ANALYSIS OF MULTISTOREY BUILDING USING 3D BEAM ELEMENT

A DISSERTATION

submitted in partial fulfilment of the
requirements for the award of the degree
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MASTER OF ENGINEERING
in
EARTHQUAKE ENGINEERING
(With Specialization in Structural Dynamics)

By

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CANDIDATES DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled "ANALYSIS OF MULTISTOREY BUILDINGS USING 3D BEAM ELEMENT" in partial fullfillment of the requirements for the award of the degree of MASTER OF ENGINEERING IN EARTHQUAKE ENGINEERING with specialization in STRUCTURAL DYNAMICS, submitted in the Department of Earthquake Engineering, University of Roorkee, ROORKEE, is an authentic record of my own work under the supervision of Dr. S.K. Thakkar, Professor and Mr. R.N. Dubey, Lecturer, Department of Earthquake Engineering, University of Roorkee, Roorkee (U.P.) India.

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ABSTRACT

3-D dynamic analysis has been carried out for three multistory buildings (3,4 and 7 storeyed) to study and evaluate the modal combination techniques proposed by different authors in CQC , AKG , DALS , DABS and SRSS .

Due to gradual development in the subject of input ground motion it has become evident that the longitudinal , transverse and vertical components of the ground motion taken simultaneously would affect the overall response of structures particularly high rise and unsymmetric structures . An attempt has been made in this thesis to highlight the contribution of different components of input ground motion taken together .

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LIST OF NOTATIONS

Notation	Description
ABS	Absolute sum of modal maxima
AKG	Prof. Ajaya Kumar Gupta's Combination
C	Damping matrix
CQC	Complete Quadratic Combination
DALS	Double Algebraic Sum
DABS	Double Absolute Sum
E_C	Short term static modulus of elasticity of concrete in N/mm^2
K	Stiffness matrix
M	Mass matrix
m_i	Modal Mass
PSa	Pseudo spectral acceleration
SRSS	Square root of sum of squares
Sd	Spectral displacement
U	Displacement vector
$\dot{U}$	Velocity vector
$\ddot{U}$	Acceleration vector
[R]	Rotation transformation matrix
f_{ck}	Characteristic cube strength of concrete in N/mm^2
i	Corresponds to ith mode of vibration
j	Corresponds to jth mode of vibration
n	No. of modes
td	Duration of white noise segment
α	Rigid response coefficient
ω	Circular frequency
ω_{Di}	Damped circular frequency in ith mode
ω_{Dj}	Damped circular frequency in jth mode
ρ_{ij}	Cross modal coefficients in CQC
ε_{ij}	Cross modal coefficients in AKG, DALS, DABS
ζ_i	Modal damping ratio

CHAPTER 1

INTRODUCTION

1.1 GENERAL

A large portion of our country is situated in active seismic zones with a varying degree of activity both in forms of frequency of occurance and maximum ground accelerations. It, therefore becomes important that structural systems be designed to resist this effect in addition to the conventional loads. The importance of appropriate seismic analysis and design assumes a predominant role for more important structures such as nuclear power plants ,dams, multistorey buildings etc., where even non-structuraldamages could lead to disastarous consequences.

Multistorey Buildings have different types of external configurations that are quite different from idealized rectangular shapes both in plan and elevation viz L, H,T, Y & U shaped buildings, assymmetric buildings, & buildings with setbacks. Set back configurations are a common vertical irregularity in building geometry and they consist of one or more abrupt reductions of floor area with the height. Generally such configurations are introduced mainly from architectural point of view. Albeit, it is desirable to avoid the unusual configurations, whenever possible,especially if it leads to coupled modes. Hence, it is essential to do the dynamic analysis of such buildings rather than pseudo static analysis, to predict the actual behaviour of buildings which depends on the distribution of mass and stiffness both in horizontal as well as in vertical planes. Additionally the foundation flexibility, nature of ground motion (Single or multi component) and direction of ground motion also affect the seismic response of the buildings. The conventional approach of structural design for earthquake forces is to use the Response Spectrum Technique, where two dimensional plane frame is analysed independently in two principal directions of structure for unidirectional ground motion. The basis for this analysis is generally valid only for those buildings which have no eccentricity. For irregular buildings the centre of mass and centre of rigidity do not coincide on the same floor as well as on different floors. This non-coincidence of Centre of Mass and Centre of Rigidity yields torsion in buildings. By plane frame analysis we can not take into account this torsion caused by eccentricity. The ground motion during earthquake essentially consists of three translational and three rolational components along three mutually perpendicular coordinate axes. In comparision to translational components, the rotational components have lesser magnitude. But these rotational components cause complex torsional response in structures. Hence, for the precise evaluation of the dynamic response of structures it is imperative to use the three dimensional dynamic analysis. This 3D dynamic analysis also takes into account the coupled translational rotational behaviour of entire structure. In this analysis the frame interaction in different planes and effect of eccentricity are also accounted for. Here the main emphasis is on the translational components of earthquake for obtaining the resultant response. There are a number of analytical alternatives which offer a wide variation in techniques i.e. deterministic, non-deterministic and empirical/semi empirical methods with varying degree of mathematical complexity, solution time and reliability of parameters in estimation of response parameters.

1.2. Response Spectrum Technique

The Response Spectrum Technique is well established in the literature of earthquake resistant design and applied widely in practice. The technique is simple, inexpensive and efficient since it does not involve a rigorous time history analysis for response for earthquake forcing function generally. Only the first few modes of vibration of the structure need be known and peak response parameters may be determined using a response spectrum curve derived for a designed intensity of earthquake this technique is applicable only for linear analysis of buildings.

Response spectrum technique used for dynamic analysis provides maximum values of any response in various modes of vibration. Maximum values in general would not occur simultaneously. The relative phasing between these maximum values is lost in the development of the spectrum and is not available for calculating the maximum combined response. Lack of time phase information in response spectrum has been a source of problem for combining responses from various modes. Extension of this method to multicomponent excitationsadds one m unknown to the problem, because it involves the uncertainty of spatial combination of the maximum in adding to uncertainties regarding combination of modal maxima. Most of the experience with this technique comes from the analysis in which only one motion component has been used. Modal combination rules based on the probablity considerations provide reasonable estimates of maximum response for a large class of earthquakes. So they are widely accepted and used in design.

When multicomponent excitations are considered the question of statistical dependence arises. In such cases the assumed that motions are independent. The assumption of statistical independence may not always be justified. Motions with a strong unidirectional character e.g. such as on hard ground at small epicentral distances and from shallow earthquake constitutes a class of excitations where components can be expected to exihibit appreciable correlations.

A third type of uncertainty is associated with design that involves more than one component of stress. For a space frame, the force quantities in such equations will generally peak at different times. The response spectrum technique, however does not provide any information that would allow estimates of maximum combined effects. Thus conservative results may be expected. A typical example of this in practice is, the peak (response spectrum) values of the force and moments in the section are used without any reduction.

The method of modal analysis in based on the fact, that for certain forms of damping the response in a mode of vibration can be computed independently and the modal responses are combined to determine the maximum total response. The response in a mode of vibration can be modelled by the response of SDOF oscillator and the maximum response can be directly computed from response spectrum.

In response spectrum technique an approximation to maximum response is obtained by combining modal maxima for the response to each component (Modal combination) and then combining the resulting partial responses (spatial combination) if there is any difference from exact solutions this error can be attributed to combination methods used, not to the differences in the spectrum ordinates.

For complex 3 - dimensional structures such as nuclear power plants, dams, piping systems and building with unusual configurations the direction of the earthquake which produces maximum stress in a particular member or at some specified point in not apparent. A number of dynamic analysis at various angles are performed in order to check all points for critical earthquake directions in time history analysis. Such elaborate study could provide a different critical input direction for each stress evaluated, but cost and solution time of such study may not allow it.

1.3 Objective and scope of study:

The objectives of this thesis are as follows :

1. Comparison of different modal combination techniques used in the thesis.

2. Comparison of responses obtained by giving multicomporent

response spectra as input.

In order to achieve aforementioned objectives 3,4 and 7 storeyed R.C. framed buildings with unusual configurations without shear wall have been considered. Soil structure interaction is not taken into account. Buildings have been analysed for unidirectional as well as for multicomponent input response spectra.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The majority of buildings are analysed and designed in accordance with the building codes the earthquake loading is defined in terms of an equivalent lateral force, and a static analysis of the building is performed. In recent years, building codes have adopted more and more features of the formal dynamic structural analysis, while retaining their original formats. The most popular and relatively rigorous building code presently in use in the profession is the Uniform Building Code. For buildings having coupled translational rotational response these are two methods of analysisas given below:

a) The uncoupled analysis may be done to find shear and moments as a first step and then the seismic torsional analysis may be performed to modify the lateral shear values due to eccentricity in the structure.

b) The second method is to consider the structure torsionally coupled translational relation of the structure subjected to either unidirectional ground motion or motion in two orthogonal directions.

If the horizontal ground motion is not uniform over the base of structure the rotational motion will occur even in symmetrical buildings. This source of rotational motion is not considered here. Samant et al (1978) showed that in multistoreyed building the chosen direction of ground motion will not cause the worst critical force in a frame element due to assymmetry of the building. Hence is is necessary to analyse the structure for vaious assumed directions of the ground motion and obtain the critical value of the design force.

Rutenberg et al (1978) proposed a scheme to calculate the effect of torsion in assymmetric buildings in the context of response spectrum technique. The scheme consists of: obtain the modal shear and torque on building by RST, compute the total modal shear forces on each frame i.e. shears due to lateral load effect and torsional effect are combined algebraically. Then these modal shear are combined in SRSS manner. Earlier SRSS shear and shear due to torsion (SRSS) were combined. Such a technque is intrisincally incorrect, Since the different phases between rotation and translation in each mode are lost. Based on their study they found that proposed scheme gives a good estimate for maximum response of assymetric building; while conventional approach tends to over estimate the response. This effect becomes more pronounced for frames which are located away from reference axis.

Gupta and Gupta (1981) had shown in their study that coupled translational and rotational frequency are changed as compared to uncoupled frequency. For the building under consideration they show that coupled frequency reduces and rotational frequency increases as compared to the uncoupled frequency of the structure.The dynamic torques and deflection of buildings are generally higher, when the bidirectional ground motion is considered, than to the response obtained under unidirectional ground excitations.

Fernandez (1982) had evaluated the effects of uneven distribution of mass and stiffness in elastic response of multistorey buildings and he discussed about what would be an adequate distribution of lateral forces, that for design purposes are assumed to be acting at each storey. This study shows that type of Earthquake does not effecttoo much response in low rise buildings ascompared with highe rise buildings. A very good behaviour of the structures in both cases viz. low rise buildings and high rise buildings is achieved when the structure has continuous variation of the stiffness or uniform weight and stiffness.

Reddy, D.P. et al (1973) had analysed dynamically a 40 storey framed tube office building using a 3-D model. The structure was subjected to a base motion associated with large magnitude earthquake. The dynamic response was compared with 3D and 2D static analysis based on UBC. The dynamic behaviour was also compared with pseudo dynamic method using 3D model. Based on their study they found that, dynamic analysis indicates the 3D behaviour, even though the building is symmetrical about two centre lines of building. This is expected because tube type building is truly a 3D structure. The dynamic response is based on five mode shapes. Although the maximum storey shear and maximum member moments for dynamic case are generally higher than UBC static case the maximum storey deflections and maximum column axial forces are lower. Axial deformation are very important for tall buildings. In present case axial deformation contributions increased the total horizontal deformations as high as 50%. The conventional 2D frame analysis results in significant error for tall buildings. The dynamic forces are above UBC design and below ultimate member strengths. Thus, except for an increase in reinforcing a few members no modification in design is recommended. The building is well conditioned for satisfactory response to earthquake input.

2.2 MODAL COMBINATION RULES

1. Sum of absolute modal maxima

Biot (1943) had given this rule which gives upper bound on the response. It assumes that all modes reach their maxima with the same sign at the same instant in time.

2 Square root of sum of square of modal maxima

Goodman et al (1953) gives a rule for combining modal maxima based on probabilistic theory. The modal maxima occurs at different times hence they can not be treated in single statistics. This rule gives most probable values of response as square root of sum of squares of modal maxima values. This rule gives lower bound to the response.

Newmark and Jenings (1960) have analysed the systems with different no. of degrees of freedom and combined the modal values by the aforesaid rules and found them acceptable. Clough R.W. (1960) has further extended the idea and compared two methods with exact analysis. First one is SRSS as proposed by Goodman et al. Second one is based on the concept that first mode contributes the major part of total response, while the higher modes essentially provide a correction to the first mode response. An appropriate factor of the second mode response is combined with the first mode response. This factor varies for different response values The ABS rule may approximate the envelope of the response values.

H.C.Merchant and D.E. Hudson (1962) had proposed the suitably weighted average of the sum of absolute values of modal maxima and the SRSS of the modes will give practical design criterion for the base shear forces in multistorey buildings. On comparing they found that the method proposed by them is applicable to limited type of structures and earthquake excitation considered. For distinctly different type of situation encountered he proposes that additional studies are required.

3. SRSS and ABS Sum Linear Combination

Arturo Arias S. and Raul Hurid L. (1963) further extended the ideas of Newmark et al (1960) and R.W. Clough (1962) and Hudson et al (1962). They proposed a formula for approximating the maximum earthquake response of shear building. This formula gives maximum shears as a linear combination of the SRSS and ABS values i.e.

$$V_k = (1-\beta)\ \Sigma\ |V_{ik}| + \beta \sqrt{\sum_i V_{ik}^2}$$

$$\beta = \frac{4}{27} \left(\frac{3}{2}\right)^{\log N/\log 2}$$

in which

V_{ik} = max^m Shear at k^{th} storey corresponding to ith mode

N = No. of stories.

β = Dimensionless coefficient which varies with the no. of stories (increases as storey nos increases).

4. I.S. Code Method

As per I.S. 1893 the lateral load Qi(r) acting at any floor level i due to rth mode of vibration is given by the following equation

$$\phi_i\ (r) = K\ \omega_i\ \phi_j\ (r)\ Cr\ \ \alpha_h\ (r)$$

in which w_i - weight of the floor

K - performance factor depending upon type of building.

$\phi_i(r)$- mode shape coefficient at floor i in rth mode of vibration

C_r - Mode participation factor

$\alpha_h(r)$ -design horizontal seismic coefficient corresponding to rth period.

The mode participation factor may be given as

$$C_r = \frac{\sum_{i=1}^{n} W_i \, \phi_i(r)}{\sum_{i=1}^{n} W_i \, \phi_i^2(r)}$$

n = No. of node.

The shear force Vi, acting in the ith storey may be obtained by superposition of first three modes as follows:

$$V_i = (1-\gamma) \sum_{i=1}^{3} V_i^{(r)} + \sqrt{\sum_{i=1}^{3} V_i(r)^2}$$

The coefficient γ depends upon the height of the buildings

	Height H (m)	γ
upto	20	0.4
	40	0.6
	60	0.80
	90	1.00

For intermediate height of buildings value of γ may be obtained by linear interpolation.

In world, different countries propose different modal combination rules, but all are related some how to aforesaid combination rules.

5. Double Sun Combination Methods.

The methods, which are under this topic, are fully discussed under chapter 3. These rules are an improvement over the SRSS rule, especially when the modal frequencies are closely spaced. These methods account for the mutual reinforcement/cancellation of modal response values.

6. Grouping Method

This is another improvement over the SRSS to account for closely spaced modes. This method is also proposed by the U.S N.R.C for nuclear buildings. In this method the modes are divided into groups, that include all modes having frequencies lying between the lowest frequency in the group and a frequency 10% higher. For each group the representative value of the response is taken as the sum of absolute values of modal maxima belonging to that group. The maximum response is then obtained as the SRSS of the representing group values.

7. 10% Method

This is another improvement over the SRSS rule in closely spaced modes. It has at least as many terms as in method(6), giving the same or more conservative result. Mathematically it is represented as

$$R = \left\{ \sum_{k=1}^{n} R_j^2 + 2 \sum_{i \neq j} R_i \, R_j \right\}^{1/2}$$

where the second summation is to be taken over all these methods satisfying the inequalities

$$\omega_i < \omega_j \leq 1.10\ \omega_i \qquad \& \quad 1 \leq i < j \leq n$$

This combination method is also proposed by USNRC.

8. NRL combination method

In the Naval research laboratory (NRL) a group had developed a new combination of model maxima values of response as, maximum of all modal maxima plus the SRSS of the rist modal values. This has been used in response studies of Submarine Structures to under water explosions as well as in seismic structural design.

9. Average of SRSS and NRL values

This gives the response estimate between SRSS and $\frac{\text{SRSS + ABS}}{2}$.

10. Advanced Response combination Technique (ARC)

N.C. Tsai (1984) had shown that modal coupling factor in CQC method, based on the the assumption of EQ ground motions are ideal stationary random processes and they are independent of values of modal frequencies, has some draw backs EQ ground motions are non stationary processes and does not contain a limited frequency band. It can be proved analytically that the combination between the responses of two modes converge to an algebraic sum when both the modal frequencies are sufficently low or high even though they may not be closely spaced.

This condition calls for ρ_{ij} to be a function of modal frequency such that it approaches the value of 1.0 at both sufficiently low and high frequencies. The advanced response combination method (ARC) has proposed that modal cross correlation factor to be as follows

$$\varepsilon_{ij} = 1 - H\,(f_{ij})\ \Delta^2_{ij} \,/\, \left\{\Delta^2_{ij} + 4\left[\zeta_j + 0.01\right]^2\right\}$$

in which H(f) is a linear function between the following coordinating points.

fij	o	1	5	15	23	33	Hz
H	0.0	1.0	1.0	0.3	0.1	0.0	

$$\Delta_{ij} = f_i - f_j / \bar{f}_{ij} \qquad \bar{f}_{ij} = (f_i + f_j)/2$$

Based on his study, Tsai found that for building having frequencies of first 3 modes as 35.0, 74.79 and 111.23 Hz, the ARC method was simply reduced to an algebraic combination of modal responses. Because, all three modal cross coefficients are equal to 1.0. As the frequency differential approaches zero, $\rho_{ij} = 1.0$ for both methods. This implies that both methods are equally adequate for closely spaced modes. Although he did not illustrate the comparison of CQC & ARC method.

Patricio Ruiz has proposed a double sum equation for combining the modal values when the modes are closely spaced. Lee C.T. et al (1988) also proposed a similar form of equation for combining modal values accounting soil condition and for horizontal and vertical direction of ground motion. In all these papers the difference lies in the definition ofmodal cross correlation coefficient. In majority of cases they found that equation proposed by Wilson, Kiureghian and Bayo is accurate and easy to use.

Peruvian seismic regulations establish as modal combination the average (AVn) of the sum of absolute maximum responses V and of the square root of sur of squares (SRSS) of the first n modes. Peruvian earthquakes are originated in the subduction zone between the Nazca and South American plates relatively close and paralled to Andean ridge. Their records have unusually high frequency contents implying that this combination adopted from areas subjected to different earthquake may not be applicable. In fact it has been found to be too conservative and SRSS to be unsafe.

Pique and Echarry (1988) proposed the following combination rule (weighted average method),

0.25 ABS + 0.75 SRSS

Based on the study of 4, 8, 12 and 15 storey high framed buildings they concluded that the weighted average combination WAn is safer than SRSS and gives a good estimate for global and local responses, specially of flexible frames, provided an adequate no. of modes are considered. AVn response falls on target and low and high values are within reasonable limit. The SRSS combination underestimates the results regardless the number of modes considered. WAn will always give lower responses than AVn and higher than SRSS a meaning neither conservative nor inadequate respones in frames taller than 8 floors. AVn may be a valid alternative for strategic structures where a 100% certainly of seismic force estimation in needed but for normal building frames it is too conservative.

Singh and Mehta (1983) showed that combination of maximum modal responses to obtain the seismic design response of a linearly behaving structural system pseudo acceleration spectra is used as seismic input. In evaluation of design response of structures with closely spaced frequencies modal correlation coefficients are considered, with assumption of white noise as input. It is shown that these correlations are not reliable for high frequency modes, but the correlation factors are important when the design response has a significant contribution from higher modes. To obtain an accurate evaluation of response, it is necessary that all modes calculated with high precision be used in the analysis. If the system is flexible relative to the frequency of the input the formulation based on white noise as input can provide an accurate value of design response. They also show that the modes with period less than the zero acceleration period can be omitted from the analysis. The zero period is the period of an oscillator below which no amplification in pseudo acceleration is obtained USNRC consider the 0.03sec as zero period.

The method proposed by Singh and Mehta considers the stationarity of ground motion. This assumption do not influence the applicability of the proposed SRSS rule than they do the existing combination rules. In this approach the effect of high frequency modes is included through a static analysis. The additional computational effort spent in static analysis, which requires the solution of a set of linear simultaneous equations, constitutes only a small part of the effort spent in the evaluation of high frequency modes in eigen value analysis.

2.2.1 Prof. A.K. Gupta's Obsevations

Gupta and Cordero (1981) had shown that in themodal cross correction coefficient given by Rosenblueth and Elorduy it is not clear what value td should be used. When the frequencies $w_1$and $w_2$are sufficiently large and for relatively large values of critical damping ratio, the term td does not play a significant part. However, when the frequencies are small, the term containing td increases the effective value of damping, thus giving a larger value of ε_{12}. It is in this case the value of $\varepsilon_{12}$is quite sensitive to the value of td. Using complete duration of ground motion does not appear to be reasonable.

Kennedy (1979) had shown that when modal frequencies are higher than maximum ground motion frequency the modal cross correlation coefficient does not hold. In fact, response time histories will be practically scaled input time histories, and would be almost perfectly correlated, in which case ε_{12}=1.0, even when $\omega_1$and $\omega_2$are sufficiently apart. They also found that even at other frequencies in the range greater than 1Hz, significant correlation between modes existed. Kennedy also pointed out that when the modal frequencies are sufficiently apart, beyond a certain point, the correlation between modal response may start increasing, rather than decrease as predicted by equation ( ). Heuristically, the reason is simply that it would be quite likely that the high frequency response can easily be maximum about the same time, when the low frequency response reaches the maximum. Gupta and Cordero, however have found no such evidence. The reason for this is that different segments of ground motion have different frequency contents. It is unlikely that same segment of motion would encite two modes with widely disparate frequencies. Gupta and Cordero (1981) had proposed another method for calculating ε_{ij}. Based on the observation of the modal responses and their combinations, a heuristic assumption is made. Any modal response Ri consists of two parts, a damped periodic response R_i^p which has characteristics similar to that obtained by using a finite segment of white noise, and a rigid response, R_i^r which is perfectly correlated with the input ground motion. It is further assumed that the two parts are mutually uncorrelated i.e.

$$R_i^2 = R_i^{p2} + R_i^{r2}$$

Thus we can write $R_i^r = \alpha_i R_i$ and $R_i^p = \sqrt{1-\alpha_i^2}\; R_i$

when the two modal responses R_1 and R_2 with frequencies ω_1 and ω_2 are combined, then the combined response is given by

$$R^2 = R^{r^2} + R^{p^2}$$

in which $R^r = \alpha_1 R_1 + \alpha_2 R_2$ $\quad R_p^2 = R_1^{p2} + R_2^{p2} + 2\,\varepsilon_{12}^p R_1^p R_2^p$

$$\text{where} \qquad \varepsilon^{p}_{12} = \left[1 + \left[\frac{\omega_2 - \omega_1}{\zeta_1\omega_1 + \zeta_2\omega_2 + C_{12}} \right]^2 \right]^{-1}$$

$$\text{Where} \quad C_{12} = (0.16 - 0.5\ \zeta_{12}) \ (1- | \omega_1^2 - \omega_2^2 |) \geq 0$$

Or we can say that value of td varies with the amount of critical damping and with $|\omega_1^2-\omega_2^2|$

$$\text{or} \qquad \varepsilon_{12} = \alpha_1\ \alpha_2 + \sqrt{(1-\alpha_1^2)(1-\alpha_2^2)} \quad \varepsilon^{p}_{12}$$

This equation gives value of ε_{12} which are quite close tonumerically calculated values for a wide range of frequencies including high frequencies. When,

$$\omega_2 \rightarrow \infty \quad \alpha_2 = 1 \quad \varepsilon^{p}_{12} = 0; \quad \varepsilon_{12} = \alpha_1$$

Hereαvarieswith the modal frequency and is a function of critical damping. The rigid response coefficient in given as

$$\alpha_i = \frac{\log f_i/f_1}{\log f_2/ f_1} \qquad 0 \leq \alpha_i \leq 1$$

$$f^1 = \frac{S_{amax}}{2\pi\ S_{rmax}} \quad \text{Hz} \qquad f^2 = (f_1 + 2f^r)/3, \text{HZ}$$

for $f_i \leq f_1$, $\alpha = 0$ and for $f_i \geq f_2$, $\alpha = 1$

Based on his study he found that even modes with a range of frequency immediately below rigid frequency continue to be perfectly correlated with the input acceleration. This correlation tends to diminish gradually. Gupta has used his method for several problems and found them acceptable as they give results close to time history methods.

A comparison of the double sum, SRSS,CQC,and the absolute sum combination rules was made by Mason et al. They analyzed the fifteen story steel moment resisting frame structure of the University of California Medical Centre located in San Francisco. Two building models were formulated. For both the models a constant 5% modal damping was used. The first was the regular building and the second was an irregular building with mass offset from the stiffness center of the building. The regular building did not have interaction between modes with closely spaced frequencies. The efore, as one would expect the double sum and the SRSS rules gave comparable results, which were also very close to the time history results for the regular building, the absolute sum rule over estimated the response values significantly. In the irregular building, the modes in the two orthogonal directions became coupled leading to interacting modes with closely spaced frequencies effective duration of the earthquake ground motion was taken to be 10 sec. The earthquake motion was applied in the east and west direction. The response in the north south direction, and the rotational torque response was generated due to the eccentricity between the mass and the stiffness centers. The parallel east west response values from the double sum and CQC are comparable; the SRSSvalues have relatively higher errors, the errors from the absolute sum calculations are the highest, Similar conclusions can be made about the torsional response, except that the absolute sum values now have much higher errors. All the combination rules have the highest errors in the orthogonal north south response. The doublė sum method using the Rosenblueth Elorduy modal correlation coefficient gives the best results.

2.3 RESPONSE TO MULTICOMPONENT EARTHQUAKES

It has been customary to design structures so that they resist the envelope of effects of various component of earthquake motion, and react instead as though these components acted one at a time. There is a growing consciousness among earthquake engineers that design should take into account the simultanous action of all components for a structure founded on a rigid base in strongly seismic area, the number of significant components can be as high as six (3 in translation and as many in rotation). Criteria for the combination of various components based on a stochastic treatment of disturbances are expounded and approximate procedure which minimizes the maximum possible errors caused by the simplifications is to be adopted.

The response of buildings under these multidirectional input motions may be quite different from usual one component analysis as the stiffness and mass distribution of buildings in two horizontal directions are unequal. For multidirectional earthquake input it will be necessary to consider sufficient no. of modes to represent any coupling between two horizontal translations and torsional rotations of the building.

2.3.1 Design Criterion for Multicomponent Input.

A response spectrum analysis for a three dimensional structure should be able to accomodate multicomponent input spectra. It is reasonable to assume that motion which take place during an earthquake has one principal direction or during a finite period of time, around the time of occurance of the maximum ground acceleration, there is a principal direction. For most structures this direction is not known and for most geographical locations it can not be estimated. Therefore, the only rational earthquake design criterion is that the structure must resist an earthquake of a given magnitude in any possible direction. There is a probability that motions normal to that direction will occur simultaneously. Also it is valid to assume that these normal motions are statistically independent because of complex nature of 3-dimensioned wave propogation.

Based on these assumptions, a statement of design criterion is "A structure must resist a major earthquake motion of magnitude S_1 for all possible angles θ and at the same point in time, resist earthquake motion of magnitude S_2 or 90^0 to the angle θ". It has also been shown that one of the normal directions of a three dimensional input would be very close to vertical. Thus would coincide with the vertical structure global axis represented as S_3.

2.3.2 Spatial Combination Rule

It is very necessary to account for direction effects produced due to multicomponent encitation. Response of a structure to such an excitation will, therefore, be the result of the corresponding components of the response.

1. Sum of three absolute values: It gives the highest response among all rules listed here and is appropriate for motion whose components are highly correlated.

2. Square root of sum of three partial responses squared (SRSS): Chu et al (1972) had suggested this rule. It is required for nuclear power plant buildings.

3. Rosenblueth and contreras (1977) have suggested this method in which the resultant response is taken as the maximum of the components plus 30% of the remaining components. (max + 30%).

4. Maximum of the three components plus the SRSS of the two (NRLS).

5. Average of 1 & 2 $\frac{\text{ABS + SRSS}}{2}$

6. Average of 2 & 4 $\frac{\text{SRSS + NRLS}}{2}$

7. Maximum of the three components plus 40% of the sum of other two (Max + 40%).

8. Maximum of the three components plus 50% of the sum of other two (Max + 50%). It is recommended for chimney stacks.

All these methods are based on the assumption that all thecomponent maxima occur at the same time, which is not true in general. These methods also assume statistical independence of the component responses. Further these methods are deterministic in nature and do not take into consideration the stochastic nature of the seismic response of a structure.

CHAPTER 3

ANALYSIS

3.1 MATRIX METHOD OF STRUCTURAL ANALYSIS

The analysis of structures, static or dynamic requires the solution of large no. of linear algebraic equations, or the calculations of eigen values and eigen vectors. Hence the problem is to be handled in systematic manner with the development of digital computer, matrix method is more useful for structural analysis because it serves two basic purposes, viz.-

i) to provide a compact and efficient notation to treat the principles and methods of structural analysis in generality with least restriction on the type.of structure.

ii) to provide a notation and organisation of the steps of structural analysis for use with a digital computer.

Thus the matrix method of analysis proceeds from part to whole. The structure is idealized into a selected system which retains the properties of the original structure. The stiffness matrix of structure consists of assembly of member stiffness matrix.

3.1.1 Assumptions

Assumptions are required for the mathematical modelling of real structure in such a manner that the behaviour of the prototype structure can be simulated. The assumptions involved in the linear structural analysis are:

a) The structural material is homogeneous and isotropic.

b) The response of the structure to the load is linear.

c) All spectral members are replaced by line members oriented along the centroidal axis of the original member.

d) The line members, however retain all properties of the original members i.e. length, inclination area and the moment of inertia.

e) The member intersection are infinitesimal in size.

f) Member having a common junction are assumed to be concentric (error so introduced either in member lengths or in inclination do not cause significant error in analysis).

3.1.2 Member stiffness matrix

The stiffness matrix method of analysis is one in which compatibility of displacements is assumed and equilibrium equations at the nodes are formulated in terms of the nodal displacement components. The stiffness matrix of a rigid frame member arbitrarily oriented in a 3D space having six degrees of freedom at each end, viz-

Translation along	X axis
Translation along	Y axis
Translation along	Z axis
Rotationabout	X axis
Rotation about	Y axis

Rotation about Z axis

The stiffness matrix can be derived by imposing a unit displacement along each degree of freedom and computing the induced forces corresponding to all other degrees of freedom. The resulting matrix is the stiffness matrix of the member in local coordinate system (Figure 1) and is as shown in Appendix A.

Here the multistorey building analysis is carried out by using 3-dimensional beam element.

3.1.3 Transformation matrix

The arbitrary orientation of rigid frame members meeting at a node in 3-dimensional space makes it different to set up equilibrium equations at nodes in terms of nodal displacements. In order to establish the equilibrium equations it is essential that force components at nodes of member meeting at the node be in the same direction. The transformation of force components from member or local coordinate system (Figure 1) achieved by means of a transformation matrix (Appendix-B).

Let R be the transformation matrix which transforms the forces from local to global coordinate system and F, d, K be the force vector displacement vector and stiffness matrix respectively.

$$\{F_G\} = [R]\ \{F_L\}$$
$$\{d_G\} = [R]\ \{d_L\}$$

Further, $\{F_L\} = [R]^T\ \{F_G\}$ Since $[R]^{-1} = [R]^T$

$$\{d_L\} = [R]^T\ \{d_G\}$$

also $\{F_L\} = [K_L]\{d_L\}$

$$[R]^T \{F_G\} = [K_L]\,[R]^T \{d_G\}$$

$$\{F_G\} = \frac{[R]\,[K_L][R]^T\{d_G\}}{[K_G]}$$

$$[K_G] = [R]\,[K_L]\,[R]^T$$

3.2 DYNAMIC ANALYSIS

3.2.1 Basic Modal equations

The Global stiffness matrix [K] is obtained as described earlier. The mass matrix to be used is shown in Appendix - C. The generalized mass matrix is assumed to be diagonal and the diagonal elements at each node corresponds to the three translational and three rotational degrees of freedom. The inertial effects due to rotational degrees of freedom have also been considered.

The dynamic equilibrium equations for a three dimensional structural system subjected to a ground acceleration,

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = -[M]\{U_b\}\,\ddot{U}_g \qquad ..3.2.1.1$$

Where C is the damping matrix. The three dimensional relative displacements, velocities and accelerations are indicated by U, $\dot{U}$, $\ddot{U}$, U_b is a displacement vector obtained by statically displacing the support by unity in the direction of the input motion; $\ddot{U}_g$ is the ground acceleration. In this mode superposition method we use the following transformation or modal superposition equation.

$$\{U\} = [\phi] \{Y\} \qquad ..3.2.1.2$$

where $[\phi]$ is the matrix containing the 3-D mode shape of the system and $\{Y\}$ is the vector of normal coordinates. The introduction of this transformation and premultiplication of equation by ϕ_i^T, yields.

$$\phi_i^T M \phi \ddot{Y} + \phi_i^T C \phi \dot{Y} + \phi_i^T K \phi = - \phi_i^T M U_b \ddot{u}_g \qquad ..3.2.1.3$$

For proportional damping the mode shape have the following properties

$$\phi_i^T M \phi_i = m_i$$

$$\phi_i^T K \phi_i = \omega_i^2 m_i$$

$$\phi_i^T C \phi_i = 2\zeta_i \omega_i m_i$$

In which ϕ_i is the i^{th} column of $[\phi]$ representing the i^{th} mode shape, m_i is the i^{th} modal mass and ζ_i is the damping ratio for mode i.

Due to the orthogonality properties of the mode shapes, all modal coupling terms of the form

$\phi_i^T A \phi_j = 0$ for $i \neq j$, so the equation 3.2.1.3 reduces to

$$\ddot{Y}_i + 2\omega_i \zeta_i \dot{Y}_i + \omega^2_i Y_i = - \gamma_i \ddot{u}_g \qquad ... 3.2.1.4$$

The method of modal analysis is based on the fact that for certain forms of damping the response in a mode of vibration can be computed independently and the modal responses are combined to determine the total response. The response in a mode can be modelled by the response of the SDOF oscillator and the maximum response can be directly computed from the response spectrum.

3.2.2 Free vibration characteristics

The equation of motion for free vibration can be expressed in the form

$$[K]\ \{\phi\} = \omega^2\ [M]\ \{\phi\} \quad \text{(Generalized eigen problem)}$$

$$\text{or} \quad [K]\ \{\phi\} = \lambda\ \{\phi\} \quad \text{(Standard eigen value problem)}$$

The solution of these equations gives us the natural frequencies and corresponding mode shapes

The forms adopted are

(i) $[M]^{-1}\ [K]\ \{\phi\} = \omega^2\ \{\phi\}$

(ii) $[K]^{-1}\ [M]\ \{\phi\} = \frac{1}{\omega^2}\ \{\phi\}$

The later form is generally preferred for the sequential determination of eigen pairs. The primary reason for the above choice lies in the fact that 'Power iterations yields the maximum roots and this provides w_{min} (or T_{max}) which is useful for determining the response from the relevant spectra. In order to evaluate successive eigen pairs, Deflation has to be adopted in such a manner so as to preserve the banded nature and Symmetry of the matrix involved because of immense computational advantage gained. Gram Schmidt orthogonalization is an obvious choice inspite of error propogation (inherent in all deflation techniques). Further economy is achieved by avoiding the actual inversion of the stiffness matrix. The method of inverse iteration technique coupled with Gram Schmidt orthogonalization (Appendix - D) has been used for the solution of eigen value problem to obtain the first six modes of vibration for each problem in this dissertation.

3.3 RESPONSE SPECTRUM ANALYSIS

After finding the natural periods of vibration and associated mode shapes, the relative displacement U of a mass along any of its six degrees of freedom in a particular mode of vibration due to horizontal component of earthquake is given by

$$U^k_{id} = \phi^k_{id} \quad Y_{ij} \quad S_{dij} \qquad ..3.3.1$$

ϕ - mode shape coefficient, k - denotes the location of mass, d - denotes the degrees of freedom, i denotes the mode of vibration, j - represents the direction of ground motion (paralled to one of the degree of freedom).

The spectral displacement S_d is equal to the maximum relative displacement of mass (relative to ground) of a SDOF system having the same period as that of modal period and damping same as that of modal damping, the pseudo spectral acceleration S_a is equal to $w_i^2 \, S_d$ for elastic spectrum. The absolute acceleration is given by

$$\{A_i\} = \gamma_i \, (PS_a)_i \, \{\phi_i\} = \gamma_i \, (S_d)_i \, \{\phi_i\} \, \omega_i^2 \qquad ..3.3.2$$

$$\{F\} = [M] \, \{A_i\} = [K] \, \{U_i\} \qquad ..3.3.3$$

The member forces can be calculated by the above equations. The maximum modal displacement is proportional to mode shape and the sign of proportionality constant is given by the sign of modal participation factor. Therefore, each maximum modal displacement has a unique sign. So for the maximum internal modal forces, the response spectrum analysis predicts the individual modal maxima, but it lacks modal time phasing information. Though the relative times at which each peak modal response occurs are different and even then we combine the modal maxima for all n modes.

3.3.1 Study of Modal Combinations

1. Square root of sum of squares of modal maxima (SRSS):- Goodman - Rosenblueth - Newmark [1953] presented this rule which is based on the assumption that modal vibrations are statistically independent i.e. vibration of any mode is not correlated with any mode. This method gives the probable maximum response equal to SRSS of modal values.

$$R = \sqrt{\sum_{i=1}^{n} R_i^2}$$

2. Double Algebraic Sum Method:

If the responses to be combined are from modes with closely spaced frequencies, the SRSS method does not give accurate results. An obvious situation is when frequencies and damping of two modes are identical. In this case the response histories of the two modes are in phase. The maximum values in two modes do occur simultaneously, and they should be combined algebraically for response history R(t) that is $R(t) = \Sigma R_i(t)$. The standard deviation of the response as follows:

$$\sigma^2 = \frac{1}{td}\int_o^{td} R(t), \quad \sigma_i^2 = \frac{1}{td}\int_o^{td} R_i^2\,(t)dt \qquad ..3.3.1.1$$

where td is the duration of input ground motion. If the earthquake is stationary ergodic process then maximum responses are given by -

$$R = \eta\,\sigma \qquad R_i = \eta_i\,\sigma_i \qquad ..3.3.1.2$$

The peak factors η & η_i are a function of frequency and varies from mode to mode for combined response. However, since we are primarily interested in modal responses with close frequencies, we make an assumption $\eta = \eta_i$ for all values of i

$$\sigma^2 = \frac{1}{td}\sum_i\sum_j\int_o^{td} R_i(t)\;R_j(t)dt$$

$$= \sum_i \sigma_i^2 + \frac{1}{td}\sum_i\sum_{j\neq i}\int_o^{td} R_i(t)\,R_j(t)dt$$

$$= \sum_i \sigma_i^2 + \sum_i\sum_{j\neq i} \varepsilon_{ij}\,\sigma_i\,\sigma_j \qquad ..3.3.1.3$$

in which ε_{ij} is called modal correlation coefficient and is defined by

$$\varepsilon_{ij} = \frac{\frac{1}{td}\int_{o}^{td} R_i(t)\ R_j(t)dt}{\sigma_i\ \sigma_j} \qquad ..3.3.1.4$$

Now equations (3.3.1.2) & (3.3.1.3) give

$$R^2 = \sum_i R_i^2 + \sum_i \sum_{j\neq i} \varepsilon_{ij}\ R_i\ R_j \qquad ..3.3.1.5$$

Rosenblueth and Elorduy assumed the earthquake ground motion to be a finite segment of white noise and assumed the response to be damped periodic of the form $e^{-\zeta wt}\sin w_D t$. They degined the cross correlation coefficient as -

$$\varepsilon_{ij} = \left[1 + \left(\frac{\omega_{Di} - \omega_{Dj}}{\zeta_i'\ \omega_i + \zeta_j'\ \omega_j} \right)^2 \right]^{-1} \qquad ..3.3.1.6$$

In which ω_i and ω_j are the circular frequences of the two modes in radian/second; ω_{Di} and ω_{Dj} are the corresponding damped frequencies.

$$\omega_{Di} = \omega_i \sqrt{1 - \zeta_i^2} \qquad \omega_{Dj} = \omega_j \sqrt{1 - \zeta_j^2} \qquad ..3.3.1.7$$

and $\zeta_i' = \zeta_i + \dfrac{2}{\omega_i\ td}$

t_d- effective duration of white noise segment. Here all the summations are algebraic. So this combination is called as double algebraic sum method. For the analysis purposes duration of the white noise segment is to be taken as 20.0 secs. in this thesis.

3. Double Absolute Sum Method (DABS):

This combination method is proposed by U.S. Nuclear Regulatory Guide. The formulas are same as given in Double Algebraic Sum method except an absolute sign placed in front of second summation. USNRC does not given any reason behind placing this absolute sign.

$$R^2 = \sum_i R_i^2 + \sum_i \sum_{j \neq i} \varepsilon_{ij} R_i R_j \qquad ..3.3.1.8$$

4. Complete Quadratic Combination Method:

Wilson, Kiureghian and Bayo(1981) had proposed a new rule as the replacement of the SRSS, when the frequencies of different modes are closely spaced. It is essential to preserve the sign of modal terms because on it the accuracy of method depends. The typical response (component expressed as)

$$R_k = \sqrt{\sum_i \sum_j R_{ki} \rho_{ij} R_{kj}} \qquad ..3.3.1.9$$

R_{ki} is the typical response component in mode i and ρ_{ij} modal cross corelation coefficient.

This combination formula is of complete quadratic form including all cross - modal terms. Hence the reason for name Complete Quadratic Combination. The cross modal terms may assume positive or negative values depending on whether the corresponding modal responses have the same or opposite signs.

The cross correlation coefficient, $\rho_{ij,}$ are functions of the duration and frequencycontent of the loading and of the modal frequencies and damping ratio of the structure If the duration of the earthquake is long as compared to the periods of the structure, and the earthquake spectrum is smooth over a wide range of frequencies then it is possible to approximate these coefficients by

$$\rho_{ij} = \frac{8\sqrt{\zeta_j \zeta_i}\ (\zeta_i + r\,\zeta_j)\ r^{3/2}}{(1-r^2)^2 + 4\,\zeta_i\,\zeta_j\,r\,(1+r^2) + 4\,(\zeta_i^2 + \zeta_j^2)r^2} \qquad ..3.3.1.10$$

where $r = \frac{\omega_j}{\omega_i}$ for constant modal damping ζ_i this expression reduces to

$$\rho_{ij} = \frac{8\,\zeta_i^2\,(1 + r)\,r^{3/2}}{(1-r^2)^2 + 4\zeta_i^2\,r\,(1+r)^2} \qquad ..3.3.1.11$$

5. Gupta's Modal Combination Method

The method presented by Prof. A.K. Gupta has the improvement over the Double Sum Combination methods. The effective duration of white noise spectrum segment can not be exactly determined from response spectrum. So Villaverde (1984) obtained values of t_d for several ground motions numerically by exploiting its relationship with the expected value of pseudo - velocities at different damping values. However, he did not specify any method for evaluating t_d for a given response spectrum. To avoid the estimation of the effective duraction t_d, Gupta and Cordero modified the equation of cross correlation coefficient as follows:

$$\varepsilon_{ij} = \left[1 + \left[\frac{\omega_{Di} - \omega_{Dj}}{\zeta_i \omega_j + \zeta_j \omega_j + C_{ij}} \right]^2 \right]^{-1} \quad ..3.3.1.12$$

On the basis of their study on 10 strong ground motion records, he suggested the following expression for C_{ij}

$$C_{ij} = (0.16 - 0.5 \zeta_{ij}) (1 - | \omega_i^2 - \omega_j^2 |) \geq 0 \quad ..3.3.1.13$$

in which ζ_{ij} is the average damping ratio for modes i and j. Since, this equation is based on the average of C_{ij} values obtained for several records, it is more appropriate to use it for a broad band earthquake input.

3.4 PARAMETRIC STUDY

In order to investiagte the 3D behaviour of the structure we give the input in different directions. They are as -

i) Horizontal ground motion in X - direction only

ii) Horizontal ground motion in X - direction coupled with vertical ground motion.

iii) Horizontal ground motion in X and Z direction acting simultaneously with vertical ground motion.

The assumptions that have been made in the above parametric study, regarding the spectra selected for various inputs are as follows:

i) The available spectra was taken as the characteristic of the horizontal component of ground motion.

ii) Both the horizontal components of the ground motions are assumed to have the same characteristic feature and hence are described by the same spectra i.e. ground motion in X and Z direction are assumed to be in perfectly correlated phase.

iii) The vertical component of ground motion has been assumed to be 50 % of the horizontal component of ground motion. Here again the correlation coefficient is assumed to be unity.

CHAPTER - 4

RESULTS AND DISCUSSION

4.1 Description of the Structures

4.1.1 The Building A is three storey symmetric in plan having 17.5 m is length and 13.0 m. in width. The building is having five bay in X direction and 3 bay in Z direction. The building has been discretized into 72 nodes, in which base nodes are fixed. The ground floor columns are 35m high and on other floors 3.0 m high. The plan and elevation of the building are shown is figure (2).

4.1.2. The building B is 4 storeyed building L shaped in plan having 4 bay in X direction and 3 bay in z direction. This building has been discretized into 72 nodes in which the base nodes are fixed. All beams are of 7.5 m in length. The plan and elevation of the building in shown in figure (3).

4.1.3 The building C is 7 storeyed building stepped in elevation having 5 bay in X direction and 4 in Z direction. The gound floor columns are 3.75 m high and on other floors 3.6 m high. The building has been discretized into 190 nodes considering base nodes as fixed. This building is the half portion of the Allahabad court Building left Block. The plan and elevation of this building are shown in figures (4-7).

4.1.4 For concrete mixes the modulus of ealsticity is calculated as per clause 5.2.3.1. of IS: 456-1978, which says that in the absence of test data, the modulus of elasticity of structural concrete may be asumed as follows:

$$E_c = 5700 \sqrt{f_{ck}}$$

Where E_c is the short term static modulus of elasticity in N/mm^2, and f_{ck} is the characteristic cube strength of concrete in N/mm^2. Here the M15 concrete is used for the beams and columns in all the building considered for the analysis purpose. f_{ck} for M15 concrete is 15 N/mm^2.

The modal damping is 5% and the input spectra has been taken from the IS code (IS- 1893-1984).

4.2 RESULTS

4.2.1 Free vibration characteristics

In the dynamic analysis of these buildings 6 mode of vibration are considered. The time period, frequencies and mode participation factors are given in table no. (1)for all the three buildings.

4.2.2 Response (Forces and Displacements)

For building A, the shear forces, and bending movements in some typical members, are given for different directions of earthquake input spectra in table no. (2). Displacements at some typical nodes are given in table no. (4). Axial forces in sometypical columns are given in table no. (3).

For building B the shear forces and bending moments in some typical members are given in table no. (5) for different directions of earthquake input spectra. Displacements at some typical nodes are given in table no. (7). Axial forces in some typical columns are given in table no. (6).

For building C the shear forces and bending moments in some typical members are given in table no. (8) for different directions of earthquake input spectra. Displacements at some typical nodes are given in table no. (10). Axial forces in some typical columns are given in table no. (9).

4.3 DISCUSSION OF RESULTS

4.3.1 Three storeyed symmetricl building

On studying the table no. (2).we observe that shear forces and bending moments given by different modal combinations are comparable. This is due to the fact that frequencies are well separated . In such cases the SRSS method gives higher values as compared to CQC, AKG, DALS on an average of 1.2%. DABS always gives higher values than the other methods. The forces are almost same if the inputs are given in (a) horizontal and (b) horizontal and vertical directions. This means that the vertical mode of vibration does not contribute to the response. In other words the participation factor in vertical mode of vibration is of negligible magnitude.On giving 3 component input the magnitude of the forces in different members which were not so significant show appreciable change both in the case of beams and columns. For beams , transvese to the input direction (one component)have forces of negligible magnitude. For three components input the displacement in transverse direction also comes into picture, where as in the longitudinal direction it remains almost same as in earlier two cases. Axial forces in different column members does not show any appreciable change for different cases of input motion . All modal combinations are giving comparable results for axial forces. In case of 3 component input the corner columns show that AKG, CQC, DALS methods are giving lower than SRSS on an average of 5.7%.

4.3.2 Four Storeyed Unsymmetrical L Shaped Building

On studying the table no. 5 we observe that in columns and longitudinal beams shear forces given by CQC, AKG, DALS are comparable and these are higher than SRSS values on an average of 3.9%. This is not true for transverse beams and corner columns in which the values are lower on an average of 3.5% and 67.5% respectively. In case of corner columns bending moments are less as compared to SRSS values as depicted in table 5. This can be altributed to the fact that in this unsymmetrical building the frequencies are close. The shear forces and bending moments are same if the inputs are given in (a) horizontal and (b) horizontal and vertical directions. This means that the contrinution of the vertical mode of vibration is of negligible magnitude. In other words the participation factors in vertical mode of vibration is negligible. On giving 3 component input the magnitude of forces in different members which were earlier not so significant show appreciable change both in the case of beams and columns. We also observe that the shear forces and bending moments in same particular members are significantly apart in vertical and transverse directions . The axial forces in different column members given by CQC, AKG, DALS methods are higher than SRSS values on an average of 6%. The corner column shows that the SRSS value is higher than CQC, AKG, DALS methods by 12% when three component input is given while in one component input it shows that SRSS is lower than CQC, AKG, DALS by 0.9% DABS always gives higher value than given by any other method. In case of three component input the displacement in the two horizontal directions also comes into picture, while the displacement in X direction remains more or less same.

4.3.3 Seven Storeyed Unsymmetric (Stepped) Building

On studying the table no.(8) we observe that in corner columns and transverse beams shear forces and bending moments given by CQC, AKG, DALS are less as compared to SRSS values remarkably for one component of input and for longitudinal beams the shear forces and bending moments are 1.2% and 10% respectively.For one component of input, the variation in shear forces and bending moments in two different directions are more pronounced. The shear forces and bending moments are almost of same magnitude, if the inputs given are in (a) horizontal and (b) horizontal and vertical direction. This because of the fact that vertical mode of vibrations do not constribute to response significantly. In other words the mode participation factors in vertical mode of vibration are of negligible magnitude.On giving three component input the shear forces and bending moments in longitudinal beams given by CQC, AKG DALS are higher than SRSS values on an average of 4% and 7% respectively. In the case of transverse beam this variation is 6% for both shear forces and bending moments. while the CQC, AKG, DALS values are lower than SRSS values. In the case of corner columns the CQC, AKG, DALS values are lower than SRSS values on an average of 18.2%. This can be attributed to the fact that the frequencies of the structure are close. Hence mutual cancellation effect occurs in corner columns and transverse beams. In the case of longitudinal beams the mutual reinforcement of values occur. In case of three component the values are increased significantly. In all cases of earthquake inputs the DABS envelopes the values given by different combinations. In case of axial forces the values given by CQC, AKG, DALS are higher than SRSS on an average of 9.83% in column members. In case of three component input the translation in z direction also comes into picture, which does not have any significant part in the earlier two cases.

CHAPTER 5

SUMMARY, CONCLUSIONS AND SCOPE FOR FURTHER STUDY

5.1 SUMMARY

To study different modal combination techniques for different direction of earthquake input spectra dynamic analysis of three multisotrey building (3, 4 and 7 storeyed building) using 3-D Beam Element has been carried out . The modal valuesare combined by different modal combination techniques viz Square Root of Sum of Squares (SRSS), Complete Quadratic Combination Method (CQC), Prof. A.K. Gupta's Combination Method (AKG), Double Algebraic Sum Method (DALS) and Double Absolute Sum Method (DABS). The different modal combination techniques are compared with each other for different input motions.

5.2 CONCLUSIONS

From the results reported herein the following conclusions can be drawn.

1. For regular symmetrical building the frequencies are well separated , hence the forces given by different modal combination techniques such as CQC, AKG, DALS are comparable although SRSS gives higher values . DABS envelopes the responses given by any other combination technique .

2. For unsymmetrical building frequencies are not so well spaced

so the results given by CQC, AKG and DALS methods are almost same but they differ from the SRSS and DABS significantly in the case of corner columns.

3. Conventional analysis consisting of single translation component input would give design forces on nonconserrvative side, the magnitude of error , depending upon number of components of ground motion neglected which is clearly observed from the different tables given in chapter 4 .

5.3 SCOPE FOR FURTHER STUDY

1. In present study, only framed building without shear walls were analysed, study should be extended for building having shear wall so that shear wall-structure interaction can be simulated.

2. In present study soil-structure interaction is not taken into account, study should be extended by considering soil structure interaction.

3. This study should also be extended for buildings of different heights , varying in plan and elevation .

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$$
\begin{bmatrix}
\frac{EA}{l} & & & & & & & & & & & \\
0 & \frac{12EI_z}{l^3(1+\Phi_y)} & & & & & & & & & & \\
0 & 0 & \frac{12EI_y}{l^3(1+\Phi_z)} & & & & & & & & & \\
0 & 0 & 0 & \frac{GJ}{l} & & & & & & & & \\
0 & 0 & \frac{-6EI_y}{l^2(1+\Phi_z)} & 0 & \frac{(4+\Phi_z)EI_y}{l(1+\Phi_z)} & & & & & & & \\
0 & \frac{6EI_z}{l^2(1+\Phi_y)} & 0 & 0 & 0 & \frac{(4+\Phi_y)EI_z}{l(1+\Phi_y)} & & & & & & \\
\frac{-EA}{l} & 0 & 0 & 0 & 0 & 0 & \frac{AE}{l} & & & & & \\
0 & \frac{-12EI_z}{l^3(1+\Phi_y)} & 0 & 0 & 0 & \frac{-6EI_z}{l^2(1+\Phi_y)} & 0 & \frac{12EI_z}{l^3(1+\Phi_y)} & & & & \\
0 & 0 & \frac{-12EI_y}{l^3(1+\Phi_z)} & 0 & \frac{6EI_y}{l^2(1+\Phi_z)} & 0 & 0 & 0 & \frac{12EI_y}{l^3(1+\Phi_z)} & & & \\
0 & 0 & 0 & \frac{-GJ}{l} & 0 & 0 & 0 & 0 & 0 & \frac{GJ}{l} & & \\
0 & 0 & \frac{-6EI_y}{l^2(1+\Phi_z)} & 0 & \frac{(2-\Phi_z)EI_y}{l(1+\Phi_z)} & 0 & 0 & 0 & \frac{6EI_y}{l^2(1+\Phi_z)} & 0 & \frac{(4+\Phi_z)EI_y}{l(1+\Phi_z)} & \\
0 & \frac{6EI_z}{l^2(1+\Phi_y)} & 0 & 0 & 0 & \frac{(2-\Phi_y)EI_z}{l(1+\Phi_y)} & 0 & \frac{-6EI_z}{l^2(1+\Phi_y)} & 0 & 0 & 0 & \frac{(4+\Phi_y)EI_z}{l(1+\Phi_y)}
\end{bmatrix}
$$

where, $\phi_Y = \frac{12EI_Z}{GA_Y\ell^2}$

$\phi_Z = \frac{12EI_Y}{GA_Z\ell^2}$

Symmetric

APPENDIX A–SPACE FRAME MEMBER STIFFNESS MATRIX

APPENDIX - B

The Relation transformation matrix R_T for a space frame member can be shown to take the following form

$$R_T = \begin{bmatrix} R & 0 & 0 & 0 \\ 0 & R & 0 & 0 \\ 0 & 0 & R & 0 \\ 0 & 0 & 0 & R \end{bmatrix}$$

where,

$$R = \begin{bmatrix} C_x & C_y & C_z \\ \dfrac{-C_x C_y \cos\alpha - C_z \sin\alpha}{\sqrt{C_x^2 + C_z^2}} & \sqrt{C_x^2 + C_z^2}\,\cos\alpha & \dfrac{-C_x C_z \cos\alpha + C_x \sin\alpha}{\sqrt{C_x^2 + C_z^2}} \\ \dfrac{C_x C_y \sin\alpha - C_z \cos\alpha}{\sqrt{C_x^2 + C_z^2}} & \sqrt{C_x^2 + C_z^2}\,\sin\alpha & \dfrac{C_y C_z \cos\alpha + C_x \cos\alpha}{\sqrt{C_x^2 + C_z^2}} \end{bmatrix}$$

This rotation matrix is expressed in terms of the direction cosines of the member (which are readily computed from the coordiates of the joints) and the angle α, which must be given as part of the description of the structure itself.

Symmetric

1	2	3	4	5	6	7	8	9	10	11	12
$\frac{1}{3}$											
0	$\frac{13}{35}+\frac{6I_z}{5Al^2}$										
0	0	$\frac{13}{35}+\frac{6I_y}{5Al^2}$									
0	0	0	$\frac{J_x}{3A}$								
0	0	$-\frac{11l}{210}-\frac{I_y}{10Al}$	0	$\frac{l^2}{105}+\frac{2I_y}{15A}$							
0	$\frac{11l}{210}+\frac{I_z}{10Al}$	0	0	0	$\frac{l^2}{105}+\frac{2I_z}{15A}$						
$\frac{1}{6}$	0	0	0	0	0	$\frac{1}{3}$					
0	$\frac{9}{70}-\frac{6I_z}{5Al^2}$	0	0	0	$\frac{13l}{420}-\frac{I_z}{10Al}$	0	$\frac{13}{35}+\frac{6I_z}{5Al^2}$				
0	0	$\frac{9}{70}-\frac{6I_y}{5Al^2}$	0	$-\frac{13l}{420}+\frac{I_y}{10Al}$	0	0	0	$\frac{13}{35}+\frac{6I_y}{5Al^2}$			
0	0	0	$\frac{J_x}{6A}$	0	0	0	0	0	$\frac{J_x}{3A}$		
0	0	$\frac{13l}{420}-\frac{I_y}{10Al}$	0	$-\frac{l^2}{140}-\frac{I_y}{30A}$	0	0	0	$\frac{11l}{210}+\frac{I_y}{10Al}$	0	$\frac{l^2}{105}+\frac{2I_y}{15A}$	
0	$-\frac{13l}{420}+\frac{I_z}{10Al}$	0	0	0	$-\frac{l^2}{140}-\frac{I_z}{30A}$	0	$-\frac{11l}{210}-\frac{I_z}{10Al}$	0	0	0	$\frac{l^2}{105}+\frac{2I_z}{15A}$

APPENDIX C GENERALIZED MASS MATRIX

APPENDIX - D

Proof of Gram - Schmidt orthogonalization

$$\bar{X} = X - \sum_{i=1}^{n} X_i X_i^T X$$

Considering convergence of vector X and root λ

$$A \bar{x} = \lambda X$$

or, $$A (X - \sum_{i=1}^{n} X_i X_i^T X) = \lambda X$$

or, $$A X - A x_1 X_1^T X - A X_2 X_2^T X - A X_3 X_3^T X ---- = \lambda X$$

If n eigen values are established and

Let $X = X_j$ where $j < n$

$$Ax_j - Ax_j X_j^T X_j = \lambda X_j$$

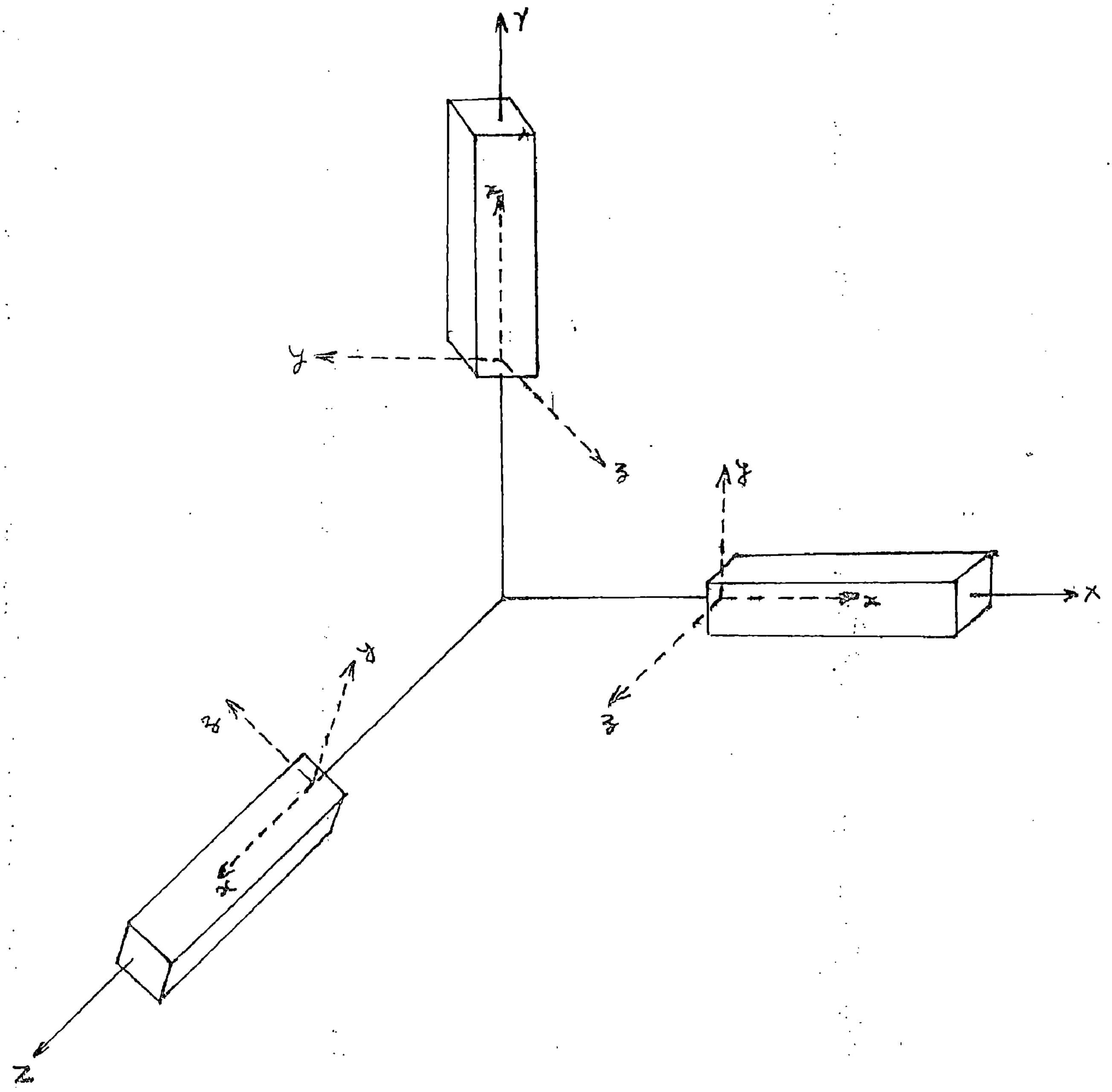
all other reducing to zero, i.e.

$$X_1^T X_j = X_2^T X_j = X_3^T X_j = ---- = X_n^T X_j = 0$$

due to orthogonality of modes

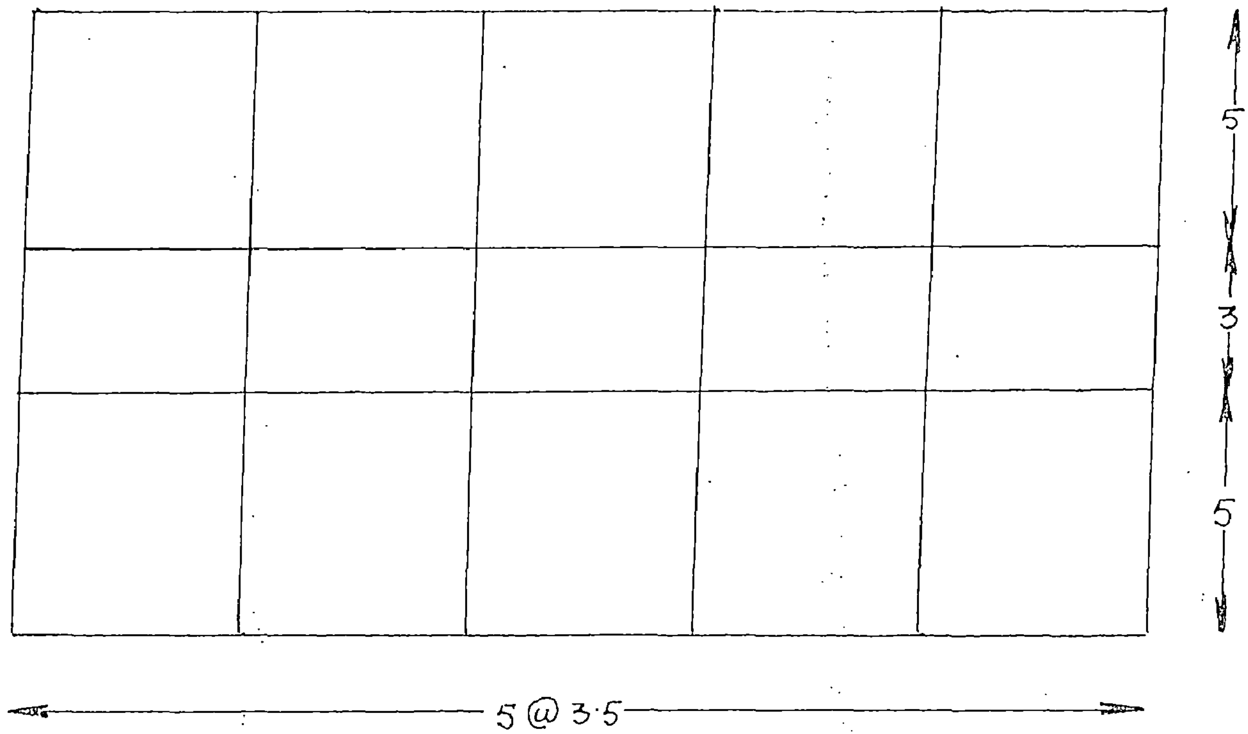
or $$AX_j - AX_j = \lambda X_j$$

Which implies $\lambda = 0$ and convergence to a trivial root is not possible hence $j>n$, in which case $AX_j = \lambda X_j$ which implies $\lambda = \lambda j$

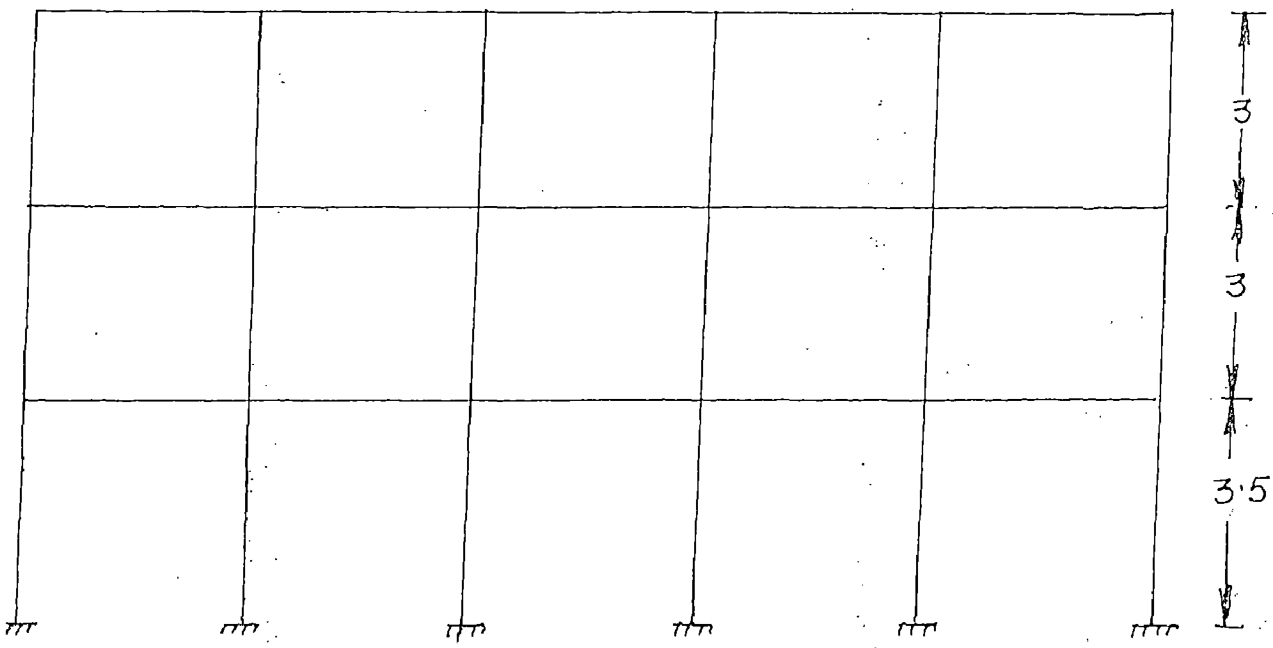


COORDINATE SYSTEM — LOCAL & GLOBAL

FIG NO 1



PLAN



ELEVATION

SYMMETRICAL BUILDING A

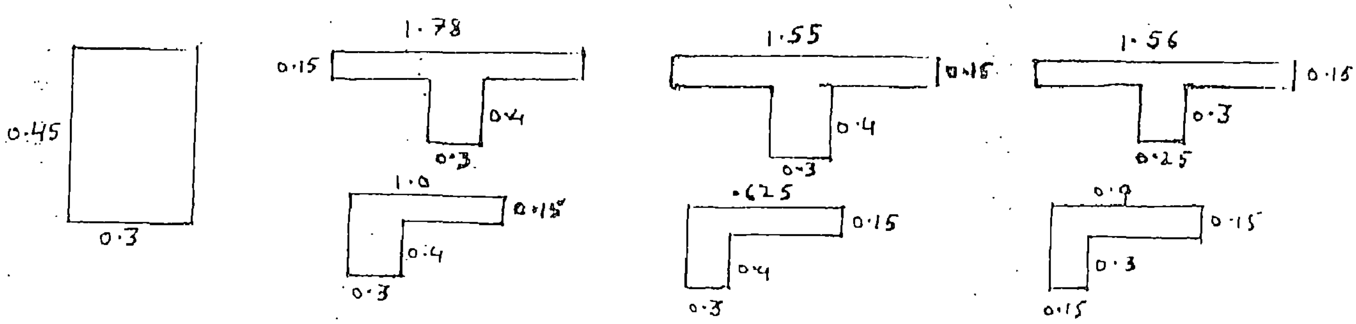
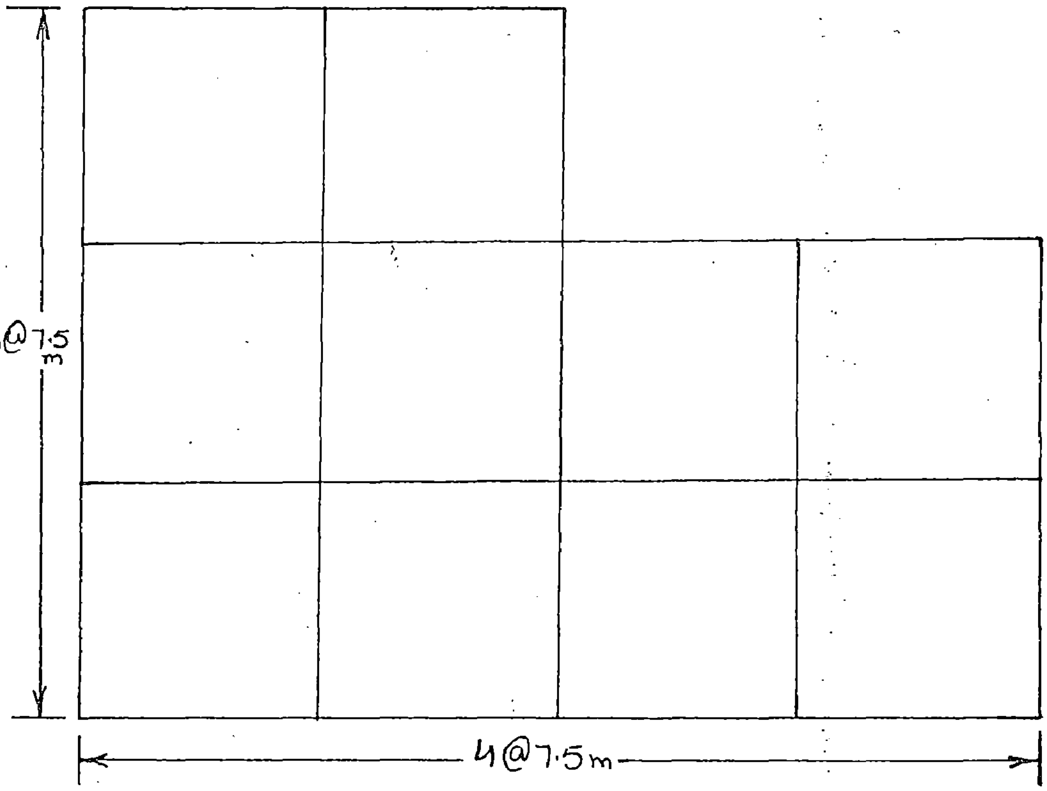


FIG NO 2



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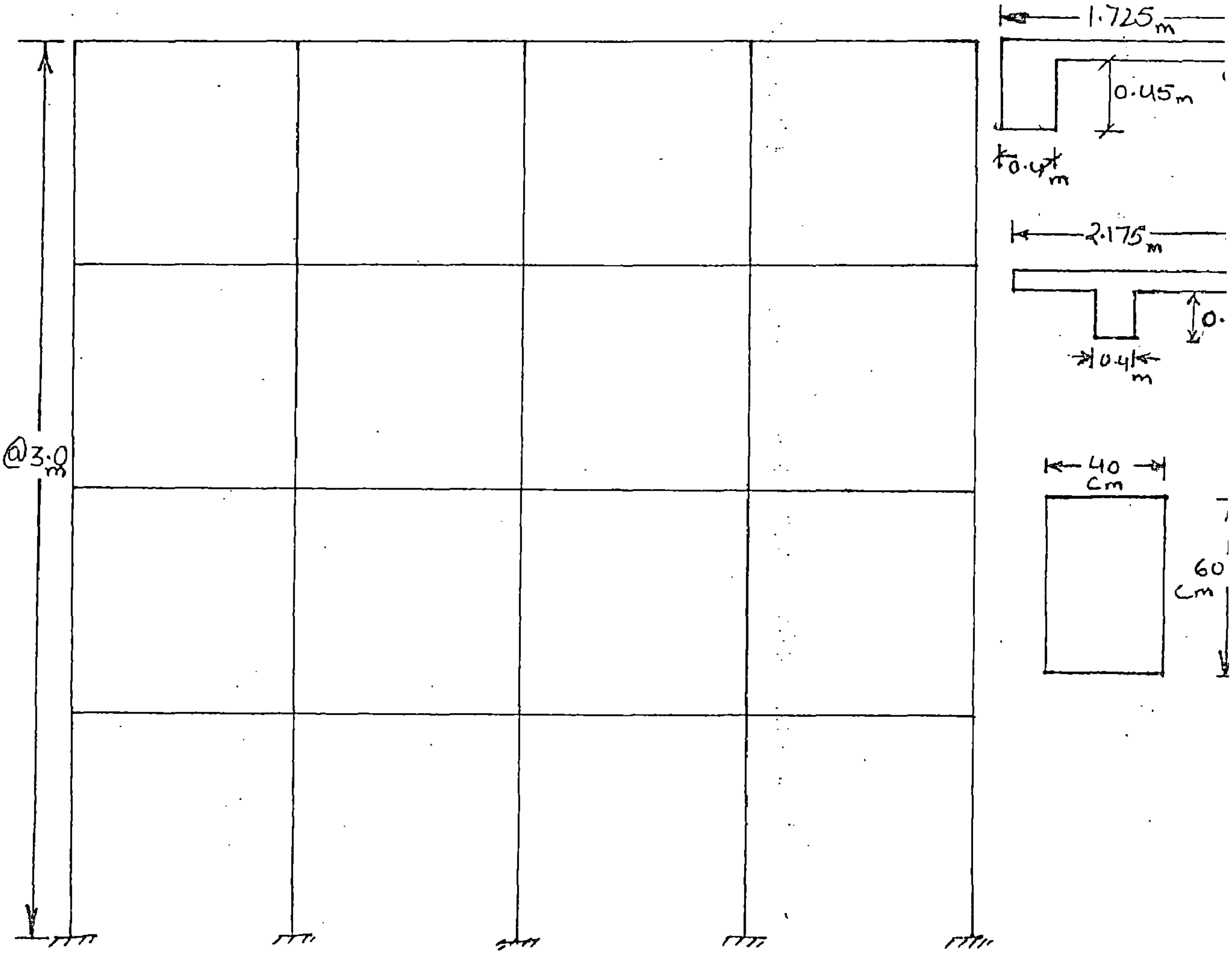


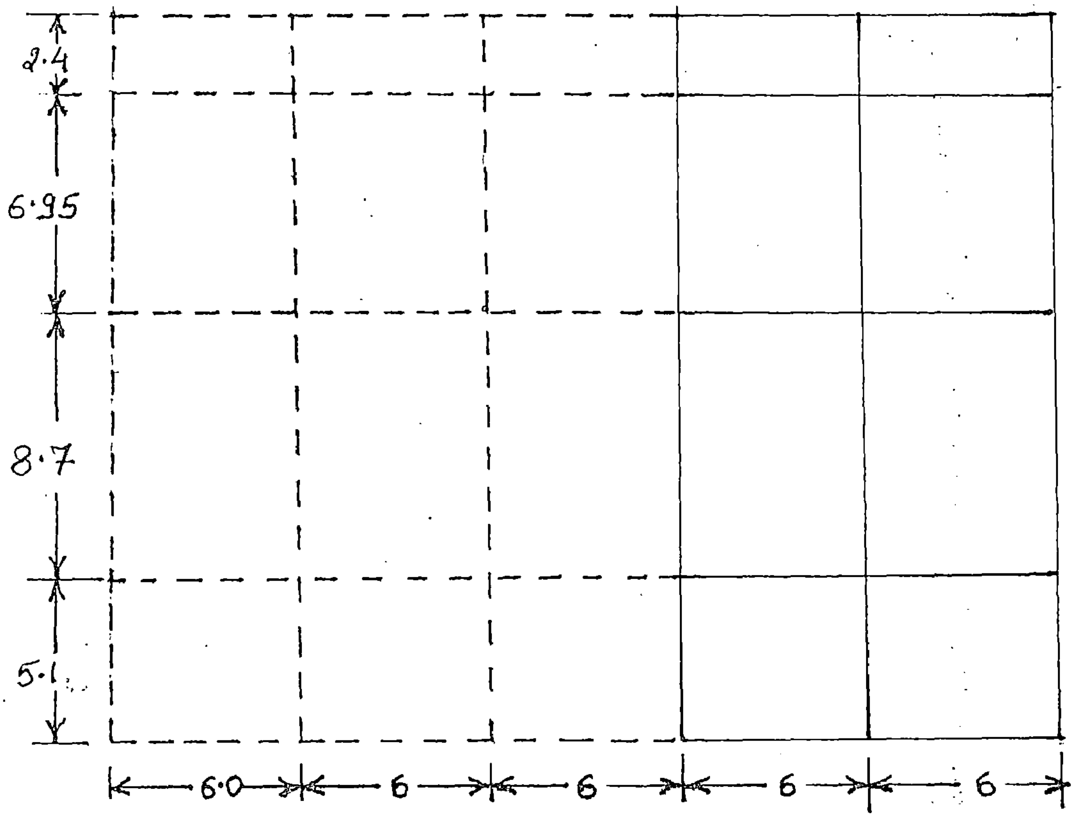
FIG NO 3

ELEVATION

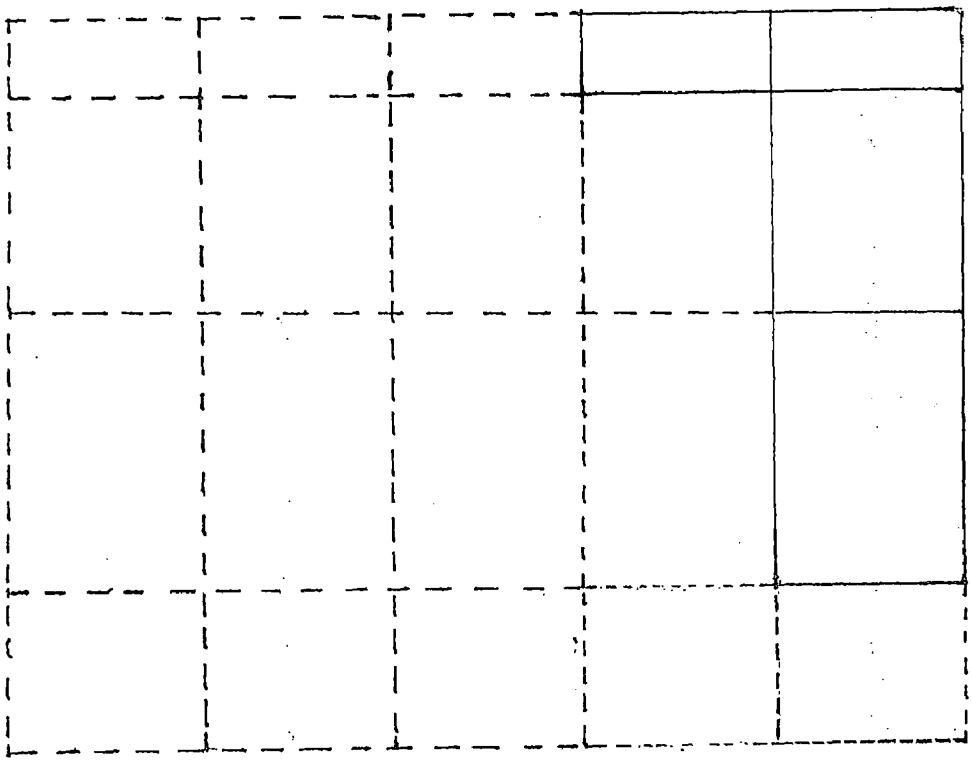
BUILDING B

UNSYMMETRICAL L-SHAPED

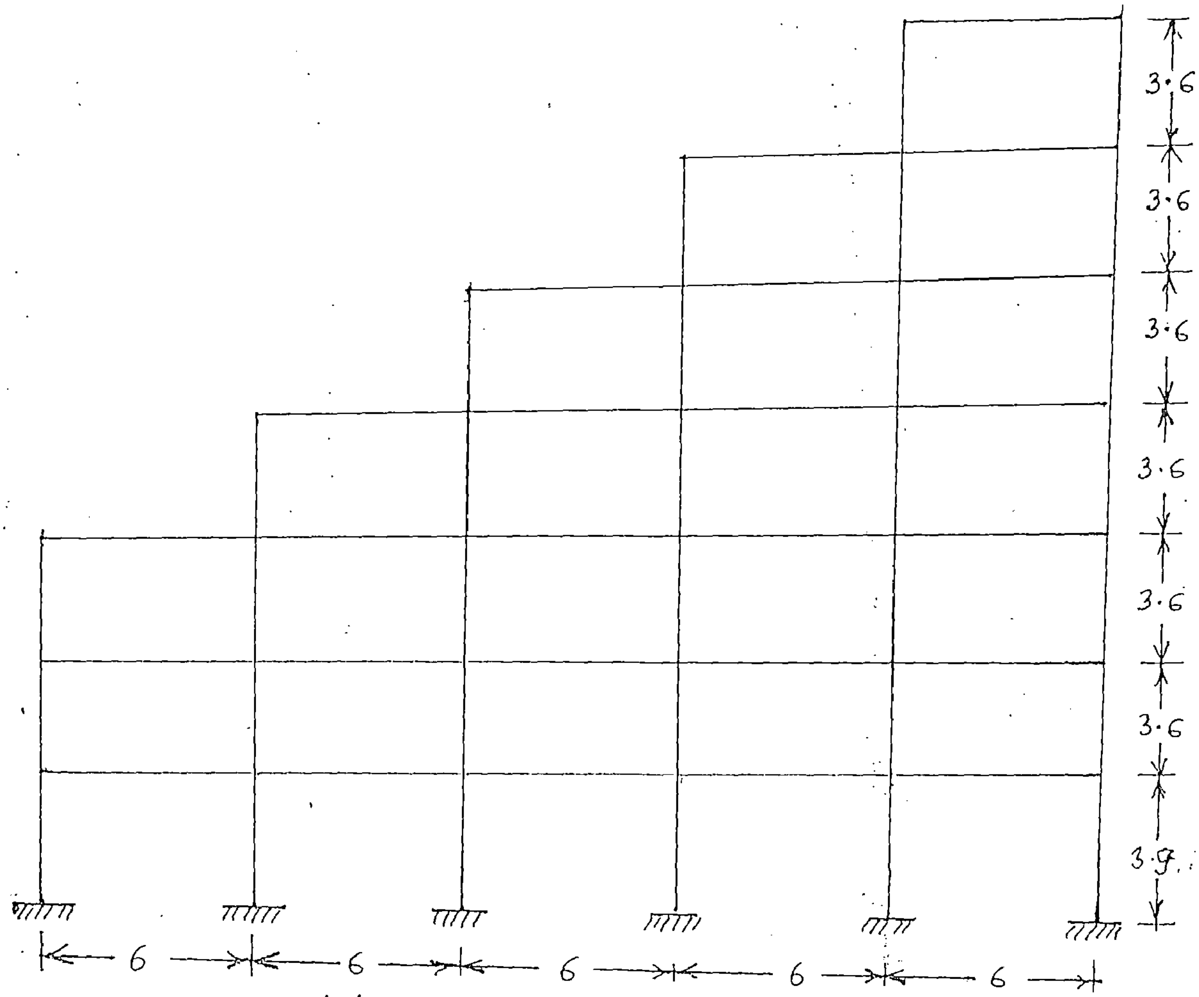
UNSYMMETRIC (STEPPED) BUILDING-C
PLAN

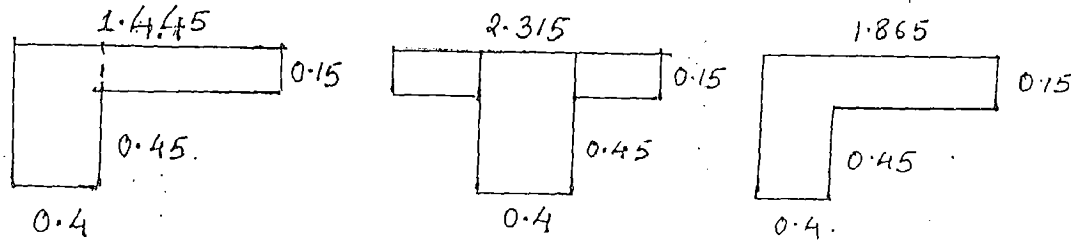


SIX FLOOR



SEVEN FLOOR

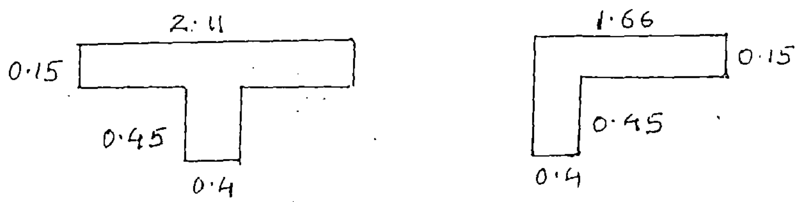




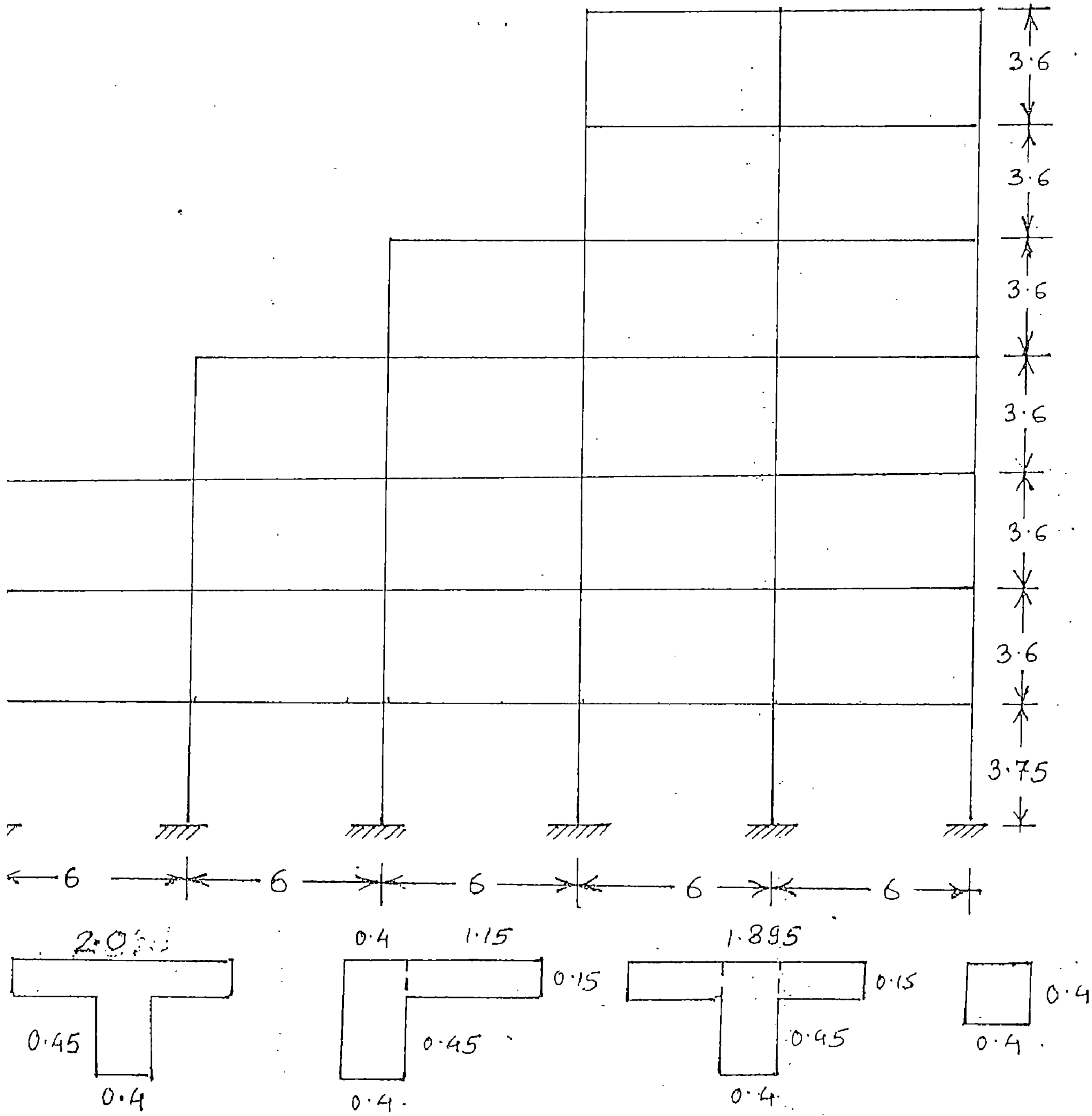
5·1 m E. BEAM · 8·7 m C. BEAM · 8·7 m E. BEAM

FRAME NO 3 IN LONGITUDINAL DIRECTION

FIG NO 5



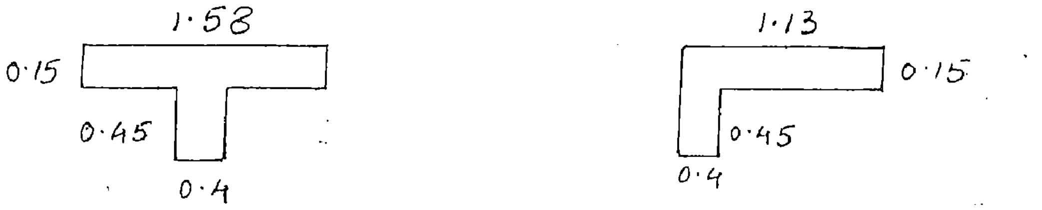
6·95 m C. BEAM · 6·95 m E. BEAM



C. BEAM. 6m E. BEAM 5.1m C. BEAM

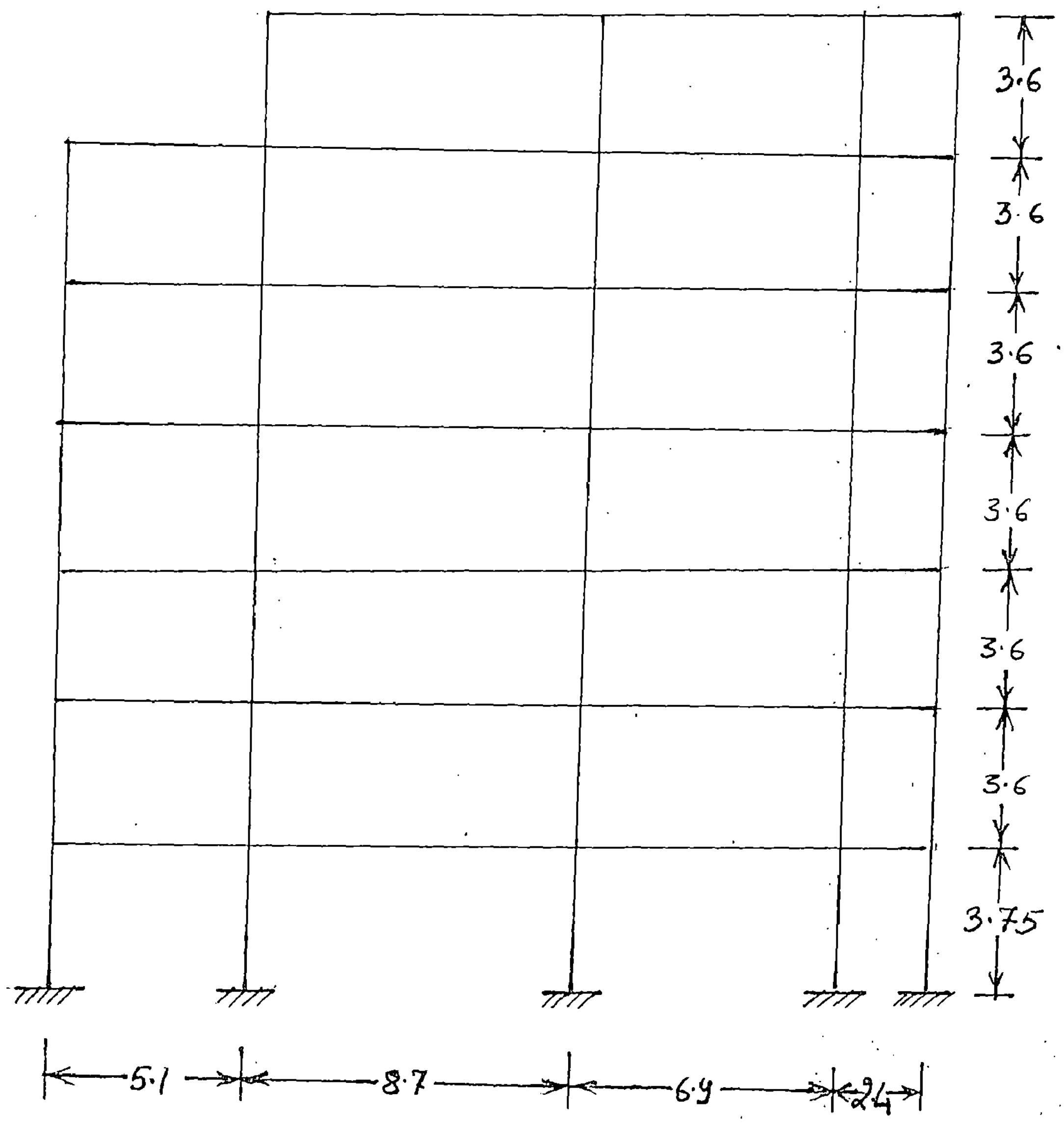
FRAME NO 4 IN LONGITUDINAL DIRECTION

FIG NO 6



2.4 m C. BEAM 2.4 m E. BEAM

FRAME NO 6 IN TRANSVERSE DIRECTION



ELEVATION

FIG NO 7

TABLE - 1

FREE VIBRATION CHARACTERISTICS OF 3 STOREYED SYMMETRIC BUILDING A

Mode	Time period (Seconds)	Frequency (rad/secs)	Mode participation factor C_x	C_y	C_z	$C_{\theta x}$	$C_{\theta y}$	$C_{\theta z}$
1	Ø.32451	19.3621	Ø.ØØ41527	-Ø.498ØE-Ø8	1.2Ø513ØØ	Ø.Ø6Ø679Ø	Ø.ØØØØ142	-Ø.ØØ1281Ø
2	Ø.25587	24.5564	1.2119Ø6Ø	-Ø.2913E-Ø8	-Ø.ØØ68855	-Ø.ØØØ3467	Ø.ØØØ7275	-Ø.373823Ø
3	Ø.13281	26.9881	-Ø.Ø16463Ø	Ø.29Ø7E-1Ø	-Ø.ØØØ17Ø3	-Ø.ØØØØ526	Ø.Ø93483Ø	Ø.ØØ52993
4	Ø.1Ø839	57.9663	Ø.ØØ1783Ø	Ø.1279E-Ø7	Ø.29817ØØ	-Ø.158353Ø	-Ø.ØØØØ761	Ø.ØØ5Ø998
5	Ø.Ø8898	7Ø.612Ø	Ø.291225Ø	Ø.968ØE-Ø6	-Ø.ØØ1945Ø	Ø.ØØ1Ø531	-Ø.ØØØØ753	Ø.835512Ø
6	Ø.Ø8182	76.7919	-Ø.ØØØ1Ø13	Ø.ØØ18Ø61Ø	-Ø.277E-Ø6	-Ø.615E-Ø7	Ø.ØØØ4573	-Ø.ØØØ29ØØ

FREE VIBRATION CHARACTERISTICS OF 4 STOREYED UNSYMMETRIC L SHAPED BUILDING B

Mode	Time period (Seconds)	Frequency (rad/secs)	Mode participation factor C_x	C_y	C_z	$C_{\theta x}$	$C_{\theta y}$	$C_{\theta z}$
1	Ø.441513	14.231Ø	Ø.ØØ83762	Ø.ØØØ1945	1.271889Ø	Ø.Ø37316Ø	-Ø.ØØ123Ø17	-Ø.ØØØ3921
2	Ø.383532	16.38Ø3	Ø.Ø897675	Ø.ØØØ1Ø63	Ø.Ø11Ø862	Ø.ØØØ2515	Ø.2Ø176375	-Ø.ØØ36899
3	Ø.365452	17.193Ø	1.3Ø27854	Ø.ØØØ2194	-Ø.Ø121Ø48	-Ø.ØØØ31Ø2	-Ø.ØØ85Ø864	-Ø.Ø538787
4	Ø.14796Ø	42.4653	Ø.ØØ28Ø73	-Ø.ØØØ58Ø1	Ø.4Ø89493	-Ø.Ø55Ø875	-Ø.ØØØ18Ø2Ø	Ø.ØØØ865Ø
5	Ø.131248	47.8726	-Ø.ØØØ7121	-Ø.ØØØØ851	Ø.Ø121777	Ø.ØØ3731Ø	Ø.Ø592318Ø	-Ø.ØØ37845
6	Ø.129557	48.4976	Ø.Ø196Ø35	-Ø.ØØØ3Ø75	-Ø.ØØ44968	Ø.ØØ23145	Ø.Ø84Ø92ØØ	Ø.ØØ19585

FREE VIBRATION CHARACTERISTICS OF 7 STOREYED UNSYMMETRIC (STEPPED) BUILDING C

Mode	Time period (Seconds)	Frequency (rad/secs)	Mode participation factor C_x	C_y	C_z	$C_{\theta x}$	$C_{\theta y}$	$C_{\theta z}$
1	Ø.85363	7.36Ø6	Ø.Ø5576Ø4	Ø.ØØ145Ø8	1.588889Ø	Ø.Ø163Ø84	-Ø.0768498	-Ø.ØØØ29385
2	Ø.721Ø3	8.7142	1.4764684	-Ø.ØØ2Ø123	-Ø.Ø437137	-Ø.ØØØ4843	Ø.ØØØ6925	-Ø.ØØ79Ø912
3	Ø.59459	1Ø.5672	-Ø.Ø146621	Ø.ØØØ8964	Ø.8ØØ189Ø	Ø.Ø1Ø8966	Ø.1377354	-Ø.ØØØØ8486
4	Ø.31Ø99	2Ø.2Ø33	Ø.2165171	-Ø.ØØ57415	Ø.7677465	-Ø.Ø275492	-Ø.Ø72Ø735	Ø.ØØ389427
5	Ø.29756	21.1157	Ø.6998749	Ø.ØØ32ØØ6	-Ø.1511648	Ø.Ø1Ø1249	Ø.Ø442145	Ø.Ø13Ø5693
6	Ø.28822	21.8ØØØ	Ø.336Ø324	Ø.ØØØ7732	-Ø.1589536	-Ø.ØØ35456	-Ø.Ø141388	Ø.ØØ646923

TABLE - 2

FORCES IN TYPICAL MEMBERS OF 3 STOREY SYMMETRIC BUILDING A

Member		SHEAR FORCES						BENDING MOMENTS					
		H Component		H+V Component		H+T+V Componemt		H Component		H+V Component		H+T+V Component	
		fyy	fzz	fyy	fzz	fyy	fzz	Myy	Mzz	Myy	Mzz	Myy	Mzz
5.	SRSS	0.5979	0.1052	0.5979	0.1052	0.5980	0.1268	0.1922	1.1719	0.1922	1.1719	0.2337	1.1720
	CQC	0.5965	0.1042	0.5965	0.1042	0.5955	0.1170	0.1904	1.1692	0.1905	1.1692	0.2155	1.1678
	AKG	0.5959	0.1042	0.5958	0.1042	0.5950	0.1168	0.1904	1.1681	0.1903	1.1681	0.2152	1.1667
	DALS	0.5957	0.1041	0.5960	0.1041	0.5946	0.1153	0.1902	1.1678	0.1902	1.1680	0.2123	1.1661
	DABS	0.6001	0.1064	0.6000	0.1064	0.6013	0.1380	0.1941	1.1760	0.1942	1.1760	0.2544	1.1778
62	SRSS	0.5756	0.0112	0.5756	0.0112	0.5756	0.3617	0.0173	1.0852	0.0174	1.0852	0.5882	1.0852
	CQC	0.5801	0.0133	0.5800	0.0133	0.5805	0.3596	0.0205	1.0921	0.0205	1.0921	0.5849	1.0928
	AKG	0.5796	0.0133	0.5796	0.0133	0.5801	0.3593	0.0205	1.0915	0.0205	1.0915	0.5845	1.0922
	DALS	0.5799	0.0134	0.5800	0.0134	0.5804	0.3589	0.0207	1.0919	0.0207	1.0919	0.5839	1.0927
	DABS	0.5828	0.0142	0.5827	0.0142	0.5833	0.3648	0.0220	1.0962	0.0220	1.0960	0.5930	1.0972
90	SRSS	0.0049	0.0662	0.0049	0.0663	0.4847	0.0771	0.1841	0.0142	0.1841	0.0142	0.2108	1.4160
	CQC	0.0041	0.0668	0.0041	0.0669	0.4839	0.0724	0.1856	0.0119	0.1856	0.0119	0.1983	1.4138
	AKG	0.0041	0.0669	0.0041	0.0667	0.4837	0.0722	0.1857	0.0118	0.1857	0.0118	0.1980	1.4130
	DALS	0.0040	0.0669	0.0040	0.0669	0.4836	0.0714	0.1858	0.0116	0.1858	0.0116	0.1958	1.4129
	DABS	0.0059	0.0670	0.0059	0.0670	0.4860	0.0835	0.1861	0.0169	0.1861	0.0170	0.2280	1.4200
103	SRSS	1.8385	0.0058	1.8385	0.0058	1.8385	0.6875	0.0092	2.8961	0.0092	2.8961	1.0816	2.8961
	CQC	1.8264	0.0052	1.8265	0.0052	1.8272	0.6870	0.0081	2.8783	0.0081	2.8783	1.0810	2.8794
	AKG	1.8261	0.0052	1.8262	0.0052	1.8269	0.6868	0.0081	2.8781	0.0881	2.8781	1.0807	2.8794
	DALS	1.8252	0.0051	1.8253	0.0051	1.8262	0.6867	0.0080	2.8768	0.0080	2.8768	1.0806	2.8783
	DABS	1.8516	0.0068	1.8517	0.0068	1.8526	0.6886	0.0107	2.9152	0.0107	2.9150	1.0833	2.9167
125	SRSS	2.2044	0.0554	2.2044	0.0555	2.2045	0.0697	0.0999	4.2040	0.1000	4.2040	0.1276	4.2041
	CQC	2.2040	0.0551	2.2037	0.0551	2.2055	0.0746	0.0994	4.2026	0.1000	4.2026	0.1369	4.2058
	AKG	2.2039	0.0552	2.2039	0.0552	2.2057	0.0749	0.0995	4.2030	0.0996	4.2030	0.1374	4.2063
	DALS	2.2039	0.0552	2.2039	0.0552	2.2060	0.0756	0.0995	4.2029	0.0996	4.2029	0.1386	4.2067
	DABS	2.2063	0.0563	2.2060	0.0563	2.2084	0.0766	0.1010	4.2076	0.1014	4.2076	0.1402	4.2115
163	SRSS	1.7982	0.0218	1.7982	0.0218	1.7982	1.0017	0.0384	3.7536	0.0384	3.7536	1.8634	3.7536
	CQC	1.7844	0.0270	1.7845	0.0268	1.7854	1.0051	0.0472	3.7292	0.0472	3.7292	1.8695	3.7311
	AKG	1.7848	0.0268	1.7840	0.0273	1.7858	1.0054	0.0468	3.7300	0.0473	3.7300	1.8700	3.7320
	DALS	1.7839	0.0272	1.7840	0.0273	1.7851	1.0060	0.0479	3.7284	0.0479	3.7284	1.8710	3.7307
	DABS	1.8155	0.0273	1.8156	0.0273	1.8167	1.0060	0.0480	3.7844	0.0480	3.7845	1.8710	3.7867

* Forces in tonne
** Moments in tonne-metre
*** Beam 5,90,125
**** Column 62,103,163

H EQ Input in X direction
T EQ Input in Z direction
V EQ Input in Y direction

TABLE - 3

THREE STOREYED SYMMETRIC BUILDING -A
AXIAL FORCES IN TYPICAL COLUMN MEMBERS

Member		H Component	H+V Component	H+T+V Component
56	SRSS	0.620	0.620	0.621
	CQC	0.618	0.619	0.625
	AKG	0.618	0.618	0.625
	DALS	0.618	0.618	0.626
	DABS	0.623	0.623	0.631
119	SRSS	2.055	2.055	2.165
	CQC	2.051	2.051	2.064
	AKG	2.050	2.050	2.061
	DALS	2.049	2.049	2.045
	DABS	2.061	2.061	2.283
163	SRSS	4.231	4.231	4.486
	CQC	4.231	4.231	4.668
	AKG	4.229	4.230	4.691
	DALS	4.229	4.229	4.722
	DABS	4.237	4.237	4.734
186	SRSS	4.228	4.228	4.484
	CQC	4.226	4.226	4.684
	AKG	4.225	4.225	4.686
	DALS	4.225	4.225	4.717
	DABS	4.235	4.235	4.732

* Forces in tonne

TABLE - 4

DISPLACEMENTS AT TYPICAL NODES IN 3 STOREYED SYMMETRIC BUILDING A

Node		H Component	H+V Component	H+V+T Component	
		uxx	uxx	uxx	wzz
	SRSS	Ø.3876	Ø.3876	Ø.3876	Ø.3135
	CQC	Ø.3858	Ø.3858	Ø.386Ø	Ø.3142
	AKG	Ø.3865	Ø.3865	Ø.3866	Ø.3138
	DALS	Ø.3866	Ø.3866	Ø.3867	Ø.3139
	DABS	Ø.3886	Ø.3886	Ø.3887	Ø.314Ø
	SRSS	Ø.3927	Ø.3927	Ø.3927	Ø.3153
	CQC	Ø.3922	Ø.3922	Ø.3924	Ø.3152
	AKG	Ø.3922	Ø.3922	Ø.3924	Ø.3152
	DALS	Ø.3922	Ø.3922	Ø.3923	Ø.3151
	DABS	Ø.3933	Ø.3933	Ø.3935	Ø.3157
2	SRSS	Ø.318Ø	Ø.318Ø	Ø.318Ø	Ø.2629
	CQC	Ø.3183	Ø.3184	Ø.3185	Ø.2618
	AKG	Ø.3184	Ø.3184	Ø.3185	Ø.2618
	DALS	Ø.3185	Ø.3184	Ø.3186	Ø.2616
	DABS	Ø.3184	Ø.3185	Ø.3186	Ø.2643
3	SRSS	Ø.3179	Ø.3179	Ø.3179	Ø.2629
	CQC	Ø.3193	Ø.3193	Ø.3195	Ø.2618
	AKG	Ø.3194	Ø.3194	Ø.3195	Ø.2618
	DALS	Ø.3195	Ø.3195	Ø.3196	Ø.2616
	DABS	Ø.3195	Ø.3195	Ø.3197	Ø.2642
4	SRSS	Ø.1782	Ø.1782	Ø.1782	Ø.1669
	CQC	Ø.1774	Ø.1774	Ø.1774	Ø.1662
	AKG	Ø.1774	Ø.1774	Ø.1774	Ø.1661
	DALS	Ø.1773	Ø.1773	Ø.1774	Ø.1661
	DABS	Ø.1794	Ø.1794	Ø.1795	Ø.1679
	SRSS	Ø.1819	Ø.1819	Ø.1819	Ø.1669
	CQC	Ø.1829	Ø.1829	Ø.183Ø	Ø.1674
	AKG	Ø.1829	Ø.1829	Ø.183Ø	Ø.1675
	DALS	Ø.183Ø	Ø.183Ø	Ø.1831	Ø.1675
	DABS	Ø.183Ø	Ø.183Ø	Ø.1831	Ø.1675

All displacements (in meter) are to be multiplied by $1Ø^{-2}$

TABLE - 5

FORCES IN TYPICAL MEMBERS OF UNSYMMETRICAL L SHAPED BUILDING B

Member		SHEAR FORCES						BENDING MOMENTS					
		H Component		H+V Component		H+T+V Component		H Component		H+V Component		H+T+V Component	
		fyy	fzz	fyy	fzz	fyy	fzz	Myy	Mzz	Myy	Mzz	Myy	Mzz
1	SRSS	Ø.8346	Ø.2551	Ø.8346	Ø.2551	Ø.8346	Ø.3224	1.112Ø	3.4974	1.112Ø	3.4974	1.3222	3.4976
	CQC	Ø.8Ø72	Ø.256Ø	Ø.8Ø72	Ø.2559	Ø.8Ø9Ø	Ø.3544	1.Ø988	3.3828	1.Ø988	3.3828	1.425Ø	3.39Ø1
	AKG	Ø.82Ø9	Ø.2555	Ø.82Ø9	Ø.2555	Ø.8218	Ø.3387	1.1Ø52	3.44Ø1	1.1Ø52	3.44Ø1	1.373Ø	3.4439
	DALS	Ø.82Ø4	Ø.2556	Ø.82Ø4	Ø.2556	Ø.8215	Ø.3425	1.1Ø53	3.4381	1.1Ø53	3.4381	1.3865	3.4426
	DABS	Ø.8486	Ø.2557	Ø.8485	Ø.2557	Ø.8498	Ø.3439	1.1211	3.556Ø	1.1211	3.5561	1.4Ø76	3.5614
4Ø	SRSS	2.778Ø	Ø.Ø38Ø	2.778Ø	Ø.Ø38Ø	2.7781	1.4Ø91	Ø.Ø6Ø9	5.ØØ13	Ø.Ø6Ø9	5.ØØ13	2.2928	5.ØØ15
	CQC	2.8379	Ø.Ø418	2.8379	Ø.Ø418	2.834Ø	1.3956	Ø.Ø668	5.1Ø94	Ø.Ø668	5.1Ø94	2.2714	5.1Ø3Ø
	AKG	2.8379	Ø.Ø417	2.838Ø	Ø.Ø417	2.8339	1.3941	Ø.Ø667	5.1Ø94	Ø.Ø668	5.1Ø94	2.2689	5.1Ø29
	DALS	2.84Ø3	Ø.Ø412	2.84Ø3	Ø.Ø412	2.8355	1.3914	Ø.Ø658	5.1136	Ø.Ø658	5.1136	2.2646	5.1Ø58
	DABS	2.84Ø6	Ø.Ø517	2.84Ø6	Ø.Ø517	2.8459	1.4276	Ø.Ø831	5.1142	Ø.Ø83Ø	5.1142	2.3224	5.1228
61	SRSS	Ø.Ø132	Ø.Ø467	Ø.Ø132	Ø.Ø467	Ø.744Ø	Ø.Ø531	Ø.533Ø	Ø.Ø594	Ø.533Ø	Ø.Ø594	Ø.5345	2.8Ø92
	CQC	Ø.ØØ97	Ø.Ø626	Ø.ØØ97	Ø.Ø626	Ø.7415	Ø.Ø614	Ø.6279	Ø.Ø456	Ø.6278	Ø.Ø45Ø	Ø.6252	2.8Ø58
	AKG	Ø.ØØ97	Ø.Ø626	Ø.ØØ97	Ø.Ø626	Ø.7415	Ø.Ø614	Ø.6279	Ø.Ø456	Ø.6279	Ø.Ø457	Ø.6249	2.8ØØØ
	DALS	Ø.ØØ95	Ø.Ø632	Ø.ØØ95	Ø.Ø632	Ø.7412	Ø.Ø6Ø7	Ø.6314	Ø.Ø453	Ø.6314	Ø.Ø453	Ø.6274	2.8Ø33
	DABS	Ø.Ø179	Ø.Ø635	Ø.Ø179	Ø.Ø635	Ø.7499	Ø.Ø761	Ø.6322	Ø.Ø81Ø	Ø.6322	Ø.Ø81Ø	Ø.6415	2.8359
73	SRSS	3.3844	Ø.2377	3.3844	Ø.2377	3.3846	1.6334	Ø.3949	6.1532	Ø.3949	6.1532	2.6939	6.1535
	CQC	3.2721	Ø.1Ø99	3.2721	Ø.1Ø99	3.2786	1.6351	Ø.1821	5.9479	Ø.1821	5.948Ø	2.696Ø	5.9594
	AKG	3.2721	Ø.1Ø98	3.2721	Ø.1Ø98	3.2786	1.6351	Ø.1818	5.9478	Ø.1817	5.9478	2.6946	5.9594
	DALS	3.2675	Ø.1Ø16	3.2676	Ø.1Ø16	3.2755	1.6348	Ø.1681	5.9396	Ø.1681	5.9396	2.6959	5.9534
	DABS	3.4978	Ø.3291	3.4978	Ø.3291	3.5Ø67	1.7549	Ø.5466	6.36Ø2	Ø.5466	6.36Ø2	2.8947	6.3758
162	SRSS	2.6384	Ø.12ØØ	2.6384	Ø.12ØØ	2.6385	Ø.1253	Ø.5265	1Ø.779Ø	Ø.5265	1Ø.771Ø	Ø.5318	1Ø.78ØØ
	CQC	2.7745	Ø.1343	2.7745	Ø.1344	2.7721	Ø.1444	Ø.5833	11.3353	Ø.5833	11.3353	Ø.6Ø36	11.3256
	AKG	2.7746	Ø.1344	2.7746	Ø.1344	2.7722	Ø.1446	Ø.5834	11.3359	Ø.5834	11.3359	Ø.6Ø4Ø	11.326Ø
	DALS	2.7799	Ø.1349	2.7799	Ø.1349	2.777Ø	Ø.1464	Ø.5856	11.3574	Ø.5856	11.3574	Ø.6Ø91	11.3454
	DABS	2.7799	Ø.1349	2.7799	Ø.1349	2.7836	Ø.1468	Ø.5857	11.3578	Ø.5856	11.3578	Ø.61Ø2	11.3727
18Ø	SRSS	5.2144	Ø.Ø442	5.2144	Ø.Ø442	5.2146	2.8186	Ø.Ø8Ø4	11.Ø14Ø	Ø.Ø8Ø4	11.Ø14Ø	5.1152	11.Ø15Ø
	CQC	5.4863	Ø.Ø367	5.4863	Ø.Ø367	5.4815	2.8131	Ø.Ø646	11.586Ø	Ø.Ø646	11.5861	5.1Ø68	11.576Ø
	AKG	5.4866	Ø.Ø367	5.4866	Ø.Ø367	5.4818	2.8145	Ø.Ø646	11.5868	Ø.Ø646	11.5868	5.1Ø9Ø	11.5764
	DALS	5.4972	Ø.Ø353	5.4971	Ø.Ø353	5.4912	2.8137	Ø.Ø622	11.6Ø89	Ø.Ø622	11.6Ø89	5.1Ø79	11.5963
	DABS	5.4973	Ø.Ø518	5.4973	Ø.Ø518	5.5Ø46	2.8327	Ø.Ø986	11.6Ø93	Ø.Ø986	11.6Ø93	5.1423	11.6249

* Forces in tonne
** Moments in tonne-metre
*** Beams 1,61,162
**** Columns 4Ø,73,18Ø

H EQ Input in X direction
T EQ Input in Z direction
V EQ Input in Y direction

TABLE - 6

FOUR STOREYED UNSYMMETRIC L SHAPED BUILDING - B
AXIAL FORCES IN TYPICAL COLUMN MEMBERS

Member		H Component	H+V Component	H+T+V Component
87	SRSS	2.713	2.713	3.003
	CQC	2.654	2.654	3.174
	AKG	2.654	2.654	3.175
	DALS	2.652	2.653	3.218
	DABS	2.781	2.781	3.218
118	SRSS	5.250	5.250	5.870
	CQC	5.285	5.285	6.380
	AKG	5.285	5.285	6.384
	DALS	5.288	5.288	6.478
	DABS	5.289	5.288	6.491
128	SRSS	5.444	5.444	5.479
	CQC	5.610	5.610	5.775
	AKG	5.610	5.610	5.776
	DALS	5.617	5.617	5.808
	DABS	5.618	5.618	5.812
163	SRSS	8.091	8.091	9.097
	CQC	8.153	8.153	9.991
	AKG	8.153	8.153	9.921
	DALS	8.157	8.157	10.070
	DABS	8.158	8.158	10.080
178	SRSS	8.102	8.102	9.091
	CQC	8.160	8.160	8.313
	AKG	8.160	8.160	8.300
	DALS	8.160	8.160	8.124
	DABS	8.189	8.189	10.090

* Forces in tonne

TABLE - 7

DISPLACEMENTS AT TYPICAL NODES IN UNSYMETRICAL L SHAPED BUILDING B

Node		H Component	H+V Component	H+V+T Component	
		uxx	uxx	uxx	wzz
1.	SRSS	Ø.8188	Ø.8188	Ø.8188	Ø.5191
	CQC	Ø.7916	Ø.7916	Ø.793Ø	Ø.5198
	AKG	Ø.8Ø51	Ø.8Ø51	Ø.8Ø59	Ø.5194
	DALS	Ø.8Ø47	Ø.8Ø47	Ø.8Ø56	Ø.5199
	DABS	Ø.8326	Ø.8327	Ø.8337	Ø.5396
15	SRSS	Ø.7772	Ø.7772	Ø.7772	Ø.5283
	CQC	Ø.794Ø	Ø.794Ø	Ø.7947	Ø.5134
	AKG	Ø.794Ø	Ø.794Ø	Ø.7948	Ø.5134
	DALS	Ø.7947	Ø.7947	Ø.7956	Ø.5117
	DABS	Ø.7946	Ø.7947	Ø.7956	Ø.568Ø
36	SRSS	Ø.6295	Ø.6295	Ø.6295	Ø.4592
	CQC	Ø.6617	Ø.6617	Ø.6611	Ø.4579
	AKG	Ø.6617	Ø.6617	Ø.6611	Ø.4579
	DALS	Ø.6629	Ø.6629	Ø.6622	Ø.4577
	DABS	Ø.6629	Ø.6629	Ø.6638	Ø.46Ø9
49	SRSS	Ø.4644	Ø.4644	Ø.4644	Ø.3278
	CQC	Ø.4745	Ø.4745	Ø.4749	Ø.327Ø
	AKG	Ø.4745	Ø.4745	Ø.475Ø	Ø.3271
	DALS	Ø.4749	Ø.4749	Ø.4754	Ø.327Ø
	DABS	Ø.4749	Ø.4749	Ø.4745	Ø.3293
63	SRSS	Ø.2175	Ø.2175	Ø.2175	Ø.1533
	CQC	Ø.2161	Ø.2161	Ø.2164	Ø.1515
	AKG	Ø.2161	Ø.2161	Ø.2164	Ø.1515
	DALS	Ø.216Ø	Ø.216Ø	Ø.2164	Ø.1513
	DABS	Ø.2189	Ø.2189	Ø.2192	Ø.1595
7Ø	SRSS	Ø.1932	Ø.1932	Ø.1932	Ø.15Ø9
	CQC	Ø.2Ø32	Ø.2Ø32	Ø.2Ø3Ø	Ø.15Ø8
	AKG	Ø.2Ø32	Ø.2Ø32	Ø.2Ø3Ø	Ø.15Ø8
	DALS	Ø.2Ø36	Ø.2Ø36	Ø.2Ø34	Ø.15Ø8
	DABS	Ø.2Ø36	Ø.2Ø36	Ø.2Ø39	Ø.1623

* All displacements (in meter) are to be multiplied by $1Ø^{-2}$

TABLE – 8

FORCES IN TYPICAL MEMBERS OF 7 STOREYED UNSYMMETRIC (STEPPED) BUILDING C

Member		SHEAR FORCES						BENDING MOMENTS					
		H Component		H+V Component		H+T+V Component		H Component		H+V Component		H+T+V Component	
		fyy	fzz	fyy	fzz	fyy	fzz	Myy	Mzz	Myy	Mzz	Myy	Mzz
8	SRSS	Ø.4515	Ø.2339	Ø.4515	Ø.2339	Ø.4644	Ø.2478	Ø.8217	2.1326	Ø.8217	2.1326	Ø.8443	2.1847
	CQC	Ø.4486	Ø.2469	Ø.4486	Ø.2469	Ø.4891	Ø.2812	Ø.8527	2.1167	Ø.8527	2.1167	Ø.9295	2.263Ø
	AKG	Ø.4478	Ø.2465	Ø.4478	Ø.2465	Ø.4882	Ø.28Ø7	Ø.8513	2.113Ø	Ø.8513	2.113Ø	Ø.928Ø	2.2591
	DALS	Ø.4472	Ø.2466	Ø.4472	Ø.2466	Ø.49Ø2	Ø.2836	Ø.851Ø	2.1Ø94	Ø.851Ø	2.1Ø94	Ø.9355	2.263Ø
	DABS	Ø.5Ø41	Ø.2537	Ø.5Ø41	Ø.2537	Ø.5563	Ø.293Ø	Ø.8678	2.358Ø	Ø.8676	2.33Ø6	Ø.9581	2.5657
19	SRSS	1.6527	Ø.3334	1.6527	Ø.3334	1.6911	Ø.6759	Ø.6159	3.Ø28Ø	Ø.6159	3.Ø277	1.2536	3.Ø979
	CQC	1.6394	Ø.1286	1.6394	Ø.1286	1.7373	Ø.5172	Ø.237Ø	3.ØØ3Ø	Ø.237Ø	3.ØØ3Ø	Ø.9622	3.1796
	AKG	1.6367	Ø.1284	1.6367	Ø.1284	1.7343	Ø.5165	Ø.2366	2.9982	Ø.2366	2.9982	Ø.96Ø8	3.1742
	DALS	1.6338	Ø.1183	1.6338	Ø.1183	1.7352	Ø.5Ø98	Ø.218Ø	2.992Ø	Ø.218Ø	2.992Ø	Ø.9485	3.1753
	DABS	1.7879	Ø.4716	1.7879	Ø.4716	1.9684	Ø.8349	Ø.8721	3.2725	Ø.8721	3.2725	1.5472	3.6Ø3Ø
71	SRSS	Ø.Ø993	Ø.2498	Ø.Ø993	Ø.2498	Ø.2952	Ø.5593	1.257Ø	Ø.4499	1.257Ø	Ø.4499	2.5624	1.4Ø83
	CQC	Ø.Ø614	Ø.19Ø8	Ø.Ø614	Ø.19Ø8	Ø.2767	Ø.5384	1.1Ø95	Ø.2794	1.1Ø95	Ø.2794	2.4165	1.3297
	AKG	Ø.Ø613	Ø.19Ø6	Ø.Ø613	Ø.19Ø6	Ø.2767	Ø.538Ø	1.1Ø82	Ø.2789	1.1Ø82	Ø.2789	2.415Ø	1.3299
	DALS	Ø.Ø578	Ø.1873	Ø.Ø578	Ø.1873	Ø.2775	Ø.5466	1.1Ø38	Ø.263Ø	1.1Ø38	Ø.263Ø	2.4769	1.3343
	DABS	Ø.131Ø	Ø.3115	Ø.1311	Ø.3115	Ø.3352	Ø.6649	1.5187	Ø.5971	1.5188	Ø.5972	2.9936	1.5949
1Ø8	SRSS	3.1697	Ø.3939	3.1697	Ø.3939	3.29Ø5	Ø.9225	Ø.7159	5.7Ø95	Ø.7159	5.7Ø95	1.6672	5.9275
	CQC	3.1Ø13	Ø.241Ø	3.1Ø13	Ø.241Ø	3.2311	Ø.816Ø	Ø.4377	5.5866	Ø.4378	5.5866	1.4794	5.8185
	AKG	3.Ø975	Ø.24Ø6	3.Ø975	Ø.24Ø6	3.2268	Ø.8156	Ø.4369	5.5798	Ø.4369	5.5798	1.4786	5.81Ø8
	DALS	3.Ø9Ø3	Ø.227Ø	3.Ø9Ø3	Ø.227Ø	3.2Ø8Ø	Ø.8139	Ø.4123	5.5669	Ø.4123	5.5669	1.4758	5.776Ø
	DABS	3.3415	Ø.5144	3.3416	Ø.5144	3.3114	1.Ø764	Ø.9313	6.Ø155	Ø.9313	6.Ø141	1.945Ø	6.8626
335	SRSS	3.66Ø8	Ø.1613	3.66Ø8	Ø.1613	3.85ØØ	Ø.1673	Ø.4967	12.729Ø	Ø.4969	12.729Ø	Ø.51Ø8	13.388Ø
	CQC	3.6975	Ø.1668	3.6974	Ø.1668	3.9634	Ø.1857	Ø.5153	12.8574	Ø.5153	12.8574	Ø.564Ø	13.782Ø
	AKG	3.6995	Ø.167Ø	3.6995	Ø.167Ø	3.9656	Ø.1862	Ø.516Ø	12.8644	Ø.516Ø	12.8644	Ø.5652	13.7898
	DALS	3.7Ø65	Ø.1676	3.Ø9Ø3	Ø.227Ø	4.ØØ11	Ø.1899	Ø.5176	12.8887	Ø.5176	12.8887	Ø.5756	13.9131
	DABS	3.721Ø	Ø.1676	3.72Ø9	Ø.1676	4.3831	Ø.1899	Ø.5177	12.9391	Ø.5177	12.9391	Ø.5759	15.243
413	SRSS	4.2312	Ø.6482	4.2312	Ø.6482	4.3524	2.7731	1.3Ø51	8.7944	1.3Ø51	8.7944	5.77ØØ	9.Ø468
	CQC	4.2176	Ø.2329	4.2176	Ø.2329	4.2795	2.6428	Ø.48Ø7	8.7665	Ø.48Ø7	8.7665	5.51Ø8	8.8944
	AKG	4.22Ø2	Ø.2325	4.22Ø2	Ø.2325	4.2815	2.6426	Ø.4799	8.7665	Ø.48Ø7	8.7665	5.5182	8.8984
	DALS	4.2197	Ø.2161	4.2197	Ø.2161	4.2535	2.627Ø	Ø.4461	8.77Ø7	Ø.4461	8.7719	5.4768	8.8395
	DABS	4.2916	Ø.9838	4.2916	Ø.9838	4.8224	2.9972	1.9766	8.9178	1.9767	8.9178	6.2166	1Ø.Ø226

* Forces in tonne
** Moments in tonne-metre
*** Beams 8, 71, 335
**** Columns 19, 108, 413

H EQ Input in X direction
T EQ Input in Z direction
V EQ Input in Y direction

TABLE - 9

SEVEN STOREYED UNSYMMETRIC (STEPPED) BUILDING - C
AXIAL FORCES IN TYPICAL COLUMN MEMBERS

Member		H Component	H+V Component	H+T+V Component
100	SRSS	2.184	2.184	2.216
	CQC	2.227	2.227	2.376
	AKG	2.227	2.228	2.376
	DALS	2.224	2.224	2.396
	DABS	2.255	2.255	2.438
157	SRSS	2.266	2.266	2.344
	CQC	2.296	2.296	2.304
	AKG	2.295	2.295	2.304
	DALS	2.292	2.292	2.279
	DABS	2.339	2.339	2.623
322	SRSS	13.753	13.750	14.050
	CQC	13.700	13.700	13.480
	AKG	13.699	13.699	13.480
	DALS	13.660	13.660	13.230
	DABS	13.920	13.920	15.260
384	SRSS	8.586	8.586	9.319
	CQC	8.663	8.663	10.410
	AKG	8.665	8.665	10.413
	DALS	8.694	8.694	10.790
	DABS	8.728	8.728	10.820
413	SRSS	14.660	14.660	20.450
	CQC	14.980	14.980	23.130
	AKG	14.981	14.981	23.132
	DALS	15.050	15.050	24.000
	DABS	15.150	15.150	24.150

* Forces in tonne

TABLE - 10

DISPLACEMENTS AT TYPICAL NODES IN 7 STOREYED (STEPPED) BUILDING C

Node		H Component	H+V Component	H+V+T Component	
		uxx	uxx	uxx	wzz
10	SRSS	Ø.2563	Ø.2562	Ø.2649	Ø.183Ø
	CQC	Ø.255Ø	Ø.255Ø	Ø.2613	Ø.175Ø
	AKG	Ø.255Ø	Ø.2549	Ø.2611	Ø.17Ø3
	DALS	Ø.2546	Ø.2546	Ø.2594	Ø.17Ø2
	DABS	Ø.26Ø2	Ø.26Ø2	Ø.2962	Ø.1686
11	SRSS	Ø.2295	Ø.2295	Ø.2421	Ø.1114
	CQC	Ø.23Ø4	Ø.23Ø4	Ø.2466	Ø.11Ø9
	AKG	Ø.23Ø3	Ø.23Ø3	Ø.2466	Ø.111Ø
	DALS	Ø.23Ø5	Ø.23Ø5	Ø.2482	Ø.1113
	DABS	Ø.2325	Ø.2325	Ø.2755	Ø.1175
42	SRSS	Ø.2122	Ø.2122	Ø.2194	Ø.Ø819
	CQC	Ø.2117	Ø.2117	Ø.2156	Ø.Ø827
	AKG	Ø.2116	Ø.2117	Ø.2156	Ø.Ø828
	DALS	Ø.2115	Ø.2115	Ø.2139	Ø.Ø834
	DABS	Ø.214Ø	Ø.214Ø	Ø.2437	Ø.Ø683
78	SRSS	Ø.1428	Ø.1429	Ø.1457	Ø.Ø5Ø8
	CQC	0.1436	Ø.1436	Ø.1486	Ø.Ø51Ø
	AKG	Ø.1437	Ø.1437	Ø.1487	Ø.Ø511
	DALS	Ø.1438	Ø.1438	Ø.1496	Ø.Ø514
	DABS	Ø.1438	Ø.1442	Ø.1589	Ø.Ø558
136	SRSS	Ø.Ø528	Ø.Ø528	Ø.Ø554	Ø.Ø349
	CQC	Ø.Ø535	Ø.Ø535	Ø.Ø571	Ø.Ø334
	AKG	Ø.Ø535	Ø.Ø535	Ø.Ø572	Ø.Ø334
	DALS	Ø.Ø536	Ø.Ø536	Ø.Ø577	Ø.Ø332
	DABS	Ø.Ø538	Ø.Ø538	Ø.Ø631	Ø.Ø375
149	SRSS	Ø.Ø534	Ø.Ø534	Ø.Ø543	Ø.Ø21Ø
	CQC	Ø.Ø533	Ø.Ø534	Ø.Ø538	Ø.Ø2Ø4
	AKG	Ø.Ø534	Ø.Ø534	Ø.Ø538	Ø.Ø2Ø4
	DALS	Ø.Ø534	Ø.Ø534	Ø.Ø536	Ø.Ø2Ø4
	DABS	Ø.Ø54Ø	Ø.Ø54Ø	Ø.Ø589	Ø.Ø233

* All displacement are to be multiplied by $1Ø^{-1}$ (mm)