SEISMIC ANALYSIS OF IRREGULAR RCC FRAME AND REVIEW OF INDIAN SEISMIC CODE

Project Report submitted in partial fulfillment of the requirement for the degree of

Bachelor of Technology

in

CIVIL ENGINEERING

under the Supervision of

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CERTIFICATE

This is to certify that the work which is being presented in the project title SEISMIC ANALYSIS OF IRREGULAR RCC FRAME AND REVIEW OF INDIAN SEISMIC CODE, in partial fulfilment of the requirements for the award of the degree of Bachelors of Technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Rohan Singhal (111671) and Shivam Pundir (111679)** during a period from August 2014 to May 2015 under the supervision of **Prof. Dr. Ashok Kr. Gupta, Head of Department** and **Mr. Abhilash Shukla, Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology** in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering to *Jaypee University of Information Technology*, Waknaghat.

The above statement made is correct to the best of my knowledge

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ABSTRACT

The destruction caused by Nepal earthquake reminds us of how unsafe our buildings are. Indian Seismic Code IS: 1893 (Part 1): 2002 has not been revised since last past 13 years even when there are substantial work in the field of earthquake resistant design of buildings. With the mission to bring upon some improvements in code of practices followed in India this work is carried forward.

This work compares the work of practices mentioned in IS 1893:2002 with those put forward by ATC-40. This work gives the procedure to produce elastic design spectrum from site specific ground displacements. This is further reduced to design response spectrum through D-A-V 4D log graph. The definition of ductility is established and ductility curvature plots are drawn by which optimum percentage of steel reinforcement is established. Equal displacement and equal energy principle are discussed which are required to evaluate the value of response reduction factor R.

The equivalent linearization suggested as per FEMA 440: 2005 is reviewed and it has been observed that Indian code need to include the hysteresis loos as per different models to account for value of R. Further, this work considers a 4 storey shopping complex with re-entrant corners with T shape plan. The complex is considered to be situated in zone V as per IS 1893: 2002 classifications and loading is provided as per IS 875 (Part1):1987. The seismic responses of complex which include top floor displacement, base shear, time period, frequency etc. of complex with T plan have been compared with structure with removal of re-entrant corners.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1. TYPES OF ANALYSIS

- Static analysis
  - Equivalent lateral base shear force (Cl. 6.4.2 of IS 1893:2002)
- Dynamic analysis
  - Time history (Cl. 7.8.3 of IS 1893:2002)
  - Response spectrum (Cl.7.8.4 of IS 1893:2002)

Choice of analysis method is influenced by seismic zone (as per fig 1. Or annex E), type of building i.e. height and irregularity. *Irregularity check is to be performed by clause 7.1.*

1.1.1.STATIC ANALYSIS

The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression.

$$V_B = A_h \, .W$$

where

A_h= Design horizontal acceleration spectrum value as per Cl. 6.4.2 using the fundamental natural period T_a as per 7.6 in the considered direction of vibration.

W = Seismic weight of the building as per 7.4.2 of IS 1893:2002

Calculation of A_h

$$A_h = \frac{Z * I * S_a}{2 * R * g}$$

where

Z = zone factor corresponding MCE (maximum considered earthquake) (Cl 3.19 of IS 1893:2002).

It is divided by 2 to convert it into DBE (design basis earthquake) (Cl 3.6 of IS 1893:2002*). Origin of Z has purely statistical basis and is defined uniquely for a particular seismic zone and design life of the structure.*

Challenge: IS 1893:2002 has given no definition of design life of the structure. If a structure is to be designed for a design period other than 50 years (suggested by other international seismic code) than what should be the value of Z used.

I = Importance factor

Table 6 of the code suggests I = 1 or 1.5 depending on the importance of structure. Greater value can be chosen. IBC (international building code) suggests I = 1.0 or 1.25 or 1.5. Value of 1.25 might help in ensuring economy.

R = Response reduction factor (Table 7 of IS 1893:2002)

It is an attempt to consider the structure's inelastic characteristics in linear analysis method. Dividing elastic design spectrum by R we get inelastic design spectrum.

NOTE: In no case $\frac{\boldsymbol{I}}{\boldsymbol{R}} \leq 1$

$\frac{S_a}{g}$ = Average response acceleration coefficient

It is a function of damping of structure, nature of soil (local ground conditions), natural time period of building.

$\frac{S_a}{g}$ = f (T, soil nature) [from fig 2 of IS 1893:2002] *damping factor multiplier [Table 3 of IS 1893:2002]

Fig. 2 in IS 1893:2002 is defined for damping ratio 0.05 for value of damping ratio other than this we have to multiply it by damping factor multiplier obtained from Table 3.

Damping factor multiplier is inversely proportional to damping ratio. It means more damping less design seismic base shear.

Here,

T = fundamental natural period

*Fundamental natural period of a building depends on its flexibility and type of frame. For qualitative estimation it can be assumed that fundamental natural time period of a building is .1*n where n is the number of stories.*

IS 1893:2002 has given following expressions;

1. For buildings composed of moment resisting frame without brick infill panels
   $T_a = 0.075h^{0.75}$ for RC building (in seconds)
   $= 0.085h^{0.75}$ for steel frame building (in seconds)

   where ‘h’ is height of building in meters.

2. For buildings including moment resisting frames with brick infill panels.

   $T_a = \frac{0.09*h}{\sqrt{d}}$ (in seconds)

   Where‘d’ base dimension of building (in m) at plinth level in the direction of considered lateral load.

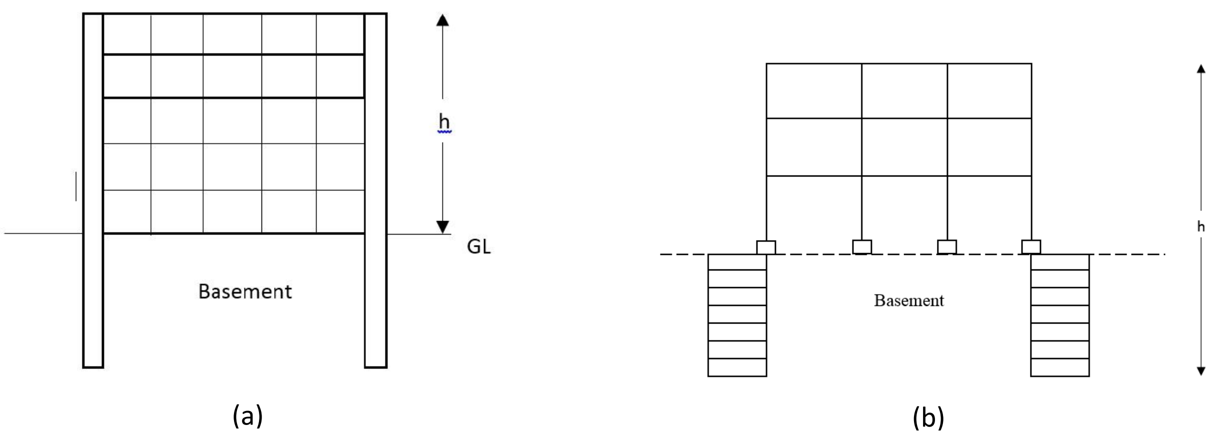


Fig.1 Height of Building

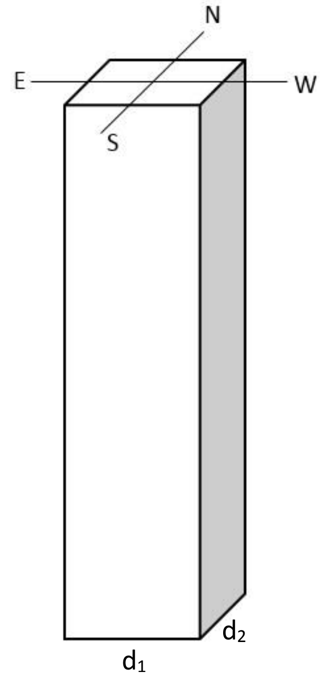


Fig.2 Base Dimension of the Building at the Plinth Level

For seismic lateral force along	**Base dimension to be used**
W-E or E-W	d_1
N-S or S-N	d_2

W = seismic weight of the building (Cl. 7.4.2 of IS 1893:2002)

W = $\sum(seismic\ weight\ of\ floors)$ (Cl. 7.4.1 of IS 1893:2002)

W_{floor} = Dead load of floor + (weight of columns + weight of walls) in inverse proportion of its distance from the floors + reduced imposed load as per clause 7.3.1 and 7.3.2 of IS 1893:2002

1.1.1.1. Reasons for considering reduction in Imposed Load

- Only a part of the maximum live load will probably be existing at the time of earthquake.
- Non-rigid mounting of the live load absorbs part of the earthquake energy.
- Specified live load include as part of it , impact effects of the loads which need not be considered since earthquake load act on mass only.

NOTE: Imposed load on roof need not be considered. If T $\leq$ .1 sec i.e. structure with very short time period.

Then $A_h \nless \frac{Z}{2}$, irrespective of the value of $\frac{I}{R}$.

A_h, modifications [Cl 6.4.4 of IS 1893:2002] this modification is valid for underground structures and foundations at depths of 30 m or below.

For depth below 30 m, $A_h^o = A_h * RF$

where RF = Reduction Factor

$= 1 - \frac{.5*d}{30}$, d$\leq 30m$

A_v = Design vertical acceleration spectrum (Cl.6.4.5 of IS 1893:2002)

$= \frac{2}{3} * A_h$

Distribution of Design force as per Cl. 7.7.1 of IS 1893:2002

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Q_i = Design lateral force at floor i

h_i = height of floor i measured from base

n= number of storeys in the building is the number of levels at which the masses are located.

Assumes a parabolic distribution of force,

1.1.1.2. Response reduction factor (R)

It has been introduced to account for conversion of elastic design spectrum to inelastic design spectrum.

INELASTIC DESIGN SPECTRA

The inelastic design spectra are generated from the elastic spectra using *the equal displacement* and *equal energy concepts*.

An inelastic structure subjected to a design level earthquake is assumed to sustain a structural displacement ductility of μ where μ is defined as the ration of the inelastic displacement $\Delta\, u$ of the structure to its yield displacement $\Delta\, y$.

$$\mu \text{ (Ductility)} = \frac{\Delta u}{\Delta y}$$

EQUAL DISPLACEMENT PRINCIPLE: [NEWMARK, 1960]

It was observed in a series of inelastic analyses that the inelastic structures had similar magnitudes of maximum displacement to those of identical structures which were considered to remain linearly elastic.

i.e. $\Delta\, u$ (maximum displacement in inelastic structure) = $\Delta\, e$(maximum displacement in elastic structure)

$$\Delta\, u = \Delta\, e$$

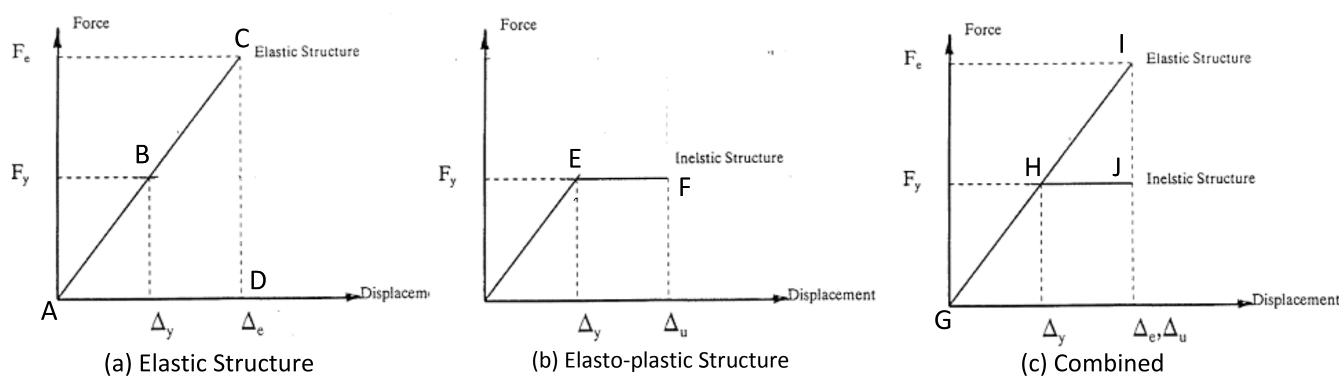


Fig.3 Equal Displacement Concept

In real life structures force does not vary linearly after yield point (B) in fig a. We observe plastic behavior such that force remains constant for a range of displacement (EF) fig b.

In fig a,

$$\frac{F_e}{\Delta\, e} = \frac{F_y}{\Delta\, y}$$

$$\frac{F_e}{\Delta u} = \frac{F_y}{\Delta y}$$

As, $\Delta e = \Delta u$

$$\frac{F_e}{F_y} = \frac{\Delta u}{\Delta y}$$

So, we have

$$\frac{F_e}{F_y} = \mu$$

$$F_y = \frac{F_e}{\mu}$$

The inertia force or base shear force generated in an elastic-plastic single degree of freedom system is $1/\mu$ of that in the matching elastic material.

For systems with very short natural periods (T < .1s), and in particular as the natural periods tends to zero, both inelastic and elastic systems appear to undergo equal accelerations and hence the forces generated in the systems are similar.

This implies that the inelastic system with a ductility of μ undergoes a displacement μ times that of the elastic system.

$\Delta u = \mu \Delta e$ responsible for straight line in [fig 2] [IS 1893:2002]

EQUAL ENERGY CONCEPT:

For systems with a natural period in an intermediate range the concept of equal energies or equal velocities has become accepted.

In case of equal energies the areas under the elastic and inelastic force displacement plots are equal. This means that the ratio of the inelastic force to the elastic force R is given by

$$R = \frac{Inelastic\ force}{Elastic\ force} = \frac{1}{\sqrt{2\mu - 1}}$$

And the ratio of the inelastic structure displacement to the elastic structure displacement is given by

$$\frac{\Delta u}{\Delta e} = \frac{\mu}{\sqrt{2\mu - 1}}$$

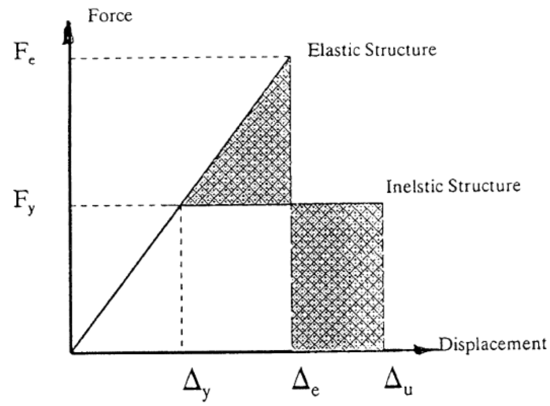


Fig.4 Equal Energy Concept

These principles have been adopted in the modifications of elastic response spectra for application to ductile structures.

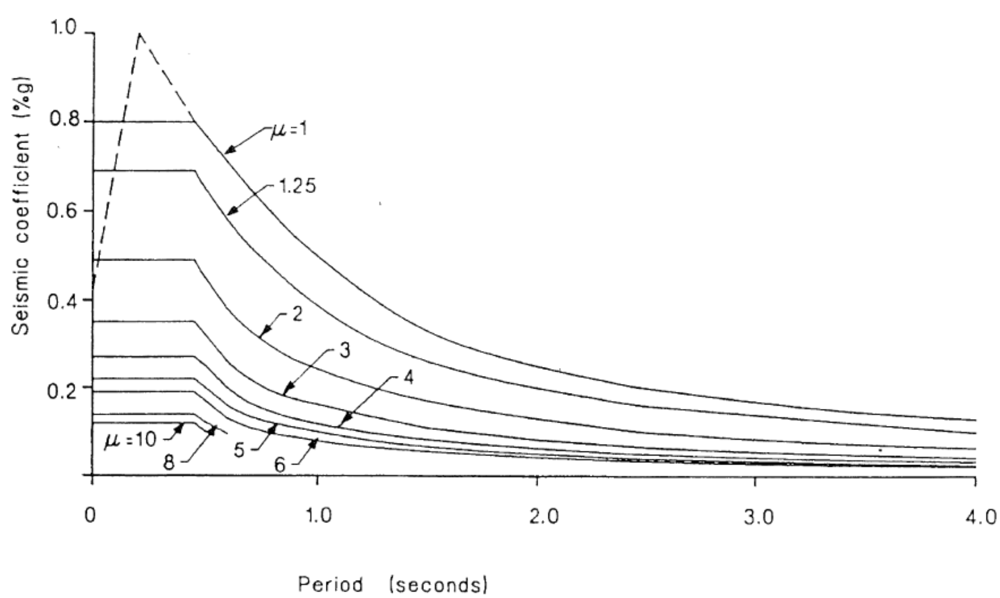


Fig.5 Basic seismic hazard acceleration coefficient for intermediate soil sites

However, in IS 1893:2002 a generalized plot of (S_a/g) with time T_a is given. Ductility is taken into account by division from response reduction factor (R).

'R' depends on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations.

1.2. DEFINITION OF DUCTILITY

1.2.1. CODAL PROVISION

The performance criteria implicit in most earthquake code provisions require that a structure be able to:

a) Resist earthquakes of minor intensity *(<DBE, as per IS 1893:2002)* without damage. A structure would be expected to resist such frequent but minor shocks within its elastic range of stresses.
b) Resist moderate earthquakes with minor structural and some non-structural damage with proper design and construction. It is believed that structural damage due to the majority of earthquakes will be limited to repairable damage.
c) Resist major catastrophic earthquake without collapse. Severe structural damage is expected.

NOTE: IS 1893:2002 call moderate earthquake as Design Basis Earthquake (DBE) and major catastrophic earthquake as Maximum Considered Earthquake (MCE).

<u>As per Cl.6.1.3 (IS 1893:2002)</u>

Actual forces that appear on structure during earthquakes are much greater than the design forces specified in this standard. However, ductility arising from inelastic material behavior and detailing and over strength arising from the additional reserve strength in structures over and above the design strength are relied upon to account for this difference in actual and design lateral loads.

1.2.2. HOW TO ENSURE DUCTILITY?

Ductility is ensured in two ways:-

1. Introduction of Response Reduction Factor (R) for the determination of seismic forces (as per *Cl.6.4.1 of IS 1893:2002*)
2. As per provisions of:
   a) *IS 4326:1993* Earthquake Resistant Design and Construction of Buildings – Code of Practice
   b) *IS 13920:1993* Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces.

1.2.3. DUCTILITY OF BEAMS

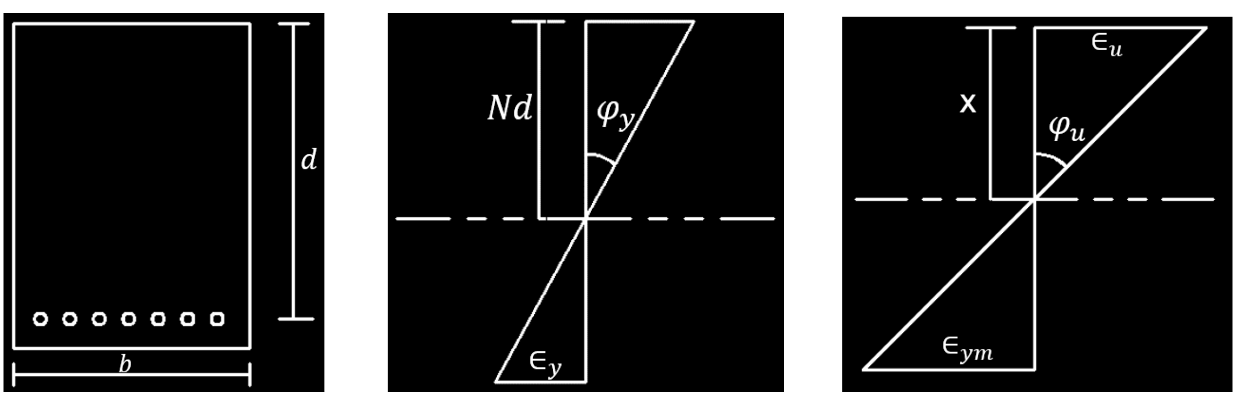
The ductility of reinforced concrete beams may be defined in terms of the behavior of individual cross-section or the behavior of entire beam. The former definition is more widely used because the behavior of cross-section is much better defined and it is easier to compute.

The ductility of a beam is often expressed in terms of curvature ductility

$$\mu = \frac{\varphi_u}{\varphi_y} \ (with\ respect\ to\ curvature)$$

where,

ϕ (Curvature) = 1/R



(a) (b) (c)

Fig.6 Rectangular Beam Cross-Section

Yield curvature using elastic theory

$$\phi_y = \frac{\epsilon_y}{d - Nd}$$

where,

ε_y = yield strain of the tensile reinforcement = σ_y/E_s

d = effective depth

Nd = depth of neutral axis computed using elastic theory

$$N = -mp + \sqrt[2]{(mp)^2 + 2mp}$$

m = modular ratio = $280/3\sigma_{cbc}$

p = tensile steel ratio = A_{st}/bd

Ultimate Curvature (ϕ_u) is calculated as per IS 456:2000 recommendations, i.e. adopting Limit state Method.

$$\phi_u = \frac{\epsilon_u}{x}$$

$$x = \frac{0.87\ \sigma_y\ A_{st}}{0.36\ \sigma_{ck}\ b} \leq x_m$$

So,

$$\mu = \frac{\epsilon_u}{\sigma_y/E_s}\left[\frac{1 + mp - \sqrt[2]{(mp)^2 + 2mp}}{x/d}\right]$$

NOTE:- $\mu = 1/(\sigma_y)^2$

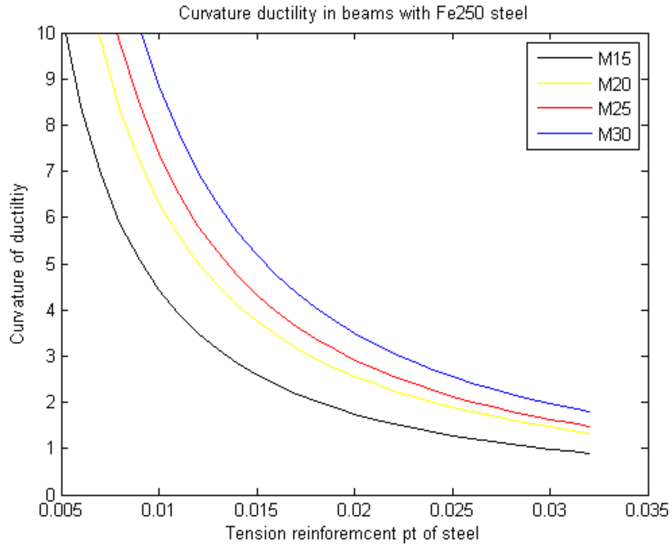


Fig. 7 Curvature of Ductility vs % tension Steel

In the case of a Doubly Reinforced Beam, a similar expression for ductility curvature can be obtained.

$$(p - p_c) \leq \left[\frac{\epsilon_u}{\epsilon_u + \mu_s \epsilon_y}\right] \frac{0.36\, \sigma_{ck}}{0.87\, \sigma_y}$$

where,

μ_s = strain ductility in steel

p_c = % of compression reinforcement

NOTE: The addition of compression reinforcement to a beam has relatively little effect on its yield curvature. It does, however, greatly increase the ultimate curvature.

With the increase in ratio of tension steel, ductility decreases. This is one of the reasons for limiting maximum reinforcement.

The maximum area of tension reinforcement shall not exceed 0.04bD. (Cl.26.5.1.1 IS 456:2000)

1.2.4. VARIABLES AFFECTING THE DUCTILITY

a) TENSION STEEL RATIO, p:

The ductility of a beam cross-section increases as the steel ratio p or (p-p_c) decreases. If excessive reinforcement is provided, the concrete will crush before the steel yields, leading to a brittle failure corresponding to $\mu = 1.0$.

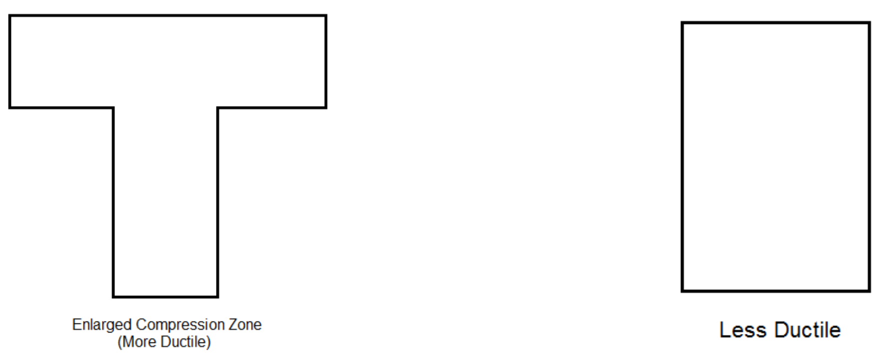
A beam should be designed as under reinforced.

b) COMPRESSION STEEL RATIO, (p-p_c):

Ductility increases with the decrease in (p-pc) value, i.e., ductility increases with the increase in compression steel.

c) SHAPE OF CROSS-SECTION

The presence of an enlarged compression Flange in a T-Beam reduces the depth of the compression zone at collapse and thus increases the ductility.



(a) (b)

Fig. 8 Shape of Cross-Section

d) LATERAL REINFORCEMENT

Lateral Reinforcement tends to improve ductility by preventing premature shear failures, restraining the compression reinforcement against buckling and by confining the compression zones thus increasing deformation capability of a reinforced concrete beam.

NOTE: Ductility increases with the increase in characteristic strength of concrete and decreases with the increase in characteristic strength of steel. (Fe 250 is more ductile than Fe450 or Fe500).

1.3. DESIGN RESPONSE SPECTRUM

As per clause 6.4.6 of IS 1893:2002

"In case design spectrum is specifically prepared for a structure at a particular project site, the same may be used for design at the discretion of the project authorities."

The code does not give any guidelines as hot to construct the design spectra from the response spectrum.

Here are the steps to be followed in order to construct design spectrum.

1.3.1.RESPONSE SPECTRUM

A plot of peak value of a response quantity as a function of the natural vibration period T_n of the system, or a related parameter such as circular frequency ω_n or cyclic frequency f_n, is called the response spectrum for that quantity . Each such plot is for SDF systems having a fixed damping ratio ζ, and several such plots for different values of ζ are included to cover the range of damping values encountered in actual structures.

Construction of response spectrum:

The response spectrum for a given ground motion component $\ddot{U}_g(t)$ can be developed by implementation of the following steps:

1. Numerically define the ground acceleration $\ddot{U}_g(t)$; typically, the ground motion ordinates are defined every 0.02 sec.
2. Select the natural vibration period T_n and damping ratio ζ of an SDF system. [use ζ = 0.05 to be use as per IS: 1893 2002 to meet other requirements]
3. As the ground motion acceleration history is known, then the equation of motion may be re-written as

$$m\ddot{x} + c\dot{x} + kx = -m\ddot{x}_g(t)$$

Substituting for c and k and dividing through by M gives

$$\ddot{x} + 2\lambda\omega\dot{x}+\omega^2 = -\ddot{x}_g(t)$$

$\lambda = \zeta$ (fraction of critical damping)

The deformation response u(t) of this SDF system due to the ground motion $\ddot{U}_g(t)$ is computed by Duhamel's integral

$$x(t) = \frac{-1}{\omega} \int_0^t \ddot{x}_g \, e^{-\lambda\omega(t-\tau)} \sin \omega_D(t-\tau)dt$$

where ,

$\omega_D = \omega\sqrt{1-\lambda^2}$ = damped circular frequency

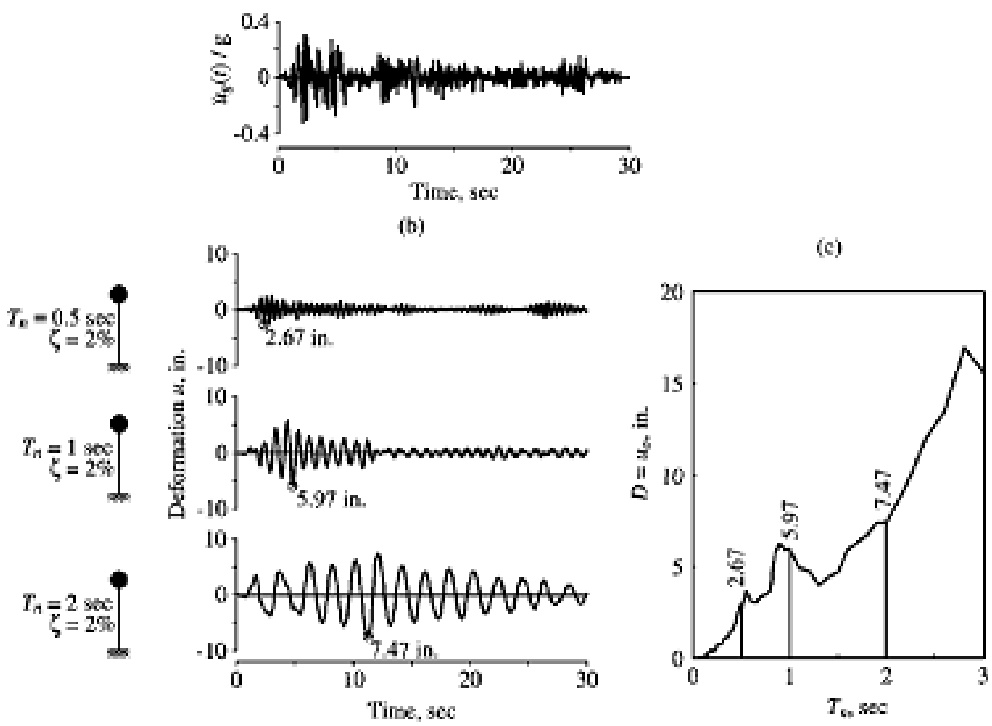


Fig.9 (a) Ground acceleration; (b) Deformation response of three SDF systems; (c) Deformation response spectrum for ξ=2%

4. Determine u_o, the peak value of u(t).
5. The spectral ordinates are $D=u_o$, $V=(2\pi/T_n)$, and $A = \omega^2_n D$

6. Repeat steps 2 to 5 for a range of T_n and ζ values covering all possible systems of engineering interest.
7. Present the results of steps 2 to 6 graphically to produce three separate spectra like fig a. or a combined spectrum like fig b.

The peak occurs during ground shaking; however, for lightly damped systems with very long periods the peak response may occur during the free vibration phase after the ground shaking has stopped. u is obtained from Duhamel's integral.

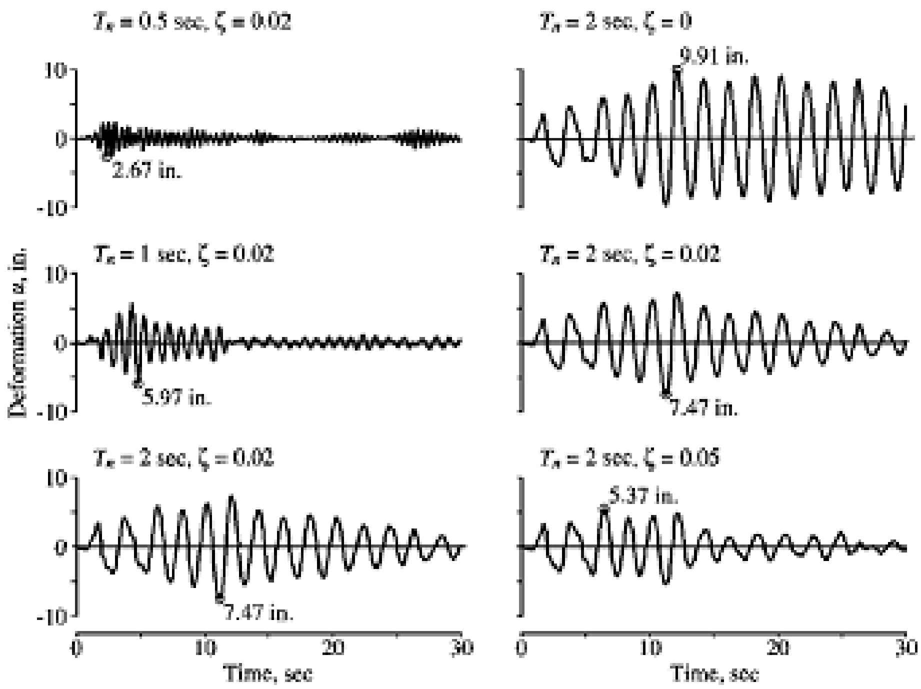


Fig.10 Deformation response of SDF systems to El Centro ground motion

We observe :

a. Longer the vibration period, the greater the peak deformation.
b. T_n of the three systems is the same, their responses display a similarity in the time required to complete a vibration cycle and in the times the maxima and minima occur.

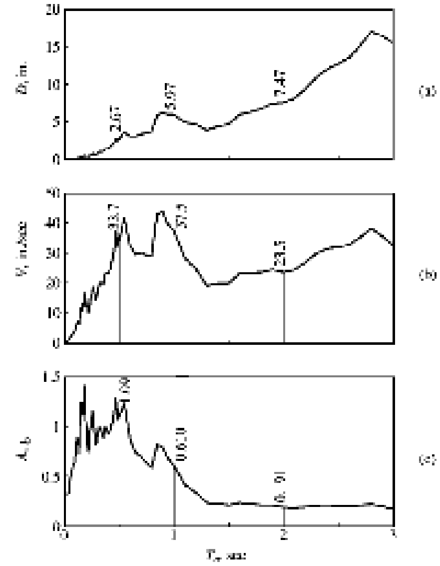


Fig. 11 (a) Displacement; (b) Psuedo-velocity; (c) Acceleration vs Natural Time Period of Structure

Consider the following peak responses:

$$u_o(T_n, \zeta) \equiv \max_t |u(t, T_n, \zeta)|$$

$$\dot{u}_o(T_n, \zeta) \equiv \max_t |\dot{u}(t, T_n, \zeta)|$$

$$\ddot{u}^t_o(T_n, \zeta) \equiv \max_t |\ddot{u}^t(t, T_n, \zeta)|$$

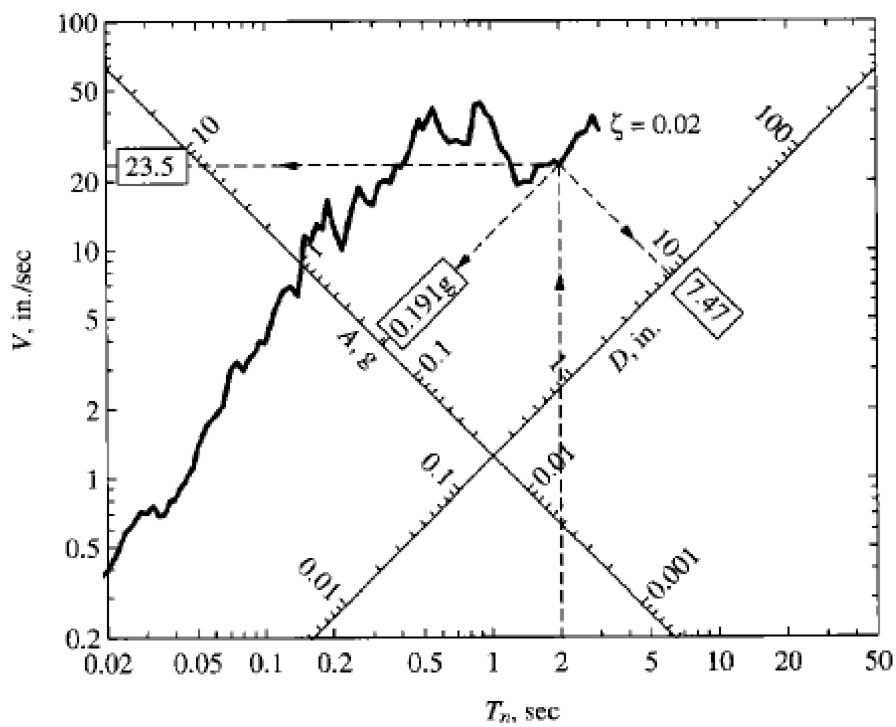


Fig.12 Combined D-V-A response spectrum for El Centro ground motion; ξ=2%

1.3.1.1. Elastic Design Spectrum

The response spectrum for a ground motion recorded during a past earthquake is inappropriate for the design of new structures. The jaggedness in the response spectrum is characteristic of that one excitation. The response spectrum for another ground motion recorded at the same site during a different earthquake is also jagged, but the peaks and valleys are not necessarily at the same periods. Similarly, it is not possible to predict the jagged response spectrum in all its detail for a ground motion that may occur in the future. Thus the design spectrum should consist of a set of smooth curves or a series of straight lines with one curve for each level of damping.

Procedure:

1. Plot the three dashed lines corresponding to the peak values of ground acceleration $\ddot{u_{go}}$, velocity $\dot{u_{go}}$ and displacement u_{go} for the design ground motion.
2. Obtain from the table, the values of α_A , α_V and α_D for the ζ selected.
3. Multiply $\ddot{u_{go}}$ by the amplification factor α_A to obtain the straight line b-c representing a constant value of pseudo – acceleration A.
4. Multiply $\dot{u_{go}}$ by α_V to obtain the straight line c-d representing a constant value of pseudo-velocity V.

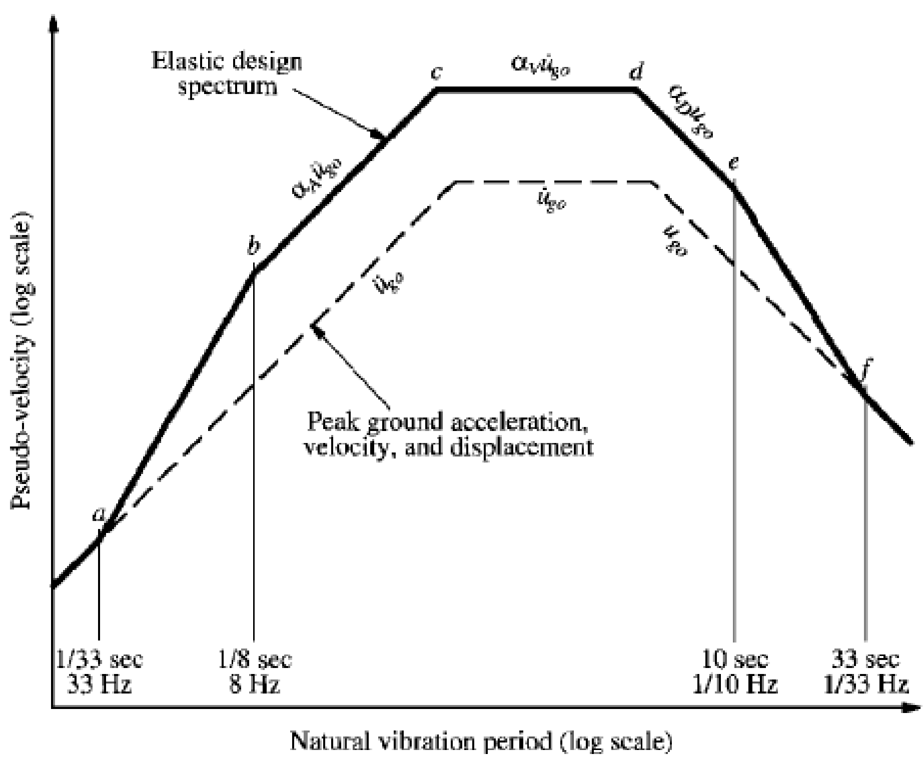


Fig.13 Elastic Design Spectrum

5. Multiply u_{go} by the amplification factor α_D to obtain the straight line d-e representing a constant value of deformation D.
6. Draw the line A=$\ddot{u}_{go}$ for periods shorter than T_a and the line D= u_{go} for periods longer than T_f.
7. The transition line a-b and e-f complete the spectrum.

AMPLIFICATION FACTORS: ELASTIC DESIGN SPECTRA

Damping, ζ (%)	Median (50 percentile)			One Sigma (84.1 percentile)		
	α_A	α_V	α_D	α_A	α_V	α_D
1	3.21	2.31	1.82	4.38	3.38	2.73
2	2.74	2.03	1.63	3.66	2.92	2.42
5	2.12	1.65	1.59	2.71	2.30	2.01
10	1.64	1.37	1.20	1.99	1.84	1.69
20	1.17	1.08	1.01	1.26	1.37	1.38

1.4. RESPONSE SPECTRUM METHOD

1.4.1. FREE VIBRATION ANALYSIS

We have to perform undamped free vibration analysis to obtain natural periods (T) and mode shapes {ϕ} of those of its modes of vibration.

1.4.2. UNDAMPED FREE VIBRATION

- No external excitation, first initial displacement or initial velocity.
- No damping mechanics.

Equations of dynamic of mass at each floor.

$m_1\ddot{x}_1 + K_1(x_1-x_0) - K_2(x_2-x_1) = f_1(t)$

$m_2\ddot{x}_2 + K_2(x_2-x_1) - K_3(x_3-x_2) = f_2(t)$

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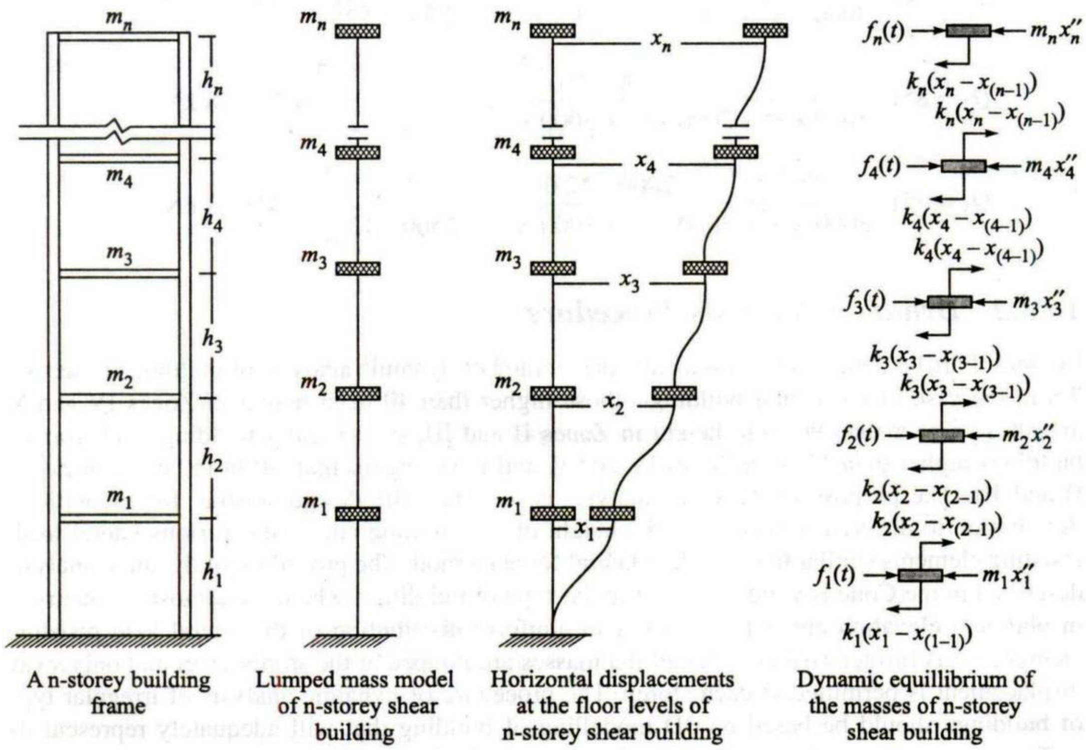


Fig.14 MDF idealization of n-storey building

$m_n\ddot{x}_n + K_n(x_n - x_{n-1}) = f_n(t)$

In matrix form

$$\begin{pmatrix} m_1 & 0 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 & 0 \\ 0 & 0 & m_3 & 0 & 0 \\ & & \cdot & & \\ & & \cdot & & \\ & & \cdot & & \\ 0 & 0 & 0 & 0 & m_n \end{pmatrix} \begin{bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \\ \ddot{x}_3 \\ \cdot \\ \cdot \\ \cdot \\ \ddot{x}_n \end{bmatrix} \quad +$$

$$
\begin{pmatrix}
K1+K2 & -K2 & 0 & 0 \\
-K2 & K2+K3 & -K3 & 0 \\
0 & -K3 & 0 & 0 \\
 & \cdot & & \\
 & \cdot & & \\
 & \cdot & & \\
0 & 0 & 0 & Kn
\end{pmatrix}
\begin{Bmatrix}
x_1 \\ x_2 \\ x_3 \\ \cdot \\ \cdot \\ \cdot \\ x_n
\end{Bmatrix}
=
\begin{Bmatrix}
F1 \\ F2 \\ F3 \\ \cdot \\ \cdot \\ \cdot \\ Fn
\end{Bmatrix}
$$

F = 0 (Free Vibration)

$M\ddot{x} + Kx = 0$

The stiffness influence coefficient K_{ij} is the force in direction of DOF I, caused by a unit displacement imposed in the direction of DOF j, while displacement in the direction of all other DOFs are kept equal to zero. The matrix of all stiffness influence coefficients K is called the **stiffness matrix**.

The mass influence coefficient m_{ij} is the force in the direction of DOF j, while acceleration in the direction of all other DOFs are kept equal to zero. The matrix of all mass influence coefficients M is called the **mass matrix**.

The displacement vector can be expressed as:

$x(t) = \phi \cos(\omega t - \theta)$

$\ddot{x}(t) = -\omega^2\cos(\omega t - \theta) = -\omega^2u(t)$

Substituting above, we get

$K\phi = \omega^2M\phi$

is called the generalized Eigen value problem.

COMMENT: A physical interpretation of equation is that a mode shape is a displacement configuration of the structure for which the elastic forces $K\phi$ are in exact equilibrium with the inertial forces $\omega^2 M\phi$.

$$(K - \omega^2 M)\phi = (K - \lambda M)\phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \mathit{Characteristic\ Equation}$$

To have non trivial solutions of ϕ

det $(K-\omega^2 M) = 0$ is called the characteristic equation.

Expanding this determinant, we obtain a polynomial equation of degree n in $\lambda_j = \omega_j^2$ for a system with n DOFs. The n roots of this equation ($\lambda_1, \lambda_2, \ldots\ldots\ldots\ldots \lambda_j, \lambda_n$) are the angular frequencies of the system and are associated with a vibration modal vector or mode shape.

$$T = \begin{pmatrix} T_1 & & & & \\ & T_2 & & & \\ & & \cdot & & \\ & & & \cdot & \\ & & & & T_n \end{pmatrix} = \begin{pmatrix} 2\pi/\omega_1 & & & & \\ & 2\pi/\omega_2 & & & \\ & & \cdot & & \\ & & & \cdot & \\ & & & & 2\pi/\omega_n \end{pmatrix}$$

For these ω is the Eigen values, we substitute it in characteristic equation to get ϕ called Eigen vector or modal vector or mode shape.

$\{\phi\} = \{\phi_1, \phi_2, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \phi_i \ldots\ldots.. \phi_n\}$

For calculating Eigen vector [mode shape], we need to put individual row equal to zero. But, this cannot be done as it always lead to more unknowns than equations.

As no external force, displacement of the structure results from initial given conditions i.e. <u>velocity and initial displacement at a particular storey level (DOF).</u>

So, one can determine relative rather than absolute displacements as initial displacement is perfectly arbitrary. We assume

$x_1 = 1$ i.e. displacement at 1st storey or 1st DOF as unity.

Interestingly, the shape of each mode is unique but the amplitude is undefined. So, the mode shape is generally normalized.

NOTE:

1. Normalization is conversion into a unit vector.
2. Largest term in the vector 1.
3. Sum of squares of the term in the vector is 1.

Vectors are normalized so the generalized mass M* is 1.0

$$M^* = \{\phi_i\}^T [M] \{\phi_i] = 1 \quad \Rightarrow \quad \{X\} = \frac{\{\phi_i\}}{\sqrt{M^*}}$$

The modal shape vector are orthogonal.

The equation of motion for the n DOF system is:

$$m\ddot{x} + kx = 0$$

the solution is assumed to be

$$x = \hat{x}\, sin\omega t$$

where $\hat{x}$ represents the vibration shape of the system.

$$\ddot{x} = -\omega^2 \hat{x}\, sin\omega t$$

therefore, we get

$$k\hat{x} - \omega^2 m\hat{x} = 0$$

Let us consider $\widehat{x_n}$, the nth modal shape vector

$$k\widehat{x_n} - \omega_n^2 m\widehat{x_n} = 0$$

By pre multiplying the above equation by the transpose of the mth modal-shape vector, $\widehat{x_m}$, we obtain

$$\widehat{x_m^T} K\widehat{x_n} - \omega_n^2 \widehat{x_m^T} m\widehat{x_n} = 0 \qquad (1)$$

Interchanging ‘m’ and ‘n’

$$\widehat{x_n^T} K \widehat{x_m} - \omega_m^2 \widehat{x_n^T} m \widehat{x_m} = 0 \tag{2}$$

Considering the symmetrical characteristics of matrices m and k

$$\widehat{x_m^T} k \widehat{x_n} = \widehat{x_n^T} k \widehat{x_m}$$

$$\widehat{x_m^T} m \widehat{x_n} = \widehat{x_n^T} m \widehat{x_m}$$

Subtracting Eq (1) from Eq (2), we get

$$(\omega_n^2 - \omega_m^2) \widehat{x_n} m \widehat{x_m} = 0$$

with the condition that ω_n^2-$\omega_m^2 \neq 0$ ($m \neq n$) then,

$$\widehat{x_n^T} m \widehat{x_m} = 0 \qquad (m \neq n)$$

Another form of expression is

$$\sum m_i \widehat{x_{\iota n}} \widehat{x_{\iota m}} = 0$$

This equation indicates that the two modal-shape vectors, $\widehat{x_n}$ and $\widehat{x_m}$, are orthogonal with respect to the mass matrix, m.

$$\widehat{x_n^T} k \widehat{x_m} = 0 \; with \; n \neq m$$

Thus the modal-shape vectors also are orthogonal to each other with respect to the stiffness matrix, k. The mode shape orthogonality properties imply that the work carried out by the inertia and elastic forces for mode I on displacements of mode j is equal to zero i.e. equations are uncoupled.

Let us assume modal-shape matric is given by

$$X = \phi Y$$

An N-DOF system contains n individual modal shapes. Arbitrary displacements, X, of the system can be expressed as the sum of the nth modal-shape vector ϕ_n multiplied by the amplitude Y_n.

$$X = \sum_{n=1}^{N} \phi_n Y_n$$

The vector Y is called the general co-ordinate vector or the normal co-ordinate of the system.

Pre multiplying above equation by $\phi_n^T m$ and considering the orthogonality condition, the amplitude corresponding to the nth modal shape, Y_n ca be derived as

$$\phi_n^T m x = \phi_n^T m \phi_n Y_n$$

$$Y_n = \frac{\sum_{i=1}^{N} m_i\, \phi_{in}^T x_i}{\sum_{i=1}^{N} m_i\, \phi_{in}^2}$$

When the equation for forced vibration is to be solved with respect to normal co-ordinates, the right side of the equation also must be expressed with respect to their co-ordinates.

$$m\ddot{x} + c\dot{x} + kx = -mI\ddot{x}_g$$

First, a unit vector I is decomposed to

$$I = \sum_{n=1}^{N} \phi_n\, \beta_n$$

Pre multiplying by $\phi_n T_m$

$$\phi_n^T m1 = \phi_n^T m \phi \beta$$

$$\beta = \frac{\phi_n^T m1}{\phi_n^T m \phi_n} = \frac{\sum_{i=1}^{N} m_i\, \phi_{in}}{\sum_{i=1}^{N} m_i\, \phi_{in}^2}$$

β_n represents the relative participation of the nth modal shape in the entire vibration of the system called Earthquake-participation factor.

IS 1893:2002, calls earthquake-participation factor as modal participation factor and denote it by p_k. According to clause 7.8.4.5(b),

$$p_k = \frac{\sum_{i=1}^{n} W_i\, \phi_{ik}}{\sum_{i=1}^{n} W_i \left(\phi_{ik}\right)^2}$$

W_i = Seismic weight of the ith floor.

ϕ_{ik} = Mode Shape coefficient = Displacement of ith floor in kth mode of vibration

NOTE:

A. Code replaces ‘i DOF’ with ‘i floor’.
B. Magnitude of p_k gets decreased as the mode no. increases. It is a function of the mode shape, the mass distribution of the structure and the direction of the earthquake excitation.
C. p_k can be negative, 0, and positive.
D. If the vibration components of the mode shape are orthogonal to the direction of the ground excitation, the p_k for that mode is zero.

SOURCE OF CONFUSION

p_k is dependent on the normalization method used in computing the mode shapes of free-vibration. So, p_k factors obtained from different computer programs should not be compared unless they both use the same method of normalization.

Number of modes to be used (Cl. 7.8.4.5(a) of IS 1893:2002) (Effective mass of each mode)

The use of those modes in analysis should be taken whose cumulative mass gives at least 90% of the total seismic weight.

Modal mass in Kth mode is determined as per

$$M_k = \frac{\left(\sum_{i=1}^{n} W_i \phi_{ik}\right)^2}{g \sum_{i=1}^{n} W_i \left(\phi_{ik}\right)^2}$$

NZS 4203:1992 recommends at least 3 modes to be considered irrespective of modal mass combination.

Determination of lateral force at each floor in each mode:

$$Q_{ik} = A_{nk} p_k \phi_{ik} W_i$$

Q_{ik} = design lateral force at floor ‘i’ in mode ‘k’

A_{nk} = design horizontal acceleration spectrum as per cl. 6.4.2 (c) time used for calculation of S_a/g is natural period of that mode (T_k)

p_k = modal participation factor in kth mode.

ϕ_{ik} = mode coefficient at ith floor in kth mode.

W_i = seismic weight of the ith floor.

$$[Qi1] = \begin{pmatrix} Ah1 & p1 & \phi 11 & W1 \\ Ah1 & p1 & \phi 11 & W1 \\ Ah1 & p1 & \phi 11 & W1 \\ . & . & . & . \\ . & . & . & . \end{pmatrix}$$

1.5. DETERMINATION OF STOREY SHEAR FORCES IN EACH MODE

The peak shear force (V_{ik}) acting in storey I in mode k is given by

$$V_{ik} = \sum_{j=i+1}^{n} Q_{ik}$$

$$V_{ik} = \sum_{j=i+1}^{n} Q_{ik} = \begin{pmatrix} V11 \\ V21 \\ V31 \\ V41 \end{pmatrix} = \begin{pmatrix} \sum_1^4 Q_{i1} \\ \sum_2^4 Q_{i1} \\ . \\ . \end{pmatrix}$$

Storey Shear Forces due to all modes considered -

The peak storey shear force (V_i) in storey I due to all modes considered is obtained by combining those due to cash mode in accordance with 7.8.4.4.

NOTE: According to cl. 7.8.4.5 (f) Lateral forces at each storey due to all modes considered – The design lateral forces, F_{roof} and F_i, at roof and at floor i:

$$F_{roof} = V_{roof} \qquad \text{and}$$

$$F_i = V_i - V_{i+1}$$

1.6. STIFFNESS INFLUENCE COEFFICIENTS

By definition, the stiffness influence coefficient k_{ij} if the force along DOF; due to a unit displacement imposed along DOF j, while keeping all other displacements equal to zero.

If we know the fixed-end reactions of trusses and beams caused by unit displacements imposed at any one support in a given direction, we can easily determine the coefficients k_{ij}.

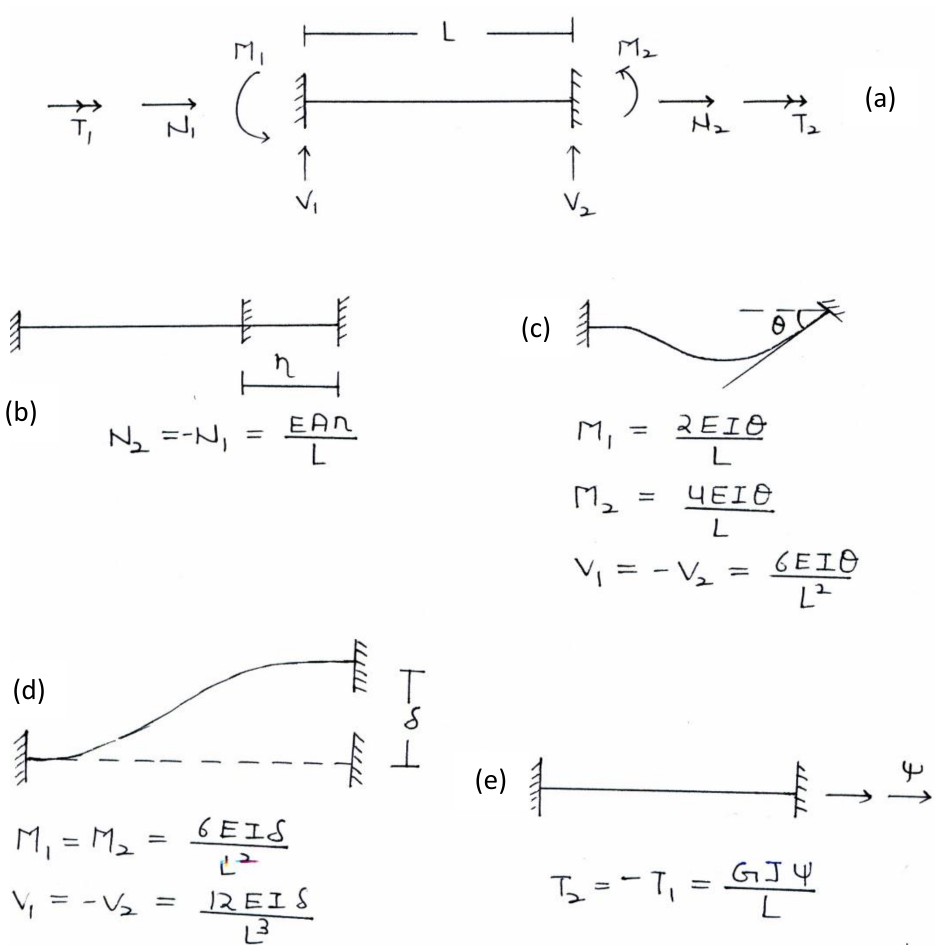


Fig. 15 Stiffness Coefficients

Fixed-end reactions caused by displacements of the supports:

a) Positive direction of support reactions and displacements
b) Longitudinal displacement x of the support 2
c) Rotation θ of support 2
d) Transverse displacement δ of support 2 and
e) Torsional rotation ϕ of support 2

1.7. ANALYSIS OF INFILLED FRAMES

The presence of infill affects the distribution of lateral load in the frames of building because of the increase of stiffness of some of the frames. The most common approximation of infilled walls is on the basic of equivalent diagonal strut i.e. the system is modelled as a braced frame and infill walls as web element. This is achieved by finding the effective width for the equivalent diagonal strut.

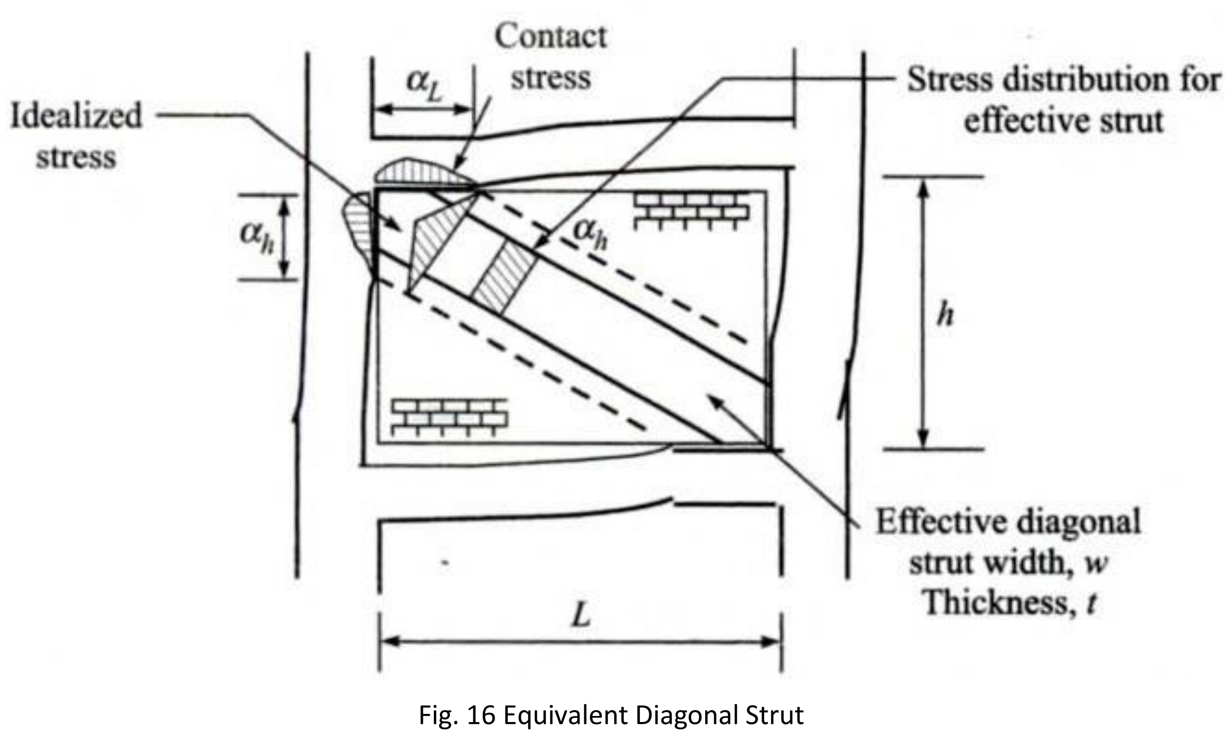


Fig. 16 Equivalent Diagonal Strut

α_h = length of contact between the wall and the columns

$$= \frac{\pi}{2} \sqrt[4]{\frac{4E_f I_c h}{E_m t \sin 2\theta}}$$

α_L = length of contact between the wall and the beam

$$= \pi \sqrt[4]{\frac{4E_f I_b L}{E_m t \sin 2\theta}}$$

where,

E_m and E_f = Elastic material of the masonry wall and frame material respectively

t,h,L = Thickness, height, length of the infill wall respectively

I_c, I_b = Moment of inertia of the column and the beam of the frame respectively

$\theta = \tan^{-1} h/L$

The equivalent or effective strut width W, where the strut is assumed to be uniform compressive stress

$$W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_L^2}$$

NOTE: NZS 4230 specifies a width equal to one quarter of its length.

Stiffness of infill panel is-

$$K_p = \frac{AE_m cos^2\theta}{L_d}$$

A = Cross-sectional area of diagonal stiffness = Wxt

L_d = Diagonal length of strut = $\sqrt{h^2 + l^2}$

NOTE: One may expect an initial lateral stiffness of the infilled frame 5 to 40 times of the respective base frame.

The stiffness of a building idealized as a shear building is always larger than the actual stiffness because additional constraints on the rotations of the node are introduced. This leads to an overestimation of the frequencies which are proportional to the stiffness.

1.8. STRUCTURES WITH MORE THAN A SINGLE MASS

The ground is assumed to move initially to the right. The base of the structure moves with the ground but the upper part of the structure has yet not followed the lower part. It takes time for the shear forces, caused by the deformation of the structure, to decelerate the masses of the upper floors.

Speed of wave propagation depends on stiffness of floors and floor masses.

When the direction of Base (ground) acceleration reverses, upper part of structures was still following the previous wave motion. This forms the complex pattern of the structure movement.

The inertial forces are derived from Newton's Law and are proportional to acceleration ($\ddot{x}$).

1.8.1. DEGREE OF FREEDOM

- The number of variables required to uniquely define the inertia forces or the displacements.
- In real life structure infinite DOF is present. But, for mathematical modelling we assumed mass of elements (structure).
- It can be reduced as a function of position (co-ordinate) with respect to a point generally joint (node).

e.g.: In Beam and Column, Lateral Displacement along length is a function (cubic variation with distance from nodes). It is helpful in computational modelling.

1.9. COMBINATION OF MODAL MAXIMA

Response Spectra Calculations have lost all information on sign or when the maximum displacement etc. occurred. Therefore, proper combinations of modes is not possible. In each mode structure members are in equilibrium and all actions in members have the appropriate of signs. However, what contribution or sign other modes should have the same time are unknown.

Let R be the modal quantity (Base shear, Nodal displacement, Nodal force, Member stress etc.). Values of R_i have been found for significant modes.

$R_{max} = \sum_{i=1}^{N} R_{i\,max}$ would be true only if all maxima occurred at the same instant of time and all had the same sign.

In general, $R_{max} \leq \sum_{i=1}^{N} R_{i\,max}$ and in almost all cases the inequality holds.

There are several accepted statistical combination methods.

a) *Maximum Possible Response:* Sum of Absolutes; this is very conservative and is very seldom used except for two or three modes for very short period structures.

$$R_{max} = \sum_{i=1}^{N} |R_{i\,max}|$$

Cl. 7.8.4.4 (b) of IS 1893:2002

b) *Maximum Likely Response:* Square Root of Sum of Squares or Root Sum Square.

$$R_{max} = \sqrt{\sum_{i=1}^{N} R_{i\,max}^{2}}$$

The Root Sum Square method was initially used when two-dimensional structural analyses were the norm.

In a two-dimensional structure no two lateral frequencies are close and so no storey correlation between modal responses is likely.

The peak response quantity due to the closely spaced modes (from (a)) is combined with those of the remaining well-separated modes by the SRSS method.

In three-dimensional structure it is likely that natural frequencies of modes in one translational direction will be similar to the natural frequencies of modes in the orthogonal translational direction or to the natural frequencies of the torsional modes.

CQC (Complete Quadratic Combination)

A number of methods are available. All the modal combination methods may be expressed in the form-

$$R_{max} = \sqrt{\sum_{i} \sum_{j} R_{i\,max} \rho_{ij} R_{i\,max}}$$

ρ_{ij} = correlation coefficient = cross-modal coefficient

The equation for the correlation coefficient due to Der Kiureghian is

$$\rho_{ij} = \frac{8\sqrt{\zeta_i\zeta_j}(\beta_{ij}\zeta_i + \zeta_j)\beta_{ij}^{3/2}}{(1-\beta_{ij}^2)^2 + 4\zeta_i\zeta_j\beta_{ij}\left(1+\beta_{ij}^2\right) + 4(\zeta_i^2 + \zeta_j^2)\beta_{ij}^2}$$

where, $\beta_{ij} = \omega_j/\omega_i$

Cl 7.8.4.4 of IS Code 1893:2002 has adopted this equation with assumption $\zeta_i=\zeta_j=\zeta$.

$$\rho_{ij} = \frac{8\,\zeta^2\left(1+\beta_{ij}\right)\beta_{ij}^{1.5}}{(1+\beta^2)^2+4\zeta^2\,\beta(1+\beta)^2}$$

1.10. TORSION

If the structure has symmetry in both stiffness and mass with respect to two orthogonal horizontal axes of the structure, then the mode shapes will uncouple the motion in the two horizontal axis direction and it may be possible to speak of , say x, and y direction mode shapes as well as torsional modes.

If these symmetries are not present and eccentricity of mass and/or stiffness is to be considered, then such a directional uncoupling is unlikely.

This means that any translation of the structure in an axis direction will involve translation in the orthogonal direction and also rotation about the vertical axis.

In, design even though the structure may appear to be symmetrical, this is not guaranteed by the materials and in the yield properties.

This is one reason for code requiring designers to make provision in the design for some torsion about the vertical axis.

As per Cl. 7.9.2

The design eccentricity, e_{di} to be used at floor i shall be taken as:

$$e_{di} = 1.5e_{si} + 0.05b_i \qquad \text{or}$$

$$e_{si} – 0.05b_i$$

Which of these gives the most severe effect in the shear of any frames where

e_{si} = static eccentricity at floor i defined as the distance between center of mass and center of rigidity ,

b_i = floor plan dimension of floor i , perpendicular to the direction of force

NOTE: The factor 1.5 represents dynamic amplification factor, while the factor 0.05 represents the extent of accidental eccentricity

The design forces calculated as in 7.8.4.5 are to be applied at the center of mass appropriately displaced at the design eccentricity (e_{di}).

Cl 7.9.3 states of IS 1893:2002 -

In case of highly irregular buildings analyzed according to 7.8.4.5, additive shears will be superimposed for a statistically applied eccentricity of $\pm$ 0.05 b_i with respect to the center of rigidity.

1.11. BASE ISOLATION

In foreword for IS 1893:2002

“Base isolation systems are found useful for short period structures , say less than 0.7 seconds including soil structure interaction”.

2. EQUIVALENT LINEARIZATION

2.1. OVERVIEW OF INELASTIC SEISMIC ANALYSIS PROCEDURES

The objective of inelastic seismic analysis procedures is to predict the expected behavior of the structure in future earthquake shaking.

The generic process of inelastic analysis is similar to conventional linear procedures in the engineer develops a model of the building or structure, which is then subjected to a representation of the anticipated seismic ground motion. The results of analysis are predictions of engineering demand parameters within the structural model that are subsequently used to determine performance based on acceptance criteria.

There are several basic inelastic analysis procedures that differ primarily on the types of structural models used for analysis and the alternatives for characterizing seismic ground shaking.

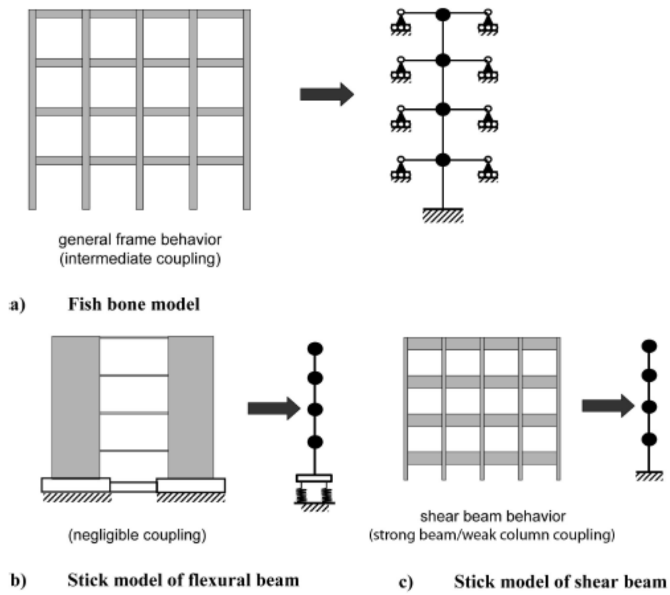
2.1.1. STRUCTURAL MODELLING

Detailed structural models for inelastic analysis are similar to liner elastic finite-element models. The primary difference is that the properties of some or all of the components of the model include post-elastic strength and deformation characteristics in addition to the inelastic properties.

In most instances with inelastic analysis, it is preferable to base the model on the best estimate of the expected properties of the structure.

While modelling engineers simply detailed structural models into equivalent multi-degree-of-freedom models. Some simplified models are:

a) Fish bone, intermediate coupling between floors is allowed.
b) Stick model, negligible rotational coupling among various vertical flexural elements
c) Shear mechanisms, simplest of all models. Beams/floor systems are rigid, factors like axial deformation of beams/columns neglected.



Forms of simplified equivalent multiple-degree-of-freedom models.

Fig. 17

2.2. CHARACTERIZATION OF SEISMIC GROUND MOTION

Ground motion records are used to define elastic response spectra, which compromise a relationship of the maximum response (acceleration, velocity, and displacement) over the entire response-history record of a single-degree-of-freedom oscillator and the frequency, or more commonly the period, of the oscillator, for a specified level of damping. Response spectral ordinates are commonly used to represent seismic demand for structural design.

Design Spectra have standardized shapes, and can be evaluated based on nationally mapped values of spectral accelerations for short and long periods.

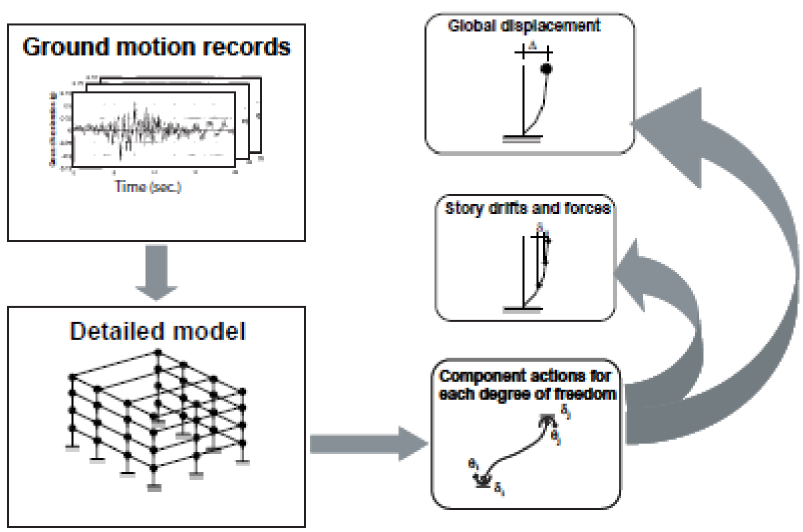
2.3. OPTIONS FOR INELASTIC ANALYSIS

Various combinations of structural model types and characterizations of seismic ground motion define a number of options for inelastic analysis. The selection of one option over another depends on the purpose of the analysis, the anticipated performance objectives, and the acceptable level of uncertainty, the availability of resources, and the sufficiency of data.

The primary decision is whether to choose inelastic procedures over more conventional linear elastic analysis. In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion of interest or when the design results in nearly uniform distribution of nonlinear response throughout the structure. In these cases, the level of uncertainty associated with linear procedures is relatively low. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and/or acceptability criteria to avoid unintended performance.

Inelastic procedures facilitate a better understanding of actual performance. This can lead to a design that focuses upon the critical aspects of the building, leading to more reliable and efficient solutions.

Nonlinear dynamic analysis using the combination of ground motion records with a detailed structural model theoretically is capable of producing results with relatively low uncertainty.



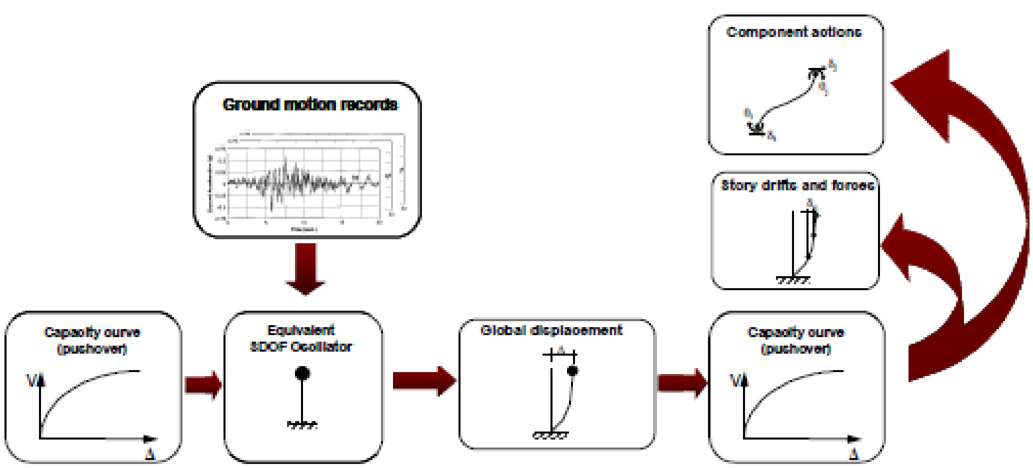
Flow chart depicting the nonlinear dynamic analysis process. Note that component actions are used to determine higher-level effects, such as story drifts and roof displacement, Δ.

Fig. 18 Nonlinear dynamic analysis process

In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model. Local as well as global demands derive directly from the basic component actions.

Simplified nonlinear dynamic analysis with equivalent multi-degree-of-freedom models also use *ground motion* records to characterize seismic demand. However, these techniques produce engineering demand parameters above the basic component level only. For example, a "stick" model produces story displacements or drifts. The engineer can estimate corresponding component actions using the assumptions that were originally the basis of the simplified model. Thus the uncertainty associated with the component actions in the simplified model is greater than those associated with the detailed mode.

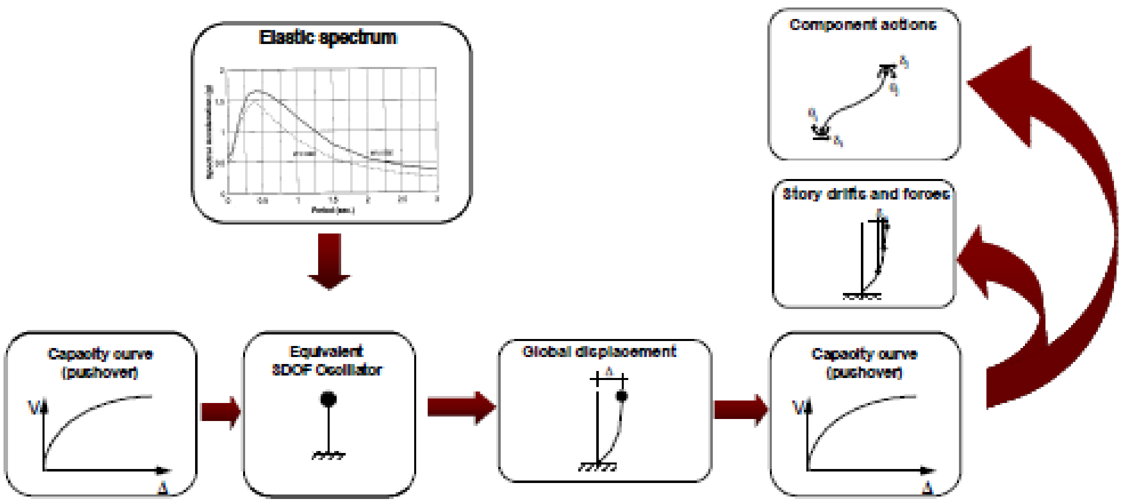
Simplified nonlinear dynamic analysis with equivalent single-degree-of-freedom (SDOF) models is a further simplification using *ground motion* records to characterize seismic shaking. The result of the analysis is an estimate of global displacement demand. It is important to recognize that the resulting lower-level engineering demands (e.g., story drifts, component actions) are calculated from the global displacement using the force-deformation relationship for the oscillator. In contrast to the use of the more detailed model, they are directly related to the assumptions, and associated uncertainties, made to convert the detailed structural model to an equivalent SDOF model in the first place. This adds further to the overall uncertainty associated with the simplified nonlinear dynamic analysis. If the SDOF model is subjected to multiple time histories a statistical representation of response can be generated.



Flow chart depicting simplified SDOF nonlinear analysis process. Note that component actions are estimated from global displacement demand using the pushover curve.

Fig. 19 SDOF Nonlinear dynamic analysis

Nonlinear static procedures (NSPs) convert MDOF models to equivalent SDOF structural models and represent *seismic ground motion with response spectra as opposed to ground-motion records*. They produce estimates of the maximum global displacement demand. Story drifts and component actions are related subsequently to the global demand parameter by the pushover or capacity curve that was used to generate the equivalent SDOF model.



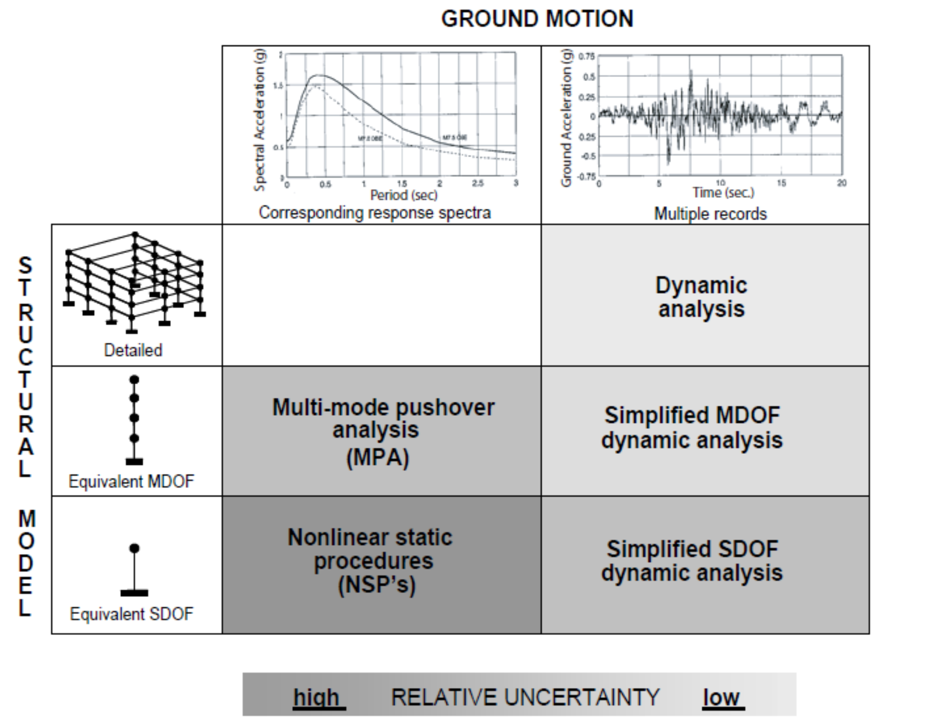
Flow chart depicting the process followed in nonlinear static procedures. Note that component actions are based on global displacement demand and a pushover/capacity curve.

Fig. 20 Nonlinear Static Procedures

Some analysis options are better than others, depending on the parameter of interest. For example, with simplified dynamic analyses, a SDOF oscillator can be subjected to a relatively large number of ground motion records to provide a good representation of the uncertainty associated with global displacement demand due to the variability of the ground motion.

On the other hand, if the engineer is comfortable with the estimate of maximum global displacement from a nonlinear static procedure, a multi-mode pushover analysis might provide improved estimates of inter-story drift that would not necessarily be available from the simplified SDOF dynamic analyses.

The below figure summarizes the relationship among the normal options for inelastic seismic analysis procedures with respect to the type of structural model and characterization of ground motion. Also noted in the figure is the relative uncertainty associated with each option.

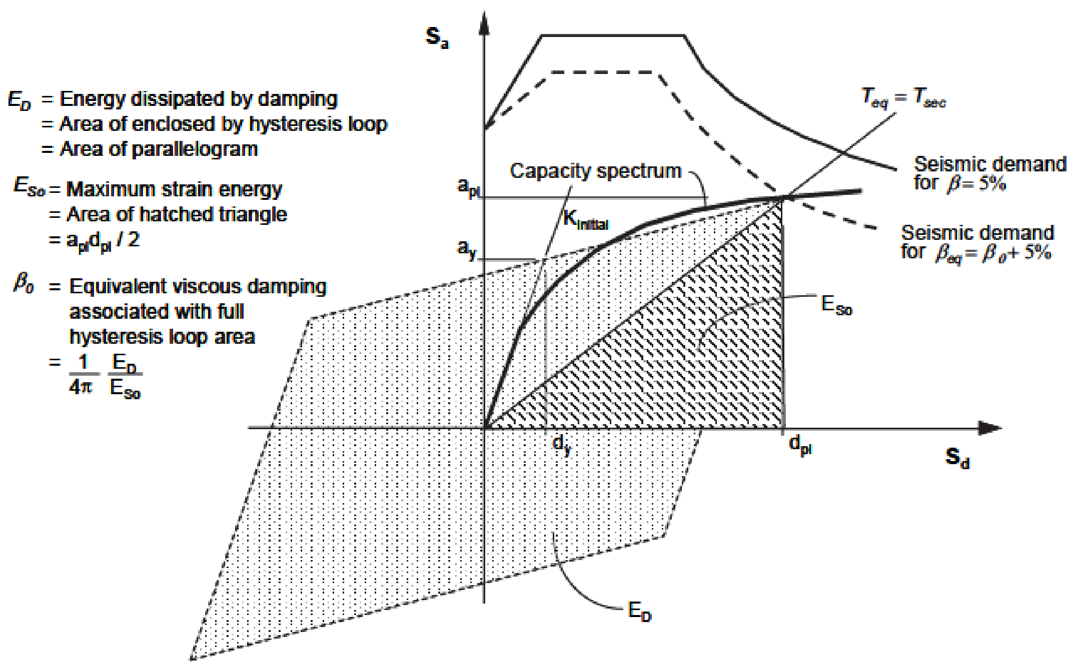


Matrix depicting possible inelastic seismic analysis procedures for various structural models and ground-

Fig. 21 Inelastic Seismic Analysis Procedures

2.4. CAPACITIES-SPECTRUM METHOD OF EQUIVALENT LINEARIZATION IN ATC-40

The basic assumption in equivalent linearization techniques is that the maximum inelastic deformation of a nonlinear SDOF system can be approximated from the maximum deformation of a linear elastic SDOF system that has a period and a damping ratio that are larger than the initial values of those for the nonlinear system. In the Capacity-Spectrum Method of ATC-40, the process begins with the generation of a force-deformation relationship for the structure. The results are plotted in acceleration- displacement response spectrum (ADRS) format. This format is a simple conversion of the base-shear-versus-roof-displacement relationship using the dynamic properties of the system, and the result is termed a capacity curve for the structure. The seismic ground motion is also converted to ADRS format. This enables the capacity curve to be plotted on the same axes as the seismic demand. In this format, period can be represented as radial lines emanating from the origin. The Capacity-Spectrum Method of equivalent linearization assumes that the equivalent damping of the system is proportional to the area enclosed by the capacity curve. The equivalent period, T_{eq}, is assumed to be the secant period at which the seismic ground motion demand, reduced for the equivalent damping, intersects the capacity curve. Since the equivalent period and damping are both a function of the displacement, the solution to determine the maximum inelastic displacement (i.e., performance point) is iterative. ATC-40 imposes limits on the equivalent damping to account for strength and stiffness degradation.



Graphical representation of the Capacity-Spectrum Method of equivalent linearization, as presented in ATC-40.

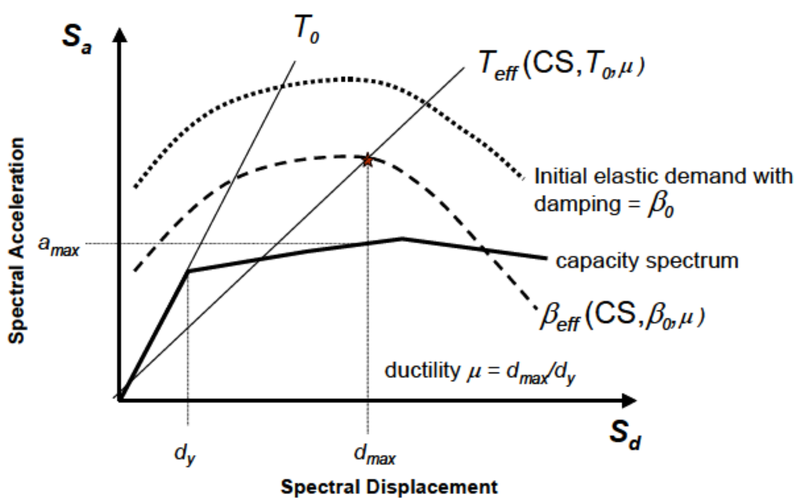
Fig. 22 Capacity-Spectrum Method

2.5. IMPROVED PROCEDURES FOR EQUIVALENT LINEARIZATION

2.5.1. INTRODUCTION

This section presents an improved equivalent linearization procedure as per modifications to the Capacity-Spectrum Method (CSM) of ATC-40.

When equivalent linearization is used as a part of a nonlinear static procedure that models the nonlinear response of a building with a SDOF oscillator, the objective is to estimate the maximum displacement response of the nonlinear system with an "equivalent" linear system using an effective period, T_{eff}, and effective damping, β_{eff}. The global force-deformation relationship for a SDOF oscillator in acceleration-displacement response spectrum (ADRS) format is termed a capacity curve. The capacity curve shown is developed using the conventional procedures of FEMA 356 or ATC-40. The effective linear parameters are functions of the characteristics of the capacity curve, the corresponding initial period and damping, and the ductility demand, μ.



Acceleration-displacement response spectrum (ADRS) showing effective period and damping paramete of equivalent linear system, along with a capacity curve.

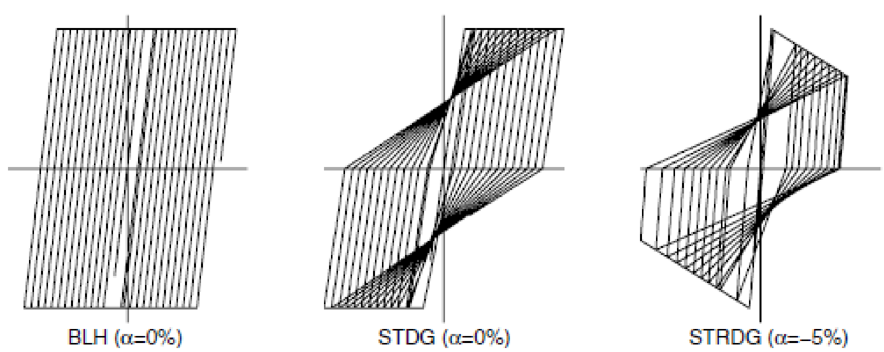
Fig. 23 Acceleration-Displacement response spectrum

FEMA 440 presents new expressions to determine effective period and effective damping. It also includes a technique to modify the resulting demand spectrum to coincide with the familiar CSM technique of using the intersection of the modified demand with the capacity curve to generate a performance point for the structural model. The reduction in the initial demand spectrum resulting from the effective damping may be determined using conventional techniques outlined in following Section. The previous limits on effective damping of ATC-40 should not be applied to the new procedures of FEMA 440. However, the user must recognize that the results are an estimate of median response and imply no factor of safety for structures that may exhibit poor performance and/or large uncertainty in behavior. The effective parameters for equivalent linearization are functions of ductility. Since ductility (the ratio of maximum displacement to yield displacement) is the object of the analysis, the solution must be found using iterative or graphical techniques.

2.5.2. BASIC EQUIVALENT LINEARIZATION PARAMETERS

Optimal equivalent linear parameters (i.e., effective period, $T_{eff,}$ and effective damping, β_{eff}) are determined through a statistical analysis that minimizes, in a rigorous manner, the extreme occurrences of the difference (i.e., error) between the maximum response of an actual inelastic system and its equivalent linear counterpart.

A variety of different inelastic hysteretic systems have been studied including bilinear hysteretic (BLH), stiffness- degrading (STDG), and strength-degrading behaviour. A negative value of the post-elastic stiffness ratio, α, is indicative of in-cycle degradation.



Types of inelastic behavior considered. BLH=Bilinear Hysteretic STDG=Stiffness Degrading, and STRDG=Strength Degrading.

Fig. 24 Type of Inelastic Behavior

2.5.3. EFFECTIVE DAMPING AND EFFECTIVE PERIOD

Equivalent periods and damping ratios consistent with target ductility ratio are rewritten in empirical equation. The coefficients A to L depend on hysteretic behaviour of a SDOF and its post yield stiffness ratio.

for $\mu < 4$:

$$\xi_{eff} = A(\mu-1)^2 + B(\mu-1)^3 + \xi$$

$$T_{eff} = \left[G(\mu-1)^2 + H(\mu-1)^3 + 1\right]T$$

for $4 \le \mu \le 6.5$:

$$\xi_{eff} = C + D(\mu-1) + \xi$$

$$T_{eff} = \left[I + J(\mu-1) + 1\right]T$$

for $\mu > 6.5$:

$$\xi_{eff} = E\left[\frac{F(\mu-1)-1}{F(\mu-1)^2}\right]\left(\frac{T_{eff}}{T}\right)^2 + \xi$$

$$T_{eff} = \left\{K\left[\sqrt{\frac{(\mu-1)}{1+L(\mu-2)}}-1\right]+1\right\}T$$

Values of the coefficients in the equations for effective damping of the model oscillators are tabulated below. Note that these are a function of the characteristics of the capacity curve for the oscillator in terms of basic hysteretic type and post-elastic stiffness, α.

Table-1

Coefficients for use in Equations for Effective Damping

Model	*α (%)*	*A*	*B*	*C*	*D*	*E*	*F*
Bilinear hysteretic	0	3.2	−0.66	11	0.12	19	0.73
Bilinear hysteretic	2	3.3	−0.64	9.4	1.1	19	0.42
Bilinear hysteretic	5	4.2	−0.83	10	1.6	22	0.40
Bilinear hysteretic	10	5.1	−1.1	12	1.6	24	0.36
Bilinear hysteretic	20	4.6	−0.99	12	1.1	25	0.37
Stiffness degrading	0	5.1	−1.1	12	1.4	20	0.62
Stiffness degrading	2	5.3	−1.2	11	1.6	20	0.51
Stiffness degrading	5	5.6	−1.3	10	1.8	20	0.38
Stiffness degrading	10	5.3	−1.2	9.2	1.9	21	0.37
Stiffness degrading	20	4.6	−1.0	9.6	1.3	23	0.34
Strength degrading	-3[a]	5.3	−1.2	14	0.69	24	0.90
Strength degrading	−5[a]	5.6	−1.3	14	0.61	22	0.90

Table-2

Coefficients for use in Equations for Effective Period

Model	α(%)	G	H	I	J	K	L
Bilinear hysteretic	0	0.11	–0.017	0.27	0.090	0.57	0.00
Bilinear hysteretic	2	0.10	–0.014	0.17	0.12	0.67	0.02
Bilinear hysteretic	5	0.11	–0.018	0.09	0.14	0.77	0.05
Bilinear hysteretic	10	0.13	–0.022	0.27	0.10	0.87	0.10
Bilinear hysteretic	20	0.10	–0.015	0.17	0.094	0.98	0.20
Stiffness degrading	0	0.17	–0.032	0.10	0.19	0.85	0.00
Stiffness degrading	2	0.18	–0.034	0.22	0.16	0.88	0.02
Stiffness degrading	5	0.18	–0.037	0.15	0.16	0.92	0.05
Stiffness degrading	10	0.17	–0.034	0.26	0.12	0.97	0.10
Stiffness degrading	20	0.13	–0.027	0.11	0.11	1.0	0.20
Strength degrading	–3[a]	0.18	–0.033	0.17	0.18	0.76	–0.03
Strength degrading	–5[a]	0.20	–0.038	0.25	0.17	0.71	–0.05

The coefficients A to L have been optimized to fit the empirical results for idealized model oscillators having well defined hysteretic behavior designated earlier in this document as Elastic Perfectly Plastic (EPP), Stiffness Degrading (SD) and Strength and Stiffness Degrading (SSD). Real buildings, comprised of a combination of many elements, each of which may have somewhat different strength and stiffness characteristics, will seldom display hysteretic behaviors that match those of the oscillators, exactly. Adaptation of these coefficients to building models with a number of components may be done with caution. If all components exhibit similar behavior (e.g., flexurally controlled concrete with stiffness degradation and strain hardening), then it is reasonable to infer that hysteretic behavior of the overall building will be similar to the behavior of the simple idealized oscillators on which this table is based. For building models in which components exhibit disparate force-deformation behavior, it is less clear which coefficients to use.

When in doubt, the practitioner should use the more generally equations that have been optimized for application to any capacity curve, independent of hysteretic model type or alpha value used for the study:

For $1.0 < \mu < 4.0$:

$$\beta_{\text{eff}} = 4.9(\mu-1)^2 - 1.1(\mu-1)^3 + \beta_0 \qquad T_{\text{eff}} = \left[G(\mu-1)^2 + H(\mu-1)^3 + 1\right]T_0$$

For $4.0 \le \mu \le 6.5$:

$$\beta_{\text{eff}} = 14.0 + 0.32(\mu-1) + \beta_0 \qquad T_{\text{eff}} = \left[I + J(\mu-1) + 1\right]T_0$$

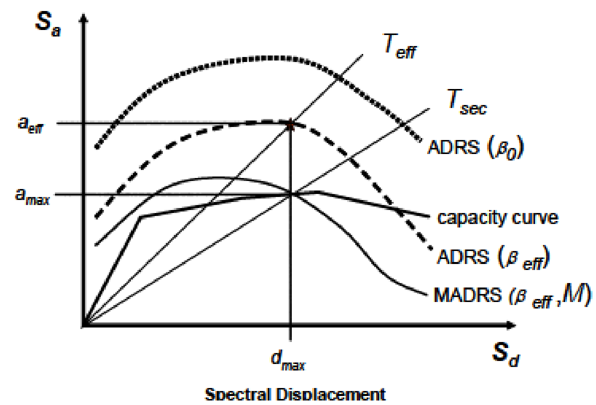
For $\mu > 6.5$:

$$\beta_{\text{eff}} = 19\left[\frac{0.64(\mu-1)-1}{\left[0.64(\mu-1)\right]^2}\right]\left(\frac{T_{\text{eff}}}{T_0}\right)^2 + \beta_0 \qquad T_{eff} = \left\{K\left[\sqrt{\frac{(\mu-1)}{1+L(\mu-2)}} - 1\right] + 1\right\}T_0$$

2.5.4. MADRS FOR USE WITH SECANT PERIOD

The conventional Capacity-Spectrum Method (ATC-40) uses the secant period as the effective linear period in determining the maximum displacement (performance point). This assumption results in the maximum displacement occurring at the intersection of the capacity curve for the structure and a demand curve for the effective damping in ADRS format. This feature is useful for two reasons. First, it provides the engineer with a visualization tool by facilitating a direct graphical comparison of capacity and demand. Second, there are very effective solution strategies for equivalent linearization that rely on a modified ADRS demand curve (MADRS) that intersects the capacity curve at the maximum displacement.

The use of the effective period and damping equations in above section generate a maximum displacement that coincides with the intersection of the radial effective period line and the ADRS demand for the effective damping. The effective period of the improved procedure, T_{eff}, is generally shorter than the secant period, T_{sec}, defined by the point on the capacity curve corresponding to the maximum displacement, d_{max}.



Modified acceleration-displacement response spectrum (MADRS) for use with secant period, T_{sec}.

Fig. 25 Modified acceleration-displacement response spectrum

The effective acceleration, a_{eff}, is not meaningful since the actual maximum acceleration, a_{max}, must lie on the capacity curve and coincide with the maximum displacement, d_{max}. Multiplying the ordinates of the ADRS demand corresponding to the effective damping, β_{eff}, by the modification factor

$$M = \frac{a_{\max}}{a_{\text{eff}}}$$

results in the modified ADRS demand curve (MADRS) that may now intersect the capacity curve at the performance point. Since the acceleration values are directly related to the corresponding periods, the modification factor can be calculated as:

$$M = \left(\frac{T_{\text{eff}}}{T_{\text{sec}}}\right)^2 = \left(\frac{T_{\text{eff}}}{T_0}\right)^2 \left(\frac{T_0}{T_{\text{sec}}}\right)^2$$

For the effective period

$$\left(\frac{T_0}{T_{\text{sec}}}\right)^2 = \frac{1+\alpha(\mu-1)}{\mu}$$

Where α is the post elastic stiffness.

3. FIBRE REINFORCED POLYMER (FRP)

3.1. INTRODUCTION

Composites are made with several components starting with man-made fibers such as glass, carbon or aramid. These fibers provide the strength and stiffness in a composite. The fibers are then combined with a polymer resin like polyester or epoxy. The resin protects the fibers from environmental attack and helps transfer the loads between the fibers.

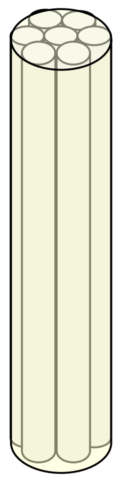


Fig. 26 Fibre Composite Matrix

The combination of the FIBER and the RESIN creates a material with attributes superior to either component alone and is critical in the performance of the composites material. Composites exhibit strength that is 5 TIMES stronger than steel at ¼ the weight, and offering corrosion resistance, and many other benefits.

The use of fiber-reinforced polymer (FRP) composites for the rehabilitation of beams and slabs started about 15 years ago and most of the work since then has focused on timber and reinforced concrete structures, although some steel structures have been renovated with FRP as well.

a) The high material cost of FRP might be a deterrent to its use, but upon a closer look, FRP can be quite competitive.

b) In addition to their resistance to corrosion, FRP have high ratios of strength and stiffness to density.
c) The light weight of FRP provides considerable cost savings in terms of labor: a worker can handle the FRP material, whereas a crane would be required for its steel equivalent.
d) FRP laminates and fabric come in great lengths, which can be cut to size in the field, as compared with welding of steel plates.
e) FRP laminates or fabric are thin, light and flexible enough to be inserted behind pipes, electrical cables, etc., further facilitating installation.
f) With heat curing, epoxy can reach its design strength in a matter of hours, resulting in rapid bonding of FRP to the structure and consequently, minimum disruption to its use.

The tensile strength of FRP can exceed 3000 MPa (compared to 400 MPa for reinforcing steel), and their stiffness ranges from slightly greater than that of steel for high-modulus carbon to about 1/3 that of steel for S-glass. FRP do not exhibit plastic yielding as steel does, however, and behave elastically up to an ultimate strain in the range of 1.5 % to *5* % (compared with a range of 15 % to 20 % for reinforcing steel). This brittle behavior must be accounted for in structural design.

The increase in strength and stiffness is sometimes realized at the expense of a loss in ductility, or capacity of the structure to deflect in elastically while sustaining a load close to its capacity.

3.2. WHERE SHOULD BE FRP REBAR USED?

a) Any concrete member susceptible to corrosion by chloride ions or chemicals.
b) Any concrete member requiring non-ferrous reinforcement due to Electro-magnetic considerations.
c) As an alternative to epoxy, galvanized, or steel bar.
d) Where machinery will “consume” the reinforced member i.e. Mining and tunneling.
e) Applications requiring Thermal non-conductivity.

3.3. TENSILE STRESS-STRAIN CHARACTERISTICS

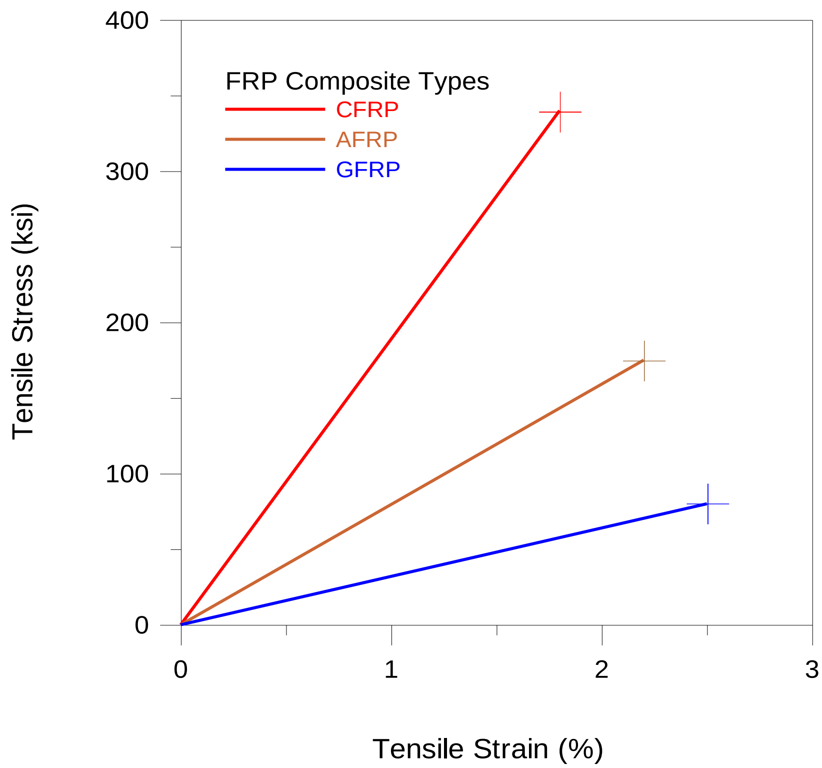


Fig. 27 Tensile Stress-Strain Characteristic

3.4. DEFINITION OF DUCTILITY FOR FRP

Ductility is a desirable structural property because it allows stress redistribution and provides warning of impending failure. Steel-reinforced concrete beams are under-reinforced by design, so that failure is initiated by yielding of the steel reinforcement, followed, after considerable deformation at no substantial loss of load carrying capacity, by concrete crushing and ultimate failure. This mode of failure is ductile and is guaranteed by designing the tensile reinforcement ratio to be substantially below the balanced ratio, which is the ratio at which steel yielding and concrete crushing occur simultaneously.

The reinforcement ratio thus provides a metric for ductility, and the ductility corresponding to the maximum allowable steel reinforcement ratio provides a measure of the minimum acceptable ductility.

The Canadian Highway Bridge Design Code (CHBDC, *2000), assesses the* ductility of FRP-strengthened sections with a performance factor equal to

$$Performance\ Factor = \frac{M_u \phi_u}{M_{0.001} \phi_{0.001}}$$

where,

M = beam moment

ϕ = curvature

subscripts u refer to the ultimate state and 0.001 to the service state that corresponds to a concrete maximum compressive strain of 0.001.

This performance factor must be greater than 4 for rectangular sections and greater than 6 for T-sections.

4. DYNAMIC ANALYSIS OF IRREGULAR BUILDING

4.1. MODEL PHYSICAL FEATURES

A five X four bay moment resisting bay frame with G+2 floors is chosen which is located in seismic zone (IV) (Shimla). The site soil condition is taken as hard soil owing to presence of rock strata in area. The length along x direction is 20 m with 4m being the length of each bay. The breath along z direction is 16 m with 4m being the breath of each bay. The total height of structure is 9.5 m with 3.5 m being the height of ground level and 3 m each the height of subsequent floors.

The plan of the structure possesses a challenge as it also contains irregularity. It contains diaphragm discontinuity as per Table 4 of IS 1893: 2002.

This requires a dynamic analysis of the building to be performed as the building falls in zone IV and contains irregularity. [Cl 7.8.1 IS 1893:2002]

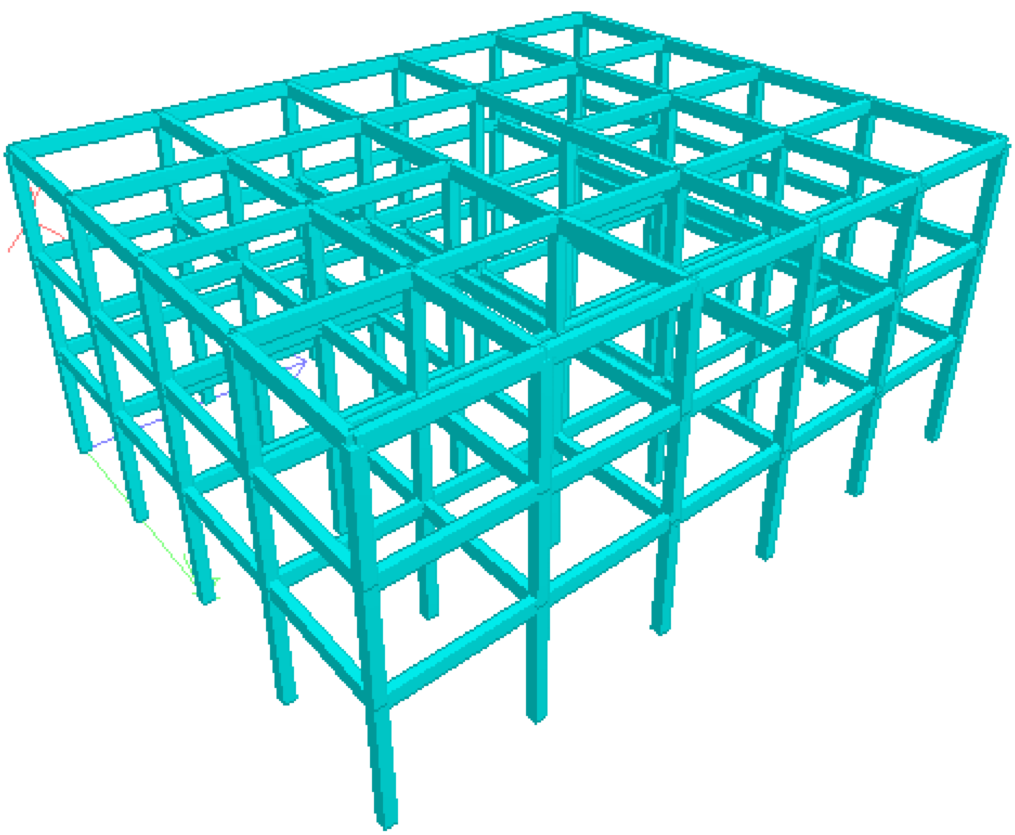
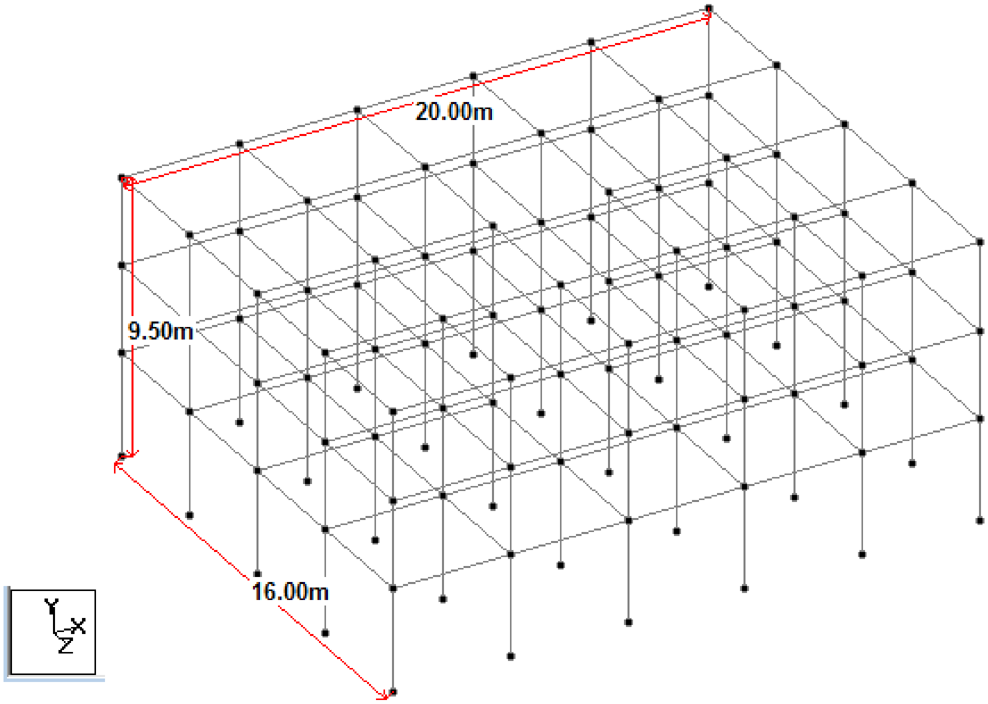
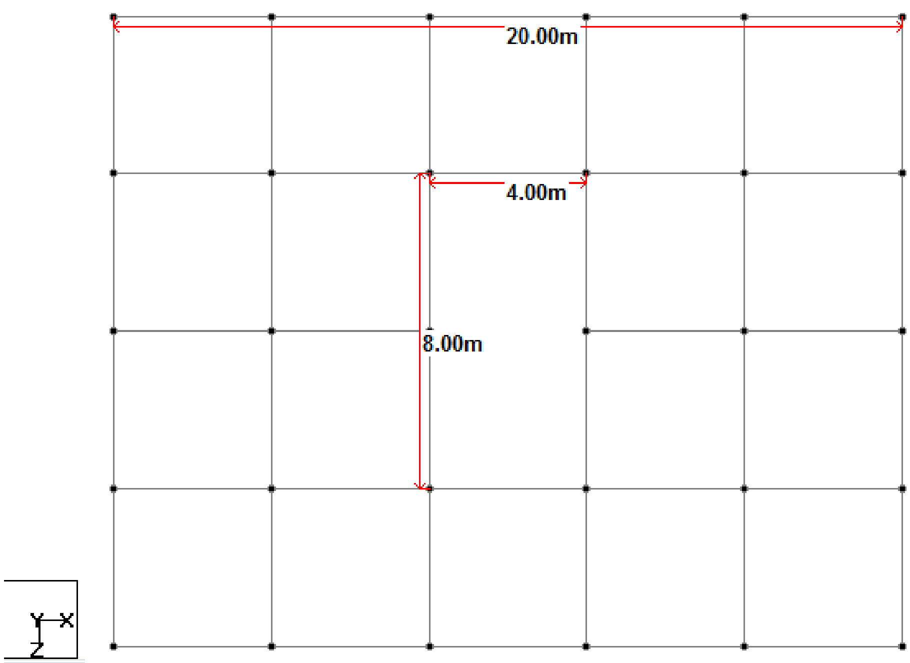


Fig.28 3D view of shopping complex



(a)



(b)

Fig.29 Top view of mall showing diaphragm discontinuity

4.2. DISCONTINUITIES

1. DIAPHRAGM DISCONTINUITY

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.

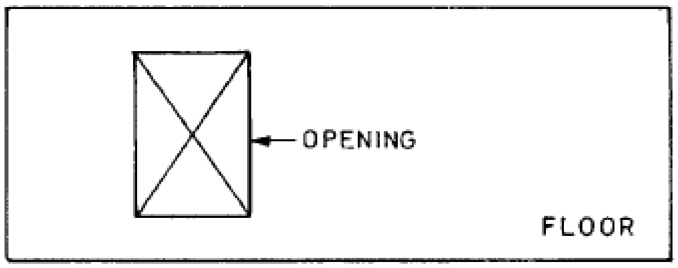


Fig. 30 Diaphragm Discontinuity

2. RE-ENTRANT CORNERS

Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction.

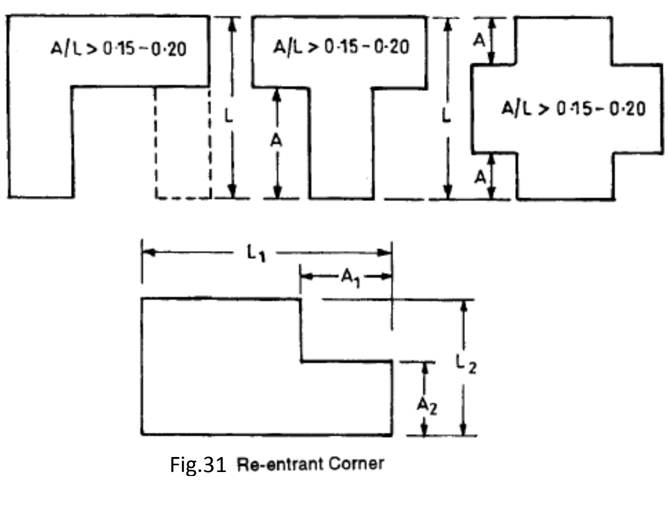


Fig.31 Re-entrant Corner

4.3. PARAMETERS USED

Seismic parameters:

Importance factor (I) is taken as 1 as building is used commercially as a shopping complex.

Response reduction factor (R) is taken as 5 corresponding to special moment resisting frame.

Time period along x axis = 0.19 sec

Time period along y axis = 0.21 sec

Load calculation:

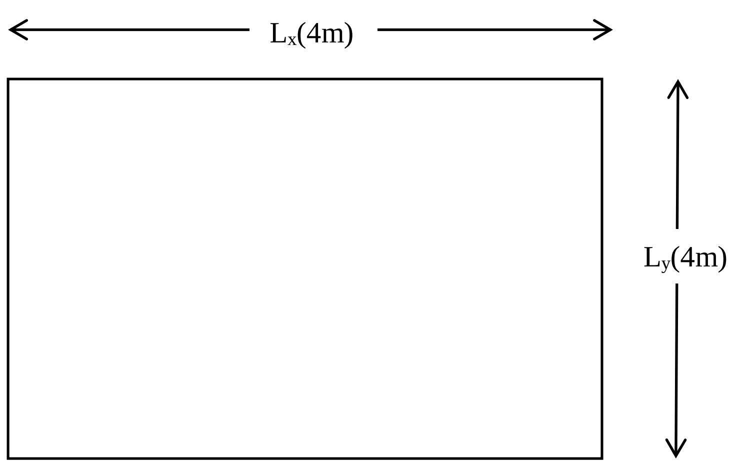
Load of brick wall per meter run = Unit weight of brick * height of wall * thickness of wall *1

= 22*3*.230*1 = 15.2 KN/m

Load of brick wall as partition wall = 4.5 KN/m

Live load on floor = 6 KN/m^2 [IS 875(PART 2) 1987]

Calculation of thickness of floor slab:



The dimensions of each bay are such that slab will be designed as a two way slab.

Depth is calculated using Limit state method:

Table3 Calculation of Slab Depth

	Ly(m)	Lx(m)	Units		
	4	4			
	Ly\Lx	1		(< 2) Two way slab	
Assume	l/d	23		Partial restraint	
	d =	173.913	mm		
Adopt	D	175 +25	mm	Clear Cover = 25 mm	
		= 200	mm		
	Dead Load	25*0.2			
		= 5	KN/m^2		
	Live Load	6	KN/m^2	IS 875(PART 2) 1987	
				Mercantile building	
	Floor finish	1	KN/m^2		
	Total load	12	KN/m^2		
	Design load	18	KN/m^2		
	Max Moment ($\alpha_x w l_x^2$)	13.536	KN-m		
	Grade of concrete	M20			
	Grade of Steel	Fe415			
	Mu_{lim}	$=0.138*20*1000*d^2$			
	d (required)	70.03	mm		
		< 175 (Assumed)	mm	OK	

Table 4 Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

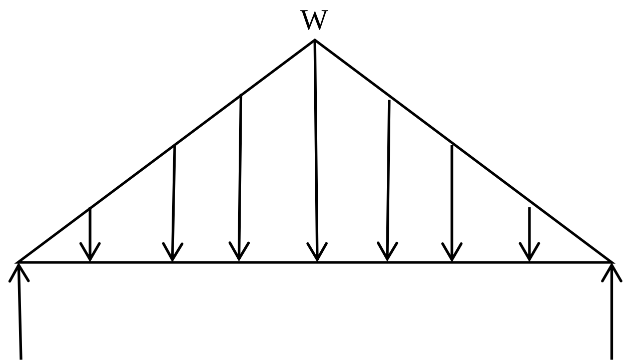
(*Clauses* D-1.1 *and* 24.4.1)

Case No.	Type of Panel and Moments Considered	Short Span Coefficients α_x (Values of l_y/l_x) 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long Span Coefficients α_y for All Values of l_y/l_x
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	*Interior Panels*:									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	*One Short Edge Continuous*:									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	*One Long Edge Discontinuous*:									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	*Two Adjacent Edges Discontinuous*:									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5	*Two Short Edges Discontinuous*:									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	—
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6	*Two Long Edges Discontinuous*:									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.045
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7	*Three Edges Discontinuous (One Long Edge Continuous)*:									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	—
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8	*Three Edges Discontinuous (One Short Edge Continuous)*:									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.057
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9	*Four Edges Discontinuous*:									
	Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

Table for bending moment coefficients α_x and α_y from IS 456:2000 Annexure D

Beam Depth calculation:

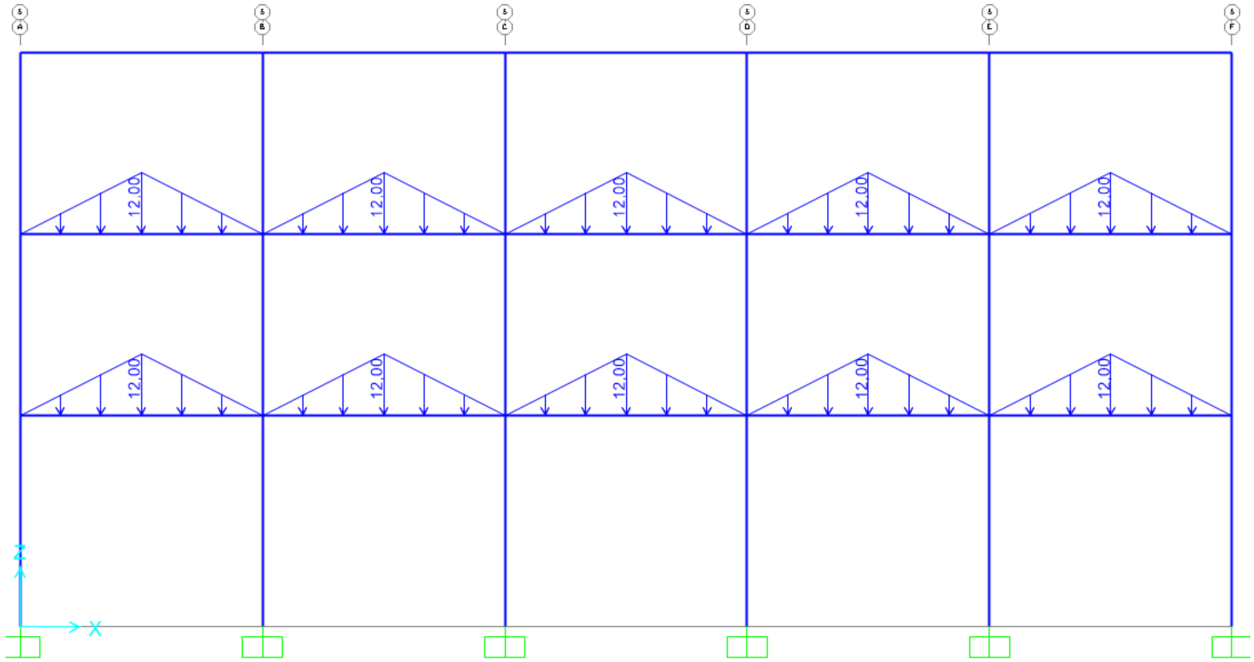
Table 5				
Total load on slab		=	18*4*4	kN
		=	288	kN
Load share of 1 triangle		=	288/4	kN
		=	72	kN



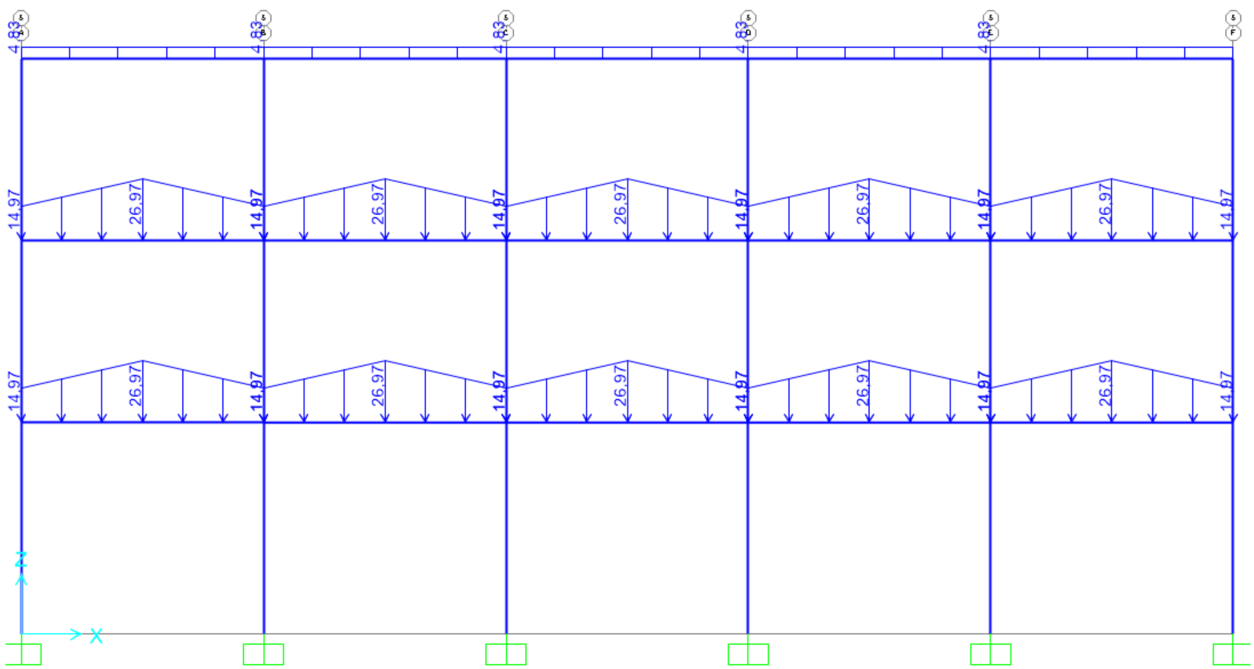
To find W:

Table 6 Load Calculation					
72	=	(1/2)*4*W			
W (Load 1)	=	36	KN/m		
Load on beam due to brick wall	=	18.85*3*.23*1		Unit weight of brick 18.85 KN/m^3	
	=	13.01	KN/m		
Load on beam due to plaster	=	(0.02+0.012)*3*20.4	KN/m		
(20 mm outer, 12 mm inner plaster)	=	1.96	KN/m	Unit weight of brick 20.4 KN/m^3	
Total UDL	=	14.97	KN/m		
Factored load (Load 2)	=	22.455	KN/m		
Maximum moment M_{max}	=	$\frac{wl^2}{12}+\frac{wl^2}{8}$			
	=	48 + 44.92			
	=	92.92	KN m		
Mu_{lim}	=	$.138*f_{ck}*b*d^2$			
$92.92*10^6$	=	$.138*20*230*d^2$			
d	=	382.6	mm		
Adopt D	=	400	mm		

Square column of dimension 300X300 mm is taken in complete building. The section is found safe as per IS 1893:2000. The verification is performed in Staad Pro concrete design command.



a. Live load



b. Dead load

Fig.32 Load on Structure

4.4. SEISMIC ANALYSIS

4.4.1. Types

We focused ourselves on the following here analysis-

- MODAL ANALYSIS: It is used to determine the vibration characteristics (natural frequencies and mode shapes) of a structure or a machine component while it is being designed. It also can be a starting point for another, more detailed, dynamic analysis, such as a transient dynamic analysis, a harmonic response analysis, or a spectrum analysis.

- P-DELTA ANALYSIS: In the **P-Δ** or **P-Delta** effect refers to the abrupt changes in ground, overturning and/or the axial distribution at the base of a sufficiently tall structure or structural component when it is subject to a critical lateral.

- RESPONSE SPECTRUM: Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of it is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

4.4.1.1.MODAL ANALYSIS

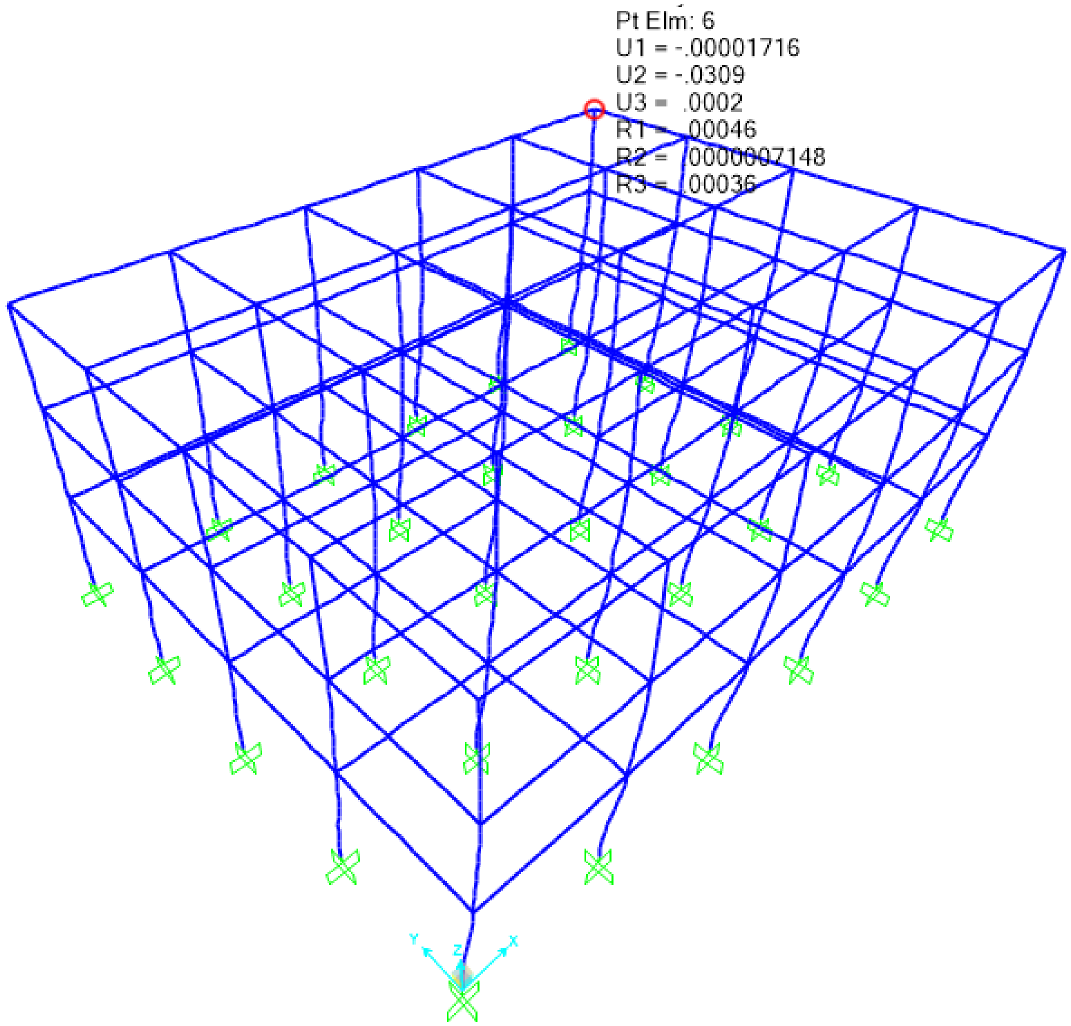
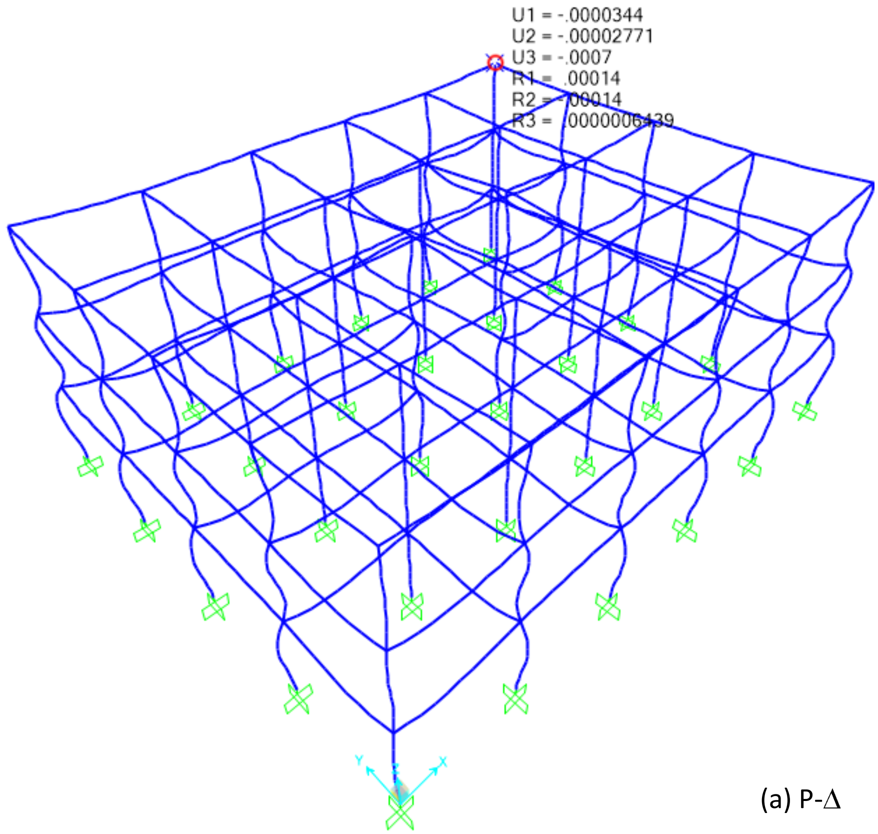


Fig.33 Deflection by Modal Analysis

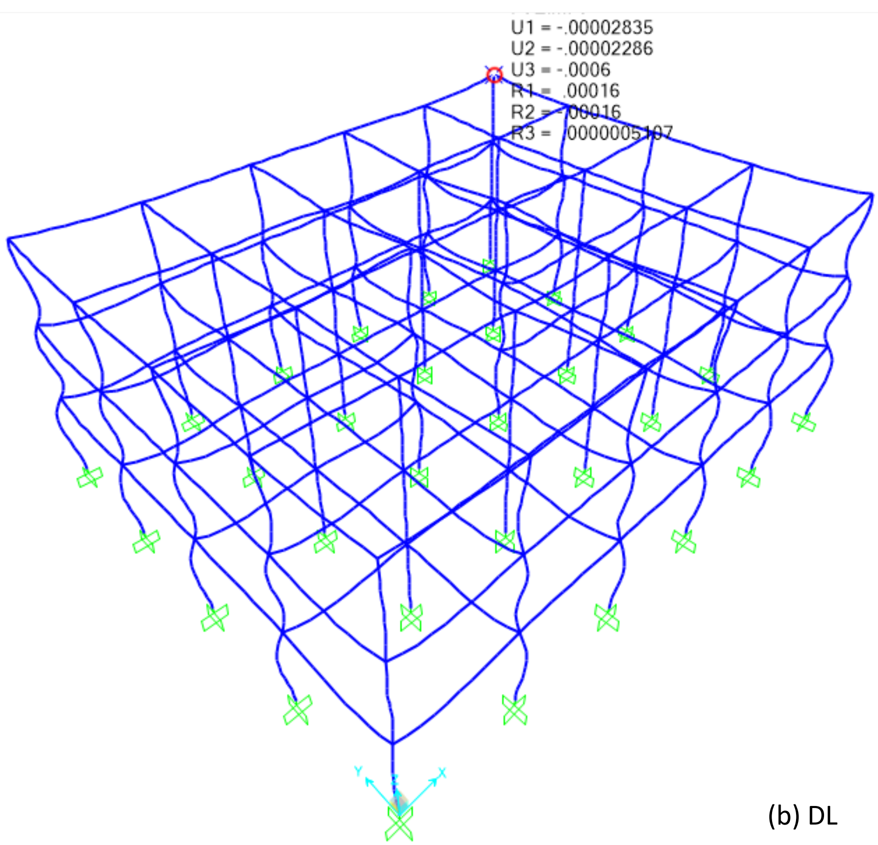
MODE NO.	TIME PERIOD (SEC)	FREQUENCY (CYC/SEC)
1	0.920598	1.0863
2	0.920436	1.0864
3	0.870629	1.1486
4	0.708701	1.411
5	0.589035	1.6977
6	0.521347	1.9181

Table 7 Time Period and Frequency of First 6 Modes

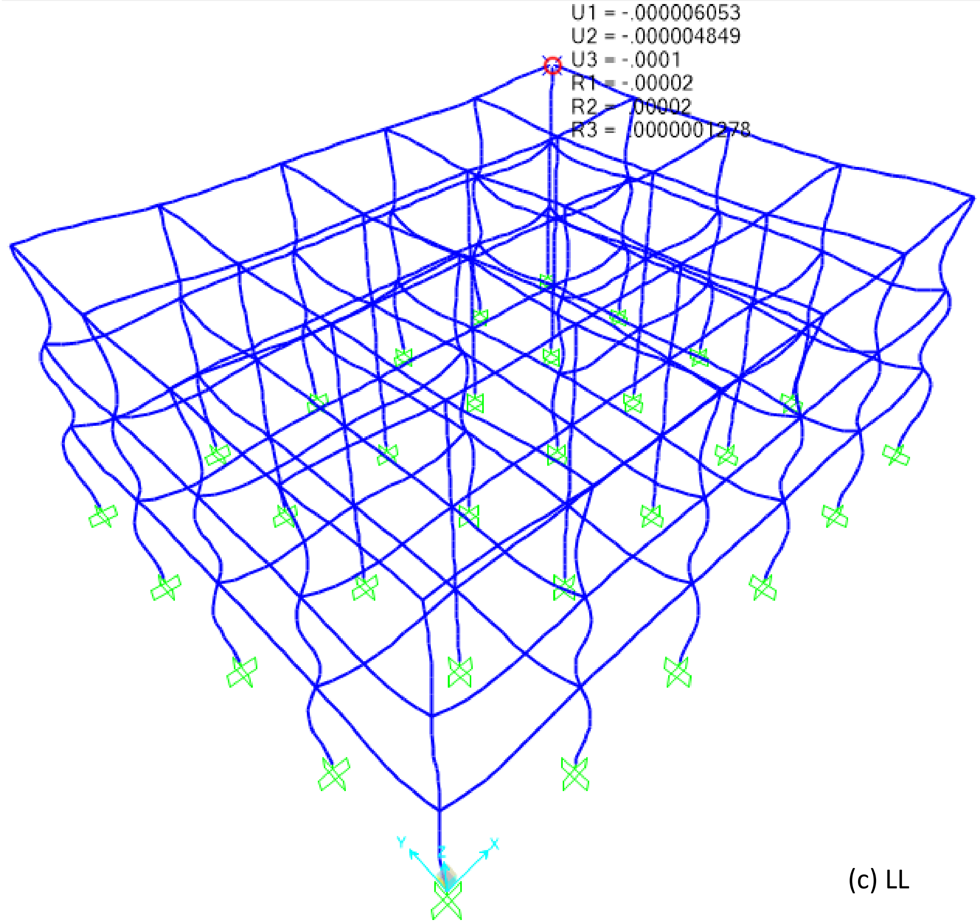
4.4.1.2.P-DELTA ANALYSIS



(a) P-Δ



(b) DL



(c) LL

Fig.34 Deflections in P-Δ Analysis

4.4.1.3.RESPONSE SPECTRUM

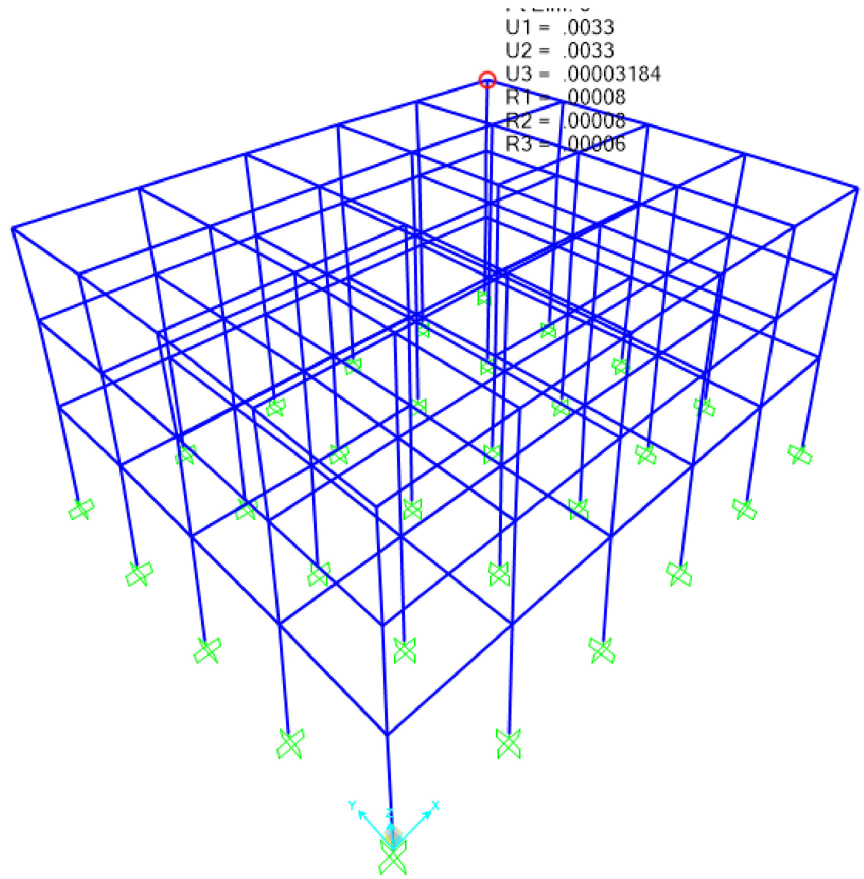
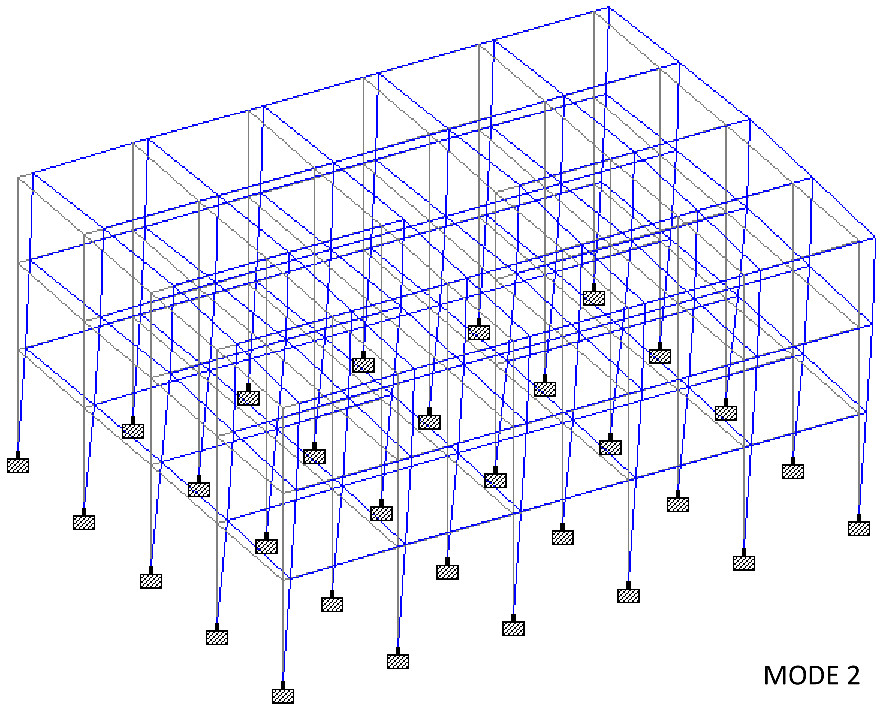


Fig.35 Deflection by Response Spectrum Analysis

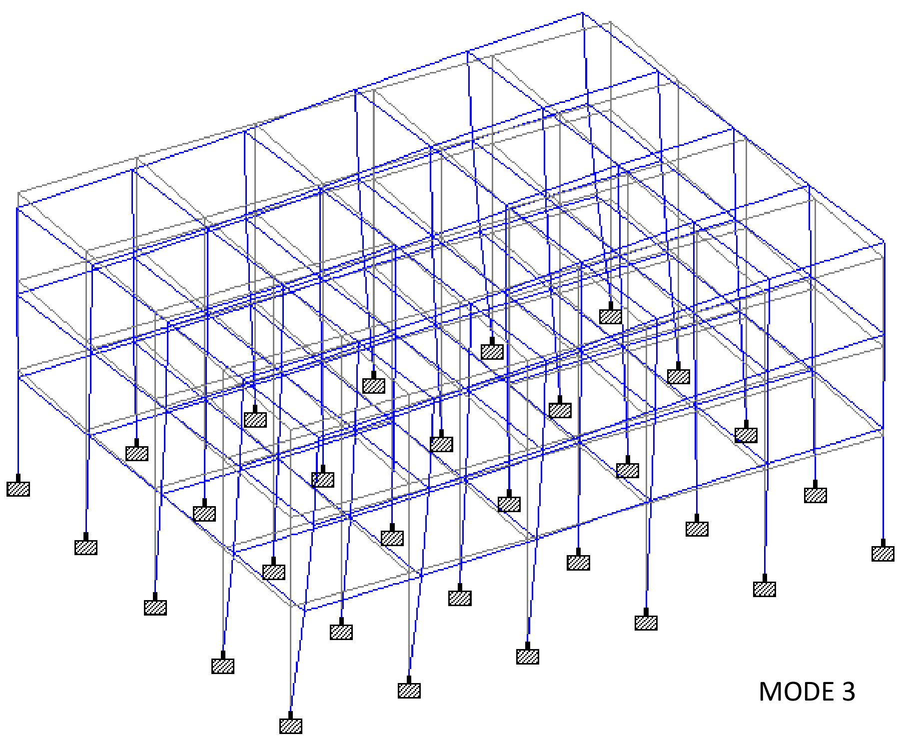
	MASS PARTICIPATION FACTORS IN PERCENT						BASE SHEAR IN KN		
MODE	X	Y	Z	SUMM-X	SUMM-Y	SUMM-Z	X	Y	Z
1	0.00	0.00	91.05	0.000	0.000	91.052	0.00	0.00	0.00
2	90.52	0.00	0.00	90.520	0.000	91.052	887.21	0.00	0.00
3	0.00	0.00	0.00	90.520	0.000	91.052	0.00	0.00	0.00
4	0.00	0.00	0.00	90.520	0.000	91.052	0.00	0.00	0.00
5	0.00	0.00	1.37	90.520	0.000	92.418	0.00	0.00	0.00
6	0.00	0.00	0.00	90.520	0.000	92.418	0.00	0.00	0.00
7	1.53	0.00	0.00	92.050	0.000	92.418	13.78	0.00	0.00
8	0.00	0.00	0.00	92.050	0.000	92.418	0.00	0.00	0.00
9	0.00	0.00	0.00	92.053	0.000	92.418	0.02	0.00	0.00
10	0.00	38.89	0.00	92.053	38.888	92.418	0.00	0.00	0.00
11	0.00	0.00	0.00	92.053	38.888	92.418	0.00	0.00	0.00
				TOTAL SRSS SHEAR			887.32	0.00	0.00
				TOTAL 10PCT SHEAR			887.32	0.00	0.00
				TOTAL ABS SHEAR			901.01	0.00	0.00
				TOTAL CSM SHEAR			901.01	0.00	0.00
				TOTAL CQC SHEAR			887.64	0.00	0.00

Table 8 Response Spectrum Results

The critical mode is mode shape 2 corresponding to the maximum base shear. The mode shape 3 has zero mass participation factors in all three directions, this shows it is a pure torsional mode.



MODE 2



MODE 3

Fig.36 Mode Shapes

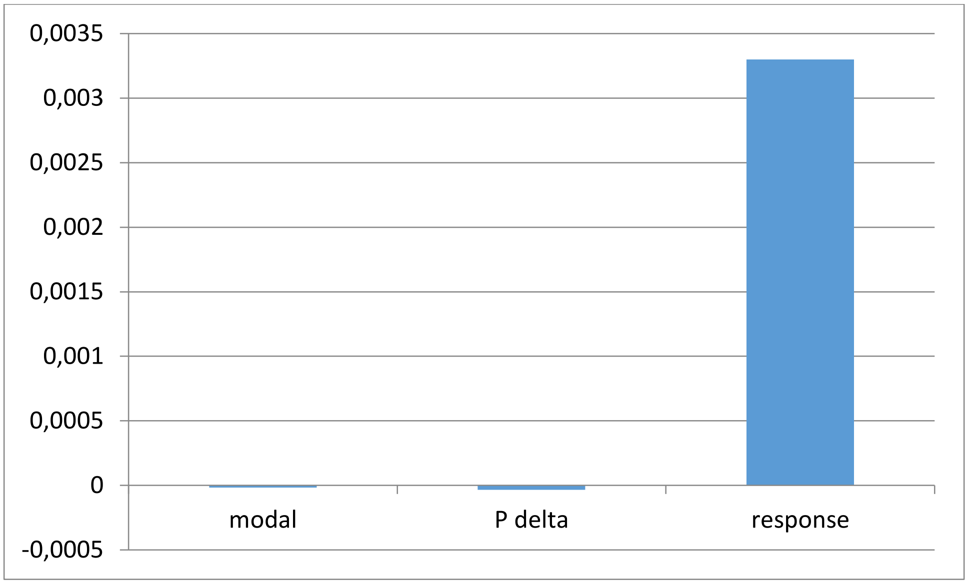


Fig.37 Roof Displacement

Comparisons of Base Reactions:

OutputCase	CaseType	StepType	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
Response Spectrum	LinRespSpec	Max	156.059	156.049	0.00004105	1094.7411	1094.8592	1998.4427
NLStatic	NonStatic	Max	5.33E-15	-1.8E-15	12785.629	102285.04	-127856.3	4.263E-14
MODAL	LinModal	Mode	1738.911	-0.00013	-2.599E-06	-0.000203	10358.121	-13911.29

Table 9 Comparison of Base Reactions

4.4.2. Observations

- Time period of structure corresponding to natural frequency are long enough to predict that structure is flexible enough.
- The significant mode shapes of structure were up to first two or three modes.
- The base shear force calculated by three methods is different.
- The base shear by The P- Δ in global Z is highest while the base shear by modal analysis in global x is highest.
- The results observed by P -Δ are at extreme ends owing to limited application in case of symmetrical structures.

4.4.3. Challenge

- To improve the accuracy of analysis.
- Remove re-entrant corners and plan irregularity.
- Reduce base shear force.

A probable solution includes modeling of structure by breaking it into parts to eliminate the plan irregularity. The structure will be broken such that an individual part remains symmetrical and do not fall in any category of irregularity. The building during construction will not be constructed monolithically rather it will appear as if individual building is constructed in proximity of each other. This will indeed reduce and total seismic weight of the building and base shear force is expected to be reduced.

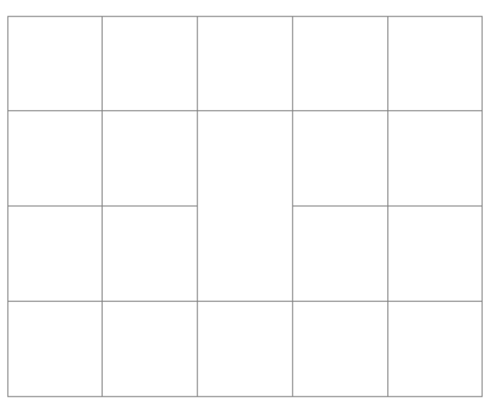


Fig.38 Original Plan

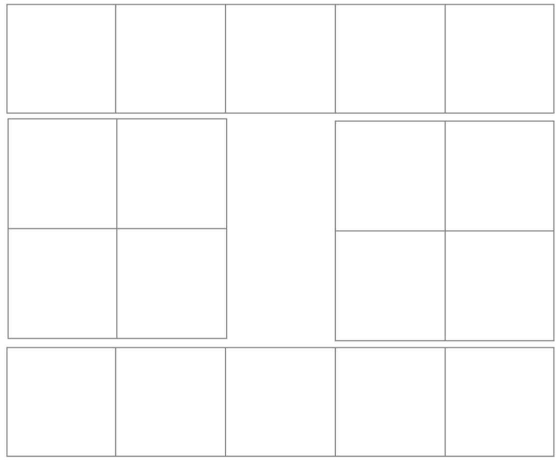


Fig. 39 Modified Plan

The structure is broken into four individual parts. Two are rectangle while two are squares. Each part has same loading and base conditions as original model. No plan or vertical irregularity exits. Also, now each part is analyzed individually.

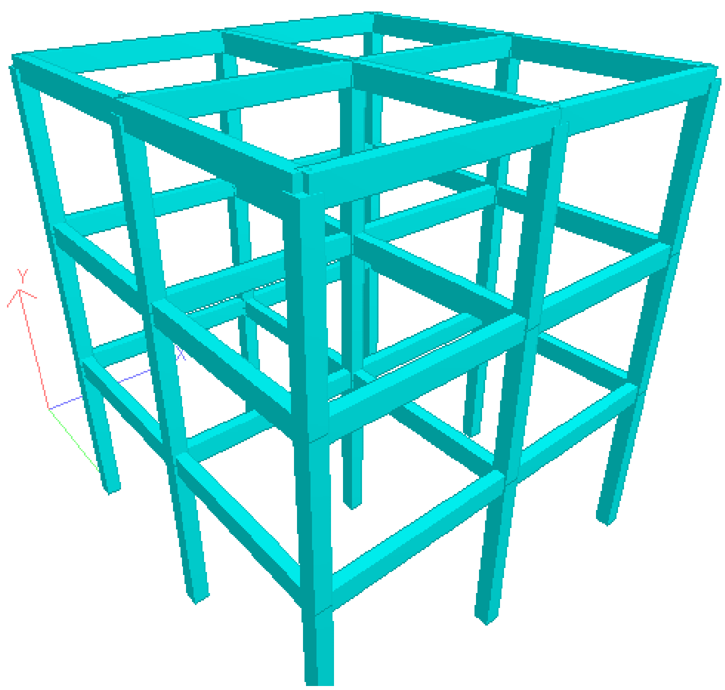


Fig. 40 3D view of Square Part

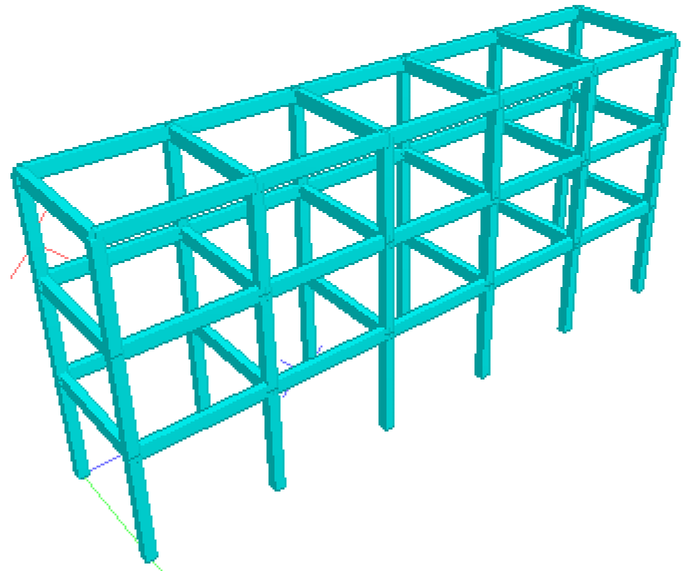


Fig.41 3D view of Rectangular Part

4.4.4. Analysis results for individual elements:

4.4.4.1.MODAL ANALYSIS

Table 10	***Square***		***Rectangle***		***Whole***	
Mode Shape	**Period(sec)**	**f(Cyc/sec)**	**Period(sec)**	**f(Cyc/sec)**	**Period(sec)**	**f(Cyc/sec)**
1	0.89673	1	0.936	1.0684	0.9206	1.0863
2	0.896728	1.1152	0.86	1.1632	0.9204	1.0864
3	0.81295	1.2301	0.8572	1.1666	0.8706	1.1486
4	0.455436	2.1957	0.8577	1.7015	0.7087	1.411
5	0.366622	2.7276	0.4985	2.0058	0.589	1.6977
6	0.366622	2.7276	0.3943	2.5357	0.5213	1.9181

4.4.4.2.P-Δ ANALYSIS

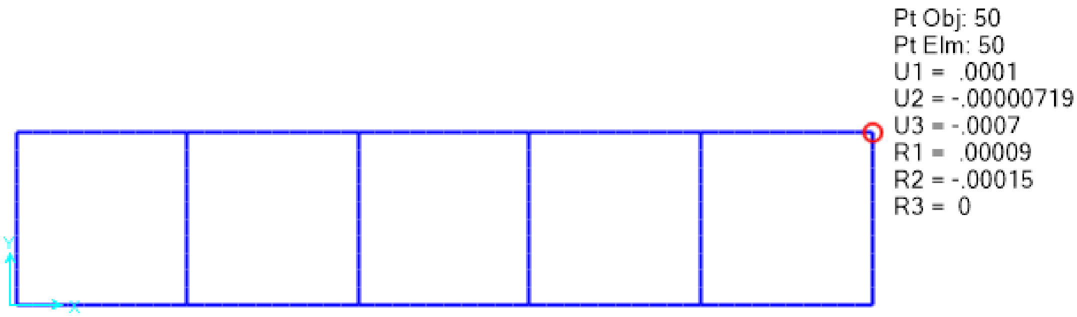


Fig.42 Top Displacement Results for Rectangular Part in P-Δ Analysis

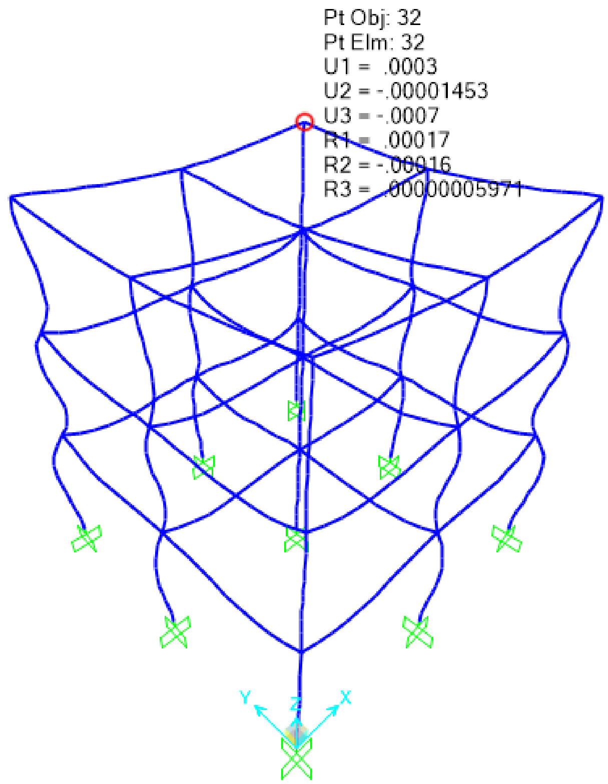


Fig.43 Top Displacement Results for Square Part in P-Δ Analysis

4.4.4.3.RESPONSE SPECTRUM

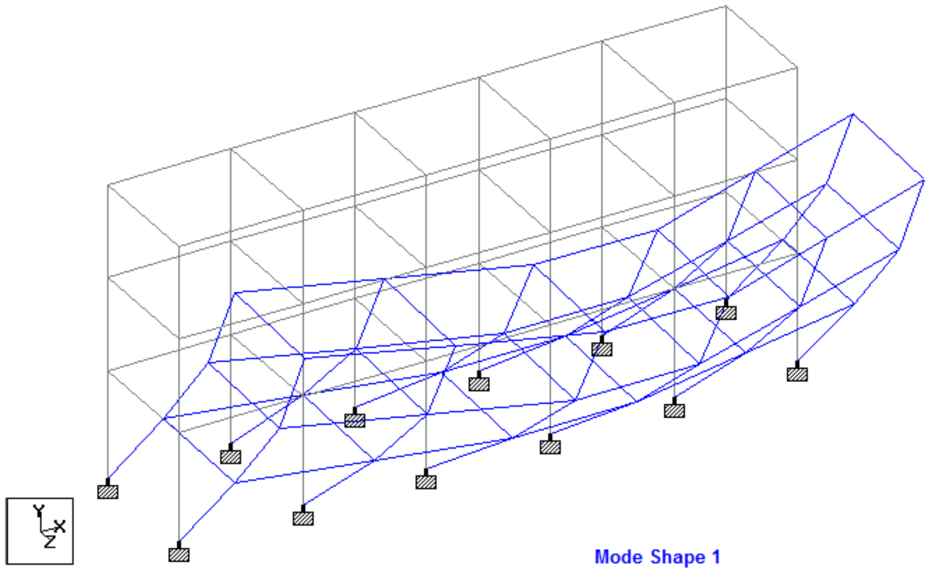


Fig.44 Critical Mode of Rectangular Part

Mass Participation factor 89.37 in Z direction

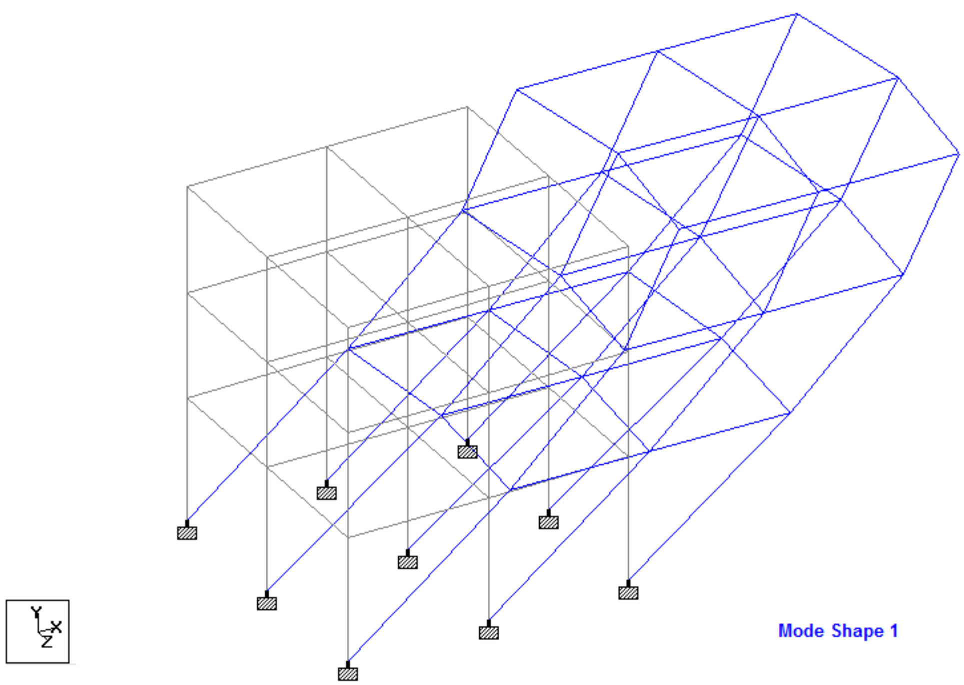


Fig.45 Critical Mode Of Square Part

Mass Participation factor 91.81 in X direction.

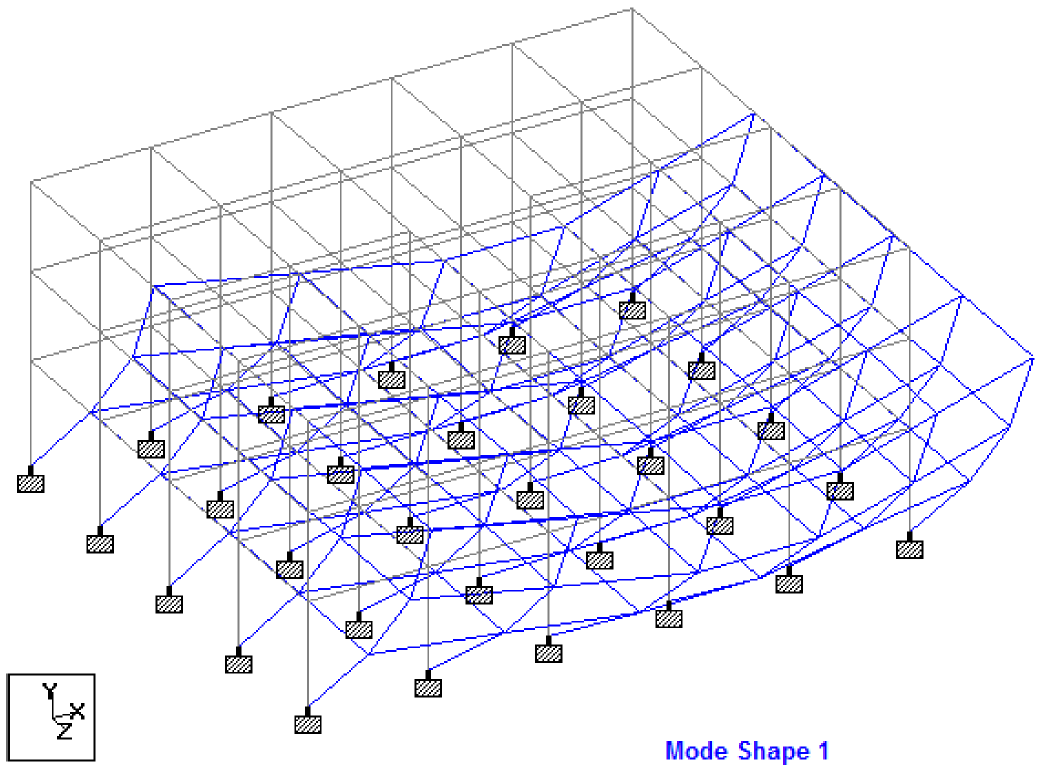


Fig.46 Critical Mode Shape of Whole Structure

Mass Participation factor 90.52 in Z direction.

4.4.4.4.COMPARISON OF BASE REACTION BETWEEN WHOLE AND INDIVIUAL PARTS

Table 11	FX	FY	FZ	MX	MY	MZ
Whole Structure	156.059	159.059	0.00004105	1094.741	1094.859	1998.443
Square Part	41.208	41.208	0.000005239	288.0627	288.0627	233.1059
Rectangle Part	55.211	54.2	0.00001081	380.9535	385.1963	545.43

4.4.4.5.DISPLACEMENT COMPARISION

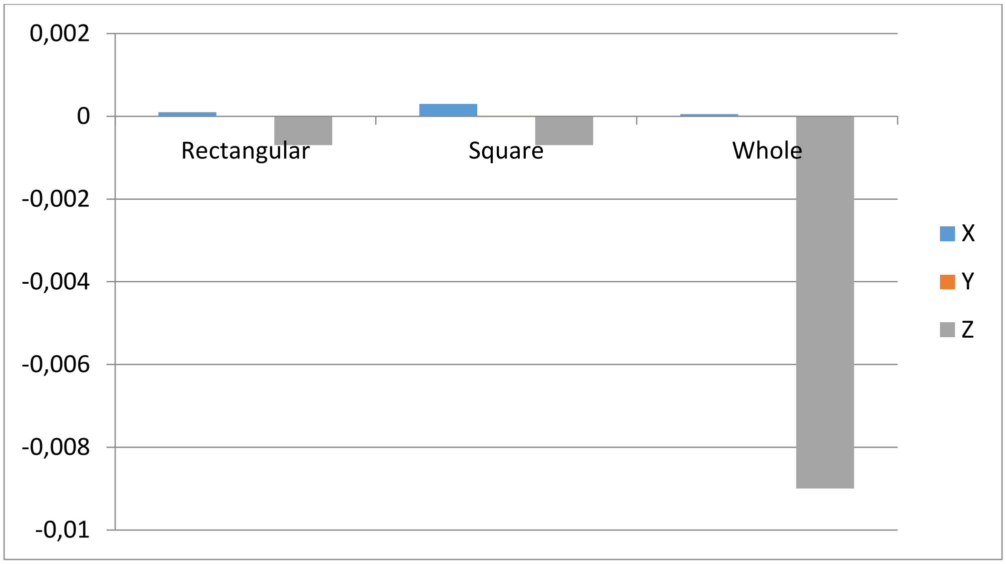


Fig.47 Displacement Comparison

4.4.4.6.FINAL OBSERVATIONS

1. Out of different seismic analysis performed Nonlinear Static analysis method has given the maximum base shear.

2. When the structure is modeled as a whole it has shown substantial flexibility which can be seen from long time periods derived from Modal analysis.

3. Response spectrum showed that the whole structure has critical base shear in mode 2 with mass participation factor maximum in Z direction while mode shape 3 is torsional mode shape.

4. Roof displacement shown by response spectrum is greatest in magnitude but in direction opposite to that given by P-Δ and modal analysis.

5. When individual non irregular structure is analyzed critical base shear gets reduced.

6. Time period of individual structure is reduced but is long enough as not to cause excessive rigidity of structure.

7. The displacement comparison shows that breaking the structure has resulted in less displacement in Z direction.

8. Reviewing IS 1893:2002 shows that Indian code do not give any logical deduction for derivation of value of S_a/g.

9. The design life of structures for which Z values are given are not mentioned.

10. **Code is silent regarding values to be used as amplification factor for conversion of elastic response spectrum to design response spectrum.**

11. Response reduction factor R does not consider nonlinear hysteresis energy dissipation mechanism.

5. CONCLUSION

5.1. CONCLUSION DRAWN ON BASIS OF WORK

a) Indian code IS 1893:2002 needs to be revised with keeping in mind the need to consider nonlinear hysteresis loss mechanism shown by materials.

b) Amplification factor used by Indian code are not derived for Indian site conditions.

c) Removing irregularity for structure reduces the base shear reactions.

d) Mode shape having zero participation factors in three directions is pure torsional mode.

e) Although the individual structure have dominant mass participation factor in different directions. The whole structure mass participation is governed by rectangular portion.

f) The forces at point where structure is broken increases for unknown reason.

g) The structure can be built in parts to have less base shear this will yield more safe and economical design.

h) All deflection are not large enough to use any retrofit technique.

5.2. FUTURE SCOPE OF STUDY

a) Capacity curves of whole structure with broken structures can be compared.

b) Work needed to be done to understand the cause of increased nodal reaction after breaking of structure.

c) More comprehensive approach to develop amplification factor for inelastic design spectrum needed to be developed.

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