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Static and Dynamic Analysis of Multistoried Building in Seismic Zone-III

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Abstract: ***Generally, the building is subjected to seismic loads the infill masonry wall is considered as nonstructural elements and their stiffness contribution are ignored during the analysis. RC frame building with open ground story is called as soft story, similarly soft story effect can be observed when soft story at different level of the structure is constructed. In recent earthquake it is observed that a building with discontinuous in stiffness and mass subjected to concentration of forces and the point of discontinuity which may leads to failure of members at the junction and collapse of building. One of the most economical ways to eliminate failure of soft story is by adding shear walls to tall buildings. Analysis is carried out by manual. The analytical model of the building includes all important component that influence the strength, stiffness, mass, and deformability of the structure. It is an attempt to study the performance of a building with open ground story along with an intermediate soft story with floating columns, type of shear wall in seismic porn areas. Fundamental time period, base shear, story displacement, story drift is calculated by equivalent static analysis (ESA) and response spectrum analysis (RSA) method and compared for all models.***

Keywords: Dynamic analysis; Equivalent static analysis; Response spectrum analysis; Seismic Zones

1. Introduction

An earthquake is the consequence of an unexpected release of energy in the Earth's crust. This sudden energy release causes the seismic waves that make the ground shake that could create seismic waves. Due to the consequence rock breaking, have result in energy waved which is seismic wave. It's kind of energy that travels all the way through the earth. Seismic waves pass through either the length of the earth's surface or through the earth's interior.

Earthquakes are usually triggered when rock underground suddenly breaks along a fault. Fault plane is the underground surface along which the rock moves and breaks. By using seismograph, it will determine the magnitude or size by measuring the amplitude of the seismic wave that occur and the distance of seismograph from the earthquake. The seismograph is consisting of a seismometer (the detector) and a recording device that located at every station of possibility of an earthquake occur. The seismometer device will electronically amplify the wave motion in earth.

Earthquakes caused too many damaging effects to the surrounding they act upon. This includes damage to man-made buildings structure and in worst cases the human death. The destruction of structures such as bridges, dams and buildings are caused by the rumbling impacts which originated from the earthquake. Besides, earthquake can also trigger landslides that have bad effect on human life and animal life.

Earthquakes usually cause dramatic changes, including ground movements, dropping, dropping, and tilting of the surface cause different in the groundwater flow. Other than producing floods and damaging the buildings, earthquakes that occur under ocean can sometimes cause tsunamis or known as tidal waves. The tsunamis' conditions are high water which travel at a short period of time. They are surely destroying area in coastlines which effect entire populations and cities.

The criteria of level adopted by codes for fixing the level of design seismic loading are generally as follows: -

a) Structures should be able to resist minor earthquakes, Design Basis Earthquake (< DBE), without damage.
b) Structures should be able to resist moderate earthquakes (DBE) without significant Structural damage but with some non- structural damage.
c) Structures should be able to resist major earthquakes Maximum Considered Earthquake (MCE) without collapse.

Design Basis Earthquake (DBE) is defined as the maximum earthquakes that reasonably can be expected to experience at the site once during lifetime of the structure. The earthquake corresponding to the ultimate safety requirements is often called as Maximum Considered Earthquakes (MCE). Generally, the DBE vertically, in all directions radiating from the epicenter. The ground accelerations cause structures to vibrate and induce inertial forces on them. Hence structures in such locations need to be suitably designed and detailed to ensure stability, strength and serviceability with acceptable levels of safety under seismic effects.

Dynamic analysis methods: - It is performed to obtain the design seismic force and its distribution to different level along the height of the building and to the various lateral load resisting elements for the regular buildings and irregular buildings also as defined in IS-1893-Part-1-2002 in clause 7.8.1.

(i) Regular building
(a) Those > than 40-meter height. in zone IVth and Vth
(b) Those > 90-meter height in zone IInd and IIIrd

(ii) Irregular building

(a) all framed building higher than 12 meter in zone IVth and Vth

(b) Those greater than 40 meter in zone IInd and IIIrd.

2. Methods of Analysis

Code-based procedure for seismic analysis: Main features of seismic method of analysis based on Indian standard 1893 (Part 1): 2002 are described as follows.

1) Equivalent static lateral force method
2) Response spectrum method
3) Absolute Sum Method (ABS)
4) Square roots of sum of squares (SRSS method).
5) Complete Quadratic Combination (CQC) Method

a) Equivalent Static Analysis

All design against seismic loads must consider the dynamic nature of the load. However, for simple regular structures, analysis by equivalent linear static methods is often sufficient. This is permitted in most codes of practice for regular, low- to medium-rise buildings. It begins with an estimation of base shear load and its distribution on each story calculated by using formulas given in the code. Equivalent static analysis can therefore work well for low to medium-rise buildings without significant coupled lateral-torsional modes, in which only the first mode in each direction is considered. Tall buildings (over, say, 75 m), where second and higher modes can be important, or buildings with torsional effects, are much less suitable for the method, and require more complex methods to be used in these circumstances.

b) Response Spectrum Analysis

The representation of the maximum response of idealized single degree freedom system having certain period and damping, during earthquake ground motions. The maximum response plotted against of un-damped natural period and for various damping values and can be expressed in terms of maximum absolute acceleration, maximum relative velocity or maximum relative displacement. For this purpose, response spectrum case of analysis has been performed according to IS 1893(part -1) -2002.

c) Absolute Sum Method (ABS) (IS 1893 Clause 7.8.4.4)

Assuming that the maximum modal responses attend peak at the same instant of time, then the maxirnum response quantity is equal to the sum of maximum absolute value of the response associated with each mode. The peak response quantity for the closely spaced modes is given by:

$$\lambda = \sum \lambda_c$$

Where λ = peak response quantity for closely spaced modes, λ_c = closely spaced modes.

This upper bound method gives very conservative estimate of maximum response, as the time of occurrence of maximum response can be different.

d) Square Root of Sum of Square (SRSS) method

In general, simultaneous occurrence of peak response in all the modes can be different. lf the natural frequencies are not very closely spaced then the peak response quantity due to all modes is obtained as:

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2}$$

where, λ_k = absolute value of quantity in mode k

r = Number modes under consideration

The peak response in each mode is squared, the squared modal peak is summed, and the square root of sum provides an estimate of peak total response with well separated natural frequencies.

e) Complete Quadratic Combination (CQC) Method

The method (CQC) for modal combination is applicable to a wider class of structures because it overcomes the limitations of SRSS rule.

$$\lambda = \sqrt{\sum_{i=1}^{r}\sum_{j=1}^{r} \lambda_i \rho_{ij} \lambda_j}$$

Where, r = Number of modes being considered.

ρ_{ij} =Cross-modal coefficient,

λ_i= Response quantity in mode i(including sign)

λ_j= Response quantity in mode j (including sign)

$$\rho_{ij} = \frac{8\zeta^2 (1+\beta_{ij})\beta_{ij}^{\ 1.5}}{(1-\beta_{ij}^{\ 2})^2 + 4\zeta^2 \beta_{ij} (1+\beta_{ij})^2}$$

ζ = Modal damping ratio

β_{ij} = Frequency ratio= ${}^{\omega_j}/_{\omega_i}$

ω_i = Circular frequency in jth mode and

ω_j = Circular frequency in jth mode.

3. Literature Review

i. Zone factor Z:(IS: 1893. Clause.6.4.2.)

Earthquake severity has been classified into zones on the basis of maximum ground acceleration based on past earthquake data. India has been divided into four seismic zones (shown in Fig.1.IS1893 page 5) for the Maximum Considered Earthquake (MCE) and service life of the structure in a zone.

The map is based on expected intensity of ground shaking but does not consider the frequency of occurrence. In seismic zone map, zone-I and zone -II of the contemporary map have been merged and assigned the level of zone-II. Zone II has lowest danger or risk while Zone - V has highest hazards.

The zone factors are given in Table 1

Table 1 Zone Factor, Z				
Seismic Zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very severe
Z	0.1	0.16	0.24	0.36

Since damage-controlled limit state method has been accepted, the zone factor, z bas been reduced to half (Z/2) of Maximum Considered Earthquake (MCE) for Design Basis Earthquake (DBE). Structures are explicitly designed for DBE and maximum considered earthquake is taken care of through over strength and ductility provisions.

ii. IS: 875 (Part 1) – 1987 for Dead Loads: Indian Standard Code of Practice for Design Loads (Other Than Earthquake)For Buildings and Structures, all permanent constructions of the structure form the dead loads. The dead load comprises of the weights of walls, partitions floor finishes, false ceilings false floors and the other permanent constructions in the buildings. The dead load loads may be calculated from the dimensions of various members and their unit weights. The unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 kN/m^3 and 25kN/m^3respectively.

iii. IS: 875 (Part 2) - 1987 for Imposed Loads, Indian Standard Code of Practice for Design Loads (Other Than Earthquake),For Buildings and Structures, imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration and dust loads. Imposed loads do not include loads due to wind, seismic activity, snow, and loads imposed due to temperature changes to which the structure will be subjected to, creep and shrinkage of the structure, the differential settlements to which the structure may undergo.

iv. IS:1893(Part-1)-2002: Indian Standard Criteria for Earthquake Resistant Design of Structures, (Part 1-GeneralProvisions and Buildings), It deals with assessment of seismic loads on various structures and earthquake resistant design of buildings. Its basic provisions are applicable to buildings; elevated structures; industrial and stack like structures; bridges; concrete masonry and earth dams; embankments and retaining walls and other structures. Temporary elements such as scaffolding, temporary excavations need not be designed for earthquake forces.

v. IS 456 -2000: Indian standard code of practice for plain and reinforced concrete (fourth revision), Bureau of Indian Standards. This standard deals with the general structural use of plain and reinforced concrete. For the purpose of this standard, plain concrete structures are those where reinforcement, if provided is ignored for the determination of strength of the structures.

vi. IS 800 -2007: Indian Standard General Construction in Steel - Code of Practice (Third Revision) This standard gives only general guidance as regards the various loads to be considered in design. For the actual loads and load combinations to be used, reference may be made to IS 875 for dead, live, snow and wind loads and to IS 1893 (Part 1) for earthquake loads. For seismic design, recommendations pertaining to steel frames only are covered in this standard.

vii. SP: 16 -1980: Design Aids for Reinforced Concrete to IS: 456-1978 (third revision), Bureau of Indian Standard. This is the explanatory hand book which covers the basis/source of each clause. The objective of these design aids is to reduce design time in the use of certain clauses in the Code for the design of beams, slabs and columns in general building structures. The charts and tables included in the design aids were used in calculation of footings and slabs.

viii. SP: 34 (S&T) –1987: Hand Book of Concrete Reinforcement and Detailing, Bureau of Indian Standards. This Handbook provides information on properties of reinforcing steel & detailing requirements, including storage, fabrication, assembly, welding and placing of reinforcement in accordance with IS: 456-1978. As a result of the introduction of limit state method of design for reinforced concrete structures and the concept of development length, detailing has become extremely important as many of the design requirements are to be met through detailing. This Handbook will be useful to concrete design engineers, field engineers and students of civil engineering.

ix. IS: 875 (Part 5) – 1987 for Load Combinations: Indian Standard Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures, the various loads should be combined in accordance with the stipulations in the relevant design codes. In the absence of such recommendations, the following loading combinations, whichever combination produces the most unfavorable effect in the building, foundation or wind, earthquake, imposed and snow loads is not likely. All members are designed for the critical forces.

x. Design Lateral Force:
The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

xi. Design Seismic Base Shear:
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 = horizontal acceleration spectrum using the fundamental natural period in the considered direction of vibration.

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

where, z = Zone factor
I - Importance factors
R - Response reduction factor

$Sa/g =$ Average acceleration response coefficient for approximate, natural period of vibration T_a to be determined.
W = seismic weight of all the floors of building

The, seismic coefficient method does not need theoretical concepts of structural dynamics and modal analysis.

xii. Fundamental Natural Period:
The approximate fundamental natural period of vibration (T_a), in seconds, of a moment resisting frame building without brick in the panels may be estimated by the following empirical expression:

$$T_a = 0.075\, h^{0.75} \text{ for RC frame building}$$
$$T_a = 0.085\, h^{0.75} \text{ for steel frame building}$$

Where, H = Height of building, in m.
This excludes the basement story's, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement stores, when they are not so connected. The approximate fundamental natural period of vibration (T), in seconds, of all other buildings, including moment-resisting frame buildings with brick lintel panels, may be estimated by the empirical Expression:

$$T_a = \frac{0.09}{\sqrt{d}}$$

Where, h = Height of building in m, and
d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

xiii. Distribution of Design Force
Vertical Distribution of Base Shear to Different Floor Level. The design base shear (V_b) shall be distributed along the height of the building as per the following expression:

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

Where, Q_i = Design lateral force at floor i,
W_i = Seismic weight of floor i,
h_i = Height of floor i measured from base, and

n = Number of story's in the building is the number of levels at which the masses are located.

4. Methodology

The building considered in the present study is G+2 storied R.C framed building of symmetrical rectangular plan with configuration of buildings having a plan area of 11 m x 4.83 m with a storey height 3.2 m each and total height of chosen building including depth of foundation is 11.5 m. Complete analysis is carried out for dead load, live load, wind and seismic load by using Mannual Calculation (Matrix Method). Response spectra method for dynamic analysis is used. Loading combinations are considered as per IS 1893:2002.

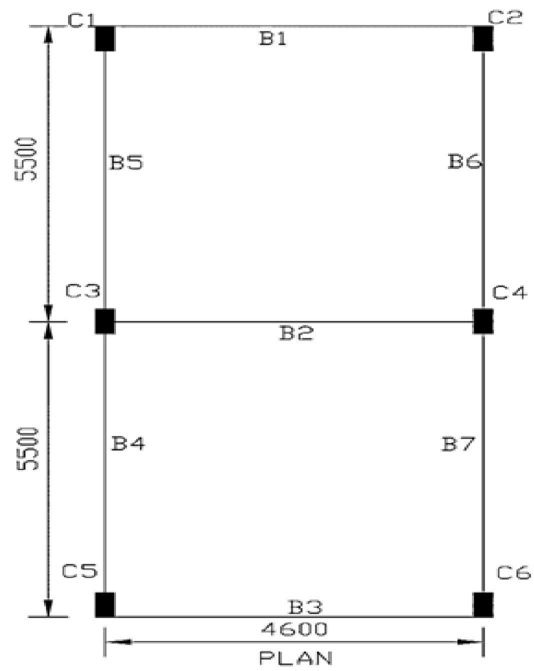
The static and dynamic analysis has done on MANNUAL CALCULATION using the parameters as per the IS1893 - 2002 Part-1 for the zone III and the post processing result obtained has summarized.

5. Building Properties

No. of story's (G + 2)
Foor to floor height = 3.2 m
Imposed Load - Roof slab = 1.5 kN/m^2
- Floor slab = 4.0 kN/m^2
Floor finish - Roof slab = 2.0 kN/m^2
- Floor slab = 1.0 kN/m^2
Thickness of infilled wall 230 mm
Beam sizes 230 mm x 380 mm
Column sizes -Ground floor 230 mm x 450 mm
First floor 230 mm x 380 mm
Second floor 230 mm x 230 mm
Thickness of slab 150 mm
Parapet 1m high and 150 mm thick
Plinth level 1 m above ground level
(Medium soil condition)
Depth of foundation for column 1.5 m below GL.
Depth of footing 0.6m
Depth of foundation for wall 1mbelowGL.
Soil type medium stiff soil
Materials Properties:
Concrete M20 Grade all
Steel HYSD reinforcement of grade Fe415conforming to IS 1786 is used throughout.
Specific weight of concrete 25 kN/m^3
Specific weight of' Masonry 20 kN/m^3

6. Seismic Properties

Seismic Zone: III
Zone factor: 0.16
Importance factor: 1
Response reduction factor: 5
Type of soil: medium soil



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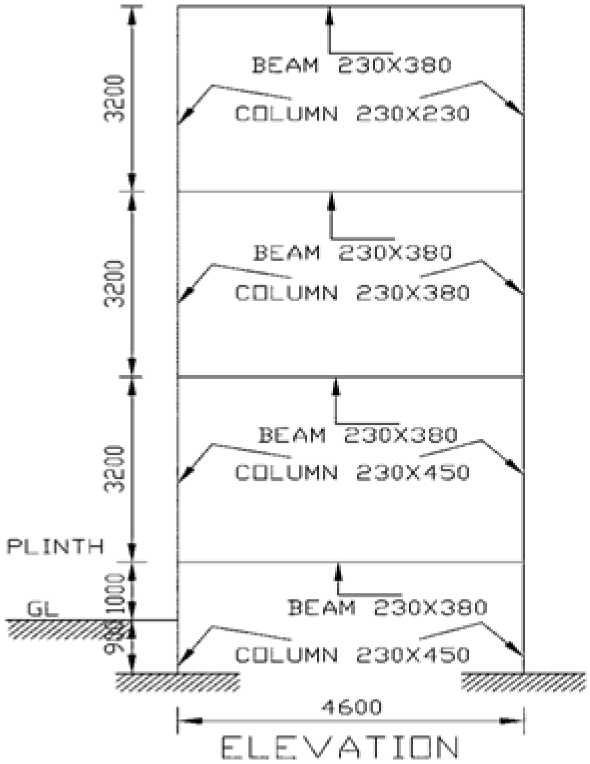
7. Calculation of Seismic Weights at varies levels

While calculating the seismic weight of each floor the weight of column and wall in any storey is equally distributed to the floors above and below that storey.

Mass	Calculation	Seismic Wt. kN
Roof		
Slab	Slab area x (γ x t + FF + LL) = 11.23 x 4.83 x (25 x 0.15 +2 +0*) = 311.89	311.89
Parapet	Perimeter x (γ x t x height) = (2 x 15.6) x (25 x 0.15 x1) = 117	117
Beam	(Perimeter) x width x (depth - t) x 25 + width (depth - 0.15) x 25 = (2 x 15.6) x 0.23 x (0.38 - 0.15) x 25 + 4.6 x 0.23 x (0.38 - 0.15) x 25 = 47.34	47.34
Wall	(Perimeter) x γ x t x net height/2 + length x width x γ x net height/2 = (2 x 15.6) x [20 x 0.23 x (3.2/2 – 0.38)] + 4.6 x 0.23 x 20 x (3.2/2 – 0.38) = 200.91	200.91
*Iposed Load on roof is not considered (Clause 7.3.2)		
Total load for Roof= 677.14 or 680		

Mass	Calculation	Seismic Wt. kN
2nd FLOOR		
Slab	Slab area x (γ x t + FF + LL) = 11.23 x 4.83 x (25 x 0.15 +1 +4/2*) = 366.13	366.13
Beam	(Perimeter) x width x (depth - t) x 25 + width (depth - 0.15) x 25 = (2 x 15.6) x 0.23 x (0.38 - 0.15) x 25 + 4.6 x 0.23 x (0.38 - 0.15) x 25 = 47.34	47.34
Wall	(Perimeter) x γ x t x net height/2 + length x width x γ x net height/2 = (2 x 15.6) x [20 x 0.23 x (3.2 – 0.38)] + 4.6 x 0.23 x 20 x (3.2 – 0.38) = 464.40	464.4
*50% of imposed load is considered (clause 7.3.2)		
Total load for Roof= 877.87 or 880		

Mass	Calculation	Seismic Wt. kN
1st FLOOR		
Slab	Slab area x (γ x t + FF + LL) = 11.23 x 4.83 x (25 x 0.15 +1 +4/2*) = 366.13	366.13
Beam	(Perimeter) x width x (depth - t) x 25 + width (depth - 0.15) x 25 = (2 x 15.6) x 0.23 x (0.38 - 0.15) x 25 + 4.6 x 0.23 x (0.38 - 0.15) x 25 = 47.34	47.34
Wall	(Perimeter) x γ x t x net height/2 + length x width x γ x net height/2 = (2 x 15.6) x [20 x 0.23 x (4.2 – 0.38)] + 4.6 x 0.23 x 20 x (4.2 – 0.38) =629.08	629.08
*50% of imposed load is considered(clause 7.3.2)		
Total load for Roof=1042.55 or 1045		
Seismic weight of building W = 680 + 880 + 1045 = 2605 kN		



8. Equivalent Static Method

Determination of fundamental natural period:
The approximate fundamental natural period of vibration T_a in seconds of a moment resisting frame building without brick infill panels is estimated by formula:

$T_a = 0.075\ h^{0.75}$ for RC frame building
Where, h = Total height of the building = 3.2 + 3.2 + 3.2 + 1 + 0.9 = 11.5 m

$T_a = 0.075 \text{ x} 11.5^{0.75} = 0.468\ sec.$ (Assuming that infill panels will not resist any load)
$Design\ base\ shear\ (V_B) = A_h\ W$

$$A_h = \frac{\frac{Z}{2}\frac{S_a}{g}}{\frac{R}{I}} = \frac{\frac{0.16}{2} x\ 2.5}{\frac{5}{1}} = 0.04$$

where, Z = 0.16 for severe seismic intensity
I = Importance factor= 1, for residential building
R= Response reduction factor= 5 for ductile detailing
S_a/g =2.5 for medium soil (0.1 ≤ T ≤ 0.55)

$\therefore Design\ base\ shear\ (V_B) = 0.04 \text{ x } 2605$
$= 104.2\ kN$

9. Vertical Di1trlbutlon or Base Shear to different Floor Levels

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

Where, Q_i = Design lateral force at floor i,
W_i = Seismic weight of floor i,
h_i = Height of floor i measured from base, and

n= Number of story's in the building is the number of levels at which the masses are located.

Story	W_i	h_i (m)	$W_i h_i^2$	$W_i h_i^2 / \sum W_j h_j^2$	Q_i	V_i (kN)
Roof	680	11.5	89930	0.506	52.75	52.75
2nd Floor	880	8.3	60623.2	0.341	35.53	88.28
1st Floor	1045	5.1	27180.45	0.153	15.94	104.22
	2605		177733.7	1.000		

The story shear, and design load using equivalent static method are shown in Fig.1

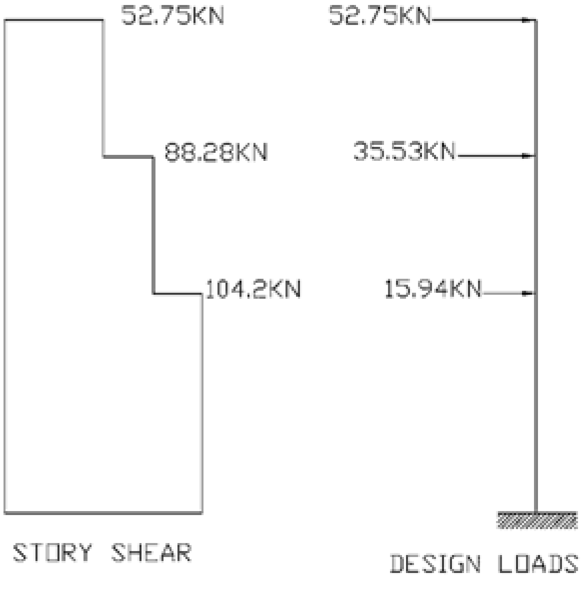


Fig 1 EQUIVALENT STATIC METHOD

10. Response Spectrum Method

Mass matrix: The masses at various floor level have already obtained early as:
M_3 = 680 kN, M_2= 880kN, M_1 = 1045 kN

To simplify the calculations the relative values of masses are taken.

Assuming, m_3 =680/680 = 1 m, m_2 = 880/ 680= 1.29 m, m_1 = 1045/680 = 1.54 m.

The positive definite property of mass is assumed because the lumped masses are non-zero in all degrees of freedom restrained in the analysis with zero lumped masses have been eliminated by static condensation. Column stiffness, relative column stiffness, lumped masses and relative lumped masses.

The mass matrix can be written as;

$$Mass\ Matrix\ [M] \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} = m \begin{bmatrix} 1.54 & 0 & 0 \\ 0 & 1.29 & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

Stiffens Matrix:
The total number of columns having the same size in each story = 6

$$\text{Therefore, k} = 6 \left[\frac{12EI}{h^3}\right]$$

$$\text{Where, E} = 5000\sqrt{f_{ck}} = 5000\sqrt{20}$$
$$= 2.236 \text{ x}10^4\ N/mm^2$$

Column stiffness for I - story k_1

$$= 6 \left[\frac{12 \text{ x } 2.236 \text{ x}10^4 \text{ x}(1.7465 \text{ x } 10^9)}{(5100)^3}\right]$$
$$= 2.119 \text{ x}10^4\ N/mm$$

Column stiffness for II - story k_2

$$= 6 \left[\frac{12 \text{ x } 2.236 \text{ x}10^4 \text{ x}(1.0517 \text{ x } 10^9)}{(3200)^3}\right]$$
$$= 5.167 \text{ x}10^4\ N/mm$$

Column stiffness for Roofk_3

$$= 6 \left[\frac{12 \text{ x } 2.236 \text{ x}10^4 \text{ x}(2.332 \text{ x } 10^8)}{(3200)^3}\right]$$
$$= 1.1457 \text{ x}10^4\ N/mm$$

To simplify the computations the relative values of stiffness are taken,
Assuming k_3= 1.1457 x 10^4 N/mm = k = 1
Thus, k_1 = 2.119 x 10^4/ (1.1457 x10^4) = 1.85 k
k_2 = 5.167 x 10^4/ (1.1457 x10^4) = 4.51 k and k_3= k

The stiffness matrix [k]

$$= \begin{bmatrix} k_1 + k_2 & -k_2 & 0 \\ -k_2 & k_2 + k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix} = k \begin{bmatrix} 6.36 & -4.51 & 0 \\ -4.51 & 5.51 & -1 \\ 0 & -1 & 1 \end{bmatrix}$$

i. Mode Shapes Equations (Characteristic Equation)

$[k - \omega_n{}^2\ m]\phi_n = 0$

The right band side of the equation is zero

$\therefore\ \emptyset_n = 0\ or\ [k - \omega_n{}^2\ m] = 0$

but,$\emptyset_n$ cannot be zero because it leads to trivial solution implying u = 0,which means no motion.

$\therefore\ [k - \omega_n{}^2\ m]$has non trivial solution, if det $[k - \omega_n{}^2\ m]$= 0

$$k \begin{bmatrix} 6.36 & -4.51 & 0 \\ -4.51 & 5.51 & -1 \\ 0 & -1 & 1 \end{bmatrix} - m\,\omega_n{}^2 \begin{bmatrix} 1.54 & 0 & 0 \\ 0 & 1.29 & 0 \\ 0 & 0 & 1 \end{bmatrix} = 0$$

$$\begin{bmatrix} 6.36 & -4.51 & 0 \\ -4.51 & 5.51 & -1 \\ 0 & -1 & 1 \end{bmatrix} - \frac{m\,\omega_n{}^2}{k} \begin{bmatrix} 1.54 & 0 & 0 \\ 0 & 1.29 & 0 \\ 0 & 0 & 1 \end{bmatrix} = 0$$

$Let\ \frac{m\,\omega_n{}^2}{k} = \lambda$ then the above equation can be written as

$$\begin{bmatrix} 6.36 & -4.51 & 0 \\ -4.51 & 5.51 & -1 \\ 0 & -1 & 1 \end{bmatrix} - \begin{bmatrix} 1.54\lambda & 0 & 0 \\ 0 & 1.29\lambda & 0 \\ 0 & 0 & \lambda \end{bmatrix} = 0$$

$$\begin{bmatrix} (6.36 - 1.54\lambda) & -4.51 & 0 \\ -4.51 & (5.51 - 1.29\lambda) & -1 \\ 0 & -1 & (1 - \lambda) \end{bmatrix} = 0$$

For non-trivial solution det.

$$\begin{bmatrix} (6.36 - 1.54\lambda) & -4.51 & 0 \\ -4.51 & (5.51 - 1.29\lambda) & -1 \\ 0 & -1 & (1 - \lambda) \end{bmatrix} = 0$$

Solving the determinant, we get,

$$\lambda^3 - 9.4\lambda^2 + 14.97\lambda - 4.19 = 0$$

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The solution of this cubic equation gives the eigen values as under:

$$\lambda_1 = 0.357, \lambda_2 = 1.57, \lambda_3 = 7.47$$

k_3= 1.1457 x 10^4 N/mm = 1.1457 x 10^7 N/m

m = 680 kN = 68000 kg

$$\sqrt{\frac{k}{m}} = \sqrt{\frac{1.1457 \text{ x } 10^7}{68000}} = 12.98$$

But,

$$\lambda = \frac{m\,\omega^2}{k} = \omega^2\left(\frac{m}{k}\right)$$

$$\therefore \omega^2 = \lambda\left(\frac{k}{m}\right) \ or\ \omega = \sqrt{\lambda} \text{x} \sqrt{\frac{k}{m}} = \sqrt{\lambda}\text{x } 12.98$$

$$i)\ {\omega_1}^2 = \lambda_1\left(\frac{k}{m}\right)$$

$$\omega_1 = \sqrt{\lambda_1}\text{x}\sqrt{\left(\frac{k}{m}\right)} = \sqrt{0.357} \text{ x } 12.98 = 7.76\ radians/sec$$

$$\therefore T_1 = \frac{2\pi}{\omega_1} = \left(\frac{2\pi}{7.76}\right) = 0.809\ sec.$$

Similarly,

$$ii)\ {\omega_2}^2 = \lambda_2\left(\frac{k}{m}\right)$$

$$\omega_2 = \sqrt{\lambda_2}\text{x}\sqrt{\left(\frac{k}{m}\right)} = \sqrt{1.57} \text{ x } 12.98 = 16.26\ radians/sec$$

$$\therefore T_2 = \frac{2\pi}{\omega_2} = \left(\frac{2\pi}{16.26}\right) = 0.386\ sec.$$

$$iii)\ {\omega_3}^2 = \lambda_3\left(\frac{k}{m}\right)$$

$$\omega_3 = \sqrt{\lambda_3}\text{x}\sqrt{\left(\frac{k}{m}\right)} = \sqrt{7.47} \text{ x } 12.98 = 35.48\ radians/sec$$

$$\therefore T_3 = \frac{2\pi}{\omega_3} = \left(\frac{2\pi}{35.48}\right) = 0.177\ sec.$$

Mode Shapes Coefficients ($\emptyset_{ik}$):

where, $\emptyset_{ik}$ =mode shape coefficients at floor i in mode k

ii. For Mode -I

$$\emptyset_1 = \begin{Bmatrix}\emptyset_{11}\\ \emptyset_{21}\\ \emptyset_{31}\end{Bmatrix} \text{ and } \lambda_1 = 0.357 \text{as obtained earlier}$$

$$\begin{bmatrix}(6.36-1.54\lambda_1) & -4.51 & 0\\ -4.51 & (5.51-1.29\lambda_1) & -1\\ 0 & -1 & (1-\lambda_1)\end{bmatrix}\begin{bmatrix}\emptyset_{11}\\ \emptyset_{21}\\ \emptyset_{31}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}5.81 & -4.51 & 0\\ -4.51 & 5.049 & -1\\ 0 & -1 & 0.643\end{bmatrix}\begin{bmatrix}\emptyset_{11}\\ \emptyset_{21}\\ \emptyset_{31}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}5.81\emptyset_{11} + (-4.51\emptyset_{21}) + 0\,\emptyset_{31}\\ -4.51\emptyset_{11} + 5.049\emptyset_{21} + (-1\emptyset_{31})\\ 0\emptyset_{11} + (-1\emptyset_{21}) + 0.643\emptyset_{31}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

Let $\emptyset_{31} = 1$

$0\emptyset_{11} + (-1\emptyset_{21}) + 0.643\emptyset_{31} = 0$

$\therefore \emptyset_{21} = 0.643$

$-4.51\emptyset_{11} + 5.049\emptyset_{21} + (-1\emptyset_{31}) = 0$

$\therefore \emptyset_{11} = 0.498$

iii. For Mode -II

$$\emptyset_2 = \begin{Bmatrix}\emptyset_{12}\\ \emptyset_{22}\\ \emptyset_{32}\end{Bmatrix} \text{ and } \lambda_2 = 1.57 \text{as obtained earlier}$$

$$\begin{bmatrix}(6.36-1.54\lambda_2) & -4.51 & 0\\ -4.51 & (5.51-1.29\lambda_2) & -1\\ 0 & -1 & (1-\lambda_2)\end{bmatrix}\begin{bmatrix}\emptyset_{12}\\ \emptyset_{22}\\ \emptyset_{32}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}3.94 & -4.51 & 0\\ -4.51 & 3.485 & -1\\ 0 & -1 & -0.57\end{bmatrix}\begin{bmatrix}\emptyset_{12}\\ \emptyset_{22}\\ \emptyset_{32}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}3.94\emptyset_{12} + (-4.51\emptyset_{22}) + 0\,\emptyset_{32}\\ -4.51\emptyset_{12} + 3.485\emptyset_{22} + (-1\emptyset_{32})\\ 0\emptyset_{12} + (-1\emptyset_{22}) + (-0.57\emptyset_{32})\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

Let $\emptyset_{32} = 1$

$0\emptyset_{12} + (-1\emptyset_{22}) + (-0.57\emptyset_{32}) = 0$

$\therefore \emptyset_{22} = -0.570$

$-4.51\emptyset_{12} + 3.485\emptyset_{22} + (-1\emptyset_{32}) = 0$

$\therefore \emptyset_{12} = -0.662$

iv. For Mode -III

$$\emptyset_2 = \begin{Bmatrix}\emptyset_{13}\\ \emptyset_{23}\\ \emptyset_{33}\end{Bmatrix} \text{ and } \lambda_3 = 7.47 \text{as obtained earlier}$$

$$\begin{bmatrix}(6.36-1.54\lambda_3) & -4.51 & 0\\ -4.51 & (5.51-1.29\lambda_3) & -1\\ 0 & -1 & (1-\lambda_3)\end{bmatrix}\begin{bmatrix}\emptyset_{13}\\ \emptyset_{23}\\ \emptyset_{33}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}-5.144 & -4.51 & 0\\ -4.51 & -4.126 & -1\\ 0 & -1 & -6.47\end{bmatrix}\begin{bmatrix}\emptyset_{13}\\ \emptyset_{23}\\ \emptyset_{33}\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

$$\begin{bmatrix}-5.144\emptyset_{13} + (-4.51\emptyset_{23}) + 0\,\emptyset_{33}\\ -4.51\emptyset_{13} + (-4.126\emptyset_{23}) + (-1\emptyset_{33})\\ 0\emptyset_{13} + (-1\emptyset_{23}) + (-6.47\emptyset_{33})\end{bmatrix} = \begin{bmatrix}0\\0\\0\end{bmatrix}$$

Let $\emptyset_{33} = 1$

$0\emptyset_{13} + (-1\emptyset_{23}) + (-6.47\emptyset_{33}) = 0$

$\therefore \emptyset_{23} = -6.47$

$-4.51\emptyset_{13} + (-4.126\emptyset_{23} + (-1\emptyset_{33}) = 0$

$\therefore \emptyset_{13} = 5.697$

Summary of mode shape coefficients:

$$\emptyset_1 = \begin{bmatrix}0.498\\0.643\\1.00\end{bmatrix}, \emptyset_2 = \begin{bmatrix}-0.662\\-0.570\\1.00\end{bmatrix}, \emptyset_3 = \begin{bmatrix}5.697\\-6.47\\1.00\end{bmatrix}$$

The mode shapes are shown in fig.2

$\omega_1 = 7.76\ radians/sec$

$\omega_2 = 16.26\ radians/sec$

$\omega_3 = 35.48\ radians/sec$

$T_1 = 0.809\ sec, T_2 = 0.386\ sec, T_3 = 0.177\ sec$

Vibration properties of the building for vibration in X-direction.

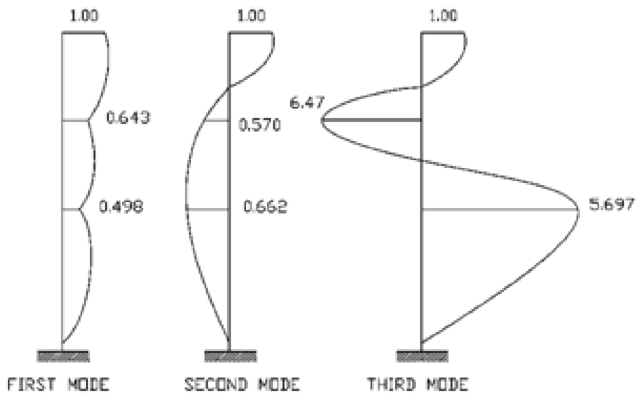

Fig.2

v. Modal Mass (M_k): (clause 3.20)

Model mass of a structure subjected to horizontal or vertical ground motion is the part of the total seismic mass of the structure that is effective in mode k of vibration. It has a unique value for a given mode irrespective of scaling of the mode shape.

The Model Mass (M_k) of mode k is given by: (clause 7.8.4.5)

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

where, W_i = seismic weight of floor i

ϕ_{ik} = mode shape coefficient at floor i in mode k

g = acceleration due to gravity

Modal Mass M_1 of Mode I

Storey level	Weight W_i	$\emptyset_{i1}$	$W_i\emptyset_{i1}$	$W_i\emptyset_{i1}{}^2$
Roof	680	1.00	680	680.00
II-Floor	880	0.643	565.84	363.84
I-Floor	1045	0.498	520.41	259.16
	2605		**1766.25**	**1303.00**

$$\therefore M_1 = \frac{\left(\sum W_i \emptyset_{i1}\right)^2}{g \sum \left(W_i \emptyset_{i1}{}^2\right)} = \frac{(1766.25)^2}{g(1303)} = \frac{2394.19}{g}$$

$$M_1 = \left[\frac{2394.19 \text{ x}1000}{9.81}\right] = 244056.07 \ kg \ or \ 2394.19 \ kN$$

Modal Mass M_2 of Mode II

Storey level	Weight W_i	$\emptyset_{i2}$	$W_i\emptyset_{i2}$	$W_i\emptyset_{i2}{}^2$
Roof	680	1.00	680.00	680.00
II-Floor	880	-0.570	-501.60	285.91
I-Floor	1045	-0.662	-691.79	457.96
	2605		**-513.39**	**1423.88**

$$\therefore M_2 = \frac{\left(\sum W_i \emptyset_{i2}\right)^2}{g \sum \left(W_i \emptyset_{i2}{}^2\right)} = \frac{(-513.39)^2}{g(1423.88)} = \frac{185.11}{g}$$

$$M_2 = \left[\frac{185.11 \text{ x}1000}{9.81}\right] = 18869.15 \ kg \ or \ 185.11 \ kN$$

Modal Mass M_3 of Mode III

Storey level	Weight W_i	$\emptyset_{i3}$	$W_i\emptyset_{i3}$	$W_i\emptyset_{i3}{}^2$
Roof	680	1.00	680.00	680.00
II-Floor	880	-6.470	-5693.60	36837.59
I-Floor	1045	5.697	5953.37	33916.32
	2605		**939.77**	**71433.91**

$$\therefore M_3 = \frac{\left(\sum W_i \emptyset_{i3}\right)^2}{g \sum \left(W_i \emptyset_{i3}{}^2\right)} = \frac{(939.77)^2}{g(71433.91)} = \frac{12.36}{g}$$

$$M_2 = \left[\frac{12.36 \text{ x}1000}{9.81}\right] = 1260.29 \ kg \ or \ 12.36 \ kN$$

vi. Modal Contribution of Various Modes

$M = 2605 \ kN, M_1 = 2394.19 \ kN, M_2 = 185.11 \ kN,$
$M_3 = 12.36 \ kN$

vii. Modal Contribution Factor:

Modal contribution factors are dimensionless quantities. They are independent of the modes. They are normalized. The sum of modal contribution factors over all modes is unity.

$$Mode - I \left[\frac{M_1}{M}\right] = \left[\frac{2394.19}{2605}\right] = 0.91907 = 91.907\%$$

$$Mode - II \left[\frac{M_2}{M}\right] = \left[\frac{185.11}{2605}\right] = 0.07105 = 7.105\%$$

$$Mode - III \left[\frac{M_3}{M}\right] = \left[\frac{12.36}{2605}\right] = 0.00474 = 0.474\%$$

$$\sum \mathbf{1.00} \sum \mathbf{100\%}$$

It will be seen that the modal contribution decreases as mode number increases. Therefore, for practical problems number of modes to be used in tile analysis should be such that the sum of modal masses of modes considered is at least 90% of the total seismic mass. Modal contributions should be carried out only for modes up to 33 Hz. The effective mass of all modes= 100%.

viii. Modal Participation Factor (P_k):

Modal participation factor of mode k of vibration is the amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motion. It is a measure of degree to which n^{th} mode participates in the response.

$$P_k = \frac{\sum (W_i \emptyset_{ik})}{\sum (W_i \emptyset_{ik}{}^2)}$$

$$P_1 = \left[\frac{1766.25}{1303.00}\right] = 1.355$$

$$P_2 = \left[\frac{-513.39}{1423.88}\right] = -0.3606$$

$$P_3 = \left[\frac{939.77}{71433.91}\right] = 0.0132$$

Design lateral force at each Floor in each mode: (clause 7.8.4.5c)

The peak lateral force Q_{ik} ,at floor i in mode k is given by:

$Q_{ik} = A_k \ P_k \ \emptyset_{ik} \ W_i$

Where, A_k = Design horizontal acceleration spectrum value (A_h) using the natural period of vibration (T_k) of mode k:

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

For medium soil in zone III, Z = 0.16, I = 1, R= 5

ix. Mode – I

T_1 = 0.809 sec

$$\frac{S_a}{g} = \frac{1.36}{T} = \frac{1.36}{0.809}$$
$$= 1.681 \ (for\ 0.55\ <= T\ < 4.0)$$
$$\therefore A_h = \frac{0.16}{2} \text{ x} \frac{1}{5} \text{ x } 1.681 = 0.0269$$
$$Q_{i1} = \mathrm{A_{h1}} \text{ x } \mathrm{P_1} \text{ x } \emptyset_{i1} \text{ x } \mathrm{W_I} = (0.0269 \text{ x } 1.355) \text{ x } \emptyset_{i1} \text{ x } \mathrm{W_I}$$

$$= 0.03644 \text{ x } \emptyset_{i1} \text{ x } \mathrm{W_I}$$

x. Mode – II

T_2 = 0.386 sec

$$\frac{S_a}{g} = 2.5 \ (for\ 0.1\ \le= T\ \le 0.55)$$
$$\therefore A_h = \frac{0.16}{2} \text{ x} \frac{1}{5} \text{ x } 2.5 = 0.04$$
$$Q_{i2} = \mathrm{A_{h2}} \text{ x } \mathrm{P_2} \text{ x } \emptyset_{i2} \text{ x } \mathrm{W_I} = (0.04 \text{ x} - 0.3606) \text{ x } \emptyset_{i2} \text{ x } \mathrm{W_I}$$

$$= (-0.0144\ )\text{x } \emptyset_{i2} \text{ x } \mathrm{W_I}$$

xi. Mode – III

T_3 = 0.177 sec

$$\frac{S_a}{g} = 2.5 \ (for\ 0.1\ \le= T\ \le 0.55)$$
$$\therefore A_h = \frac{0.16}{2} \text{ x} \frac{1}{5} \text{ x } 2.5 = 0.04$$
$$Q_{i3} = \mathrm{A_{h3}} \text{ x } \mathrm{P_3} \text{ x } \emptyset_{i3} \text{ x } \mathrm{W_I} = (0.04 \text{ x } 0.0132 \text{ x } \emptyset_{i3} \text{ x } \mathrm{W_I}$$

$$0.000528 \text{ x } \emptyset_{i3} \text{ x } \mathrm{W_I}$$

Calculation of Peak Lateral force Q_i and storey shear for Mode I

Storey level	Weight W_i	$A_h P_k$	$\emptyset_{i1}$	Q_{i1}	V_{i1}
Roof	680	0.03644	1.00	24.78	24.78
II-Floor	880	0.03644	0.643	20.62	45.40
I-Floor	1045	0.03644	0.498	18.96	64.36

Calculation of Peak Lateral force Q_i and storey shear for Mode II

Storey level	Weight W_i	$A_h P_k$	$\emptyset_{i2}$	Q_{i2}	V_{i2}
Roof	680	-0.0144	1.00	-9.79	-9.79
II-Floor	880	-0.0144	-0.57	7.22	-2.57
I-Floor	1045	-0.0144	-0.662	9.96	7.39

Calculation of Peak Lateral force Q_i and storey shear for Mode III

Storey level	Weight W_i	$A_h P_k$	$\emptyset_{i3}$	Q_{i3}	V_{i3}
Roof	680	0.000528	1.00	0.36	0.36
II-Floor	880	0.000528	-6.47	-3.01	-2.65
I-Floor	1045	0.000528	5.697	3.14	0.50

$\omega_1 = 7.76\ rad/sec$
$\omega_2 = 16.26\ rad/sec$
$\omega_3 = 35.48\ rad/sec$

Lowest frequency (i.e., fundamental frequency) = 7.76 rad/sec

$0.9 \text{ x}\omega_1 = 0.9 \text{ x } 7.76 = 6.98\ rad/sec$
$1.1 \text{ x}\omega_1 = 1.1 \text{ x } 7.76 = 8.54\ rad/sec$

It will be seen that ω_2 and ω_3 differ from ω_{i1} by more than 10%.
All the modes are well separated.

The same problem is solved by all the three methods as under

xii. **Absolute Sum Method (ABS):**

$$V(Roof) = [24.78 + \ (-9.79) + \ 0.36] = 15.35\ kN$$
$$V(II - Floor) = [45.40 + \ (-2.57) + \ (-2.65)] = 40.18\ kN$$
$$V(I - Floor) = [64.36 + \ 7.39 + \ 0.5] = 72.25\ kN$$

xiii. Square Root of Sum of Squares (SRSS) Method:

Peak storey shear forces due to all modes:

$$V(Roof) = \sqrt{24.78^2 + (-9.79)^2 + 0.36^2} = 26.65\ kN$$
$$V(IInd\ Floor) = \sqrt{45.4^2 + (-2.57)^2 + (-2.65)^2} = 45.55\ kN$$
$$V(Ist\ Floor) = \sqrt{64.36^2 + 7.39^2 + 0.5^2} = 64.78\ kN$$

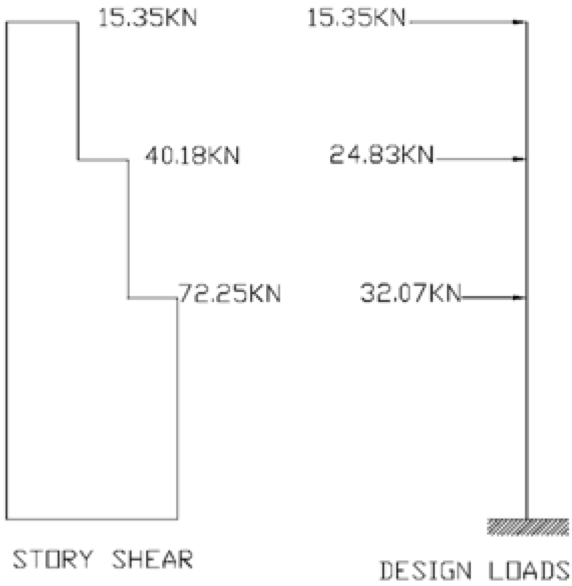

Fig. 3 ABSOLUTE SUM METHOD(ABS)

xiv. Complete Quadratic Combination (CQC):

The method (CQC) for modal combination is applicable to a wider class of structures because it overcomes the limitations of SRSS rule.

$$\lambda = \sqrt{\sum_{i=1}^{r} \sum_{j=1}^{r} \lambda_i \rho_{ij} \lambda_j}$$

Where, r = Number of modes being considered.
ρ_{ij} =Cross-modal coefficient,
λ_i= Response quantity in mode i (including sign)
λ_j= Response quantity in mode j (including sign)

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

ζ = Modal damping ratio
β_{ij} = Frequency ratio= ${}^{\omega_j}/_{\omega_i}$
ω_i = Circular frequency in j[th] mode and

ω_j = Circular frequency in j[th] mode.

For 5% damping, $\zeta = 0.05$

$$\beta_{ij} = \begin{bmatrix} \beta_{11} & \beta_{12} & \beta_{13} \\ \beta_{21} & \beta_{22} & \beta_{23} \\ \beta_{31} & \beta_{32} & \beta_{33} \end{bmatrix} = \begin{bmatrix} \omega_1/\omega_1 & \omega_2/\omega_1 & \omega_3/\omega_1 \\ \omega_1/\omega_2 & \omega_2/\omega_2 & \omega_3/\omega_2 \\ \omega_1/\omega_3 & \omega_2/\omega_3 & \omega_3/\omega_3 \end{bmatrix}$$

$\omega_1 = 7.76\, rad/sec$

$\omega_2 = 16.26\, rad/sec$

$\omega_3 = 35.48\, rad/sec$

$$\therefore \beta_{ij} = \begin{bmatrix} 7.76/7.76 & 16.26/7.76 & 35.48/7.76 \\ 7.76/16.26 & 16.26/16.26 & 35.48/16.26 \\ 7.76/35.48 & 16.26/35.48 & 35.48/35.48 \end{bmatrix}$$

$$= \begin{bmatrix} 1.00 & 2.095 & 4.572 \\ 0.477 & 1.00 & 2.182 \\ 0.219 & 0.458 & 1.00 \end{bmatrix}$$

$$\rho_{11} = \frac{8 \text{ x } 0.05^2(1+1.0)1.0^{1.5}}{(1-1.0^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 1.0(1+1.0)^2} = 1.0$$

$$\rho_{21} = \frac{8 \text{ x } 0.05^2(1+0.477)0.477^{1.5}}{(1-0.477^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 0.477(1+0.477)^2} = 0.0160$$

$$\rho_{31} = \frac{8 \text{ x } 0.05^2(1+0.219)0.219^{1.5}}{(1-0.219^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 0.219(1+0.219)^2} = 0.00275$$

$$\rho_{12} = \frac{8 \text{ x } 0.05^2(1+2.095)2.095^{1.5}}{(1-2.095^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 2.095(1+2.095)^2} = 0.0161$$

$$\rho_{22} = \frac{8 \text{ x } 0.05^2(1+1.0)1.0^{1.5}}{(1-1.0^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 1.0(1+1.0)^2} = 1.0$$

$$\rho_{32} = \frac{8 \text{ x } 0.05^2(1+0.458)0.458^{1.5}}{(1-0.458^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 0.458(1+0.458)^2} = 0.0143$$

$$\rho_{13} = \frac{8 \text{ x } 0.05^2(1+4.572)4.572^{1.5}}{(1-4.572^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 4.572(1+4.572)^2} = 0.00274$$

$$\rho_{23} = \frac{8 \text{ x } 0.05^2(1+2.182)2.182^{1.5}}{(1-2.182^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 2.182(1+2.182)^2} = 0.0143$$

$$\rho_{33} = \frac{8 \text{ x } 0.05^2(1+1.0)1.0^{1.5}}{(1-1.0^2)^2 + 4 \text{ x } 0.05^2 \text{ x } 1.0(1+1.0)^2} = 1.0$$

$$\therefore \rho_{ij} = \begin{bmatrix} \rho_{11} & \rho_{12} & \rho_{13} \\ \rho_{21} & \rho_{22} & \rho_{23} \\ \rho_{31} & \rho_{32} & \rho_{33} \end{bmatrix} = \begin{bmatrix} 1.0 & 0.0161 & 0.00274 \\ 0.0160 & 1.0 & 0.0143 \\ 0.00275 & 0.0143 & 1.0 \end{bmatrix}$$

Here the terms λ_1, λ_2 and λ_3, represents the response of different modes of certain storey level

$$\begin{Bmatrix} V_1 \\ V_2 \\ V_3 \end{Bmatrix} = \sqrt{[\lambda_1 \quad \lambda_2 \quad \lambda_3] \left[\begin{bmatrix} \rho_{11} & \rho_{12} & \rho_{13} \\ \rho_{21} & \rho_{22} & \rho_{23} \\ \rho_{31} & \rho_{32} & \rho_{33} \end{bmatrix}\right] \begin{Bmatrix} \lambda_1 \\ \lambda_2 \\ \lambda_3 \end{Bmatrix}}$$

Using matrix notation Storey shear for each mode and other storey are as under:

Storey Shear for Roof are as under:

Mode 1 Storey Shear= 24.78 kN, Mode2 Storey Shear= -9.79 kN. Mode3StoreyShear =0.36 kN

$V(Roof) =$

$$\sqrt{[24.78 \quad -9.79 \quad 0.36] \begin{bmatrix} 1.0 & 0.0161 & 0.00274 \\ 0.0160 & 1.0 & 0.0143 \\ 0.00275 & 0.0143 & 1.0 \end{bmatrix} \begin{Bmatrix} 24.78 \\ -9.79 \\ 0.36 \end{Bmatrix}}$$

$$= \sqrt{[[24.78 \quad -9.79 \quad 0.36]] \begin{Bmatrix} 24.62 \\ -9.39 \\ 0.29 \end{Bmatrix}}$$

$$= \sqrt{702.116} = 26.49\ kN$$

Storey ShearforII - Floor are as under:

Mode 1 Storey Shear= 45.40 kN, Mode 2StoreyShear= -2.57 kN. Mode3StoreyShear =-2.65 kN

$V(II - Floor) =$

$$\sqrt{[45.40 \quad -2.57 \quad -2.65] \begin{bmatrix} 1.0 & 0.0161 & 0.00274 \\ 0.0160 & 1.0 & 0.0143 \\ 0.00275 & 0.0143 & 1.0 \end{bmatrix} \begin{Bmatrix} 45.40 \\ -2.57 \\ -2.65 \end{Bmatrix}}$$

$$= \sqrt{[[45.40 \quad -2.57 \quad -2.65]] \begin{Bmatrix} 45.35 \\ -1.88 \\ -2.56 \end{Bmatrix}}$$

$$= \sqrt{2070.506} = 45.50\ kN$$

Storey Shear for I - Floor are as under:

Mode 1 Storey Shear= 64.36 kN, Mode 2StoreyShear= 7.39 kN. Mode3StoreyShear =0.5 kN

$V(I - Floor) =$

$$\sqrt{[64.36 \quad 7.39 \quad 0.5] \begin{bmatrix} 1.0 & 0.0161 & 0.00274 \\ 0.0160 & 1.0 & 0.0143 \\ 0.00275 & 0.0143 & 1.0 \end{bmatrix} \begin{Bmatrix} 64.36 \\ 7.39 \\ 0.5 \end{Bmatrix}}$$

$$= \sqrt{[64.36 \quad 7.39 \quad 0.5] \begin{Bmatrix} 64.48 \\ 8.426 \\ 0.783 \end{Bmatrix}}$$

$$= \sqrt{4212.592} = 64.90\ kN$$

Floor Level	Static Analysis	Daynamic Analysis		
		ABS	SRSS	CQC
Roof	52.75	15.35	26.65	26.49
IInd Floor	88.28	40.18	45.55	45.50
Ist Floor	104.2	72.25	64.78	64.90

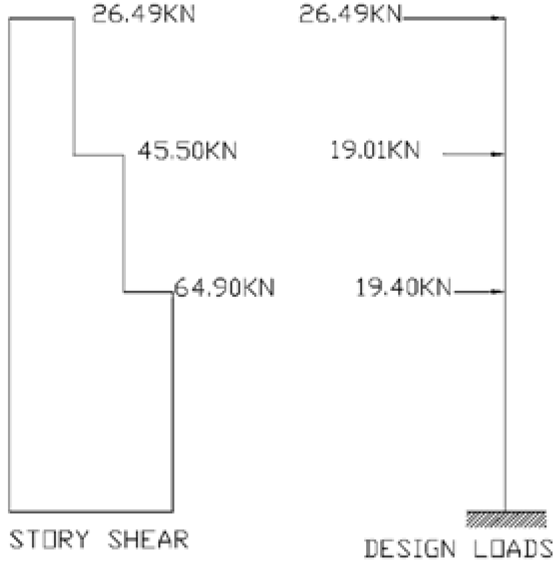

Fig. 5 CQC METHOD

Since V_B (= 64.78 kN) $< \bar{V}_B$ 104.2 kN

The response quantities obtained from SRSS method will be scaled up i.e., to be multiplied by $\frac{\bar{V}_B}{V_B}$

$$\therefore \text{Scaling factor} = \left[\frac{104.2}{64.78}\right] = 1.609$$

The details are shown in fig. 6

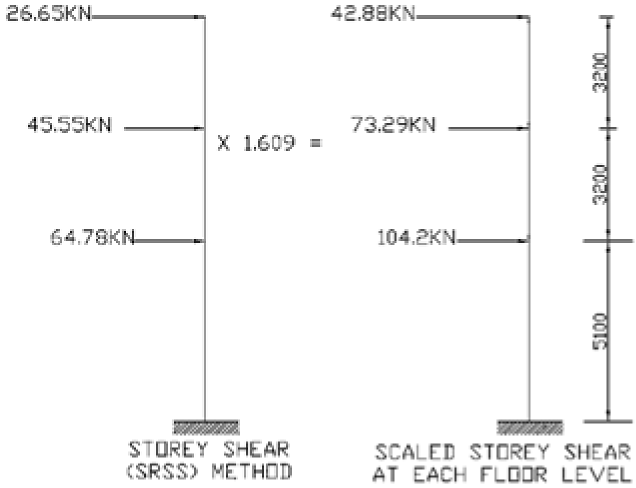

Fig. 6 SCALED STOREY SHEAR-SRSS METHOD

11. Result

Storey shear for static and scaled SRSS dynamic method

Floor Level	Storey shear kN	
	Static (ABS)	Daynamic (SRSS)
Roof	52.75	42.88
IInd Floor	88.28	73.29
Ist Floor	104.2	104.2

12. Conclusion

Storey shear are significantly affected by change in load distribution. Advantage of dynamic analysis is to reduce storey shear. Seismic analysis carried out by static method gives higher values of storey shear.

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