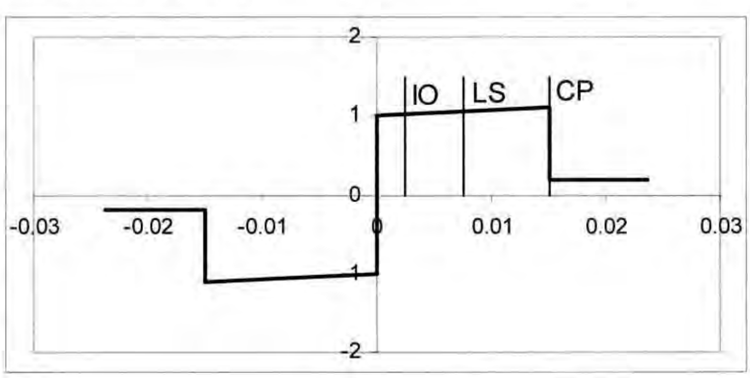
IO	LS	CP
0.005	0.011	0.020

[1] Average values of the four rows of conforming transverse reinforcement (Table 4.8).
[2] Average values of the four rows of conforming transverse reinforcement (Table 3.6).

4.6.2 Reinforced Concrete Column - M2/M3 Hinge

Column model for major/minor axis moment and it's acceptance criteria as used in the thesis has been defined below. Considered that the columns are controlled by flexure and conforming transverse reinforcement.

Point	Moment/SF	Rotation/SF[3]
E	0.2	0.024
D	0.2	0.015
C	1.1	0.015
B	1	0
A	0	0
B'	-1	0
C'	-1.1	-0.015
D'	-0.2	-0.015
E'	-0.2	-0.024



Acceptance criteria[4]

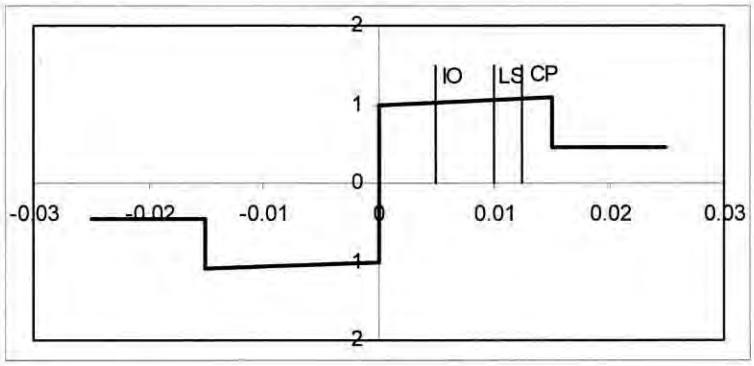
IO	LS	CP
0.0025	0.0075	0.0150

[3] Average values of the four rows of conforming transverse reinforcement (Table 4.9).
[4] Average values of the four rows of conforming transverse reinforcement (Table 3.7).

4.6.3 Reinforced Concrete Beams - Shear Hinge

Coupling beams controlled by shear. Conventional longitudinal reinforcement with conforming transverse reinforcement.

Point	Force/SF	Displacement/SF[5]
E	0.45	0.025
D	0.45	0.015
C	1.1	0.015
B	1	0
A	0	0
B'	-1	0
C'	-1.1	-0.015
D'	-0.45	-0.015
E'	-0.45	-0.025



Acceptance criteria[6]

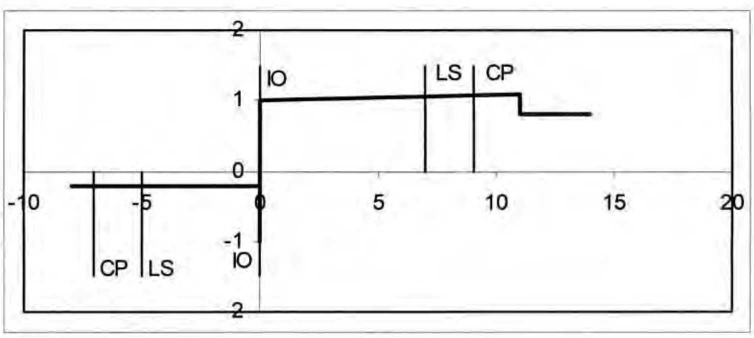
IO	LS	CP	
0.0050	0.0100	0.0125	1.5

[5] Average values of the two rows of Item 2- Conforming transverse reinforcement (Table 4.11).
[6] Average values of the two rows of Item 2 -Conforming transverse reinforcement (Table 3.8).

4.6.4 Reinforced Concrete Column-Axial Hinge[7]

Column model for axial hinge and it's acceptance criteria as used in the thesis has been defined below.

Point	Force/SF	Displacement/SF
E	0.8	14
D	0.8	11
C	1.1	11
B	1	0
A	0	0
B'	-1	0
C'	-1	-0.01
D'	-0.2	-0.01
E'	-0.2	-8



Acceptance criteria [8]

	IO	LS	CP
+ve	0	7	9
-ve	0	-5	-7

[7] From Table 4.10
[8] From Table 3.9

Chapter 5

SEISMIC PERFORMANCE EVALUATION OF STRUCTURES THROUGH PUSHOVER ANALYSIS

5.1 Introduction

In the previous chapters an outline of the procedure for structural performance evaluation in the light of ATC-40 and FEMA 356 has been described. For the performance evaluation purposes Dhaka is selected as the site and seismic demand for Dhaka has been estimated as per guideline of ATC-40. Structural performances of twelve numbers of medium-rise structures of height ranging from 20m to 30m with different plan configuration have been investigated. This height range has been selected because of the fact that buildings with this height range are very common in Dhaka city. All structures have regular geometry and stiffness.

The performances of the structures as evaluated through pushover analysis have been presented through capacity curves and capacity spectrums described in the sections that follow.

5.2 Assumption Pertaining to the Structures Under Analysis

5.2.1 Loading conditions and Material Properties

For basic design of the structures the following loading conditions have been considered.

Self-weight of the structure has been assumed as per geometric dimension of the structural elements with the unit weight of the concrete has been taken as 24 kN/m^3. 125mm partition wall of solid brick assumed on the beams resulting a uniform load of 7.30 kN/m and on the roof parapet load considered as 4.25 kN/m. Code specified floor finish 1.0 kN/m^2 has been considered on the floors and live load considered as 2.0 kN/m^2 irrespective to different use/inhabitable area. Wind load has been considered as per BNBC with basic wind speed of 210 km/h, Structure Importance of 1.0 with Exposure A. Seismic load has been considered as per BNBC. Equivalent Static Load method has been used with response modification factor, $R = 12$. No live load has been considered in calculating Seismic Dead Load. Other coefficients used in seismic load calculation are $Z = 0.15$, $I = 1.0$, $S = 1.2$. Earthquake load at any level equally distributed among all the nodes of in that level.

The material properties and relevant features are as follows:

- The structure was designed with load combination defined in the BNBC/93 with
  - Cylinder strength of concrete, f'_c = 21 MPa
  - Yield strength of steel, f_y = 413 MPa

  Sections of the columns have been chosen so as to provide maximum 4.5% reinforcement of the gross concrete Section.
- All supports are considered as fixed support.

5.2.2 Geometry and Stiffness

A total of 12 numbers of framed structures having regular geometry and stiffness were chosen for investigation of the performance under a given seismic event. All structures are designed with load and load combination as per BNBC, 1993 and they all are residential buildings. The structural detailing has been assumed to be as per Section 8.3 Special Provision For Seismic Design of BNBC. All structures are assumed to have strong column-weak beam by measuring that flexure strength of column shall satisfy the relation- $\Sigma M_c \geq 1.2\ \Sigma M_g$ where,

ΣM_c = sum of moments at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint.

ΣM_g = sum of moments at the centre of the joint, corresponding to the design flexural strengths of the girder framing into the joint.

All structures considered are bare frames i.e. presence of infill walls or soft story effects have not been considered.

5.2.3 Assumption for Pushover Analysis

The following assumptions relate to the pushover analysis of the structure

- Moment(M3) and Shear(V2) hinges are considered at the ends of beam members and P-M-M and Shear hinges are considered at the ends of the column members. All hinges are Conforming as per ATC 40 document which is described in Section 4.6 of Chapter 4 along with the performance limits.
- Pushover analysis has been done using load pattern of BNBC/93 equivalent static load. Load intensities have been normalized with the base shear. Geometric non-

linearity (P-Δ effect) of the structure was considered with full dead loads and 25% of the live load.

- In each case, the horizontal displacement of the left top most node of the structure has been selected for performance monitoring of roof displacement.
- The general-purpose finite element program SAP 2000 Nonlinear(CSI, 2000) has been used as the tool for modeling the structures and study its behavior in terms of capacity and performance. Non-linear Static Pushover analysis has also been done using the same program.

5.3 Description of Structures Considered for Performance Evaluation

5.3.1 Salient Features of Structure 1

Structure 1 is a 6 Story 3 x 2 bay residential building as shown in Fig. 5.1. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns are 0.35 x 0.55m and exterior columns are 0.3 x 0.45m with grade beam of dimension 0.3 x 0.5m and all floor beams are of dimension 0.25 x 0.45m. Reinforcement in the various floor levels of the interior column varied from 1925 mm^2 (1%) to 7200 mm^2 (3.75%) and that of exterior column from 1350mm^2 (1%) to 5300 mm^2(3.93%). Other geometric parameters are described in Table 5.1.

Table 5.1 Geometric parameters of structure 1

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 1	20000	15005	18000	12000	3 x 2	0.90	0.60	1.50

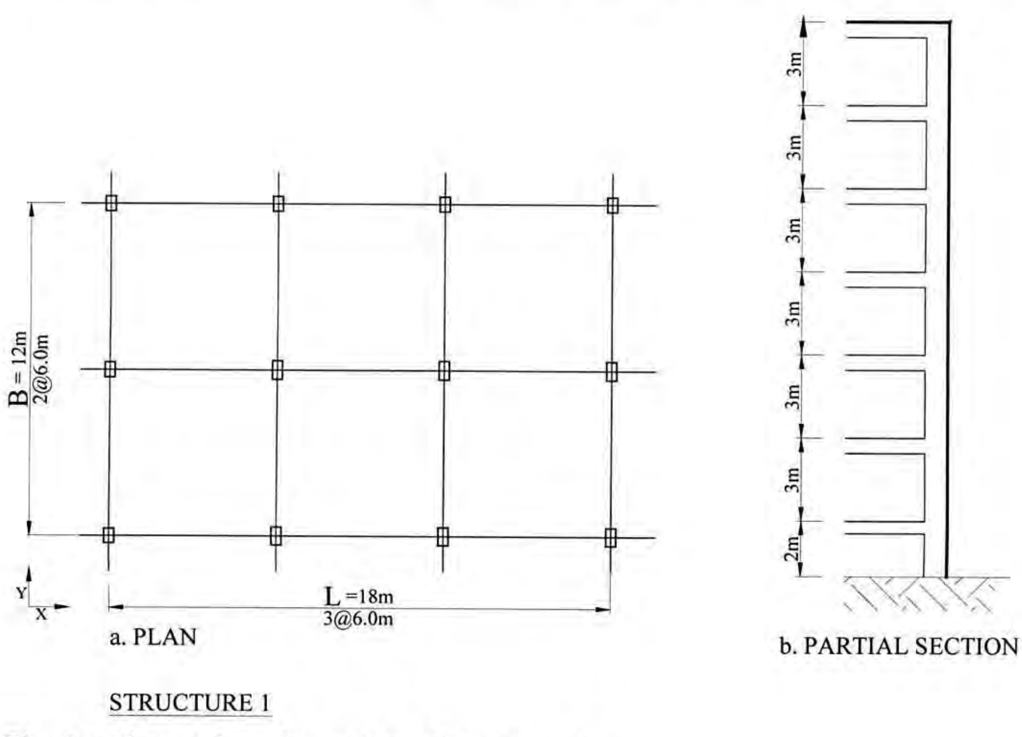


Fig. 5.1 Plan and partial section of structure 1

5.3.2 Salient Features of Structure 2

Structure 2 is a 6 story residential building. It has equal 3 x 3 bays. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns are 0.45 x 0.45m and exterior columns are 0.35 x 0.35m with grade beam of dimension 0.325 x 0.45m and all floor beams are of dimension 0.25 x 0.5m. Reinforcement in the various floor levels of the interior columns varied from 2025 mm^2 (1%) to 5560 mm^2 (2.75%) and that of exterior columns from 1230mm^2 (1%) to 3860 mm^2 (3.15%). Other geometric parameters are as described in Table 5.2.

Table 5.2 Geometric parameters of structure 2

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 2	20000	17776	18000	18000	3 x 3	0.90	0.90	1.00

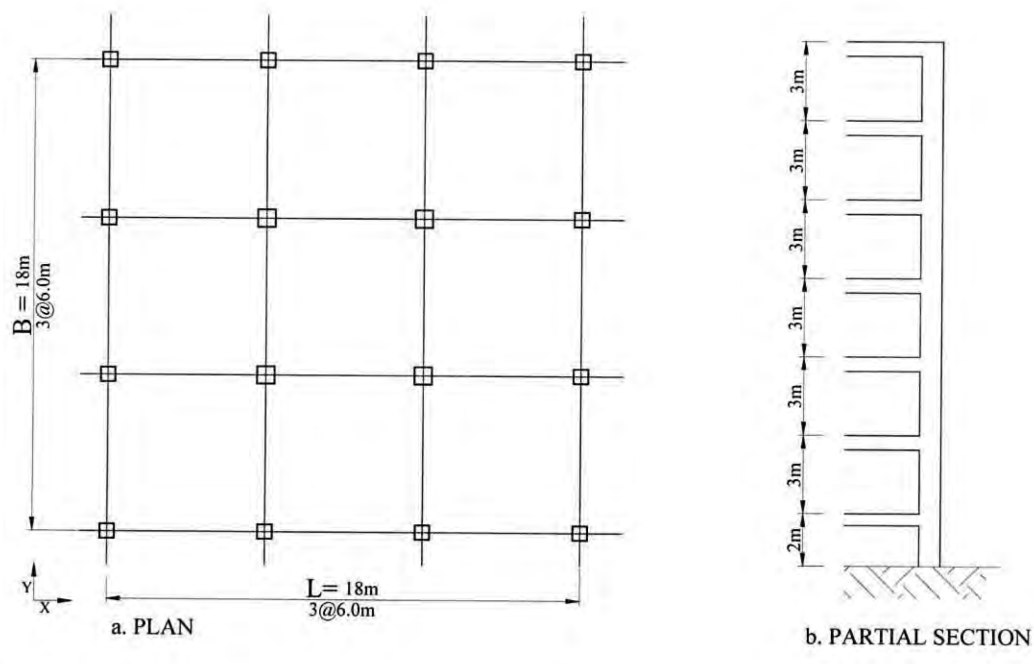


Fig. 5.2 Plan and partial section of structure 2

5.3.3 Salient Features of Structure 3

Structure 3 is a 7 story 4 x 4 equal bays residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns are 0.45 x 0.45m and exterior columns are 0.35 x 0.35m with grade beam of dimension 0.325 x 0.45m and all floor beams are of dimension 0.25 x 0.50m. Reinforcement in the various floor levels of the interior columns varied from 2025 mm^2 (1%) to 8290 mm^2 (4.1%) and that of exterior columns from 1230mm^2 (1%) to 4900 mm^2 (4.0%). Other geometric parameters are as described in Table 5.3.

Table 5.3 Geometric parameters of structure 3

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 3	23000	35477	24000	24000	4 x 4	1.04	1.04	1.00

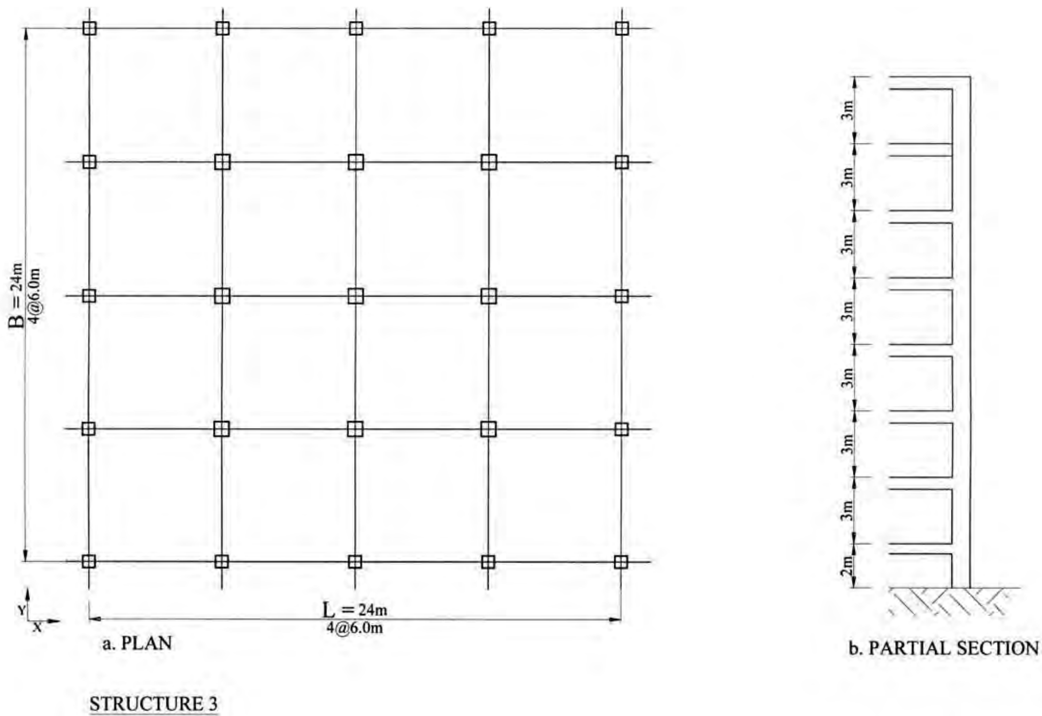


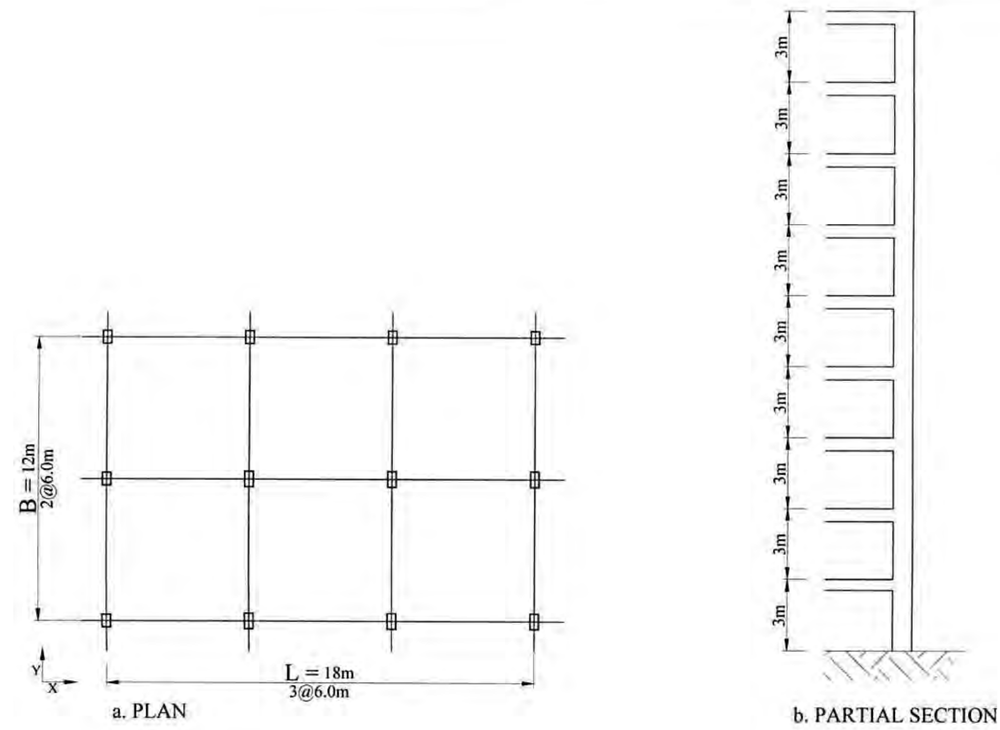
Fig. 5.3 Plan and partial section of structure 3

5.3.4 Salient Features of Structure 4

Structure 4 is an 8 story 3 x 2 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns are 0.4 x 0.8m ~ 0.3 x 0.6m and exterior columns are 0.3 x 0.6m ~ 0.3 x 0.5m with grade beam of dimension 0.325 x 0.45m and all floor beams are of dimension 0.25 x 0.50m. Reinforcement in the various floor levels of the interior columns varied from 1800 mm^2 (1%) to 10100 mm^2 (3.15%) and that of exterior columns from 1500mm^2 (1%) to 5400 mm^2 (3.0%). Other geometric parameters are as described in Table 5.4

Table 5.4 Geometric parameters of structure 4

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 4	27000	25877	18000	12000	3 x 2	0.67	0.44	1.50



STRUCTURE 4

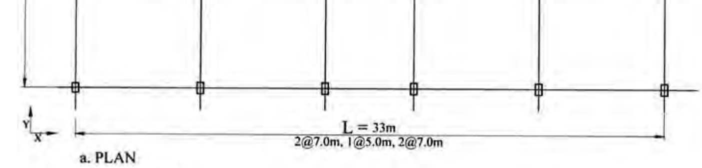
Fig. 5.4 Plan and partial section of structure 4

5.3.5 Salient Features of Structure 5

Structure 5 is a 9 story 5 x 3 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns are 0.4 x 0.7m ~ 0.4 x 0.6m and exterior columns are 0.35 x 0.65m ~ 0.3 x 0.5m with floor beam of dimension 0.25 x 0.60m in X-direction and floor beams of dimension 0.25 x 0.50m in Y-direction. Reinforcement in the various floors of the interior columns varied from 2400 mm^2 (1%) to 11100 mm^2 (4%) and that of exterior columns from 1500mm^2 (1%) to 8500 mm^2 (3.73%). Other geometric parameters are as described in Table 5.5.

Table 5.5 Geometric parameters of structure 5

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 5	30000	46729	33000	18000	5 x 3	1.10	0.60	1.83



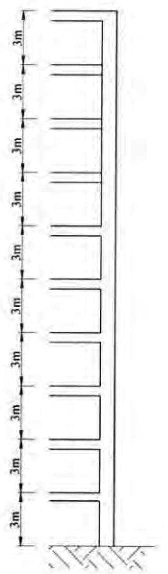


Fig. 5.5 Plan and partial section of structure 5

5.3.6 Salient Features of Structure 6

Structure 6 is a 6 story 3 x 3 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns are 0.45 x 0.45m and exterior columns are 0.45 x 0.30m with grade beam of dimension 0.325 x 0.45m and floor beams of dimension 0.25 x 0.50m. Reinforcement in the various floors of the interior columns varied from 2025mm^2 (1%) to 9900mm^2 (4.9%) and that of exterior columns from 1350mm^2 (1%) to 5100 mm^2 (3.77%). Other geometric parameters are as described in Table 5.6.

Table 5.6 Geometric parameters of structure 6

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 6	20500	22226	18000	21000	3 x 3	0.88	1.02	0.86

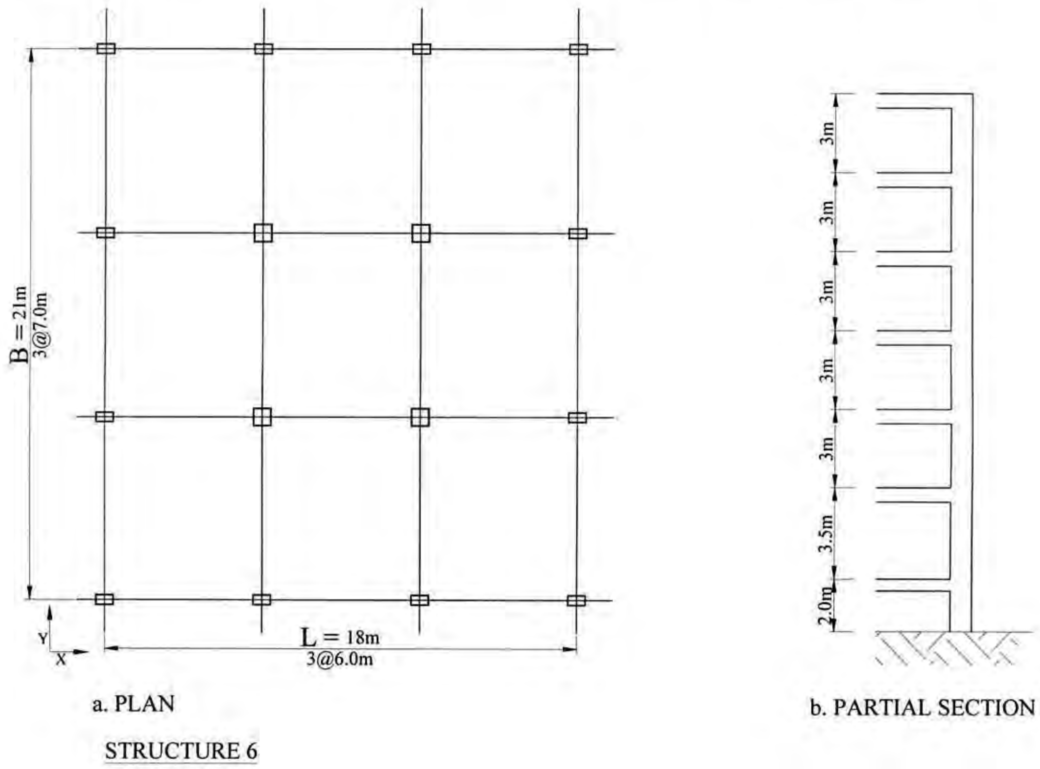


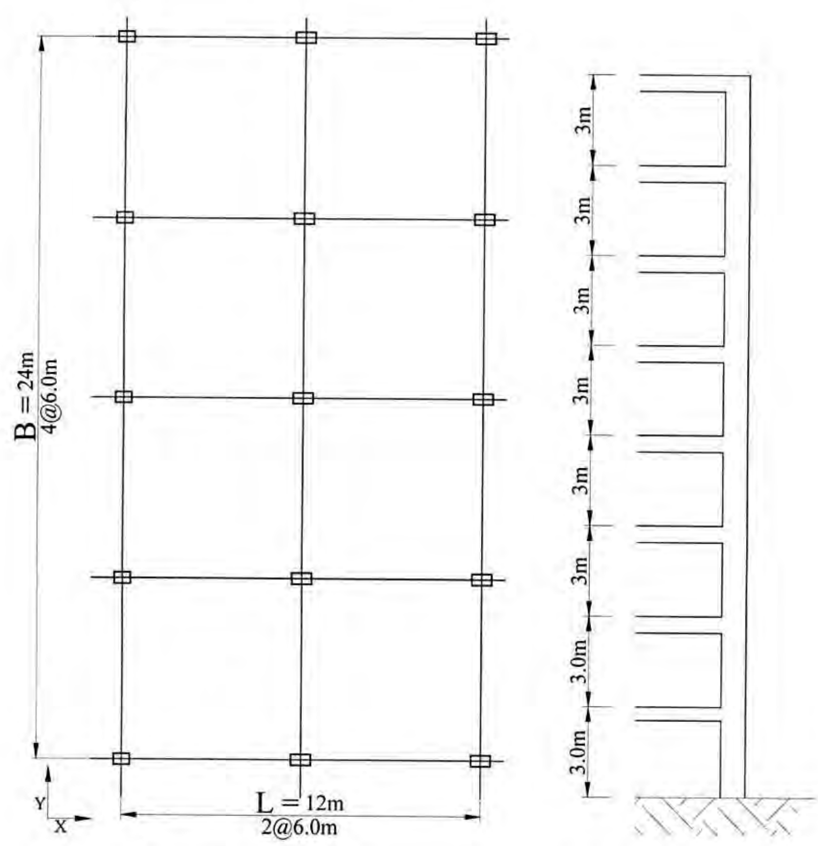
Fig. 5.6 Plan and partial section of structure 6

5.3.7 Salient Features of Structure 7

Structure 7 is a 7 story 2 x 4 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns of the structure are 0.7 x 0.45m ~ 0.6 x 0.35m and exterior columns are 0.60 x 0.45m ~ 0.50 x 0.35m with grade beam of dimension 0.325 x 0.45m and floor beams of dimension 0.25 x 0.50m. Reinforcement in the various floor levels of the interior columns varied from 2100 mm^2 (1%) to 8500 mm^2 (2.7%) and that of exterior columns from 1750mm^2 (1%) to 5300 mm^2 (2%). Other geometric parameters are as described in Table 5.7.

Table 5.7 Geometric parameters of structure 7

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 7	24000	20902	12000	24000	2 x 4	0.50	1.00	0.50



a. PLAN b. PARTIAL SECTION

STRUCTURE 7

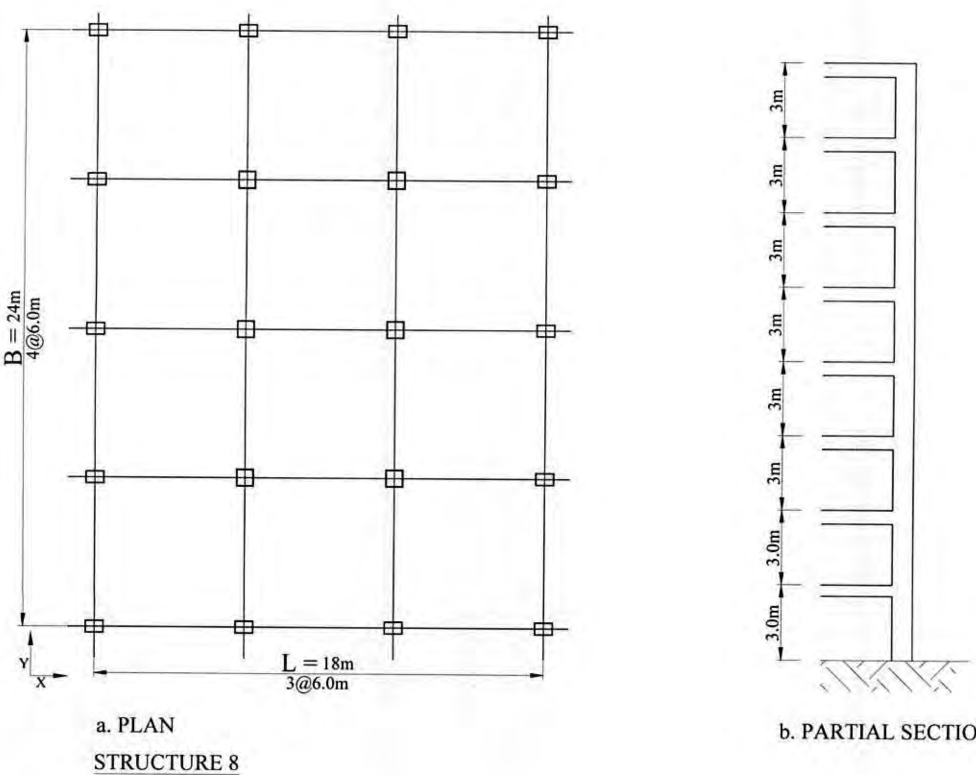
Fig. 5.7 Plan and partial section of structure 7

5.3.8 Salient Features of Structure 8

Structure 8 is a 7 story 3 x 4 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns of the structure are 0.45 x 0.45m and exterior columns are 0.50 x 0.35m with grade beam of dimension 0.325 x 0.50m and floor beams of dimension 0.25 x 0.50m. Reinforcement in the various levels of the interior columns varied from 2025 mm^2 (1%) to 8400 mm^2 (4.15%) and that of the exterior columns from 1750mm^2 (1%) to 5300 mm^2 (3.1%). Other geometric parameters are as described in Table 5.8.

Table 5.8 Geometric parameters of structure 8

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 8	24000	29419	18000	24000	3 x 4	0.75	1.00	0.75



a. PLAN
STRUCTURE 8

b. PARTIAL SECTION

Fig. 5.8 Plan and partial section of structure 8

5.3.9 Salient Features of Structure 9

Structure 9 is a 7 story 5 x 3 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns of the structure are 0.45 x 0.60m and exterior columns are 0.35 x 0.50m with grade beam of dimension 0.325x 0.45m and floor beams of dimension 0.25 x 0.50m. Reinforcement in the various floor levels of the interior columns varied from 2700 mm^2 (1%) to 9800 mm^2 (3.63%) and that of exterior columns from 1750mm^2 (1%) to 6400 mm^2 (3.65%). Other geometric parameters are as described in Table 5.9.

Table 5.9 Geometric parameters of structure 9

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 9	24000	44146	30000	18000	5 x 3	1.25	0.75	1.67

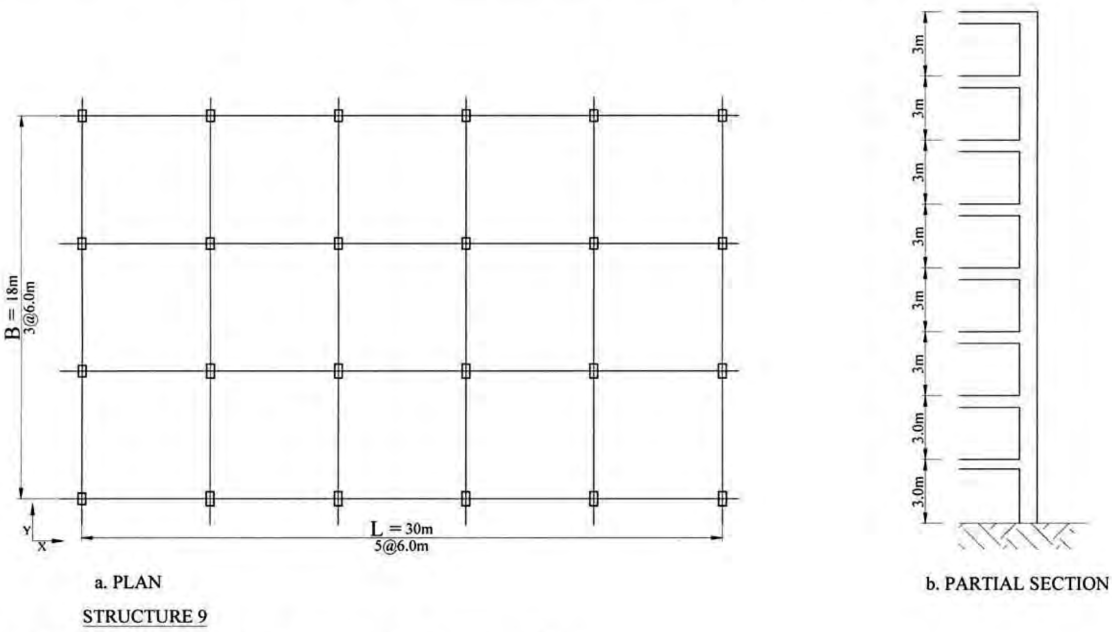


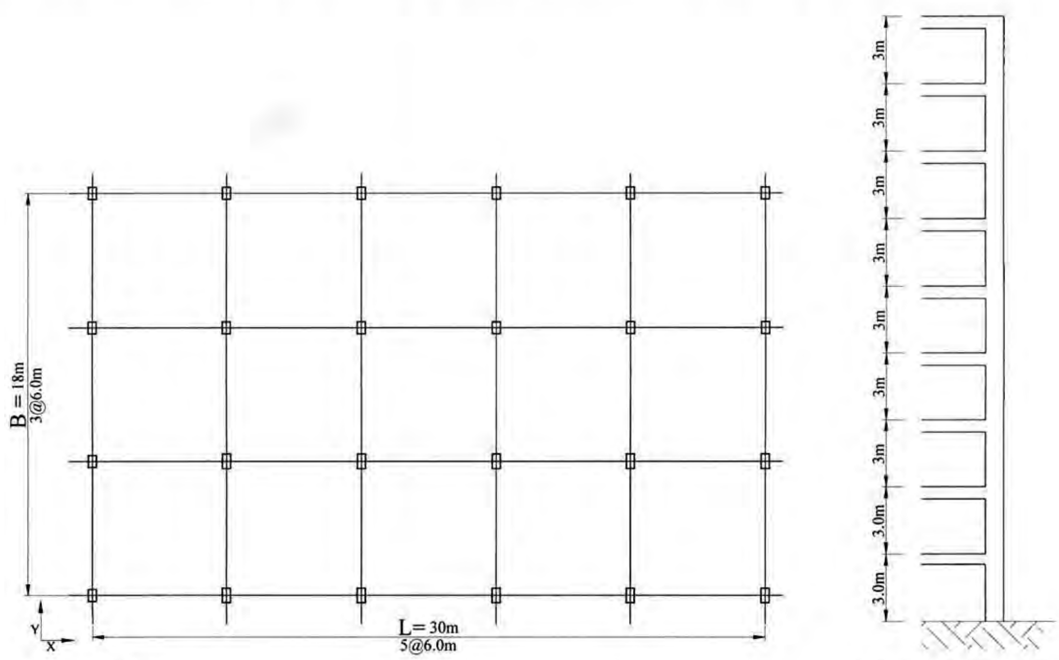
Fig. 5.9 Plan and partial section of structure 9

5.3.10 Salient Features of Structure 10

Structure 10 is an 8 story 5 x 3 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns of the structure are 0.4 x 0.8m and exterior columns are 0.30 x 0.60m with grade beam of dimension 0.325 x 0.450m and floor beams of dimension 0.25 x 0.50m. Reinforcement in the various floor levels of the interior columns varied from 3200 mm^2 (1%) to 10900 mm^2 (3.4%) and that of the exterior columns from 1800mm^2 (1%) to 7700 mm^2 (4.3%). Other geometric parameters are as described in Table 5.10.

Table 5.10 Geometric parameters of structure 10

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 10	27000	50382	30000	18000	5 x 3	1.11	0.67	1.67



a. PLAN

b. PARTIAL SECTION

STRUCTURE 10

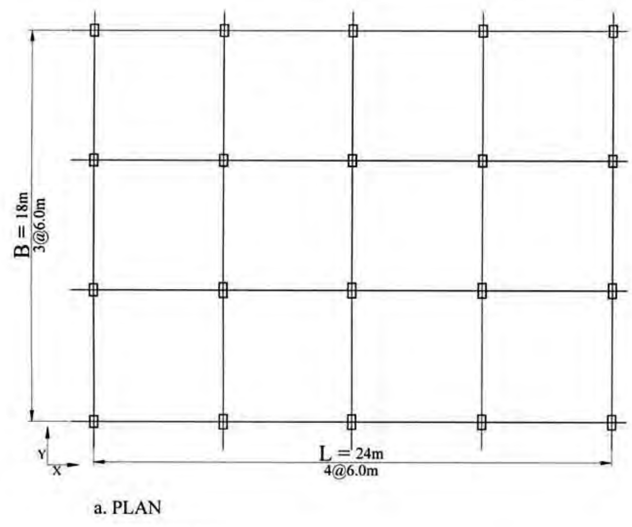
Fig. 5.10 Plan and partial section of structure 10

5.3.11 Salient Features of Structure 11

Structure 11 is a 10 story 4 x 3 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns of the structure are 0.45 x 0.75m ~ 0.45 x 0.65m and exterior columns are 0.40 x 0.60m ~ 0.30 x 0.50m with floor beam of dimension 0.25 x 0.60m. Reinforcement in the various floor levels of the interior columns varied from 2925 mm^2 (1%) to 16300 mm^2 (4.83%) and that of the exterior column from 1500mm^2 (1%) to 8900 mm^2 (3.70%). Other geometric parameters are as described in Table 5.11.

Table 5.11Geometric parameters of structure 11

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 11	34000	49862	24000	18000	4 x 3	0.71	0.53	1.33



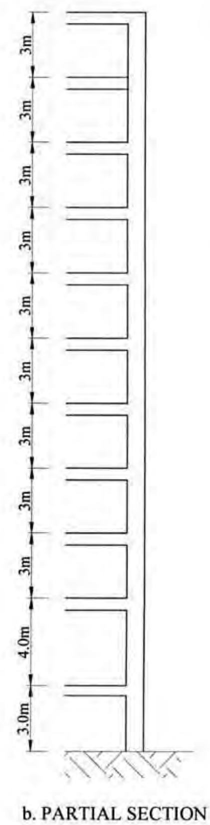


Fig. 5.11 Plan and partial section of structure 11

5.3.12 Salient Features of Structure 12

Structure 12 is a 9 story 4 x 3 bay residential building. It has been designed as frame structure with load and other parameters as specified in the Section 5.2. Interior columns of the structure are 0.50 x 0.75m ~ 0.40 x 0.65m and exterior columns are 0.35 x 0.50m ~ 0.30 x 0.45m with floor beam of dimension 0.25 x 0.60m. Reinforcement in the various levels of the interior columns varied from 2600 mm^2 (1%) to 12600 mm^2 (3.36%) and that of the exterior columns from 1350mm^2 (1%) to 7750 mm^2 (4.42%). Other geometric parameters are as described in Table 5.12.

Table 5.12 Geometric parameters of structure 12

	Roof height, mm	Seismic dead weight, kN	Geometric Parameter					
			L mm	B mm	Bay	L/H	B/H	L/B
Structure 12	31000	45157	24000	18000	4 x 3	0.77	0.58	1.33

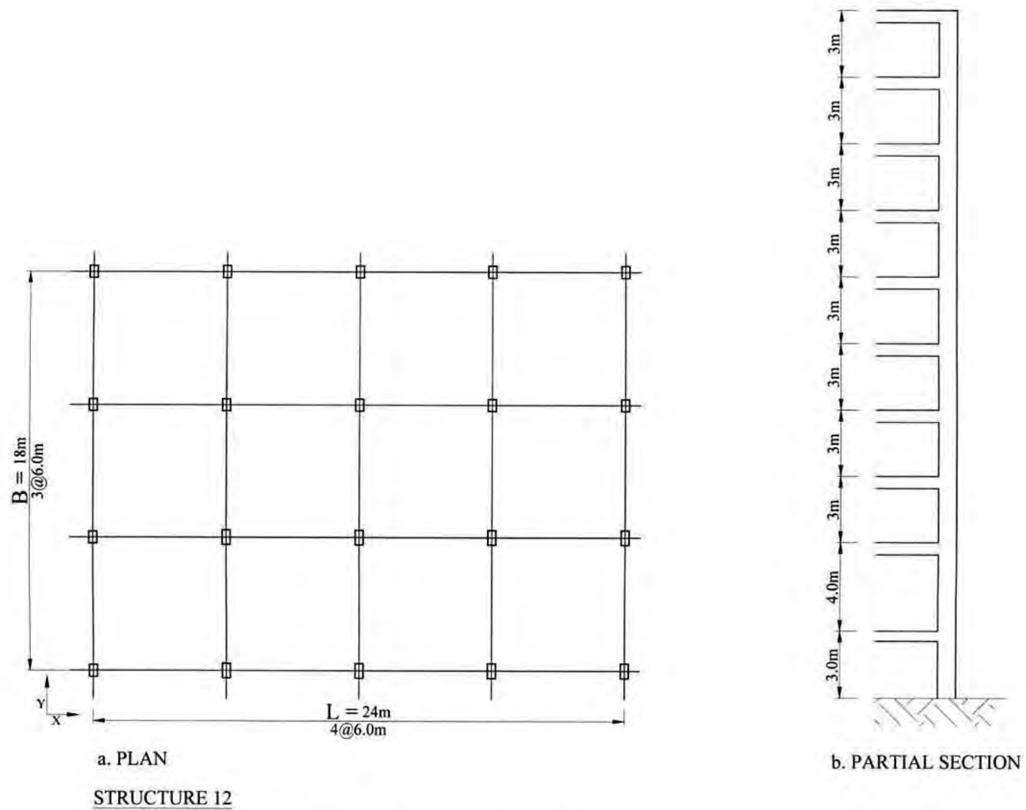


Fig. 5.12 Plan and partial section of structure 12

5.4 Performance Evaluation of the Structures

The structures described in Section 5.3 have been modeled with the parameters defined in Section 5.2 and Chapter 4. These structures are analyzed and designed using SAP 2000. After designing of the structures, hinges as defined in Chapter 4 have been assigned to the respective members and pushover analyses have been performed to develop capacity curves for each structure. The capacity curves have been transformed to capacity spectrums as per method described in Chapter 3. Site specific seismic demand for the structures has been established in Chapter 4. Accordingly performance points of the structures for the estimated seismic demand have been calculated. Resulting output for each structures are presented next. Hinges states near the performance point have been shown in color code. Different performance levels are determined as per Table 3.6 to Table 3.12 of Chapter 3. A general graphical representation of the various performance levels is given in Fig. 5.13.

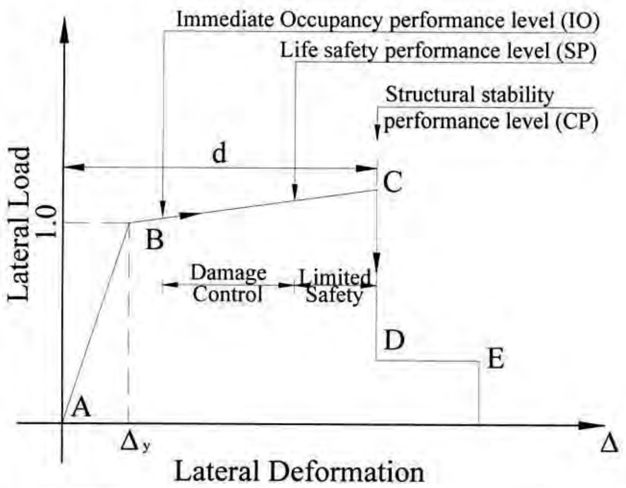


Fig. 5.13 Typical load-deformation acceptance criteria (FEMA-356, 2000)

To find the performance of the structure for the specific seismic demand the 5% elastic demand needs to be scaled down by some factors to recognize the increase of damping due hysteretic damping for inelastic deformation. The equivalent viscous damping, β_{eq} resulting from viscous damping and hysteretic damping may be calculated by formula:

$$\beta_{eq} = \beta_0 + 0.05 \quad \dots \dots \dots \qquad (5.1)$$

Where,

β_0 = hysteretic damping represented as equivalent viscous damping

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

The term β_0 can be calculate as (Chopra, A.K. 1995) :

$$\beta_{eq} = \frac{1}{4\pi}\frac{E_D}{E_{SO}} \quad \text{.........} \qquad (5.2)$$

Where,

E_D = energy dissipated by damping

E_{SO} = maximum strain energy

The physical significance of the terms E_D and E_{SO} is illustrated in Fig. 5.14.

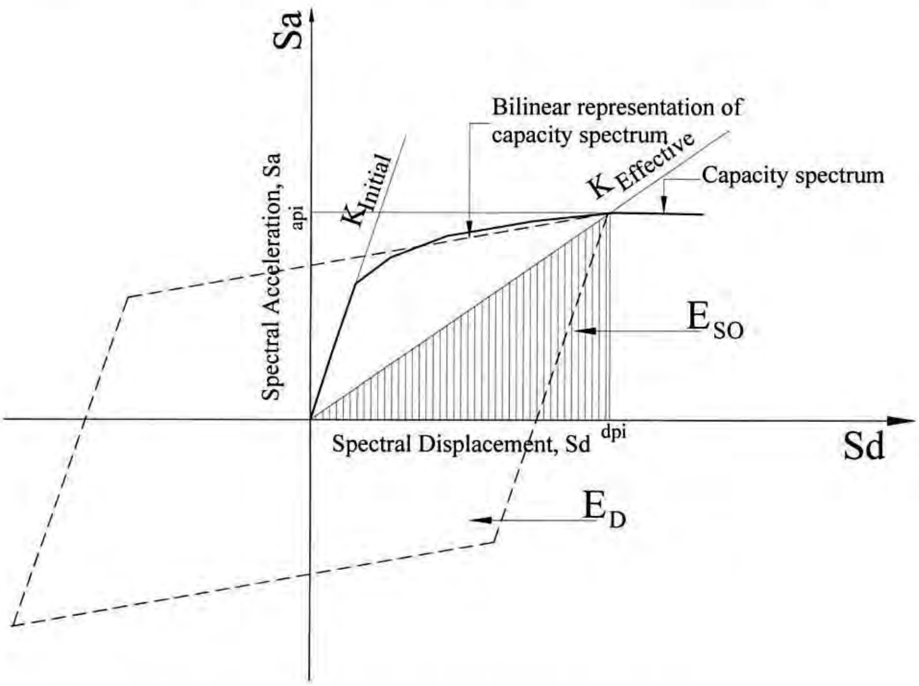


Fig. 5.14 Derivation of damping for spectrum reduction

The equation for the reduction factors SR_A and SR_V are given by (ATC 40, 1996):

$$SR_A \approx \frac{3.21 - 0.681\ln(\beta_{eff})}{2.12} \quad \text{.........} \qquad (5.3)$$

$$SR_V \approx \frac{3.21 - 0.41\ln(\beta_{eff})}{1.65} \quad \text{.........} \qquad (5.4)$$

Where,

SR_A = reduction factor for short-period system in the acceleration domain

SR_V = reduction factor for long-period system in the velocity domain

5.4.1 Capacity Curves and Capacity Spectrums of the Structures

a. Structure 1: This is a six-storied structure. Detailed configuration is given in Section 5.3.1 under this chapter. Well defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.15. Capacity of the structure in the Y-direction is more because structural capacity of the members in Y-direction is more than that in the X-direction.

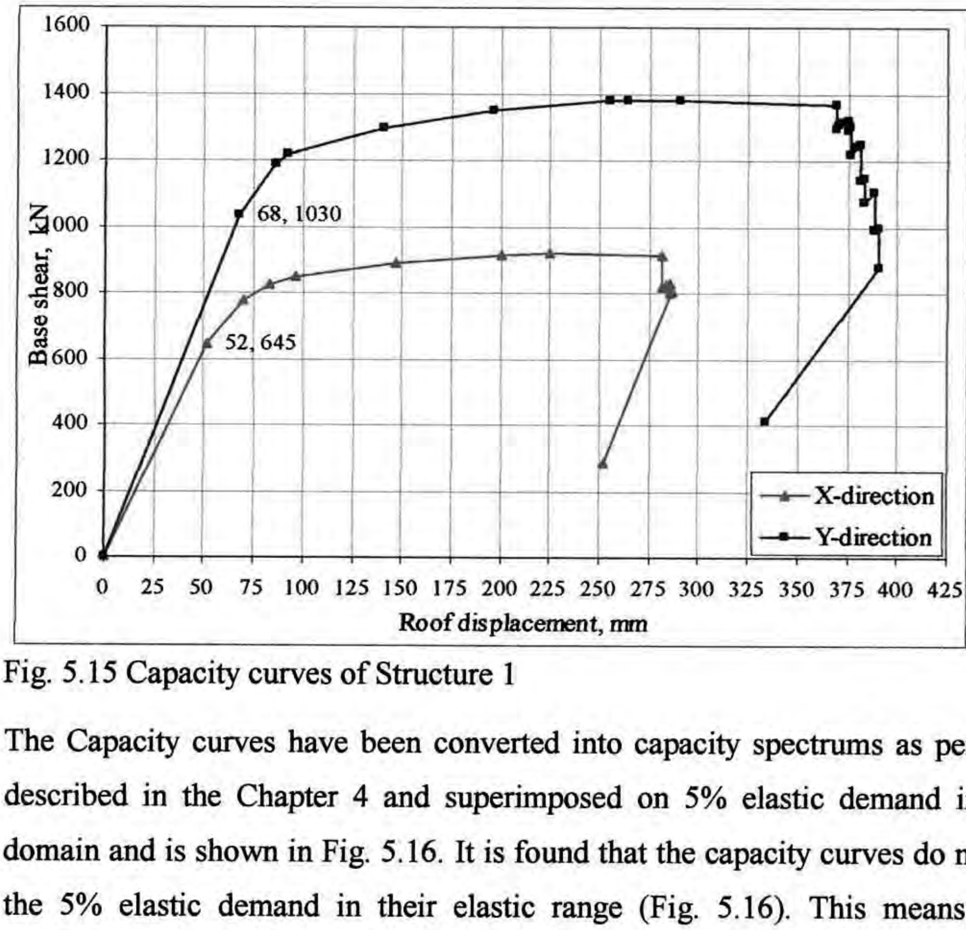


Fig. 5.15 Capacity curves of Structure 1

The Capacity curves have been converted into capacity spectrums as per procedure described in the Chapter 4 and superimposed on 5% elastic demand in the same domain and is shown in Fig. 5.16. It is found that the capacity curves do not intersect the 5% elastic demand in their elastic range (Fig. 5.16). This means that some inelastic deformation will be demanded to meet the specific seismic demand.

The reduction factors for reduced seismic demand for inelastic response have been calculated and shown in Table 5.13. Other terms in the Table 5.13 have been defined in Section 4.3 of Chapter 4. Reduced seismic demand have been constructed by multiplying the 5% damped elastic demand by the calculated reduction factors for Structure 1 and presented in Fig. 5.16 along with 5% elastic demand and capacity spectrums for comparison.

Table 5.13 Calculation of reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		Direction		
		X	Y	Unit
Effective Damping, β_{eff}	=	23.5	16.5	
Spectral Reduction Factor, SR_A	=	0.50	0.61	
Spectral Reduction Factor, SR_V	=	0.62	0.70	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.376	0.461	*g*
T_A	=	0.164	0.153	*sec*
T_S	=	0.818	0.763	*sec*

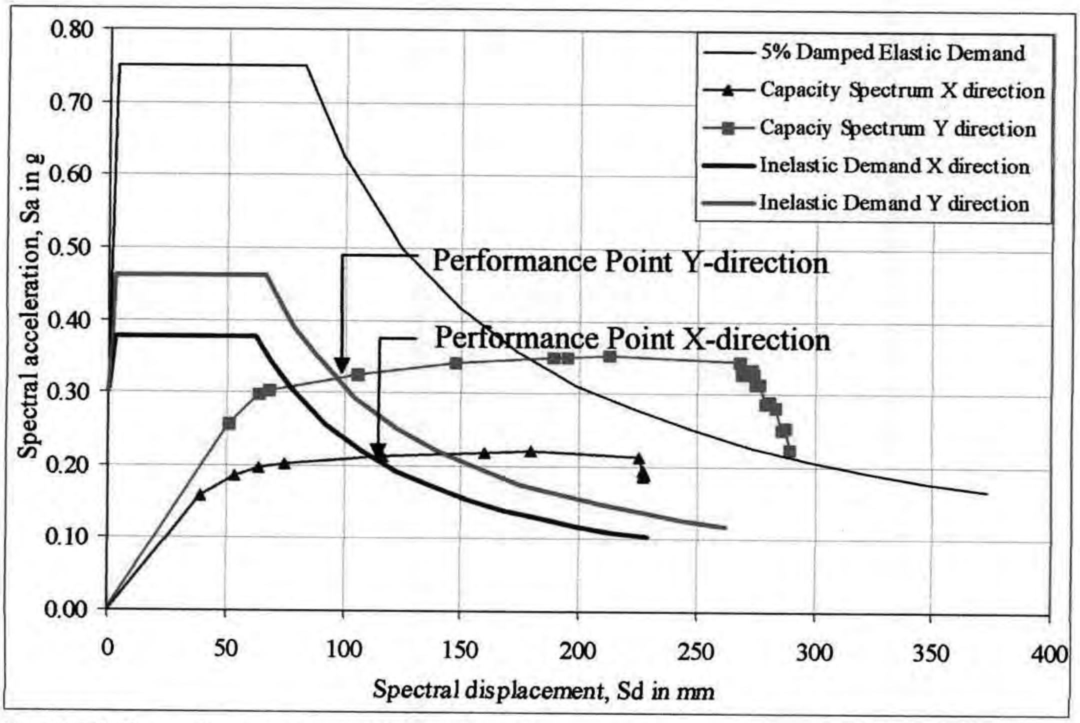


Fig. 5.16 Capacity spectrum of Structure 1

b. **Structure 2:** This is a six-storied structure symmetrical in both directions. Detailed configuration is given in Section 5.3.2 under this chapter. As the structure is symmetrical in both directions, the capacity curves have been found identical in two orthogonal directions (Fig. 5.17).

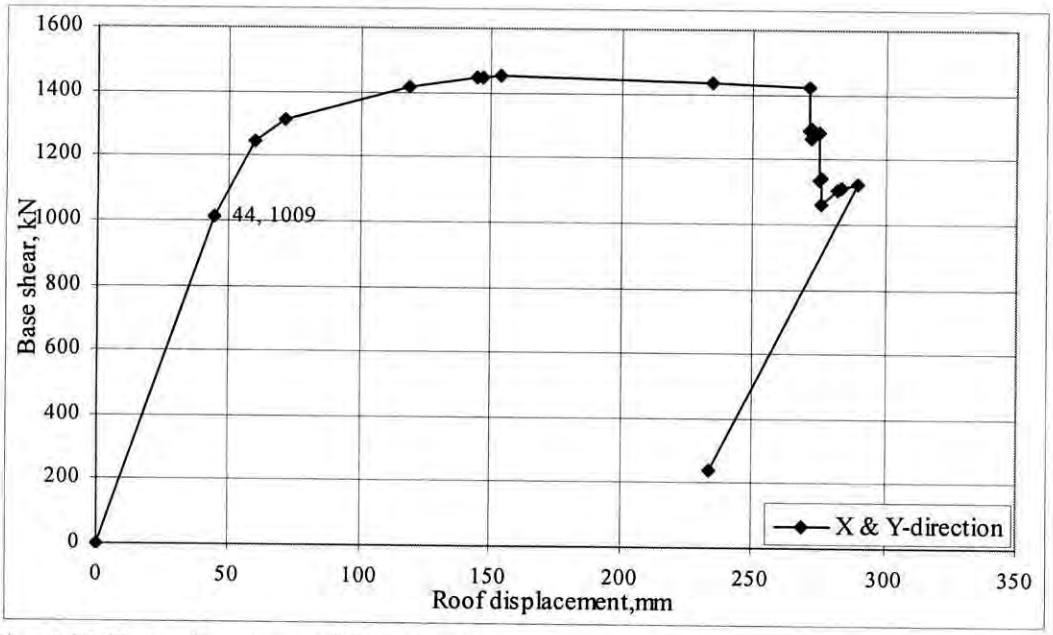


Fig. 5.17 Capacity curve of Structure 2

The capacity curve has been converted into capacity spectrum and superimposed on the 5% elastic demand in the same domain (Fig. 5.18). It has been found that the capacity curve does not intersect the 5% elastic demand in their elastic range. This means that some inelastic deformation will be demanded to meet the specific seismic demand for finding the performance point.

Thus to find the performance of the structure, reduced seismic demand has been constructed (Fig. 5.18) by multiplying 5% elastic demand by the structural reduction factors. Calculated structural reduction factors are shown in Table 5.14. The reduced seismic demand is presented in Fig. 5.18 along with 5% elastic demand and capacity spectrum for comparison and evaluation of performance point.

Table 5.14 Calculation of spectral reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		X and Y-direction	Unit
Effective Damping, β_{eff}	=	24.2	
Spectral Reduction Factor, SR_A	=	0.49	
Spectral Reduction Factor, SR_V	=	0.61	
Effective peak ground acceleration (EPA)	=	0.3	*g*
Average value of peak response	=	0.369	*g*
T_A	=	0.165	*sec*
T_S	=	0.824	*sec*

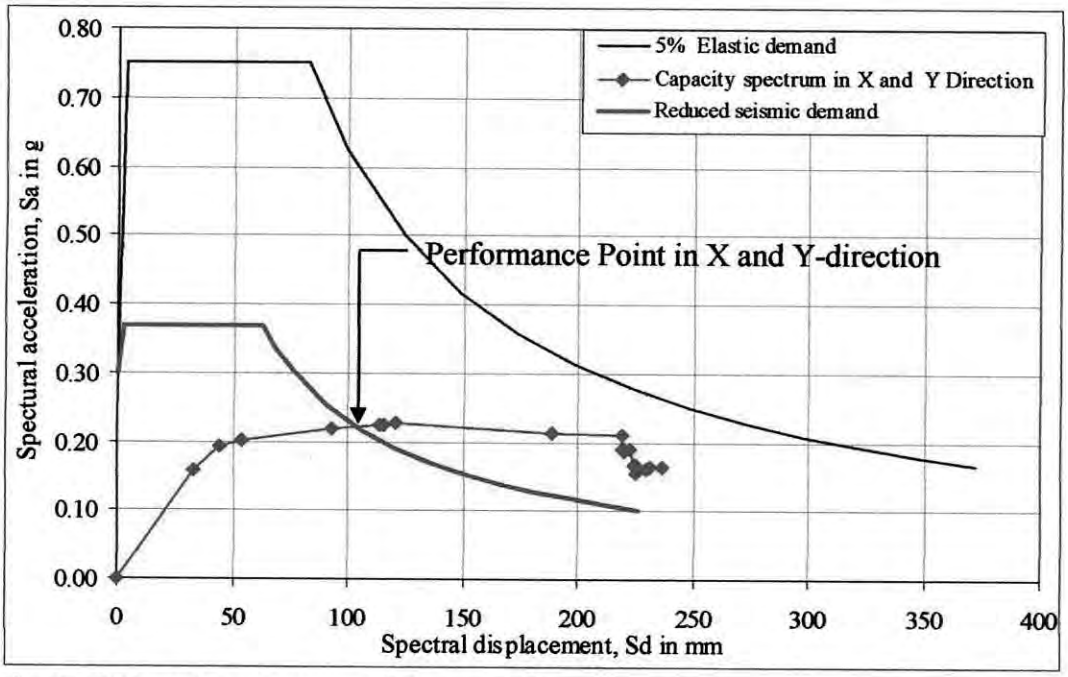


Fig. 5.18 Capacity spectrum of Structure 2

c. **Structure 3:** This is a seven-storied structure. Detailed configuration is given in Section 5.3.3 under this chapter. As the structure is symmetrical in both directions, the capacity curves have been found identical in two orthogonal directions (Fig. 5.19).

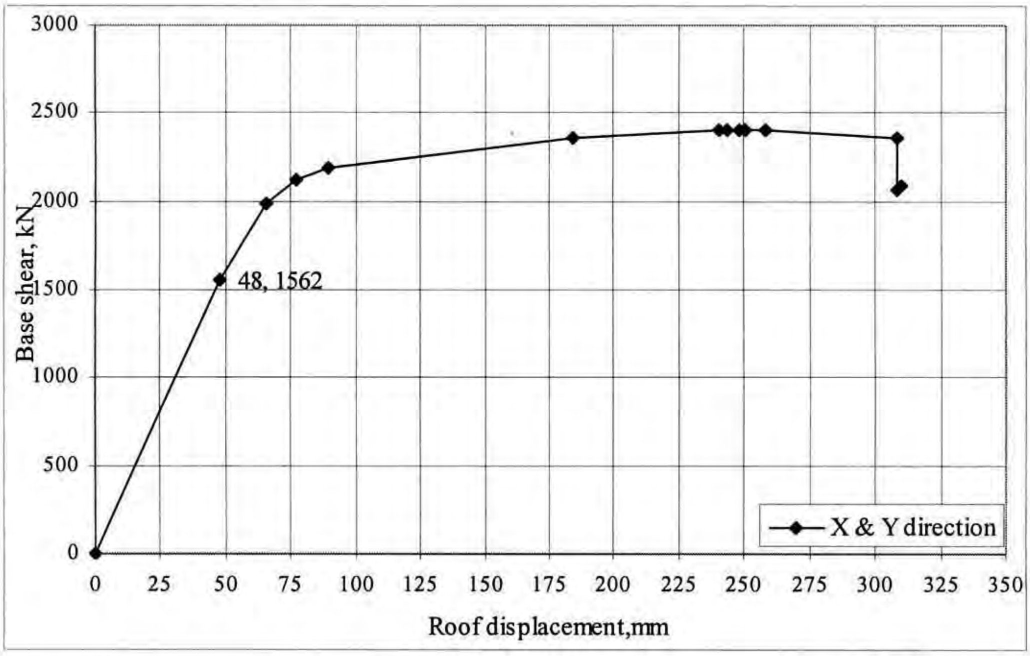


Fig. 5.19 Capacity curve of Structure 3

The capacity curve converted into capacity spectrum as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.20. It is found that the capacity curves do not intersect the 5% elastic demand in its elastic range. This means that some inelastic deformation will be demanded to meet the specific seismic demand for finding the performance point.

Reduced seismic demand has been constructed by multiplying 5% elastic demand by spectral reduction factor for inelastic deformation. Calculated structural reduction factors are shown in Table 5.15. Reduced seismic demand has been presented in Fig. 5.20 along with 5% elastic demand and capacity spectrum for comparison and evaluation of performance point.

Table 5.15 Calculation of Spectral reduction factors

Calculation of Spectral Reduction Factor		X and Y-Direction	Unit
Effective Damping, β_{eff}	=	22.5	
Spectral Reduction Factor, SR_A	=	0.52	
Spectral Reduction Factor, SR_V	=	0.63	
Effective peak ground acceleration (EPA)	=	0.3	*g*
Average value of peak response	=	0.387	*g*
T_A	=	0.162	*sec*
T_S	=	0.810	*sec*

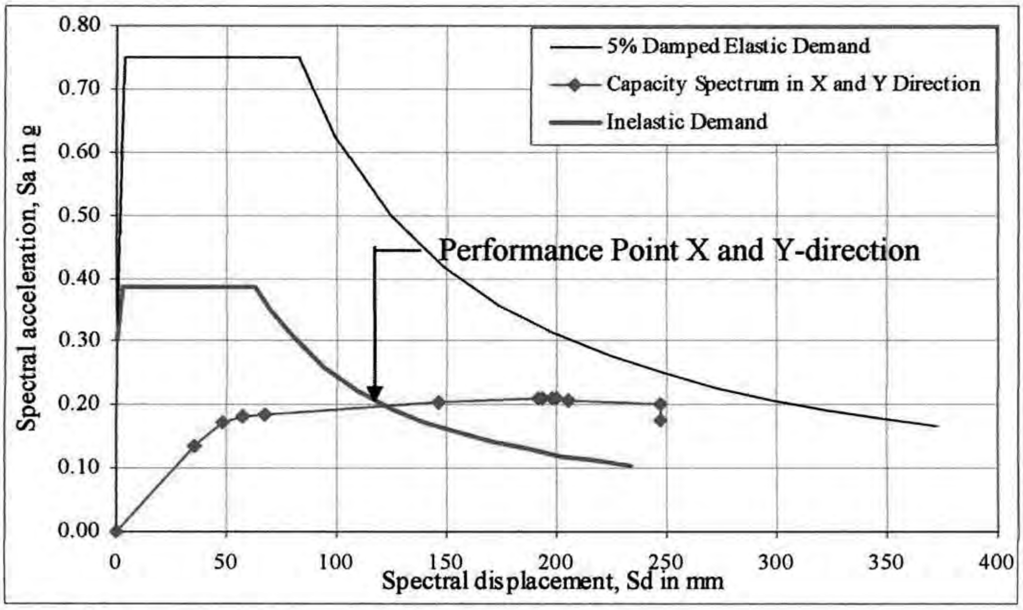


Fig. 5.20 Capacity Spectrum of Structure 3

d. Structure 4: This is an eight-storied structure. Detailed configuration is given in Section 5.3.4 under this chapter. Well defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.21. Capacity of the structure in the Y-direction is more because structural capacity of the members in Y-direction is more than that in the X-direction.

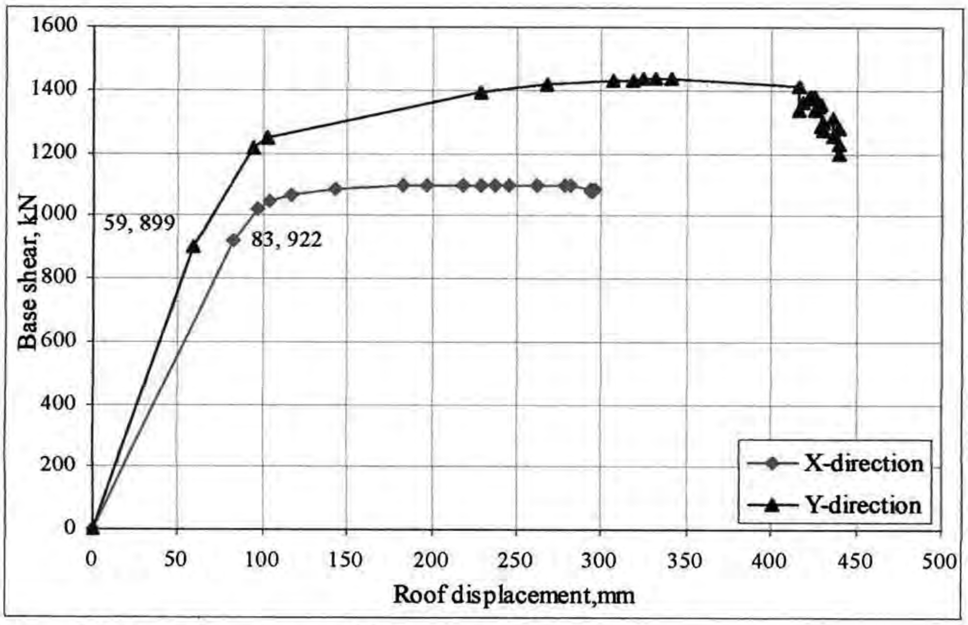


Fig. 5.21 Capacity curves of Structure 4

The capacity curves converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.22. It is found that the capacity curves do not intersect the 5% elastic demand spectrum in their elastic range. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus, to find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors. The reduction factors for reduced seismic demand have been calculated and shown in Table 5.16. Reduced seismic demand for Structure 4 has been presented in Fig. 5.22 along with 5% elastic demand and capacity spectrums for comparison and evaluation of performance point.

Table 5.16 Calculation of spectral reduction factors for reduced seismic demand

		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	23.3	18.1	
Spectral Reduction Factor, SR_A	=	0.50	0.59	
Spectral Reduction Factor, SR_V	=	0.62	0.68	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.378	0.439	*g*
T_A	=	0.163	0.155	*sec*
T_S	=	0.817	0.775	*sec*

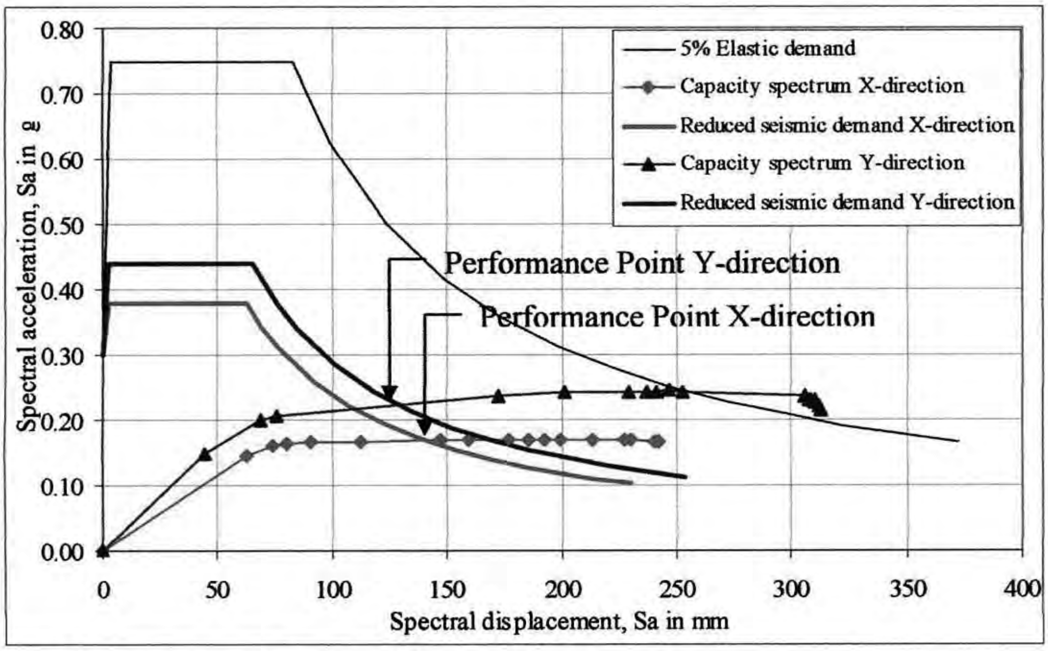


Fig. 5.22 Capacity spectrums of Structure 4

e. **Structure 5:** This is a nine-storied structure. Structural configuration is given in Section 5.3.5 under this chapter. Well-defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.23. Capacity of the structure in the Y-direction is more because structural capacity of the members in the Y-direction is more than that in the X-direction.

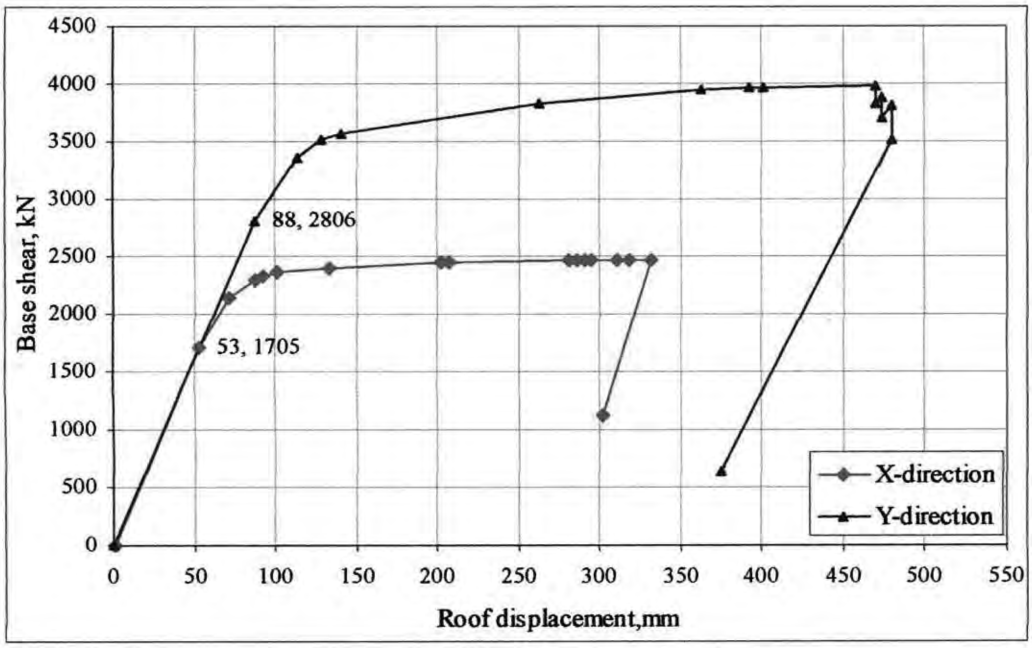


Fig. 5.23 Capacity curves of Structure 5

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.24. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus, to find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors to accommodate the inelastic deformation. The reduction factors for reduced seismic demand calculated and shown in Table 5.17. Reduced seismic demand presented in Fig. 5.24 along 5% elastic demand and the capacity spectrums for comparison and evaluation of performance point.

Table 5.17 Calculation of reduction factors for reduced seismic demand

		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	27	16.3	
Spectral Reduction Factor, SR_A	=	0.46	0.62	
Spectral Reduction Factor, SR_V	=	0.58	0.71	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.343	0.464	*g*
T_A	=	0.170	0.152	*sec*
T_S	=	0.848	0.761	*sec*

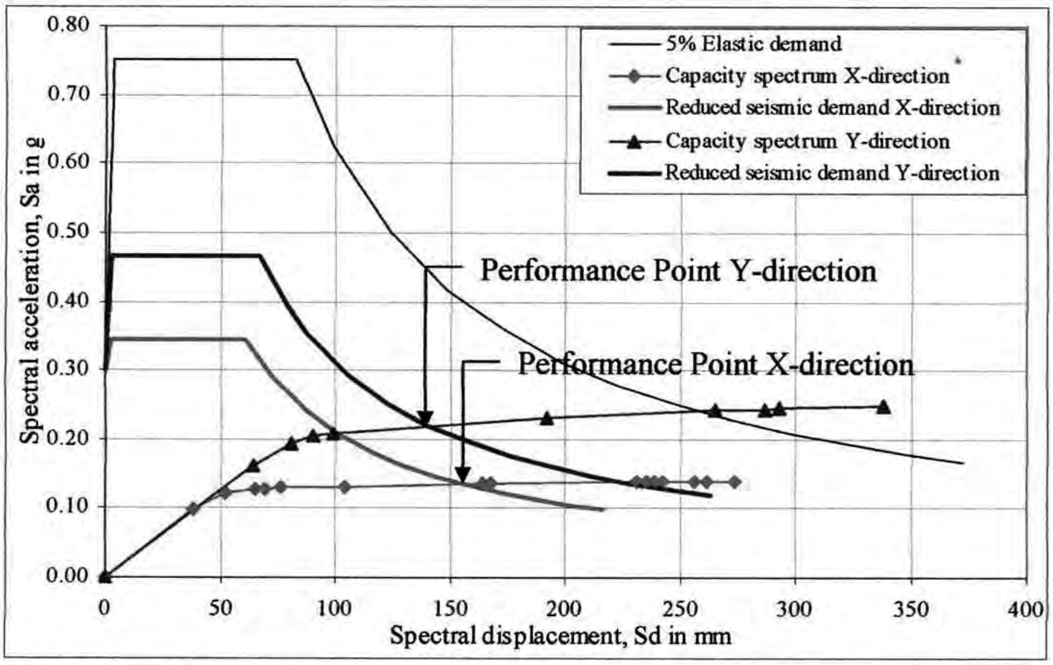


Fig. 5.24 Capacity spectrums of Structure 5

f. **Structure 6**: This is a six-storied structure. Structural configuration is given in Section 5.3.6 under this chapter. Well defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.25. Capacity of the structure in the Y-direction is more because structural capacity of the members in Y-direction is more than that in the X-direction.

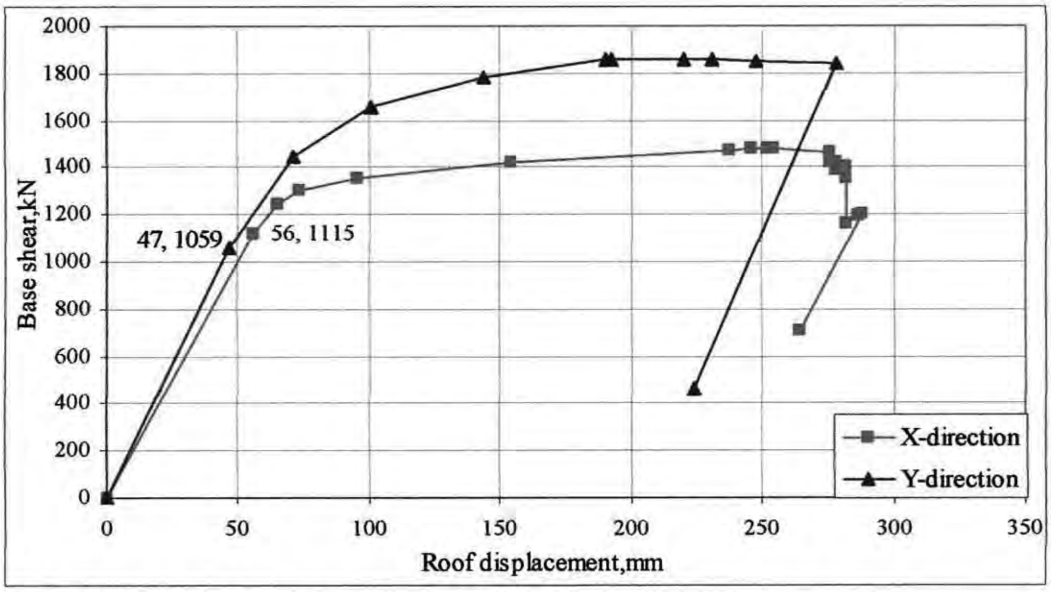


Fig. 5.25 Capacity curves of the Structure 6

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.26. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus to find the performance of the structure for the specific seismic demand the 5% elastic demand needs to be scaled down by some factors. The reduction factors for reduced seismic demand have been calculated and shown in Table 5.18. Reduced seismic demand for Structure 6 is presented in Fig. 5.26 along with 5% elastic demand and the capacity spectrums for comparison and evaluation of performance point.

Table 5.18 Calculation of reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	17.8	14.2	
Spectral Reduction Factor, SR_A	=	0.59	0.66	
Spectral Reduction Factor, SR_V	=	0.68	0.74	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.443	0.497	*g*
T_A	=	0.155	0.149	*sec*
T_S	=	0.773	0.745	*sec*

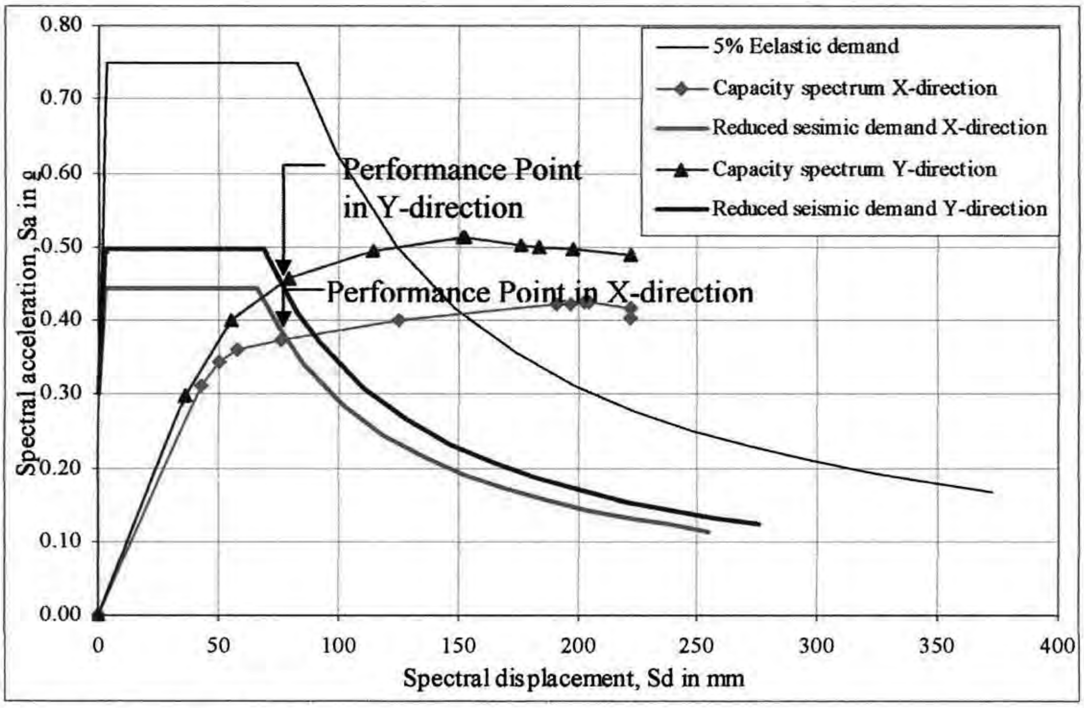


Fig. 5.26 Capacity spectrums of Structure 6

g. Structure 7: This is a seven-storied structure. Detailed configuration is given in Section 5.3.7 under this chapter. Well defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.27. Capacity of the structure in the X-direction is more because structural capacity of the members in X-direction is more than that in the Y-direction.

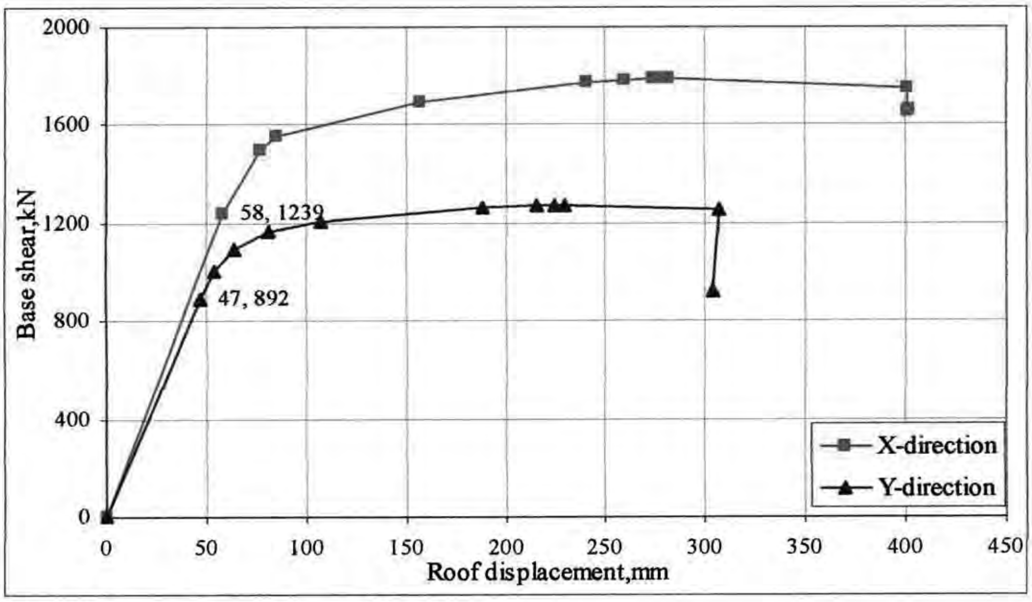


Fig. 5.27 Capacity curves of Structure 7

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.28. It is found that when superimposed, the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus, to find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors. The reduction factors for reduced seismic demand have been calculated and shown in Table 5.19. Reduced seismic demand for Structure 7 is presented in Fig. 5.28 along with 5% elastic demand and capacity spectrums for comparison and evaluation of performance point.

Table 5.19 Calculation of reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	20.5	25.1	
Spectral Reduction Factor, SR_A	=	0.55	0.48	
Spectral Reduction Factor, SR_V	=	0.65	0.60	
Effective peak ground acceleration (EPA)	=	0.3	0.3	g
Average value of peak response	=	0.409	0.360	g
T_A	=	0.159	0.166	*sec*
T_S	=	0.794	0.831	*sec*

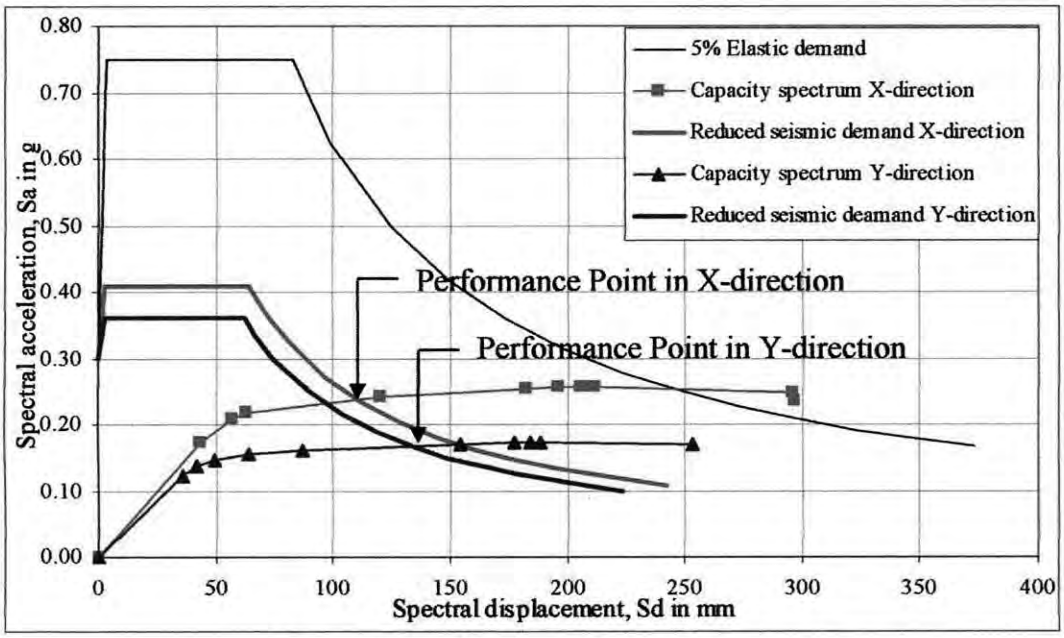


Fig. 5.28 Capacity spectrums of Structure 7

h. Structure 8: This is a seven-storied structure. Detailed configuration is given in Section 5.3.7 under this chapter. Well defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.29. Capacity of the structure in the X-direction is more because structural capacity of the members in X-direction is more than that in the Y-direction.

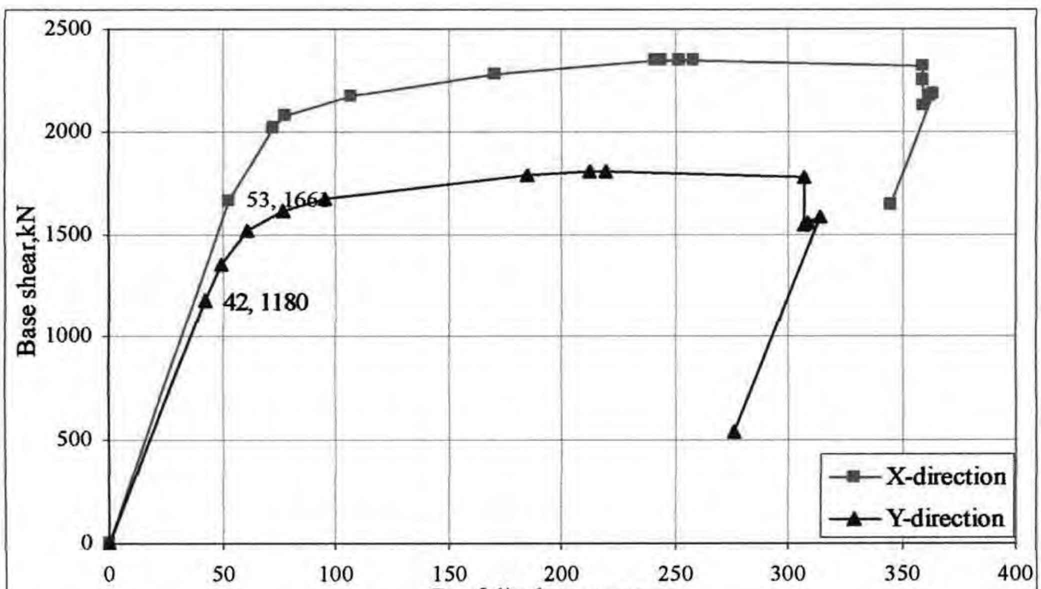


Fig. 5.29 Capacity curves of Structure 8

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in

Fig. 5.30. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus, to find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors to account the inelastic deformation. The reduction factors for reduced seismic demand have been calculated and shown in Table 5.20. Reduced seismic demands have been constructed by multiplying the 5% elastic demand by the calculated reduction factors for Structure 8 and presented in Fig. 5.30 along with 5% elastic demand and capacity spectrums for comparison and evaluation of performance point.

Table 5.20 Calculation of reduction factors for reduced seismic demand

		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	22.1	25	
Spectral Reduction Factor, SR_A	=	0.52	0.48	
Spectral Reduction Factor, SR_V	=	0.63	0.60	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.391	0.361	*g*
T_A	=	0.161	0.166	*sec*
T_S	=	0.807	0.831	*sec*

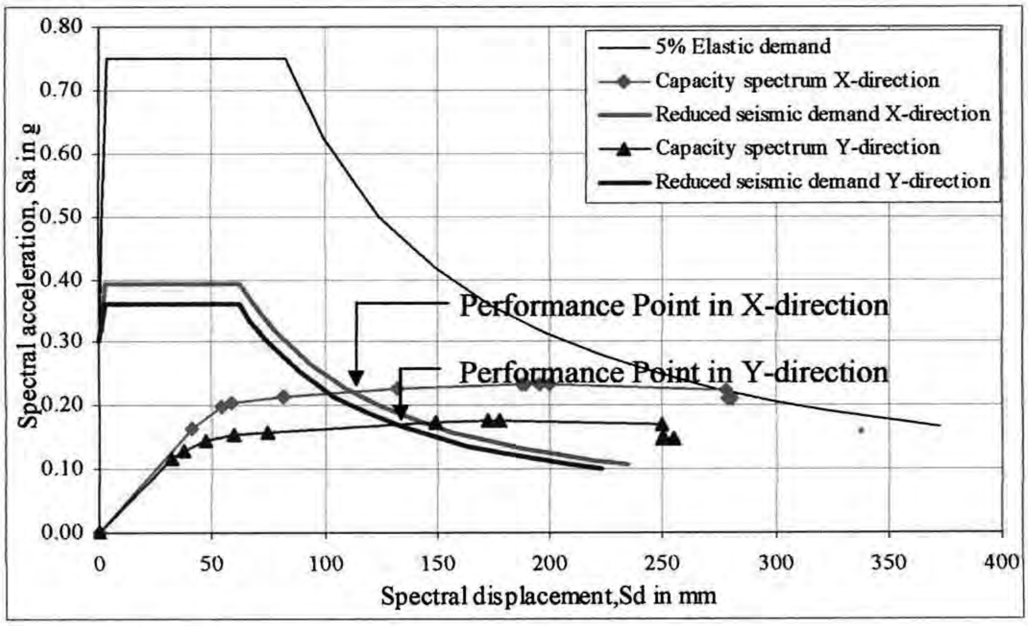


Fig. 5.30 Capacity spectrums of Structure 8

i. Structure 9: This is a seven-storied structure. Detailed configuration is given in Section 5.3.9 under this chapter. Well-defined capacity curves have been found in two orthogonal directions, which are shown in Fig. 5.31. Capacity of the structure in the Y-direction is more because structural capacity of the members in Y-direction is more than that in the X-direction.

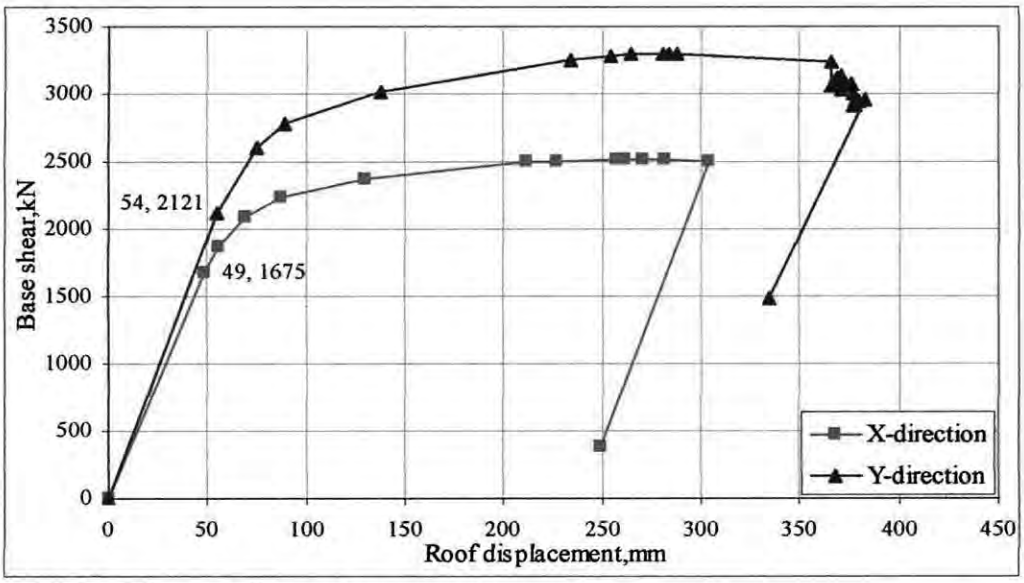


Fig. 5.31 Capacity curve of structure 9

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.32. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus, to find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors to account the gain in damping for inelastic deformation. The reduction factors have been calculated and shown in Table 5.21. Reduced seismic demands been constructed by multiplying the 5% elastic demand with the calculated reduction factors for Structure 9 and presented in Fig. 5.32 along with 5% elastic demand and capacity spectrums for comparison and evaluation of performance point.

Table 5.21 Calculation of reduction factors for reduced seismic demand

		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	24.1	20.8	
Spectral Reduction Factor, SR_A	=	0.49	0.54	
Spectral Reduction Factor, SR_V	=	0.61	0.65	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.370	0.406	*g*
T_A	=	0.165	0.159	*sec*
T_S	=	0.823	0.796	*sec*

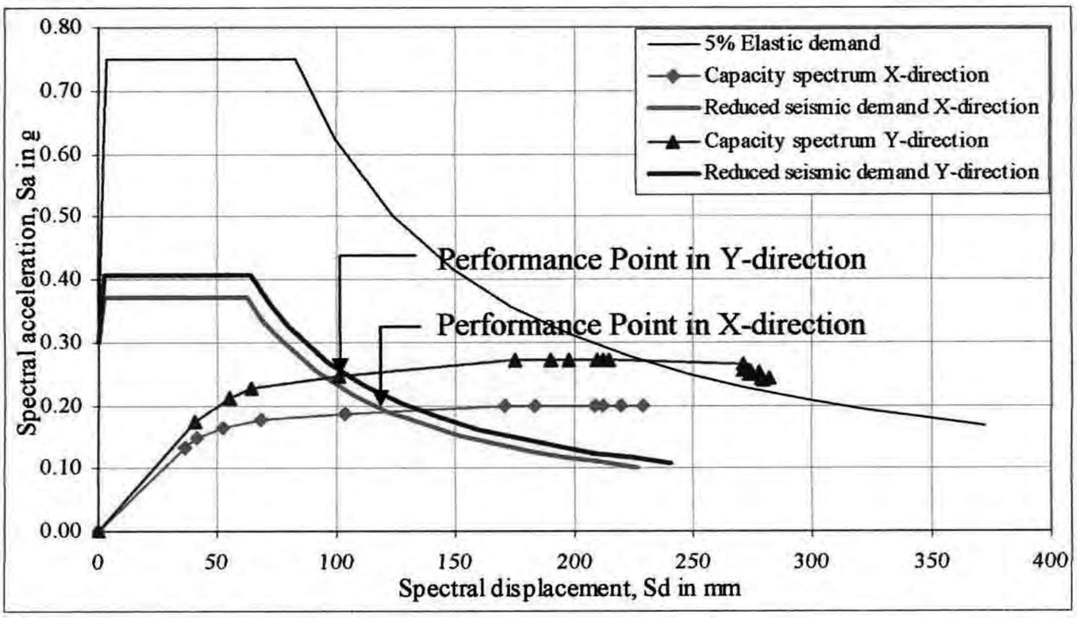


Fig. 5.32 Capacity spectrums of Structure 9

j. **Structure 10:** This is an eight-storied building. Detailed configuration is given in Section 5.3.10 under this chapter. Well-defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.33. Capacity of the structure in the Y-direction is more because structural capacity of the structural members in Y-direction is more than that in the X-direction.

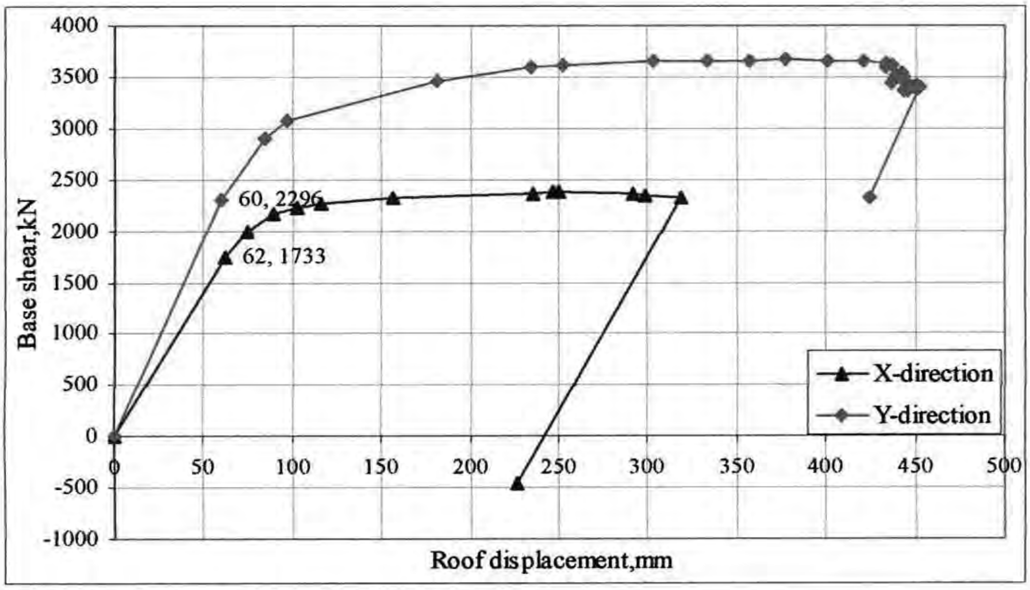


Fig. 5.33 Capacity curves of Structure 10

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on 5% elastic demand and is shown in Fig. 5.34. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

Thus, to find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors to account the gain in damping due to inelastic deformation. The reduction factors have been calculated and shown in Table 5.22. Reduced seismic demands have been constructed by multiplying the 5% elastic demand with the calculated reduction factors for Structure 10 and presented in Fig. 5.34 along with 5% elastic demand and the capacity spectrums for comparison and evaluation of performance point.

Table 5.22 Calculation of reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	25.0	18.7	
Spectral Reduction Factor, SR_A	=	0.48	0.57	
Spectral Reduction Factor, SR_V	=	0.60	0.67	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.361	0.431	*g*
T_A	=	0.166	0.156	*sec*
T_S	=	0.831	0.780	*sec*

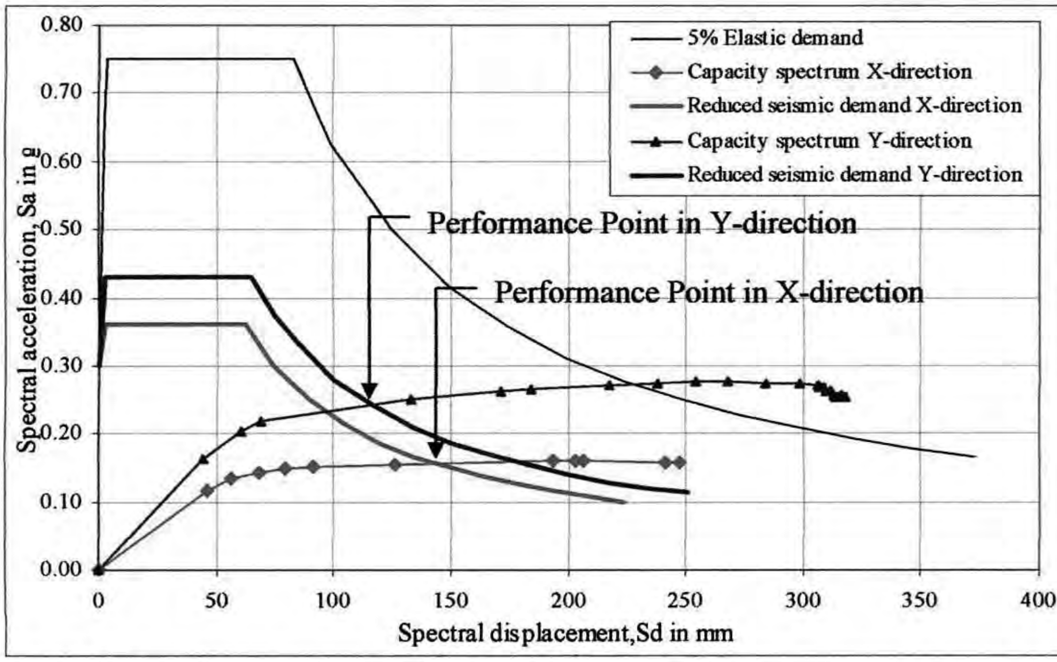


Fig. 5.34 Capacity spectrums of Structure 10

k. Structure 11: This is a ten-storied structure. Detailed configuration of this structure is given in Section 5.3.11 under this chapter. Well-defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.35. Capacity of the structure in the Y-direction is more because structural capacities of the members in Y-direction are more than that of the X-direction.

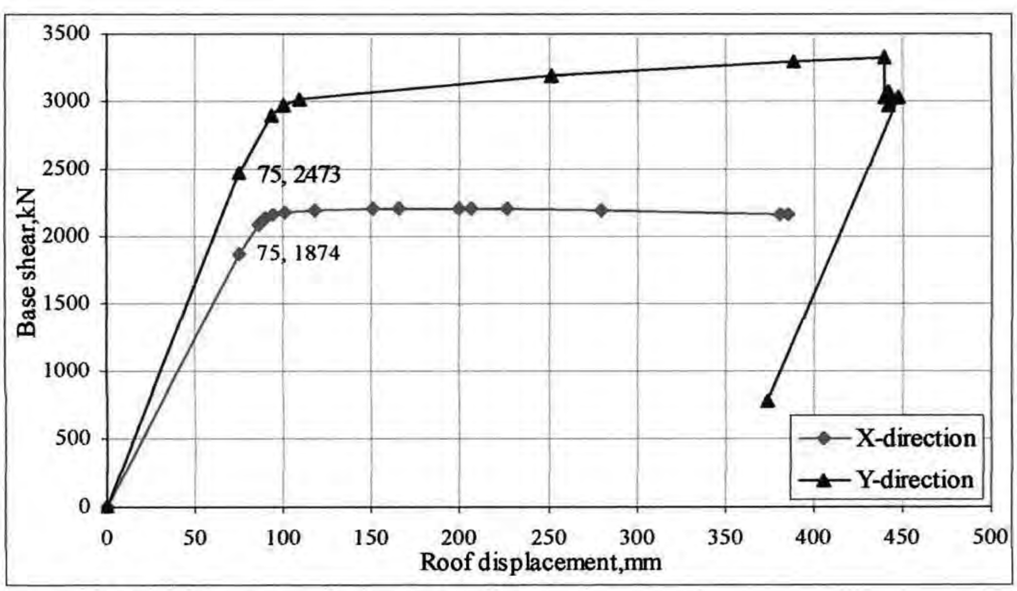


Fig. 5.35 Capacity curves of Structure 11

The capacity curves have been converted in to capacity spectrums as per procedure described in the Chapter 3 and superimposed on the 5% elastic demand and is shown in Fig. 5.36. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

To find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors to account the gain in damping due to inelastic deformation. The reduction factors have been calculated and shown in Table 5.23. Reduced seismic demands have been constructed by multiplying the 5% elastic demand with the calculated reduction factors for Structure 11 and presented in Fig. 5.36 along with 5% elastic demand and capacity spectrums for comparison and evaluation of performance point.

Table 5.23 Calculation of reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	26.5	18.1	
Spectral Reduction Factor, SR_A	=	0.46	0.59	
Spectral Reduction Factor, SR_V	=	0.59	0.68	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.347	0.439	*g*
T_A	=	0.169	0.155	*sec*
T_S	=	0.843	0.775	*sec*

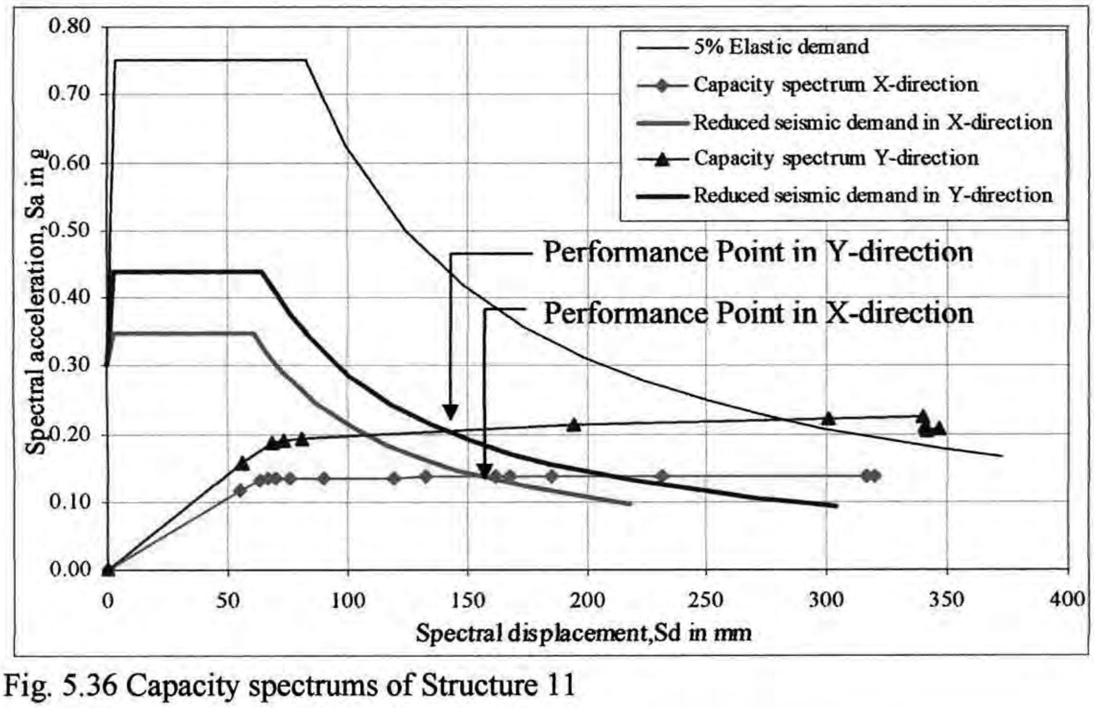


Fig. 5.36 Capacity spectrums of Structure 11

l. **Structure 12:** This is a nine-storied structure. Detailed configuration is given in Section 5.3.12 under this chapter. Well-defined capacity curves have been found in two orthogonal directions which are shown in Fig. 5.37. Capacity of the structure in the Y-direction is more because structural capacities of the members in Y-direction are more than that of the X-direction.

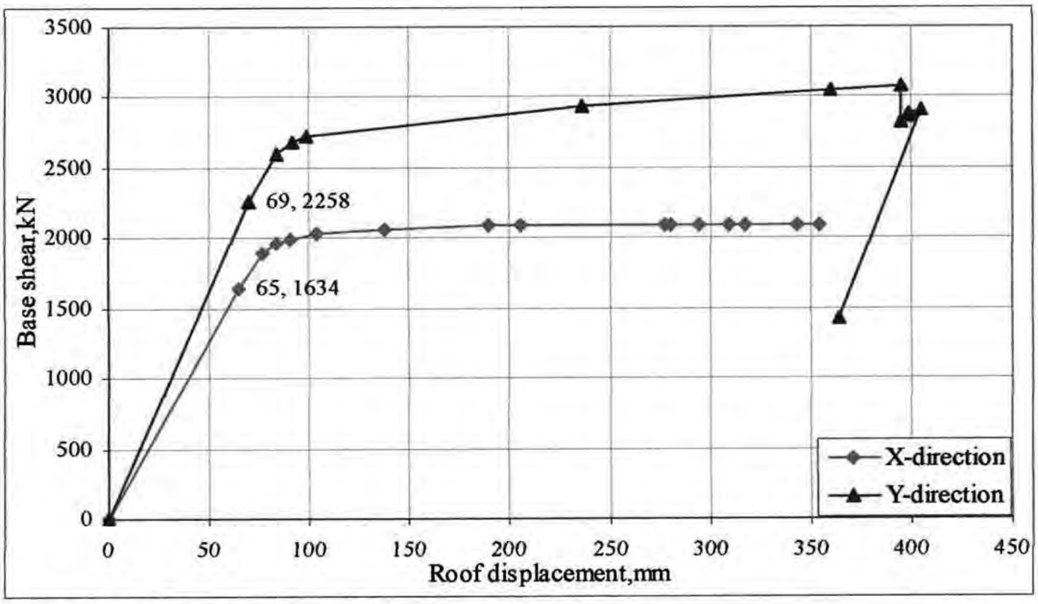


Fig. 5.37 Capacity curves of Structure 12

The capacity curves have been converted into capacity spectrums as per procedure described in the Chapter 5 and superimposed on 5% elastic demand which is shown in Fig. 5.38. It is found that the capacity curves do not intersect in their elastic range to the 5% elastic demand. This means that some inelastic deformation will be demanded to meet the specific seismic demand.

To find the performance of the structure for the specific seismic demand, the 5% elastic demand needs to be scaled down by some factors to account the reduction of seismic demand for inelastic deformation. The reduction factors have been calculated and shown in Table 5.24. Reduced seismic demands have been constructed by multiplying the 5% elastic demand with the calculated reduction factors for Structure 12 and presented in Fig. 5.38 along with 5% elastic demand and capacity spectrums for comparison and evaluation of performance point.

Table 5.24 Calculation of reduction factors for reduced seismic demand

Calculation of Spectral Reduction Factor		X-Direction	Y-Direction	Unit
Effective Damping, β_{eff}	=	26.3	18.8	
Spectral Reduction Factor, SR_A	=	0.47	0.57	
Spectral Reduction Factor, SR_V	=	0.59	0.67	
Effective peak ground acceleration (EPA)	=	0.3	0.3	*g*
Average value of peak response	=	0.349	0.430	*g*
T_A	=	0.168	0.156	*sec*
T_S	=	0.842	0.781	*sec*

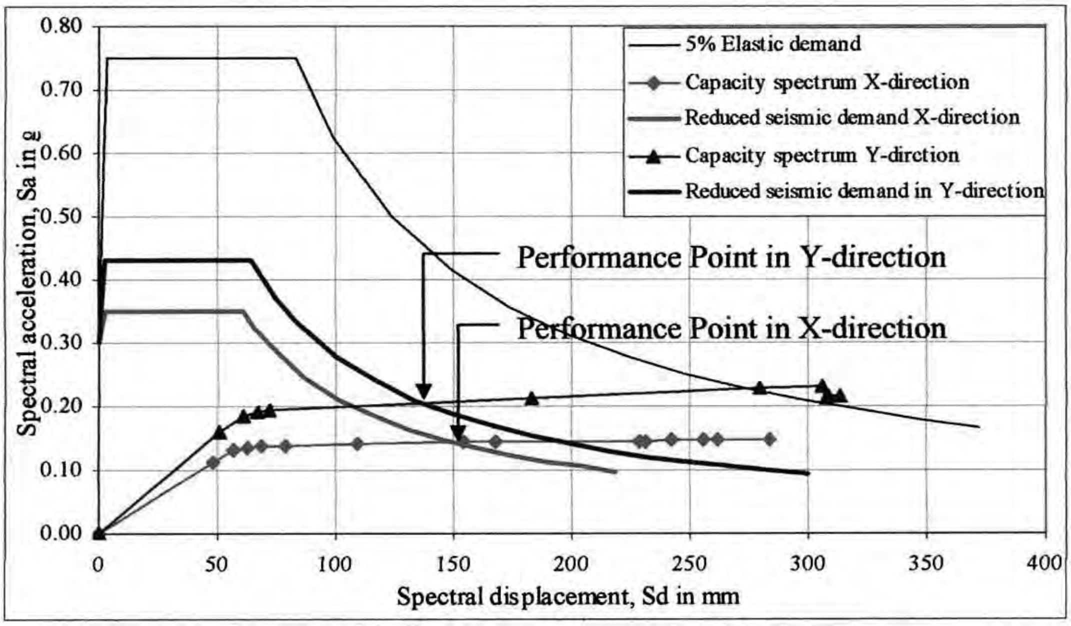


Fig. 5.38 Capacity spectrums of Structure 12

5.4.2 Comparison of the Structural Parameters

a. **Structure 1:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.25 along with the corresponding quantities obtained from elastic analysis. Also shown the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node that defined in Section 5.2. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.25 Comparison of time period and base shear quantities for Structure 1

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
20000	15005	0.69	555	0.037	0.96	645	52.0	0.86	0.87	1030	67.8	0.54

b. **Structure 2:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.26 along with the corresponding quantities obtained from elastic analysis. Table 5.26 also shows the corresponding roof deflections Δ in both directions at the left topmost node that defined in Section 5.2. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.26 Comparison of time period and base shear quantities for Structure 2

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities			
					X and Y-Directions			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	$V_{x/y}$ *kN*	Δ *mm*	$V/V_{x/y}$
20000	17776	0.69	427	0.024	0.90	1009	44.2	0.42

c. **Structure 3:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.27 along with the corresponding quantities obtained from elastic analysis. Table 5.27 also shows the corresponding roof deflection Δ at the left topmost node that defined in Section 5.2. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.27 Comparison of time period and base shear quantities for Structure 3

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities			
					X and Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_{xy} *kN*	Δ_{xy} *mm*	V/V_{xy}
23000	35477	0.77	794	0.022	0.99	1562	47.7	0.51

d. **Structure 4:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.28 along with the corresponding quantities obtained from elastic analysis. Table 5.28 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost. Structural periods that have been calculated in the elastic analysis are the first mode period in that direction.

Table 5.28 Comparison of time period and base shear quantities for Structure 4

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
27000	25877	0.87	535	0.021	1.27	922	82.9	0.58	1.07	899	59.1	0.60

e. **Structure 5:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.29 along with the corresponding quantities obtained from elastic analysis. Table 5.29 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.29 Comparison of time period and base shear quantities for Structure 5

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
30000	46729	0.94	916	0.020	1.21	1705	52.7	0.54	1.23	2806	87.8	0.33

f. **Structure 6:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.30 along with the corresponding quantities obtained from elastic analysis. Table 5.30 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are of the first mode in that direction.

Table 5.30 Comparison of time period and base shear quantities for Structure 6

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
20500	22226	0.70	527	0.024	0.71	1115	55.9	0.47	0.67	1059	46.8	0.50

g. **Structure 7:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.31 along with the corresponding quantities obtained from elastic analysis. Table 5.31 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.31 Comparison of time period and base shear quantities for Structure 7

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
24000	20902	0.79	458	0.022	0.99	1239	57.7	0.37	1.06	892	46.7	0.51

h. **Structure 8:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.32 along with the corresponding quantities obtained from elastic analysis. Table 5.32 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.32 Comparison of time period and base shear quantities for Structure 8

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
24000	29419	0.79	645	0.022	0.98	1661	53.3	0.39	1.29	1180	42.5	0.55

i. **Structure 9:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.33 along with the corresponding quantities obtained from elastic analysis. Table 5.33 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.33 Comparison of time period and base shear quantities for Structure 9

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
24000	44146	0.79	968	0.022	1.02	1675	48.5	0.58	0.95	2121	54.5	0.46

j. **Structure 10:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.34 along with the corresponding quantities obtained from elastic analysis. Table 5.34 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.34 Comparison of time period and base shear quantities for Structure 10

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
27000	50382	0.87	1041	0.021	1.21	1733	61.7	0.60	1.03	2296	60.2	0.45

k. **Structure 11:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.35 along with the corresponding quantities obtained from elastic analysis. Table 5.35 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.35 Comparison of time period and base shear quantities for Structure 11

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
34000	49862	1.21	918	0.018	1.32	1874	75.0	0.49	1.16	2473	75.1	0.37

l. **Structure 12:** The calculation of structural period and base shear as per BNBC is presented in the Table 5.36 along with the corresponding quantities obtained from elastic analysis. Table 5.36 also shows the corresponding roof deflections Δ_x and Δ_y in the X and Y direction respectively at the left topmost node. Structural periods that have been calculated in the elastic analysis are the first mode in that direction.

Table 5.36 Comparison of time period and base shear quantities for Structure 12

Roof height, mm	Seismic dead weight W, kN	BNBC Quantities			Calculated Elastic Quantities							
					X-Direction				Y-Direction			
		T *sec*	*V* *kN*	*V/W*	*T* *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*T* *sec*	V_y *kN*	Δ_y *mm*	V/V_y
31000	45157	0.96	871	0.019	1.25	1634	65.3	0.53	1.10	2258	69.4	0.39

5.4.3 Hinge States of the Structures at Performance Point

a. Structure 1: Hinge states of the yielded members at the performance point are shown in the Fig. 5.39 and Fig. 5.40. Section 3.9 of Chapter 3 and Section 4.4 and 4.6 of Chapter 4 define different states of the hinges. From Fig. 5.39 and 5.40, it may be seen that beam hinges are in the 'B' and 'IO' range and only hinges formed in the base of the columns and are in the 'B' range.

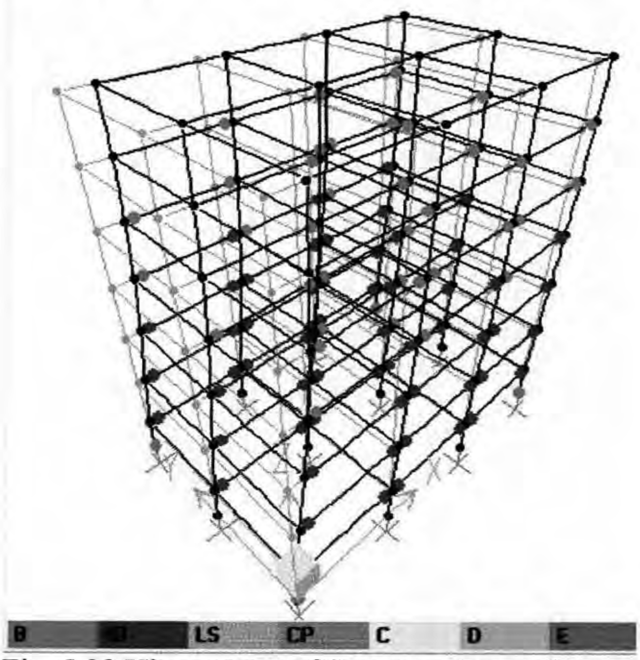


Fig. 5.39 Hinge states of Structure 1 at performance point in X-direction

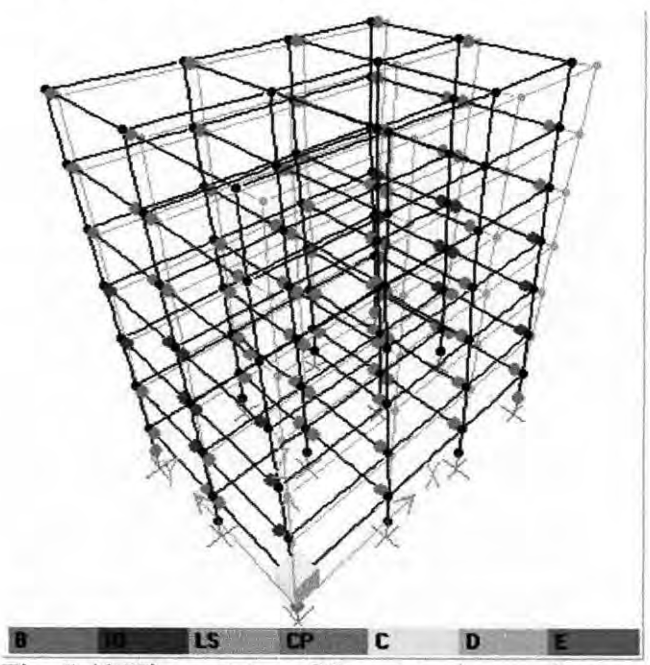


Fig. 5.40 Hinge states of Structure 1 at performance point in Y-direction

b. Structure 2: Fig. 5.41 and Fig. 5.42 represent the hinge states of the structure in X and Y-directions at the performance point (corresponding to performance point deflection of 131.4mm). It may be seen that beam and column hinges are in the 'B' and 'IO' ranges.

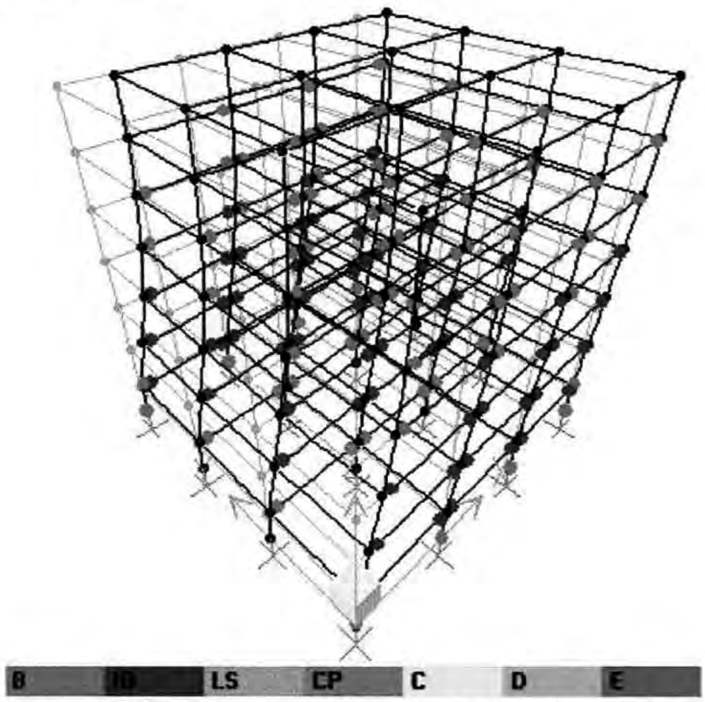


Fig. 5.41 Hinge states of Structure 2 at performance point in X-direction

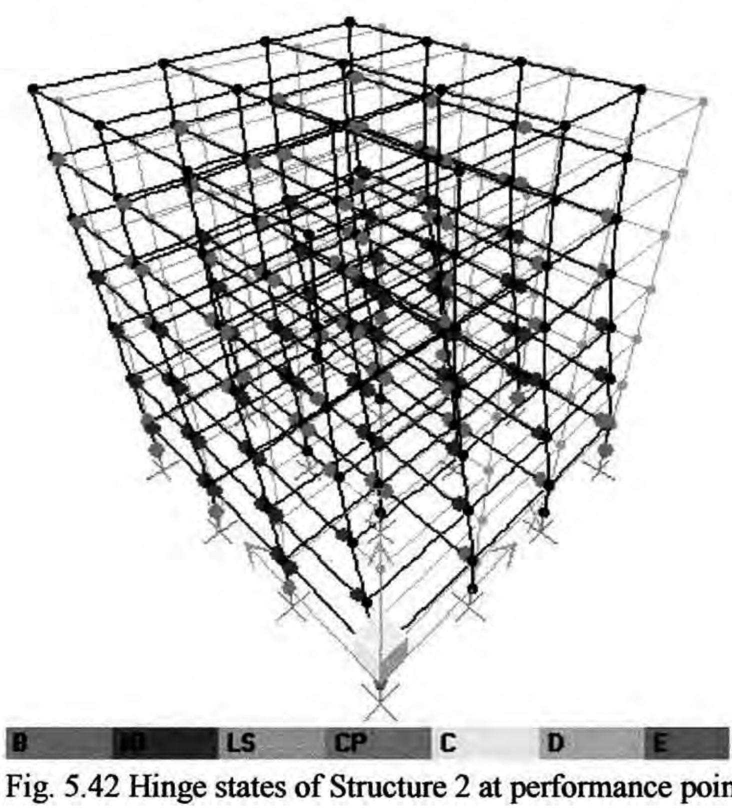


Fig. 5.42 Hinge states of Structure 2 at performance point in Y-direction

c. Structure 3: Fig. 5.43 shows the hinge states of the structure in X-direction at the performance point (corresponding performance point deflection at that direction 150.7mm). The developed hinges are in 'B' and Immediate Occupancy (IO) range.

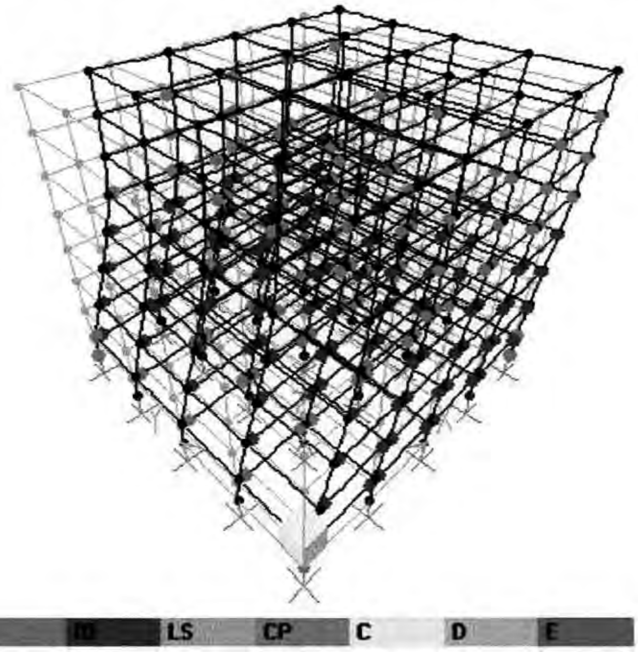


Fig. 5.43 Hinge states of Structure 3 at performance point in X-direction

Fig. 5.44 shows the hinge states of the structure in Y-direction at the performance point (corresponding to performance point deflection at that direction 150.7mm). The developed hinges are in 'B' and Immediate Occupancy (IO) ranges.

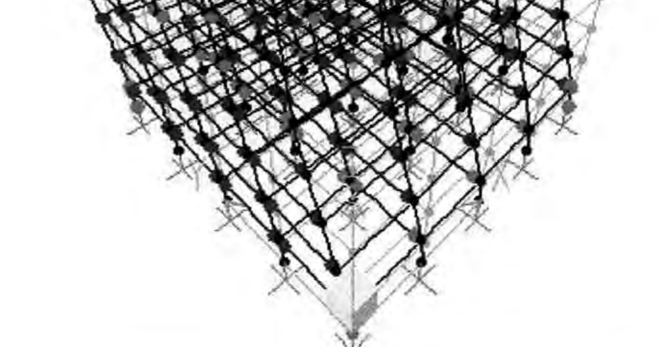


Fig. 5.44 Hinge states of Structure 3 at performance point in Y-direction

d. Structure 4: Fig. 5.45 shows the hinge states of the structure in X-direction at the performance point (corresponding to performance point deflection in that direction 174mm). It may be seen that beam hinges are in the 'B' and 'IO' range and only hinges formed in the base column and are in the 'B' range.

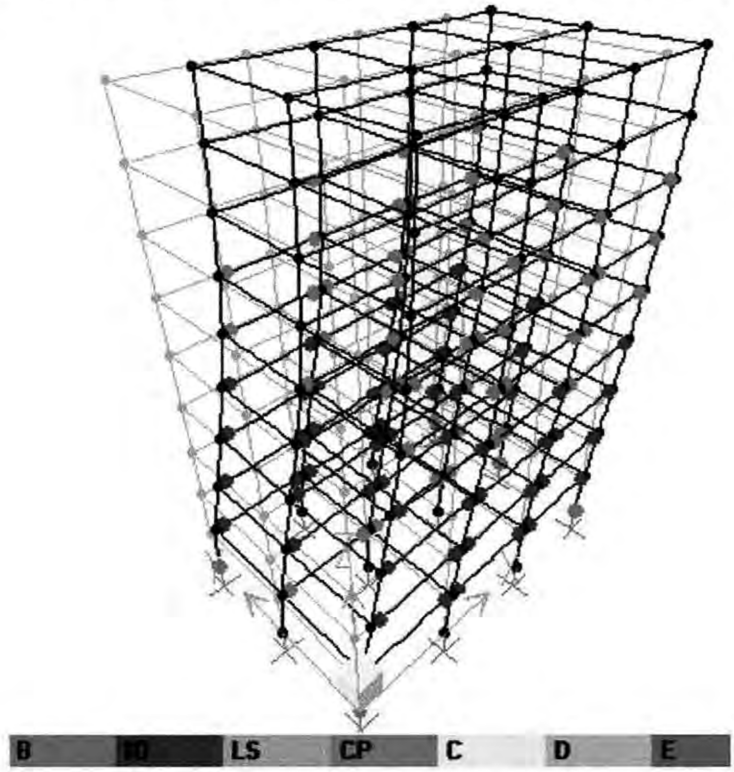


Fig. 5.45 Hinge States of Structure 4 at performance point in X-direction

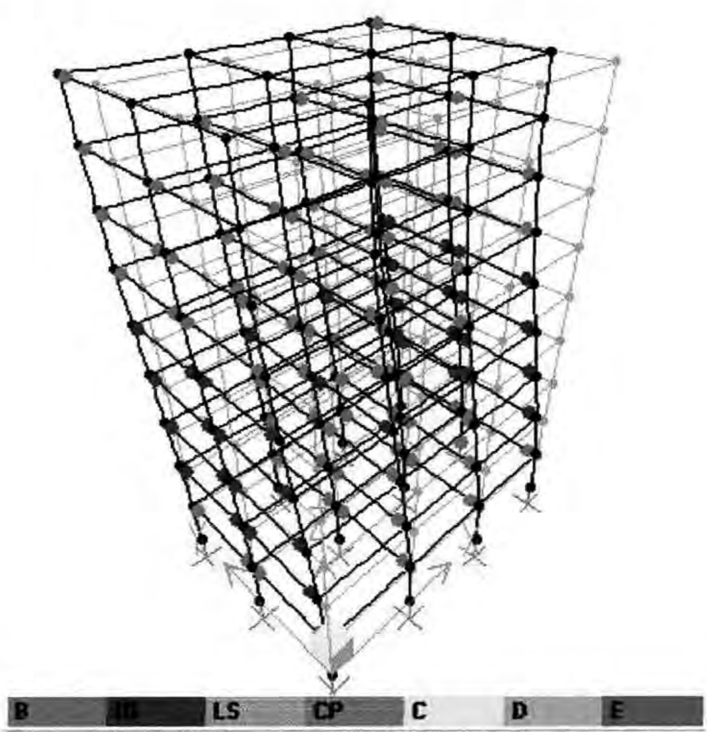


Fig. 5.46 Hinge States of Structure 4 at performance point in Y-direction

Fig. 5.46 shows the hinge states of the structure in Y-direction at the performance point (corresponding to performance point deflection 165.9mm in that direction). The Fig. shows that the developed hinges are in 'B' and 'IO' range. Hinges are formed in the base of the columns only and are in 'B' range.

e. Structure 5: Hinge states of the yielded members at the performance point for X and Y directions are shown in the Fig. 5.47 and Fig. 5.48 respectively.

Fig. 5.47 shows the hinge states of the structure in X-direction at the performance point (corresponding to performance point deflection at that direction 191.1mm). The Fig. 5.47 shows that the edge beams in the second story reaches 'LS' range. Other hinges in the beams and columns are in 'B' and 'IO' range.

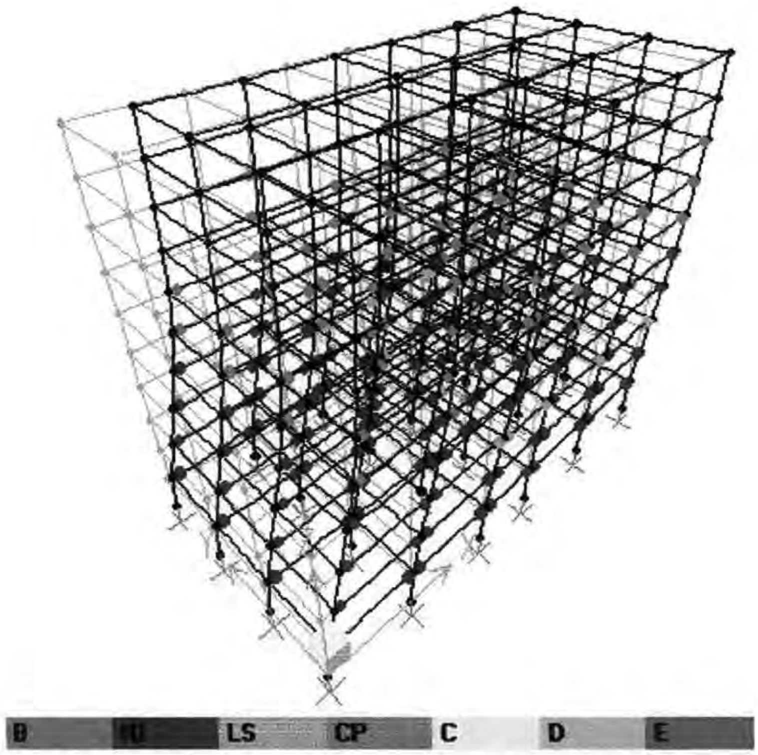


Fig. 5.47 Hinge states of Structure 5 at the performance point in X-direction

Fig. 5.48 shows the hinge states of the Structure 5 in Y-direction at the performance point (corresponding to performance point deflection at that direction 191.8mm). The Fig. 5.48 shows that the hinges are in 'B' and 'IO' range.

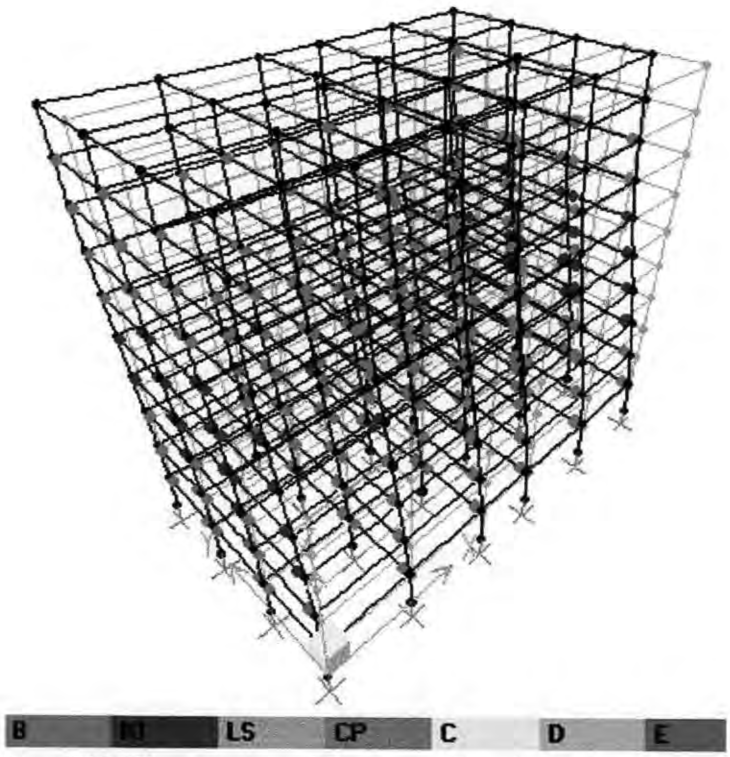


Fig. 5.48 Hinge states of Structure 5 at the performance point in Y-direction

f. Structure 6: Hinge states of the yielded members at the performance point for X and Y directions and are shown in the Fig. 5.49 and Fig. 5.50 respectively.

Fig. 5.49 shows the hinge states of the structure in X-direction at the performance point (corresponding to performance point deflection at that direction 97.4mm). Fig. 5.50 shows the hinge states of the structure in Y-direction near the performance point (corresponding to performance point deflection at that direction 96.7mm). It may be seen from Fig. 5.49 and Fig. 5.50 that the hinges are in 'B' range.

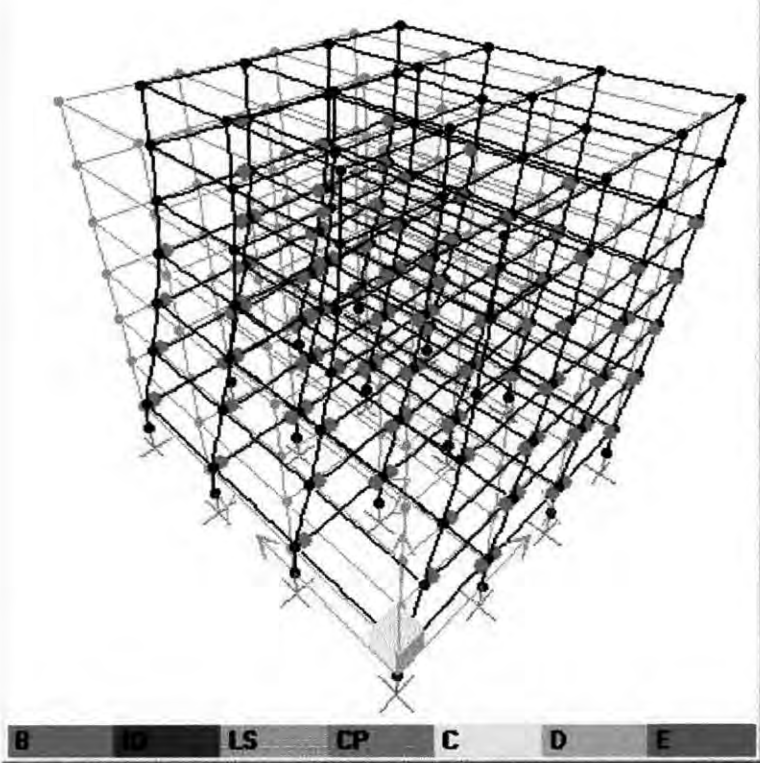


Fig. 5.49 Hinge states of Structure 6 at performance point in X-direction

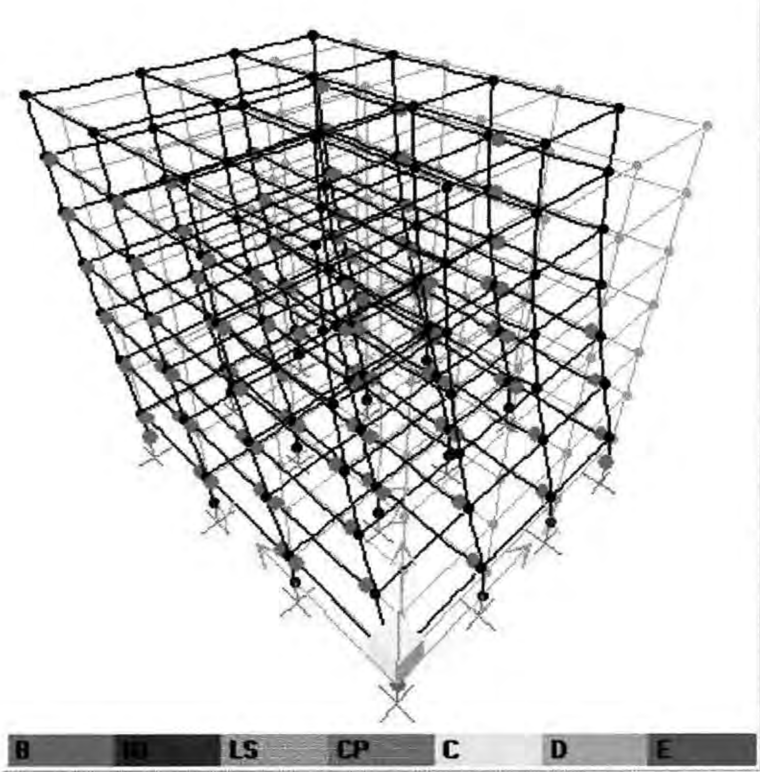


Fig. 5.50 Hinge states of Structure 6 at performance point in Y-direction

g. Structure 7: Hinge states of the yielded members at the performance points are shown in the Fig. 5.51 and Fig. 5.52.

Fig. 5.51 shows the hinge states of the structure in X-direction at the performance point (corresponding to performance point deflection at that direction 144.7mm). It may be seen from Fig. 5.51 that the beam hinges are in 'B' and 'IO' range. Column hinges are formed in the base columns only and are in 'B' range.

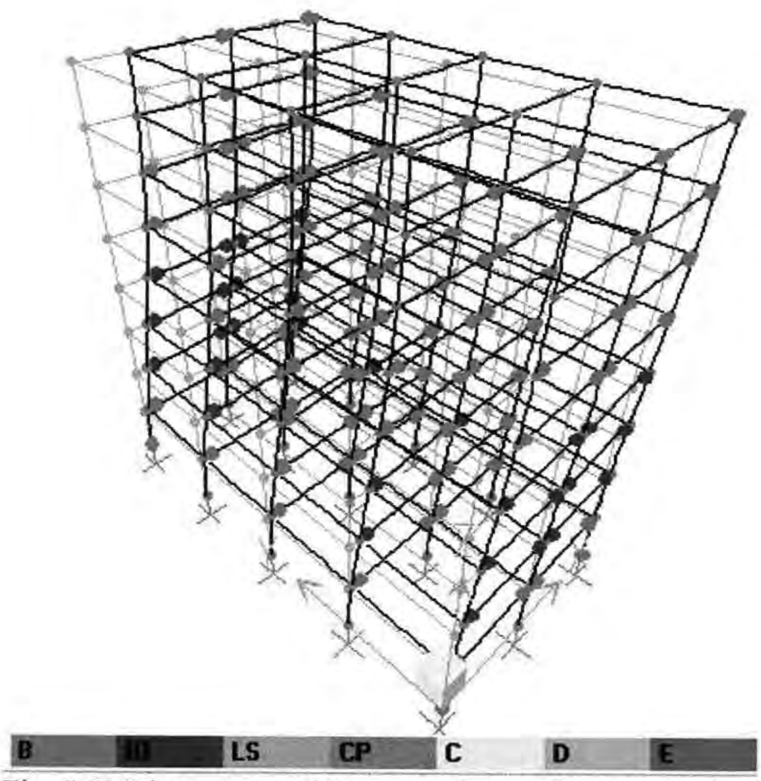


Fig. 5.51 Hinge states of Structure 7 at performance point in X-direction

Fig. 5.52 shows the hinge states of the structure in Y-direction near the performance point (corresponding to performance point deflection at that direction 159.0mm). The Fig. 5.52 shows that the beam hinges are in 'B' and 'IO' range and the base columns only are yielded and in 'B' range.

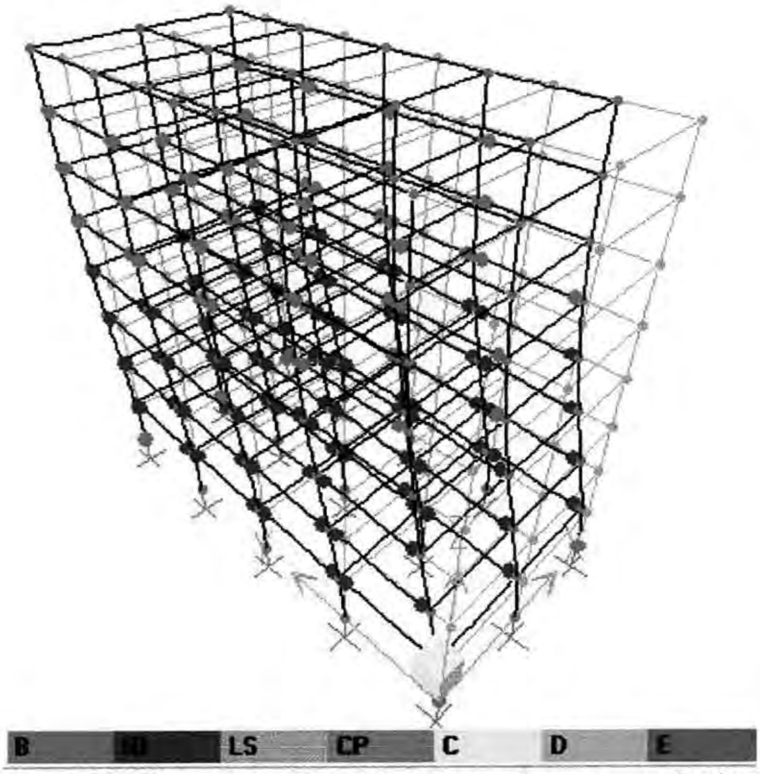


Fig. 5.52 Hinge states Structure 7 at performance point in Y-direction

h. **Structure 8:** Hinge states of the yielded members at the performance point are shown in Fig. 5.53 and Fig. 5.54.

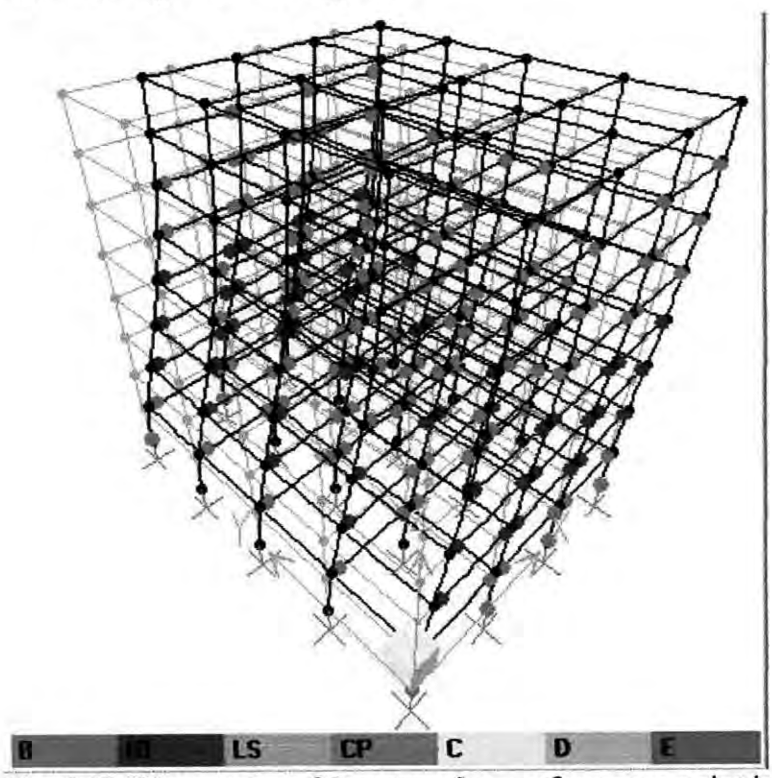


Fig. 5.53 Hinge states of Structure 8 at performance point in X-direction

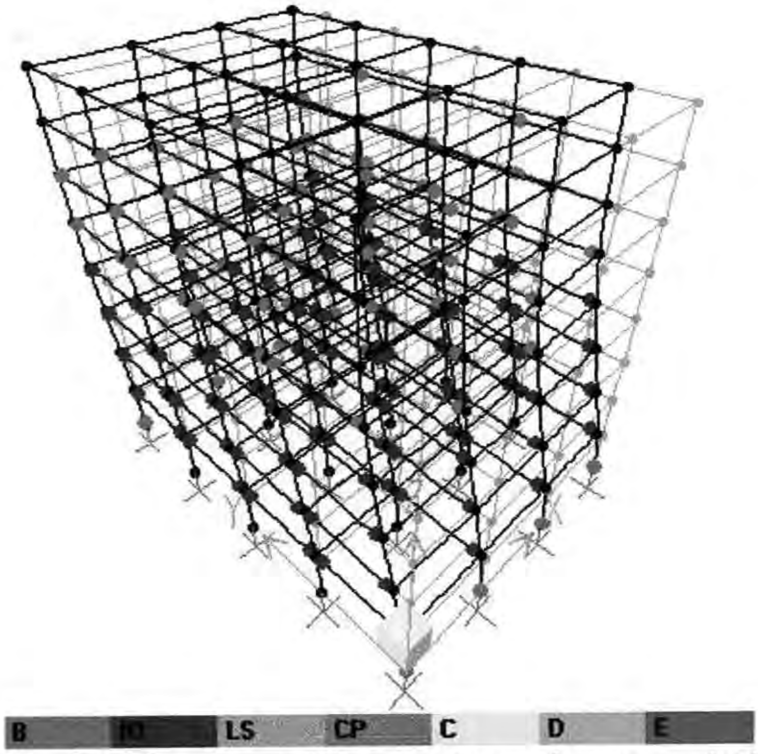


Fig. 5.54 Hinge states of Structure 8 at performance point in Y-direction

Fig. 5.53 and Fig. 5.54 show that the developed hinge in the structural members both in X-direction and Y-direction at the performance point (corresponding to performance point deflection in X-direction 142.30mm and Y-direction 160.0mm) are in the 'B' and 'IO' range.

i. **Structure 9:** Fig. 5.55 shows the hinge states of Structure 9 in X-direction at the performance point (corresponding to performance point deflection 149.30 mm in that direction.). The Fig. 5.55 shows that the hinges in the structural members are in 'B' and Immediate Occupancy (IO) range or below.

Fig. 5.56 shows the hinge states of Structure 9 in Y-direction at the performance point (corresponding to performance point deflection 141.40mm in that direction). The Fig. 5.56 shows that the hinges in the structural members are in the 'B' and 'IO' range.

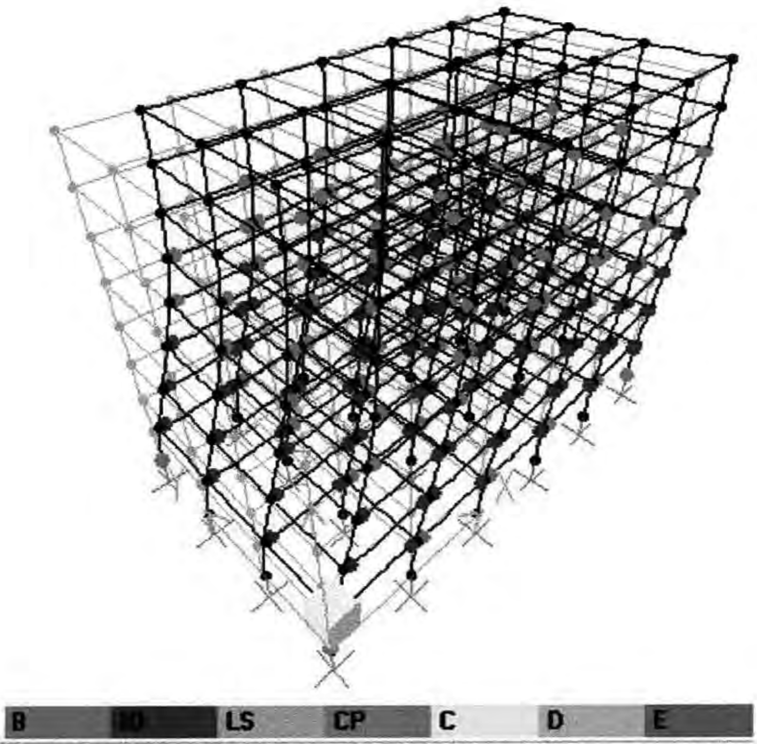


Fig. 5.55 Hinge states of Structure 9 at performance point in X-direction

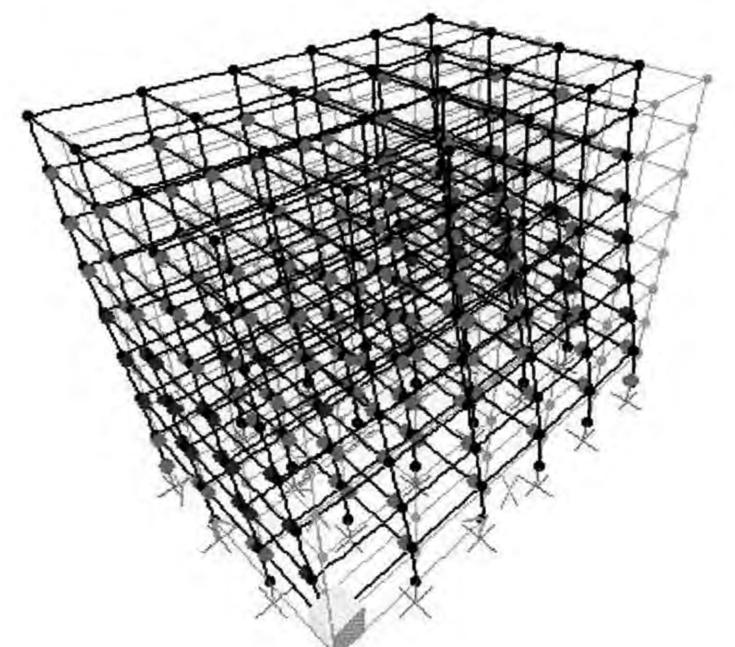


Fig. 5.56 Hinge states of Structure 9 at performance point in Y-direction

j. Structure 10: Fig. 5.57 shows the hinge states of Structure 10 in X-direction at the performance point (corresponding to performance point deflection in that direction 175.30mm). The Fig. 5.57 shows that the hinges in the beams are in 'B' and 'IO' range and no hinges are formed in the columns.

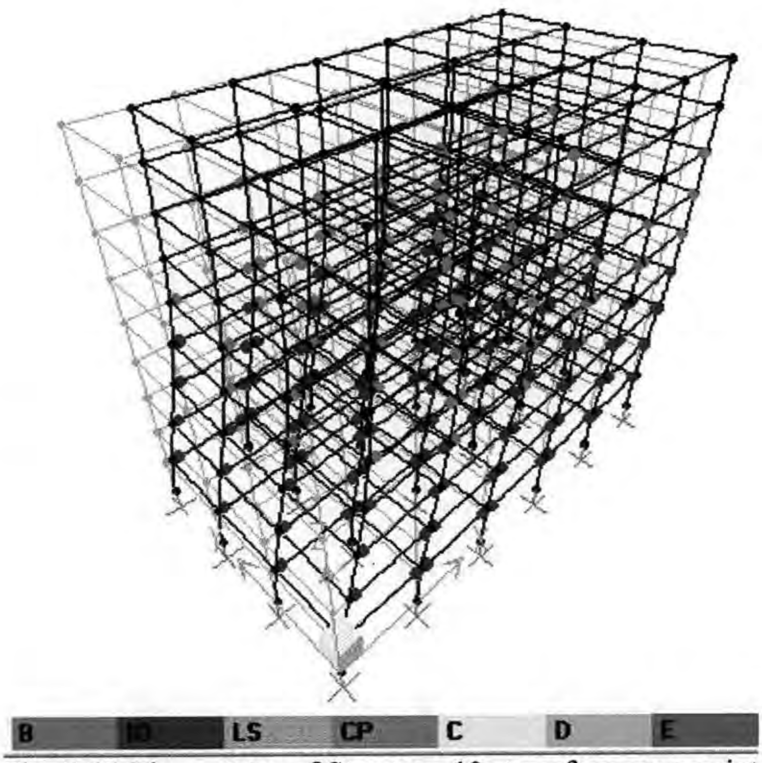


Fig. 5.57 Hinge states of Structure 10 at performance point in X-direction

Fig. 5.58 shows the hinge states of the structure in Y-direction at the performance point (corresponding to performance point deflection in that direction 158.30mm). The Fig. 5.58 shows that the hinges in the beams are in 'B' and 'IO' range and no hinges are formed in the columns.

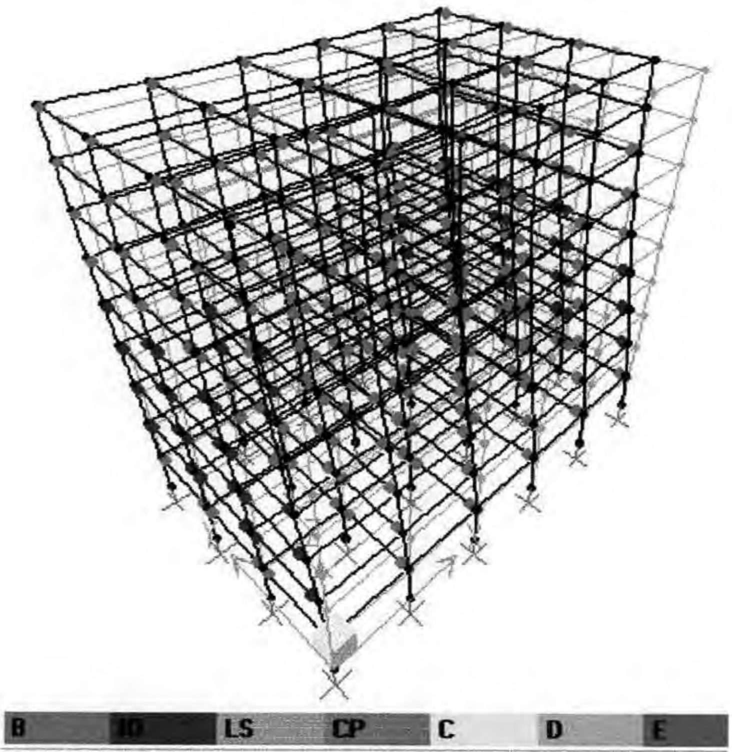


Fig. 5.58 Hinge states of Structure 10 at performance point in Y-direction

k. Structure 11: Fig. 5.59 shows the hinge states of the structure in X-direction at the performance point (corresponding to performance point deflection at that direction 192.3mm). The Fig. 5.59 shows that the structural members are just yielded ('B' range) and in the Immediate Occupancy (IO) range. No hinges are formed in the columns.

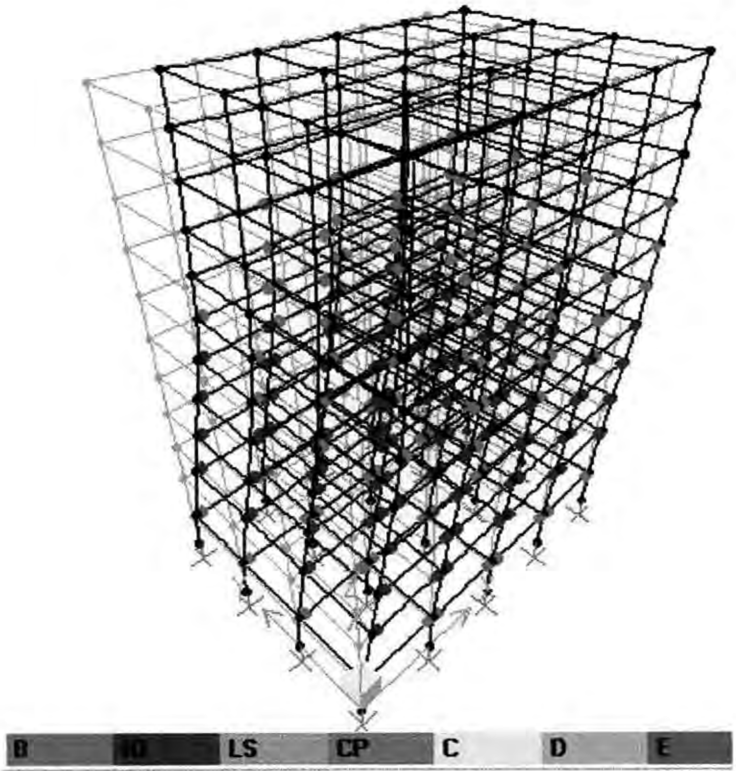


Fig. 5.59 Hinge states of Structure 11 at performance point in X-direction

Fig. 5.60 shows the hinge states of the structure in Y-direction at the performance point (corresponding to performance point deflection at that direction 176.60mm). The Fig. 5.60 shows that the hinges formed in the structural members are in 'B' and 'IO' range.

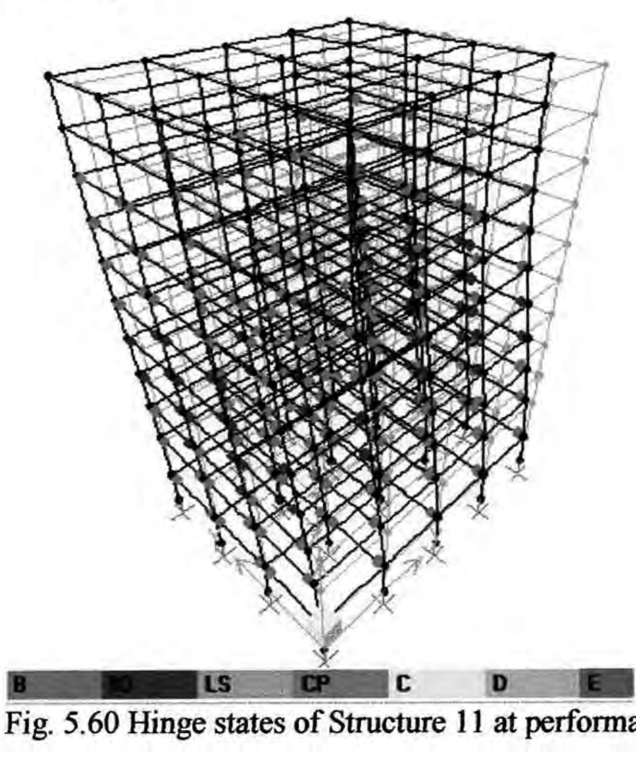


Fig. 5.60 Hinge states of Structure 11 at performance point in Y-direction

l. **Structure 12:** Fig. 5.61 shows the hinge states of Structure 12 in X-direction at the performance point (corresponding to performance point deflection at that direction 184.9mm). Fig. 5.61 shows that the beam members are just yielded ('B' range) and in the Immediate Occupancy (IO) range. No hinges are formed in the columns.

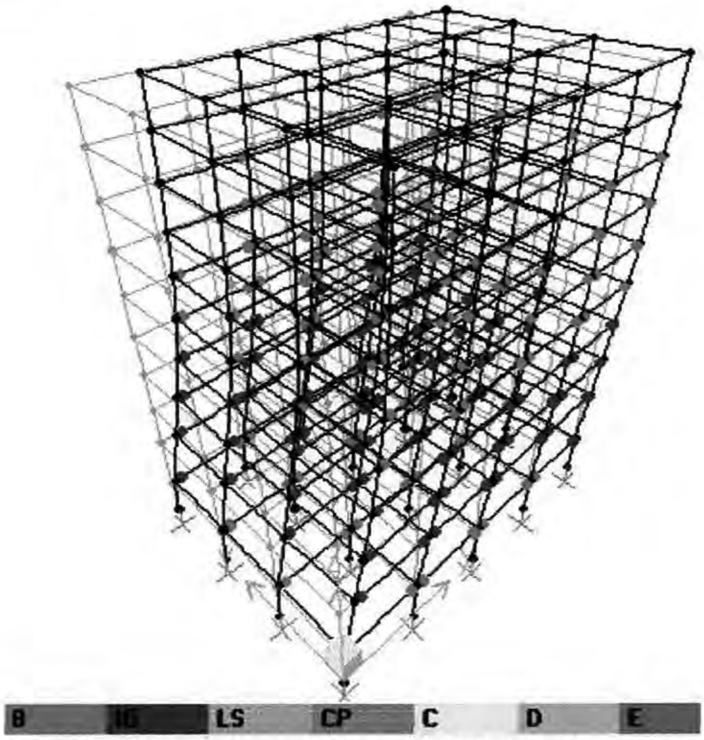


Fig. 5.61 Hinge states of Structure 12 at performance point in X-direction

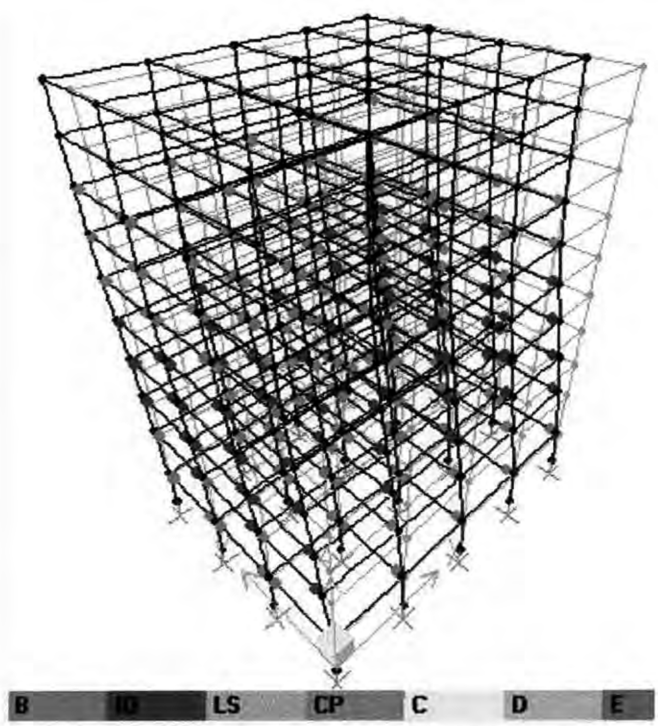


Fig. 5.62 Hinge states of Structure 12 at performance point in Y-direction

Fig. 5.62 shows the hinge states of Structure 12 in Y-direction at the performance point (corresponding to performance point deflection at that direction 169.8 mm). Fig. 5.62 shows that the beam members are just yielded ('B' range) and in the Immediate Occupancy (IO) range. No hinges are formed in the columns.

5.4.4 Structural Performance

a. **Structure 1:** Performance of Structure 1 as calculated has been presented in the Table 5.37 and Table 5.38. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.007 in the X-direction and 0.0064 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0044 in X-direction and 0.003 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, the global performance of the structure meets Immediate Occupancy (IO) performance level in both directions.

Table 5.37 Performance of Structure 1 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
1.45	887	139.9	0.63	0.0070	Immediate Occupancy	0.0044	Immediate Occupancy

Table 5.38 Performance of Structure 1in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.09	1274	127.2	0.44	0.0064	Immediate Occupancy	0.0030	Immediate Occupancy

b. **Structure 2**: Performance of Structure 2 as calculated has been presented in the Table 5.39. It has been found that the maximum total drift ratio (total deflection by total height) is 0.0066 in both directions and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0044 in both directions. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy (IO) performance level in both directions.

Table 5.39 Performance of Structure 2 in X and Y-direction

At Performance Point							
X and Y-Direction							
T_{eff} sec	V_{xy} kN	Δ_{xy} mm	V/V_{xy}	Max^m total drift ratio		Max^m inelastic drift ratio	
1.38	1432	131.4	0.30	0.0066	Immediate Occupancy	0.0044	Immediate Occupancy

c. **Structure 3:** Performance of Structure 3 as calculated has been presented in the Table 5.40. It has been found that the maximum total drift ratio (total deflection by total height) is 0.0066 in both directions and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0045 in both directions. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.40 Performance of Structure 3 in X and Y-Direction

At Performance Point							
X and Y-Direction							
T_{eff} *sec*	V_{xy} *kN*	Δ_x *mm*	V/V_{xy}	*Max^m total drift ratio*		*Max^m inelastic drift ratio*	
1.52	2296	150.7	0.35	0.0066	Immediate Occupancy	0.0045	Immediate Occupancy

d. **Structure 4:** Performance of Structure 4 as calculated has been presented in the Table 5.41 and Table 5.42. It has been found that the maximum total drift ratio (total deflection by total height) is 0.0064 in the X-direction and 0.0061 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0034 in X-direction and 0.004 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.41 Performance of Structure 4 in X-direction

At Performance Point							
X-Direction							
T_{eff} *sec*	V_x *kN*	Δ_x *mm*	V/V_x	*Max^m total drift ratio*		*Max^m inelastic drift ratio*	
1.82	1092	174.0	0.49	0.0064	Immediate Occupancy	0.0034	Immediate Occupancy

Table 5.42 Performance of Structure 4 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} *sec*	V_y *kN*	Δ_y *mm*	V/V_y	*Max^m total drift ratio*		*Max^m inelastic drift ratio*	
1.47	1320	165.9	0.41	0.0061	Immediate Occupancy	0.0040	Immediate Occupancy

e. **Structure 5:** Performance of Structure 5 as calculated has been presented in the Table 5.43 and Table 5.44. It has been found that the maximum total drift ratio at performance point (total deflection by total height) is 0.0064 in the X-direction and 0.0064 in the Y-direction and that of maximum inelastic drift ratio(inelastic deflection by total height) is 0.0046 in X-direction and 0.0035 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.43 Performance of Structure 5 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
2.14	2439	191.1	0.38	0.0064	Immediate Occupancy	0.0046	Immediate Occupancy

Table 5.44 Performance Structure 5 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.57	3680	191.8	0.25	0.0064	Immediate Occupancy	0.0035	Immediate Occupancy

f. **Structure 6:** Performance of Structure 6 as calculated has been presented in the Table 5.45 and Table 5.46. It has been found that the maximum total drift ratio at performance point (total deflection by total height) is 0.0048 in the X-direction and 0.0047 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) at performance point is 0.002 in X-direction and 0.0024 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.45 Performance of Structure 6 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
0.91	1354	97.4	0.39	0.0048	Immediate Occupancy	0.0020	Immediate Occupancy

Table 5.46 Performance Structure 6 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
0.82	1625	96.7	0.32	0.0047	Immediate Occupancy	0.0024	Immediate Occupancy

g. **Structure 7:** Performance of Structure 7 as calculated has been presented in the Table 5.47 and Table 5.48. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.006 in the X-direction and 0.0066 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0036 in X-direction and 0.0047 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.47 Performance of Structure 7 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
1.36	1670	144.7	0.27	0.006	Immediate Occupancy	0.0036	Immediate Occupancy

Table 5.48 Performance of Structure 7 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.75	1242	159.0	0.37	0.0066	Immediate Occupancy	0.0047	Immediate Occupancy

h. **Structure 8:** Performance of Structure 8 as calculated has been presented in the Table 5.49 and Table 5.50. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.0059 in the X-direction and 0.0067 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0037 in X-direction and 0.0049 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.49 Performance of Structure 8 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
1.40	2233	142.3	0.29	0.0059	Immediate Occupancy	0.0037	Immediate Occupancy

Table 5.50 Performance of Structure 8 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.73	1753	160.0	0.37	0.0067	Immediate Occupancy	0.0049	Immediate Occupancy

i. **Structure 9:** Performance of Structure 9 as calculated has been presented in the Table 5.51 and Table 5.52. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.0062 in the X-direction and 0.0059 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0042 in X-direction and 0.0036 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.51 Performance of Structure 9 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
1.58	2403	149.3	0.40	0.0062	Immediate Occupancy	0.0042	Immediate Occupancy

Table 5.52 Performance of Structure 9 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.30	3024	141.4	0.32	0.0059	Immediate Occupancy	0.0036	Immediate Occupancy

j **Structure 10:** Performance of Structure 10 as calculated has been presented in the Table 5.53 and Table 5.54. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.0065 in the X-direction and 0.0059 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0042 in X-direction and 0.0036 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.53 Performance of Structure 10 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
1.91	2332	175.3	0.45	0.0065	Immediate Occupancy	0.0042	Immediate Occupancy

Table 5.54 Performance of Structure 10 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.38	3351	158.3	0.31	0.0059	Immediate Occupancy	0.0036	Immediate Occupancy

k. **Structure 11:** Performance of Structure 11 as calculated has been presented in the Table 5.55 and Table 5.56. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.0057 in the X-direction and 0.0052 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0035 in X-direction and 0.003 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.55 Performance of Structure 11 in X-direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
2.14	2207	192.3	0.42	0.0057	Immediate Occupancy	0.0035	Immediate Occupancy

Table 5.56 Performance of Structure 11 in Y-direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.59	3097	176.6	0.30	0.0052	Immediate Occupancy	0.0030	Immediate Occupancy

l. **Structure 12:** Performance of structure 12 as calculated has been presented in the Table 5.57 and Table 5.58. It has been found that at performance point, the maximum total drift ratio (total deflection by total height) is 0.006 in the X-direction and 0.0055 in the Y-direction and that of maximum inelastic drift ratio (inelastic deflection by total height) is 0.0039 in X-direction and 0.0032 in the Y-direction. As per the acceptance limit given in Table 3.4 of Chapter 3, global performance of the structure meets Immediate Occupancy performance level in both directions.

Table 5.57 Performance of Structure 12 in X-Direction

At Performance Point							
X-Direction							
T_{eff} sec	V_x kN	Δ_x mm	V/V_x	Max^m total drift ratio		Max^m inelastic drift ratio	
2.05	2080	184.9	0.42	0.0060	Immediate Occupancy	0.0039	Immediate Occupancy

Table 5.58 Performance of Structure 12 in Y-Direction

At Performance Point							
Y-Direction							
T_{eff} sec	V_y kN	Δ_y mm	V/V_y	Max^m total drift ratio		Max^m inelastic drift ratio	
1.55	2831	169.8	0.31	0.0055	Immediate Occupancy	0.0032	Immediate Occupancy

5.5 Comments on the Behavior of the Structures at Performance Point

From the analyses and the results presented in the preceding sections, the following general behaviors are observed:

1. For medium-rise structures, examined in this study, ranging from six to nine story buildings designed and detailed as per provisions of BNBC have well defined capacity curves in both directions when pushed monotonically by load pattern similar to code specified Equivalent Static Load procedure. Base shear capacity (elastic) of the structures considered in this thesis are in the range of 1.67 to 3.0 times than that of the code specified base shear calculated by empirically formula (See Table 5.25 to Table 5.36).

2. Capacity of a structure in any direction basically depends on the capacity of the structural members in that direction. For the cases studied in this thesis, the numbers of bays present in that direction do not significantly affect the capacity of the structure.

3. Elastic fundamental periods calculated from the present analysis are found to be 20% to 40% more than that estimated by BNBC empirical formula. Code formula is thus regarded to be reasonably conservative.

4. Global structural performance of the buildings designed as per provision of BNBC meets the Immediate Occupancy (IO) performance level in Design Earthquake (DE).

5. At the performance point, the deformation demand (rotational and displacement) of the hinges satisfy the requirement specified for the Immediate Occupancy (IO) range.

6. At performance point hinges have been formed at the base of the columns only. For beams it has been found that hinges are formed in the grade beams first and limited to the lower stories only.

Chapter-6

CONCLUSIONS

6.1 Conclusions

In this thesis a review of the existing seismic design provisions of BNBC has been made. Based on the review a number of shortcomings of the seismic design provisions of BNBC have been identified. Subsequently, a total of twelve medium rise buildings designed as per the provisions of BNBC, 1993 have been analyzed using nonlinear static procedure upto collapse. The structural performances of these structures have been evaluated as per the procedure of ATC-40. The conclusions derived from the study are subject to the limitations as stated in Section 6.2. The observation and conclusions of the study are summarized as follows:

A. Observation on the Limitations of BNBC

A.1 It appears that the seismic zoning and zone coefficients of BNBC are not consistent with those of the neighboring country, India. Recently India has updated its zoning map (See Fig. 2.8) in consistency with recent data of seismic activities in the region. It is felt that a review of the current seismic zoning of BNBC is required for possible updating.

A.2 The distribution of base shear along the building height is linear which is not consistent with the fundamental mode shapes. Nonlinear distribution matching the first mode shape may be proposed as has been done in UBC, (1997) and IS-1893, (2002).

A.3 Response modification factor, R does not quantitatively relate to the detailing required for achieving the assumed level of ductility.

A.4 The provision of seismic detailing required for different types of framing system is not explicitly defined. For easy conceptualization a graphical representation of the different types of detailing may be adopted.

B. Conclusion on Structural Performance

B.1 For medium-rise structures, examined in this study, ranging from six to nine story buildings designed and detailed as per provisions of BNBC have well defined capacity curves in both directions when pushed monotonically by load pattern similar to code specified Equivalent Static Load procedure. Base shear capacity (elastic) of the structures considered in this thesis are in the range of 1.67 to 3.0 times than that of the code specified base shear calculated by empirically formula (See Table 5.25 to Table 5.36).

B.2 Capacity of a structure in any direction basically depends on the capacity of the structural members in that direction. For the cases studied in this thesis, the numbers of bays present in that direction do not significantly affect the capacity of the structure.

B.3 Elastic fundamental periods calculated from the present analysis are found to be 20% to 40% more than that estimated by BNBC empirical formula. Code formula is thus regarded to be reasonably conservative.

B.4 Global structural performance of the buildings designed as per provision of BNBC meets the Immediate Occupancy (IO) performance level in Design Earthquake (DE).

B.5 At the performance point, the deformation demand (rotational and displacement) of the hinges satisfy the requirement specified for the Immediate Occupancy (IO) range.

B.6 At performance point hinges have been formed at the base of the columns only. For beams it has been found that hinges are formed in the grade beams first and limited to the lower stories only.

6.2 Limitation of the Study

The buildings under the study are all residential structure of very regular geometry. Stiffness irregularities in both horizontal and vertical directions have not been considered. Only 'Structural Performance' has been evaluated for Design Earthquake (DE). Pushover analysis has been done with load pattern as per earthquake load calculated by empirical formula of BNBC (Equivalent Static Load Method). Structural variation has been done only in numbers of bays and numbers of story.

6.3 Recommendation for Future Research

The following recommendations for future study can be made from the present study:

a) BNBC calculates structural period as a function of height. But structure's fundamental period is a function of mass and stiffness which may vary in different directions. Again dynamic behavior of a structure depends on the structural period. So, a study may be attempted to formulate structural period on the basis of structure's lateral stiffness giving due consideration to the spatial variation in stiffness.

b) First mode shape of a structure is not a straight line as assumed in BNBC in its recommendation for the distribution of earthquake lateral loads. A study may be made to represent the Equivalent Static Force as nonlinear variation along the height representing the first mode shape of the structure as is currently done in UBC (1997) and IS-1893 (2002).

c) Performance of a structure depends on the deformation capabilities of the members. A study may be made to relate the deformation capabilities of a member to a specific detailing requirement.

d) Structural performances of only very regular structures of limited heights have been studied. Further investigation may be made based on geometric irregularity, soft story mechanism and different seismic demand for the structures.

e) Reliability analysis of the structures incorporating probabilistic approach may be done.

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