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A proposed draft for IS: 1893 provisions on seismic design of buildings – Part II^{+}: Commentary and examples

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The provisions for seismic design of buildings contained in IS:1893-1984 need to be revised in view of many deficiencies that are currently being felt. Part I of this paper discussed a proposed draft on provisions for seismic design of buildings for inclusion in the next edition of the code. This paper provides a detailed commentary to explain the proposed codal provisions.

Based on the detailed review[1,2] of IS:1893-1984[3] provisions on seismic design of buildings, a revised draft for the same has been presented in Part I of this paper[4]. In order to explain these provisions and to give the intent behind some of the clauses, this paper provides a detailed commentary.

In the following sections, clause numbers are as in Part I of the paper. For instance, clause C3.4.1 of this paper contains discussion about clause 3.4.1 of Part I. Only those clauses of Part I which require discussion are included in the commentary. Figures and tables of Part II are given numbers starting with C. Thus, for example "Table 4" refers to the Table 4 of Part I of this paper, while "Table C4" refers to Table C4 of Part II of the paper.

COMMENTARY

Symbols (C2.2): The 1984 edition of the code considers variation in seismic risk in different parts of the country through "basic horizontal coefficient" (α_0) in the seismic coefficient method and through "seismic zone factor" (F_0) in the response spectrum method. There is really no need for defining two different parameters for the same purpose; in fact F_o is simply five times α_0. Hence, in the new provisions, a single parameter "zone factor" (Z) has been defined.

Symbol "A" has been assigned to represent the design horizontal acceleration spectrum arrived at after considering all the relevant factors such as the importance factor (I), zone factor (Z), response reduction factor (R), and soil profile factor (S). This is the spectrum to be finally used for design of a particular type of building at that site, irrespective of the analysis procedure used (i.e., static or dynamic).

Ground Motion (C3.1.1) :

The Northridge earthquake of January 17,1994 in southern California has clearly shown the vulnerability of prestressed horizontal members to vertical component of ground motion. To check the structure for vertical component of motion, it may be sufficient to consider the structure, except for the large-span structures, as rigid for vertical vibrations and to subject it to zero-period vertical accelerations, with no reduction factor (i.e., the seismic coefficient as 0.5 ZIS).

Assumptions (C3.2):

C3.2(c): The elastic modulus for materials such as concrete and masonry is difficult to specify. Its value varies with stress level, loading conditions (static versus dynamic),

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material strength, age of the material, etc. Hence, there tends to be a very large variation in the value of elastic modulus specified by different codes even for a specific grade of concrete under static condition.

For instance, ACI-318[5] recommends E as $4700\sqrt{f'_c}$ (MPa), while IS : 456-1978[6] suggests $5700\sqrt{f_{ck}}$ (MPa) or about $6370\sqrt{f'_c}$ (f_{ck} = characteristic cube strength, f'_c = characteristic cylinder strength = $0.8 f_{ck}$); i.e., E given by the IS code is about 1.4 times the value given by the ACI code for the same grade of concrete. Further, actual concrete strength in a structure is usually more than the specified 28-day strength and it also increases with time. There are further difficulties with choosing the value of modulus of elasticity for concrete of seismic analysis. The value given in the codes, such as ACI-318[5] and IS:456[6] is often the secant modulus; its value is prescribed with a view to obtain a conservative estimate of deflections, i.e., lower stiffness. On the other hand, the dynamic modulus of concrete refers to almost pure elastic effects and is equal to the initial tangent modulus and is appreciably higher than the secant modulus. When a structure is new and subjected to low amplitude of ground motion, the dynamic modulus of elasticity may be applicable. However, long time exposure of the structure to wind pressures may overcome the initial stiffness properties, and the modulus of elasticity of concrete may tend to be close to the secant modulus. The value of modulus of elasticity to be used in analysis has two opposite implications on seismic design. For calculation of the design seismic force it is unconservative to have low value of modulus of elasticity; this leads to high time period and lower design seismic coefficient. However, for the drift criteria (deflection condition) it is unconservative to make a higher estimate of the stiffness.

Hence, there are no easy answers to the question of what value of modulus of elasticity should be used for seismic analysis. Considering the enormous variations, this clause allows the designer to use elastic modulus as for static condition. However, a safeguard has been introduced (4.4.3 and 4.6.2) against using a very high value of natural period for calculation.

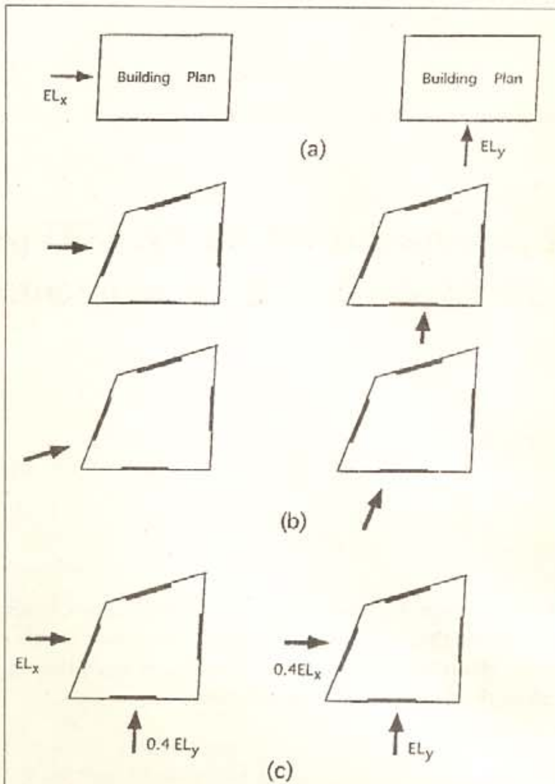


FIG.C1 (a) EARTHQUAKE LOAD CONDITION FOR DESIGN OF BUILDINGS WITH LATERAL LOAD RESISTING SYSTEMS ORIENTED ALONG TWO PRINCIPAL DIRECTIONS;

(b) TO ENSURE NO ELEMENT IS UNDER-DESIGNED, EARTHQUAKE FORCE SHOULD BE CONSIDERED IN ALL POSSIBLE DIRECTIONS IN WHICH THE ELEMENTS ARE ORIENTED;

(c) ALTERNATIVE TO CONDITION (b), THE BUILDING MAY BE DESIGNED CONSIDERING FULL DESIGN LOAD IN ONE DIRECTION AND 40% DESIGN LOAD IN THE OTHER DIRECTION, ACTING SIMULTANEOUSLY AND VICEVERSA

Load Combinations and Increase in Permissible Stresses (C3.3) :

C3.3.1: The design ground motion can occur along any direction of a building. Moreover, the motion has different directions at different time instants. The earthquake ground motion can be thought of in terms of components in the two horizontal and one vertical directions. For buildings with lateral force resisting elements oriented along two principal directions, it is usually sufficient to design the building for the earthquake force acting in x- and y- directions separately; i.e., not for forces acting in both the directions simultaneously (Fig. C1(a)). During earthquake shaking, when the resultant ground motion is in a direction other than x and y, the motion can be resolved into the x-and y-components, as elements in the two principal directions which are normally able to withstand, except for the corner columns for which this may be unconservative.

However, when the lateral force resisting elements are not oriented along the x- and y- directions, design based on earthquake force in x- and y- directions, separately, leads to underdesign of the elements. In such a case, one should design the structure for earthquake force acting along all possible directions in which the seismic load resisting elements are oriented (Fig. C1(b)). One way to get around the difficulty of having to consider too many possible earthquake directions is to design the structure for (Fig. C1(c)):

i. Full design force in the x-direction (EL_x) acting simultaneously with 40% of the design force in the y-direction (EL_y); i.e., $EL_x + 0.4\, EL_y$), and ii Full design force in the y-direction (EL_y) acting simultaneously with 40% of the design force in the x-direction (EL_x); i.e., ($0.4\ EL_x + EL_y$).

This combination ensures that elements oriented in any direction will have sufficient lateral strength. It is also a good practice to design the corner columns of otherwise orthogonal system as per these combinations.

Design Spectrum (C3.4):

C3.4.2: The present code[2] provides different design spectra for use in the seismic coefficient and the response spectrum methods. The draft provisions provide for a common design spectrum which is applicable irrespective of whether the design force is calculated by the static or dynamic procedure. Several important changes have been introduced in the new design spectrum:

a. The performance factor (K) in the earlier version, has been replaced by a response reduction factor (R). The soil-foundation factor (β) has been replaced by a soil-profile factor (S), and the basic horizontal coefficient (α_0) and seismic zone factor (F_0) have been replaced by the zone factor (Z). The terms representing the importance of structure (I) and the structure flexibility effect (C) are the same.

b. In the earlier version, the code directly specified the design seismic force; this was often misunderstood as the maximum expected force on the structure. In line with the world-wide trend in this regard, the code now tries to distinguish the two. The terms ($Z\ I\ C\ S$) represent the spectrum corresponding to the maximum expected earthquake force, if the structure is to respond elastically, and the design force is arrived at by dividing this force by R. The term R gives a clear indication of the level of overstrength and ductility that a structure is expected to have[8].

c. The term Z now represents the realistic values, as fraction of acceleration due to gravity, of the expected peak ground acceleration in different seismic zones. For instance, the code specifies zone IV for areas which are likely to sustain shaking of intensity VIII on the Modified Mercalli scale. The value of Z (= 0.30) for zone IV gives the value of peak ground acceleration as $0.30g$ which may be reasonably expected in shaking intensity VIII.

d. Adoption of realistic values of peak ground acceleration as the seismic zone factor has also rationalized the relative values of design force for different seismic zones. As the intensity of shaking goes up one level on the MM scale (say from VI to VII), the peak ground acceleration almost doubles. In earlier code this was not duly reflected since the seismic force in different zones varied in the ratio 1:2:4:5:8.

e. Another change introduced is that the soil-foundation factor (β) has been replaced by the soil-profile factor. Factor β, depending on the type of soil and the type of foundation, was intended to increase the design force for systems that are more vulnerable to differential settlements. However, in real earthquake situations, buildings do not suffer higher earthquake-induced inertia force on account of vulnerability to differential settlement. Also, the problem of differential settlement cannot be addressed by increasing the design seismic force on the building; instead it has to be addressed by a proper choice of the foundation. On the other hand, records obtained in the past earthquakes clearly show that the average acceleration spectrum tends to be different for sites with different soil profiles (Fig.C2). The soil-profile factor (S) considers this variation.

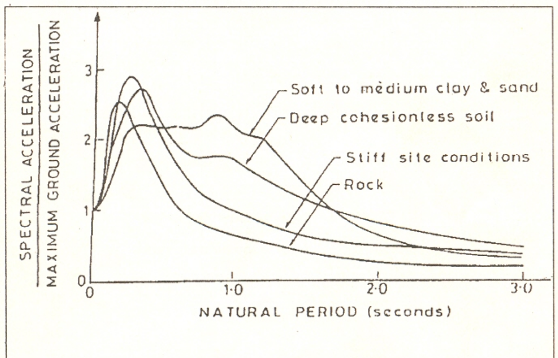


FIG. C2 EFFECT OF SOIL PROFILE ON SHAPE OF RESPONSE SPECTRUM

f. The product of terms (C) and (S), shown in Fig.2 of the draft code, represents the shape of design spectrum with peak ground acceleration scaled to the value of 1.0. This shape is same as the average shape of acceleration response spectrum, except in the period range 0 - 0.1 sec (Fig.C3). In this range, the plot of Fig.2 is at a constant value as against the response spectrum which varies from 1.0 at zero period to the maximum value (equal to 2.0) at a period of around 0.1 sec. The shape of design spectrum is modified in this range in view of the fact that ductility does not help in reducing the maximum force on stiff structures with fundamental period in the range 0 to 0.1 sec[9,10]. However, it is acceptable if one were to use the shape of response spectrum in this range, for modes other than the fundamental mode (Fig.C3).

g. The basic philosophy of earthquake-resistant design is that a structure should not collapse during a severe earthquake, although it may undergo some structural as well as nonstructural damage. Hence, a building is usually designed for a much less force than what would be required if the building were to remain elastic during a severe earthquake shaking. Fig. C4 shows

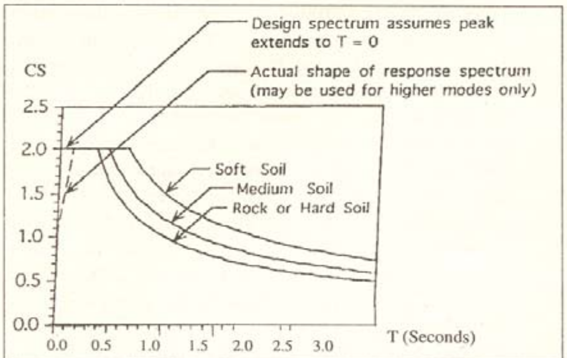


FIG. C3 SHAPE OF DESIGN SPECTRUM

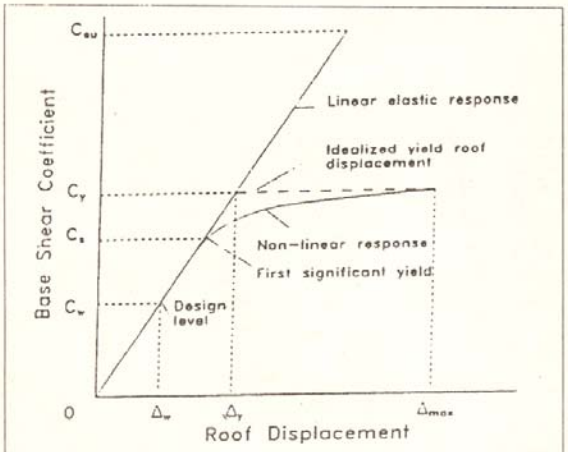


FIG. C4 TYPICAL LOAD DISPLACEMENT RESPONSE OF A BUILDING

global structural response of a building in terms of base-shear coefficient versus roof-displacement during a severe ground shaking. The response reduction factor, R represents the ratio of the forces that will develop by design-level intensity of ground shaking, if the structure were to respond elastically, to the force that is considered appropriate for design. Factor R consists of essentially two factors; the ductility reduction factor (R_μ) and the overstrength (Ω)[8]. Ductility reduction factor (R_μ) reduces the elastic demand force to the level of maximum yield strength of the structure ($R_\mu = C_{eu}/C_Y$); this reduction basically depends on ductility and time period of the structure, and hence on its energy dissipation capacity. Overstrength (Ω) accounts for the additional strength over the design force that is inherently introduced in the code-designed structures and is defined as the ratio between the maximum lateral strength of t he structure and the code prescribed unfactored design base shear force ($\Omega = C_Y/C_w$). Hence,

$$R = R_\mu \ \Omega \tag{1}$$

Due to the variation in ductility and overstrength for different building systems, the value of R may vary significantly; it would be ideal to evaluate R with a realistic nonlinear analysis. However, considering practical difficulties with prescribing such a sophisticated analysis for design, representative values of R are specified for general class of structures, based on observed performance of buildings in the past earthquakes, expected ductility (toughness) and overstrength, and on practices in other countries.

C3.4.6: Site-specific studies are often conducted to prescribe design spectrum to be used for important projects, such as a nuclear power plant. It will be unrealistic to design structures for forces lower than those for which a similar building located at the same site has to be designed as per the code. Hence, this clause provides that irrespective of the site-specific spectrum, the building should still comply with the minimum requirements of this code.

Buildings (C4.0)

Performance of a building in an actual earthquake depends on its overall configuration, lateral stiffness, ductility, and lateral strength.

Configuration: Buildings with simple and regular configuration with direct load transfer path perform much better during strong shaking. While additional analysis requirements are usually provided for buildings with irregular configuration, sophistication in analysis is not necessarily a solution to the problems caused by irregular configuration. Hence, the seismic configuration is an important consideration at the stage of architectural planning of a building[11], and all efforts should be made to achieve a regular, or nearly a regular configuration. The cost of compliance with seismic code provisions[12,13] also depends very strongly on building configuration.

The major factors[12,13] influencing the cost of complying with the provisions are:

1. The complexity of the shape and structural framing system for the building. (It is much easier to provide seismic resistance in a building with a simple shape and framing plan.)
2. The cost of the structural system in relation to the total cost of the building
3. The stage in design at which the provision of seismic resistance is first considered. (The cost can be inflated greatly if no attention is given to seismic resistance until after the configuration of the building, the structural framing plan, and the materials of construction have already been chosen).

Another important requirement for good seismic performance is redundancy in structural system. In systems without redundant components, every single component must remain functional to ensure the overall integrity of structure. However, in buildings with high redundancy, several members may fail and yet may not lead to collapse of the building. Considering the uncertainties in evaluating ground motion parameters and in behaviour of structural members, there is a distinct possibility of failure of at least some of the members during strong shaking. Thus, one should avoid situations where lateral load resistance in a direction is provided by a structural system without any redundancy.

Stiffness: Even though the shape of design spectrum suggests that stiff systems attract more earthquake force, it is desirable to have high lateral stiffness in a building. Higher lateral stiffness leads to lower deformation during strong ground shaking and improves the seismic performance in a number of ways:

a. Lower drift leads to reduced inelastic strains in the structural members in the event of strong shaking; this implies less damage to the structure.

b. Large lateral deformations lead to substantial secondary moments (P-Δ effect), given by gravity load times the lateral displacement, for which the building may not have been designed.

c. Lower deformations cause less damage to non-structural elements and to structural non-seismic elements. The repair and replacement cost of these elements is usually quite substantial. Moreover, many of these elements share significant part of seismic force and with damage to such members, this load-carrying capacity is lost.

d. The occupants undergo less scare and trauma if the building deformation during the earthquake is less.

Ductility: As discussed elsewhere, the code relies on inelastic response, i.e., ductility of structure during strong ground shaking.

Design Live Load for Earthquakes (C4.1):

C4.1.2: Live load on roof is not considered in evaluating the seismic weight, as the roof is likely to be carrying the live load at the time of earthquake shaking. However, the contents of water tanks placed at the roof are not to be treated as a live load for this purpose.

Seismic Weight (C4.2): Seismic weight is the total weight of the building and that part of the service load which may reasonably be expected to be attached to the building at the time of an earthquake shaking. This includes permanent and movable partitions, permanent equipment, etc. Buildings designed for storage purposes are likely to have larger percent of the service load present at the time of earthquake shaking.

Design Lateral Force (C4.3)

C4.3.2: The earthquake force is an inertia force (mass times acceleration), caused by ground acceleration on the building mass. Since most of the mass is located at the floors, most of the inertia force is generated there. At any instant the earthquake force at the floor is the total floor mass times the floor acceleration. This force is then transferred by the floor to the supporting elements (Fig.C5). The amount by which different vertical elements share this force depends on the in-plane floor diaphragm action, lateral stiffness of the vertical elements, and other factors such as torsion in the building caused by eccentricity between the centre of mass and centre of stiffness. Hence, this clause requires that the design earthquake force be first evaluated for the entire building as a whole and then this force be distributed to different vertical elements. This caution is considered necessary because at times a designer may erroneously calculate the design seismic force for a frame in isolation of the entire building, by considering the tributary mass shared by that frame; such a procedure has a serious problem that only a fraction of the building mass is considered in the seismic load calculation.

Design Base Shear (C4.3.3):

The equation $V = A\ W$ gives the overall design force to be applied to the entire building. Depending on the value of fundamental time period in the two directions, the design seismic force may be different for the two principal directions. This is not the maximum expected force on the structure during a strong shaking. Design of members and connections based on linear elastic analysis of the building using this force, alongwith other parameters, is expected to give acceptable performance of a building during the earthquake shaking of expected intensity. The assumptions involved in the "static" procedure reflected in this expression are: (a) Fundamental mode of the building makes the most significant contribution to base shear, and (b) The total building mass is considered as against the modal mass that would be used in a dynamic procedure. Both these assumptions are quite appropriate for low-and medium-rise buildings. Hence, this expression is not as approximate as the term "static" may indicate, because even here the most

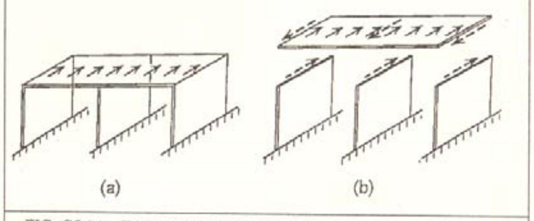


FIG. C5 (a) EARTHQUAKE CAUSES INERTIA FORCE AT THE FLOORS;

(b) FLOORS TRANSFER THE INERTIA FORCE TO THE VERTICAL ELEMENTS (e.g., WALLS OR FRAMES), WHICH TRANSFER IT TO THE GROUND significant property of the building, i.e., its fundamental period, is being duly considered.

Dual Systems (C4.3.4): Consider a building with moment resisting frames and shear walls. Usually the shear walls are much more stiff as compared to the frames, and hence resist most of the lateral force, with very little force going to the frames. The concern is that if shear walls fail, there may be sudden collapse of the building; this is particularly so since most buildings will have only small number of the shear walls with very little redundancy. Hence, it is required that the frame should be able to resist at least 25% of the design force. This way a secondary resisting system is available which in the event of shear wall failure, can continue to support the gravity loads after the earthquake. As an example, say the lateral stiffness of the shear walls and of the frames in one lateral direction of the building are in the ratio 9:1; and that the floor diaphragm is rigid in its own plane. The analysis in this case will give the total force in the walls as 90% of the design force, and in the frames as 10% of the design force. This clause requires that in such a situation while the walls should be designed for 90% of the design force, the frames should be designed to carry 25% of the design force. However, note that in this case the storey drifts for clause 4.7 should be calculated by analyzing frame for 10% of the design force, and not for 25% of the design force.

Fundamental Period (C4.4)

C4.4.1 & C4.4.2: The present code provides two expressions for estimating fundamental period of a building: (i) $T = 0.1\ n$ for moment resisting frames without bracing or shear walls, and (ii) $T = 0.09\ H/\sqrt{d}$ for all other building systems (n = number of storeys; H = building height; and d = maximum base dimension in direction parallel to the applied seismic force, (Fig. C6). These expressions were adopted from earlier versions of the Uniform Building Code provisions which were based on vibration tests conducted on actual buildings in California in the sixties.

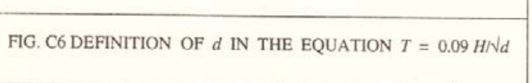


FIG. C6 DEFINITION OF d IN THE EQUATION $T = 0.09\ H/\sqrt{d}$

In India, is is a common practice to treat the brick infill walls as non-structural and to ignore the strength and stiffness contributed by them. Hence, many moment resisting frame buildings with brick infills have been designed with fundamental period based on the expression $T = 0.1\ n$. Experimental observations on models [14] and on prototypes[15] show that in such buildings the brick infills contribute significant lateral stiffness, and therefore the expression $T = 0.1\ n$ is not applicable to such buildings. In fact, the expression $T = 0.09\ H/\sqrt{d}$ gives a much better estimate of the fundamental period when compared to the results of ambient vibration tests on such buildings. Hence, the present draft specifically suggests that for buildings with brick infill panels, the expression $T = 0.09\ H/\sqrt{d}$ is applicable.

Even for moment resisting frame buildings without brick infills, the expression for fundamental period has been changed to $T = 0.075\ h^{3/4}$. This is because it can be shown analytically that period is better related linearly to three-fourth power of overall building height[12, 13] The coefficient value of 0.075 is based on the value used in the U.S. codes ($T = 0.030\ h^{3/4}$, h in feet) applicable to reinforced concrete frames. This equation has been obtained on the basis of actual recorded motions on multistorey buildings during the 1971 San Fernando earthquake[12,13]

On the whole, the new expressions are expected to reflect the stiffness of Indian buildings better than the earlier expressions. However, clearly there is an urgent need to conduct an extensive experimental investigation of the fundamental period of as-built buildings in India.

C4.4.3 There are significant uncertainties in analytically evaluating time period of buildings. This is because of numerous factors including stiffness contribution of non-structural and structural non-seismic elements, and uncertainties in evaluating modulus of elasticity of concrete and moment of inertia of a concrete member. Depending on whether or not the stiffness contribution of structural non-seismic, and non-structural members have been modelled, and what material and section properties are chosen, one can get large variation in the value of fundamental period. It is now an established fact that the analysis of "bare" frame, i.e., ignoring the stiffness contribution of the infills and other non-structural elements, usually gives fundamental period which may be higher than the empirical value or the experimentally evaluated value. On the other hand, empirical expressions in the code are based on observations of actual as-built structures, and hence these expressions, even though empirical and approximate, are more reliable than a dynamic analysis based on questionable parameters and assumptions. Hence, the current trend is to put an upper-bound value of fundamental period that can be used to calculate the design force, or to put a lower-bound limit on the overall seismic design force, based on empirical formulae for fundamental period. This clause intends to do the same through an upper bound on the fundamental period to be used in design.

The clause requires that if fundamental period is calculated analytically, its value should not exceed $C_a T_a$, where C_a is a coefficient whose value ranges from 1.2 to 1.6 depending on the seismic zone, and T_a is the time period by code-specified empirical formula. Variation in the value of C_a accounts for the fact that buildings on which experimental observations have been conducted usually lie in high seismic zones, and that buildings in the lower seismic zones may be considerably more flexible. The value of 1.2 for C_a in zone V recognizes the fact that the empirical values are indeed not exact and that it is possible to obtain a reasonable assessment of fundamental period by a careful dynamic analysis. The relative value of C_a for different zones is based on an analytical study[16] and is consistent with the values used in the NEHRP code[12, 13]. Commentary to the NEHRP provisions justifies this clause as:

> "If one ignores the contribution of nonstructural elements to the stiffness of the structure . . . the calculated period is lengthened, leading to a . . . decrease in the design force. Nonstructural elements do not know that they are nonstructural. They participate in the behaviour of the structure even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the unconservative side. The limitation of $C_a T_a$ is imposed as a safeguard".

Another important modification is that the benchmarking of the minimum design force on the basis of empirical value of the fundamental period is applicable to all buildings irrespective of the configuration. This was different in the present code which encouraged the designer to use the value of fundamental period obtained from dynamic analysis, but offered the empirical values in the absence of such an analysis and "required" dynamic analysis for the irregular buildings. One could argue about the applicability of empirical expressions for irregular buildings. However, in the absence of any better alternative for reliable calculation of fundamental period of irregular buildings, empirical expressions are the best option which the codes around the world are now adopting. Consider, for instance, the regular building (Fig. C7(a)) which is designed on the basis of empirical expression $T = 0.075\, h^{0.75} = 0.59$ secs. Now consider the building in Fig. C7(b) which is similar to the building of Fig. C7(a) except that a mezzanine floor has been introduced. It is obvious that the building has now become irregular requiring dynamic analysis; also, it is now more stiff. Say a dynamic analysis of this building gives the value of fundamental period as 1.3 secs. In this case, it will be unreasonable to design the second building for a much higher period (and hence for lower force) just because a dynamic analysis has been performed even when we know that its fundamental period is less than that of the first building.

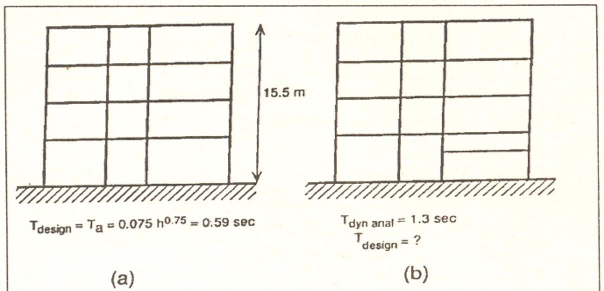


FIG. C7: (a) A REGULAR FRAME BUILDING DESIGNED BY EMPIRICAL VALUE OF $T = 0.59$ SEC;

(b) SAME BUILDING NOW HAS AN ADDITIONAL MEZZANINE FLOOR AND NOW TREATED AS AN IRREGULAR BUILDING, AND ITS DYNAMIC ANALYSIS GIVES $T = 1.3$ SEC; (CLAUSES 4.4.3 AND 4.6.2 PROTECT AGAINST BUILDING BEING DESIGNED FOR LOW SEISMIC FORCE CORRESPONDING TO $T = 1.3$ SEC.)

Distribution of Design Force (C4.5)

Vertical Distribution of Base Shear to Different Floor Levels (C4.5.1): The distribution of lateral forces with building height depends on natural periods and mode shapes and on the shape of design spectrum. In low-and medium-rise buildings, the fundamental mode dominates the contribution to overall forces; moreover, for such buildings with regular distribution of mass and stiffness (with building height) the fundamental mode shape is similar to a straight line. However, for tall buildings, contribution of higher mode can also be significant even though fundamental mode may still be the most significant mode. Hence, for k, the expression for load distribution [12, 13] is:

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

where $k = 1$ for $T \leq 0.5$ sec, and $k = 2$ for $T \geq 2.5$ sec; the value of k varies linearly for T in the range of 0.5 sec to 2.5 sec. In contrast, IS:1893-1984 provides for $k = 2$ for all buildings and the draft code has retained the same expression.

Horizontal Distribution of Design Lateral Force to Different Lateral Force Resisting Elements (C4.5.2):

C4.5.2.1: Horizontal floor diaphragms play a very important role in seismic response of a building. When the building vibrates in the horizontal direction, a monolithic reinforced concrete floor acts as a beam with depth equal to the building width (or building length), and the width equal to the floor thickness (Fig. C8). Due to significant width or length of the building, quite often the floor cannot have any

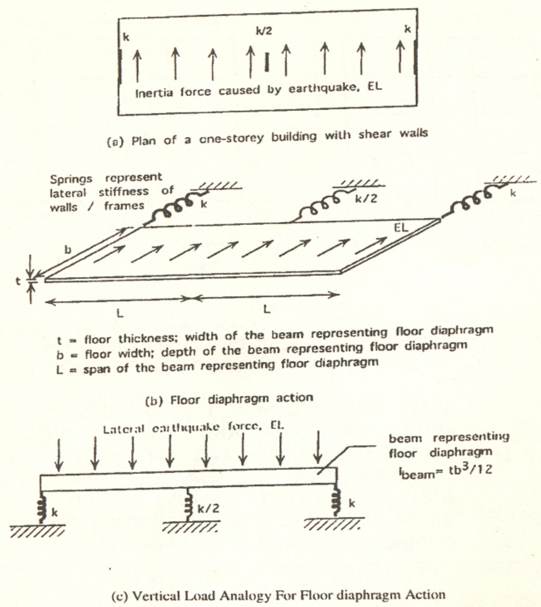


FIG. C8 FLOOR DIAPHRAGM ACTION

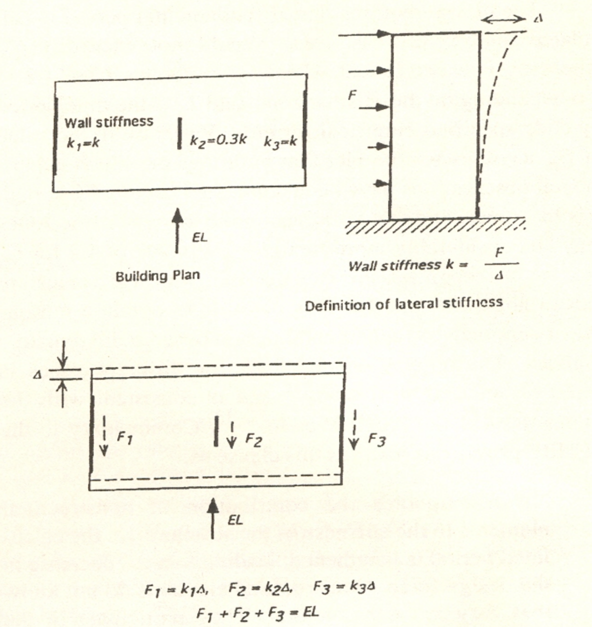


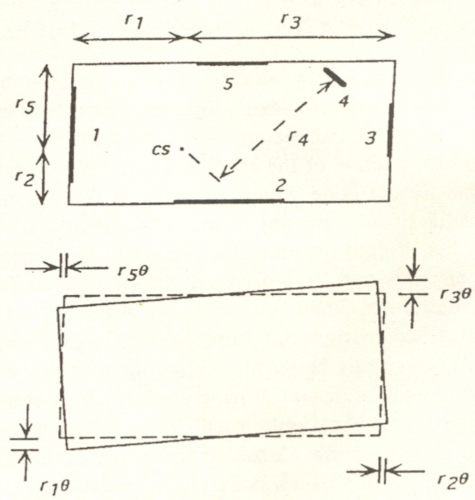
FIG.C9 LATERAL LOAD DISTRIBUTION DUE TO RIGID FLOOR DIAPHRAGM (SYMMETRICAL CASE – NO TORSION)

significant deformation in its own plane. Thus the floor distributes the lateral force to the frames/walls such that the displacement compatibility of no in-plane floor deformation is satisfied. This is termed as the ''rigid floor diaphragm'' action[17].

In symmetrical buildings (i.e., buildings with rigid floor diaphragm and with no torsional coupling), the floors move as a rigid body in the two principal horizontal directions. This means that the lateral load resisting elements share the seismic force in proportion to their lateral stiffness (Fig.C9). However, in case of torsional coupling, the floors undergo lateral translation and rotation, and the walls/frames share the load such that the displacement compatibility condition requiring no in-plane floor deformation is still satisfied (Fig. C10). It is obvious that the floor diaphragm action has a significant bearing on the load distribution, and that a three-dimensional finite element analysis which models only the beams and the columns and not the floor diaphragm action, nor representing floors by realistic elements may give erroneous load distribution to different vertical elements.

The in-plane rigidity of floors is sometimes misunderstood to mean that the columns are not free to rotate at their ends. From Fig. C11 it is obvious that the in-plane floor stiffness has no role to play with regard to column end condition which is governed by the beam and floor stiffness in a plane perpendicular to the plane of the floor.

C4.5.2.2 Numerous situations [12,13] occur when the floor diaphragm does not act as entirely rigid, due to:



k_i = Lateral stiffness of the i^{th} element
r_i = Perpendicular distance of the i^{th} element from centre of stiffness
θ = Rotation of the floor diaphragm in its own plane

Displacement of i^{th} element, in its own plane, due to rotation θ about centre of stiffness $\quad \Delta_i = r_i \theta$

Resisting force in i^{th} element $\quad F_i = k_i r_i \theta$

Restoring moment by force in i^{th} element $\quad M_i = f_i r_i = k_i r_i^2 \theta$

By moment equilibrium $\quad M_t = \theta \Sigma k_i r_i^2$

Force in the i^{th} element $\quad F_i = \dfrac{k_i r_i}{\Sigma k_i r_i^2} M_t$

FIG. C10 ANALYSIS OF FORCES INDUCED BY TWISTING MOMENT (RIGID FLOOR DIAPHRAGM)

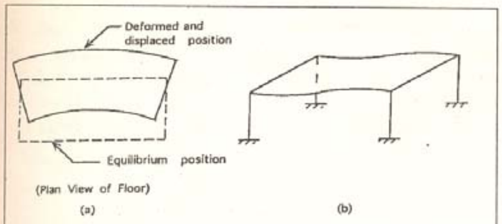


FIG. C11 (a) IN-PLANE FLOOR DEFORMATION, (b) OUT-OF-PLANE FLOOR DEFORMATION

a. Diaphragm is very flexible as compared to the vertical members. In this case, the vertical members act independent of each other. In such a case the load could be distributed by treating the vertical members as rigid; i.e., by analyzing the diaphragm as a continuous horizontal beam on rigid supports. However, the lateral shear in any vertical element should not be taken less than that based on "tributary areas".

b. Where the horizontal diaphragm is not continuous, the storey shear should be distributed to the vertical elements based on their tributary areas.

c. In situations where diaphragm is neither completely rigid nor very flexible, the load distribution should explicitly consider diaphragm deformations and satisfy equilibrium and compatibility conditions. Fig. C12 illustrates one such simple static procedure

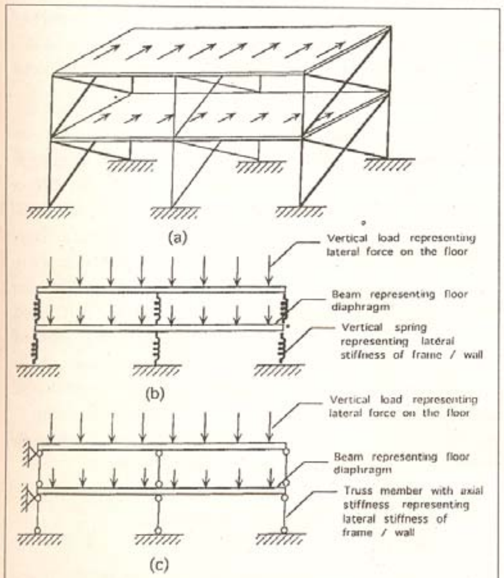
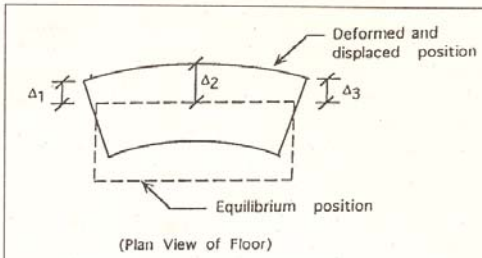


FIG. C12 LATERAL LOAD DISTRIBUTION CONSIDERING FLOOR DIAPHRAGM DEFORMATION : VERTICAL ANALOGY METHOD



In-plane flexibility of diaphragm to be considered when

$$\Delta_2 > 1.5\left[0.5\left(\Delta_1 + \Delta_2\right)\right]$$

FIG. C13 DEFINITION OF FLEXIBLE DIAPHRAGM (CLAUSE 4.5.2.2)

("vertical analogy method"[18]). Alternatively, the design forces should be the envelop of the two sets of forces resulting from the two extreme assumptions, i.e., rigid and very flexible floor diaphragm.

Clause 4.5.2.2 also provides for a test to decide when explicit consideration of floor flexibility is important (Fig. C13).

Dynamic Analysis(C4.6)

C4.6.1 The static procedure for calculating the overall seismic force on a building (clause 4.3.3) and distributing it to different floor levels (clause 4.5.1) is essentially based on the assumptions that (a) the fundamental mode of vibration has the most dominant contribution to seismic force, and (b) the mass and stiffness are evenly distributed in the building, thus giving a regular mode shape. However, in tall buildings the contribution of higher modes may be important, in irregular buildings the mode shape may not be regular, and in industrial buildings with large spans and heights both these conditions may not be valid. Hence, this clause recommends a dynamic analysis for such buildings.

C4.6.2 : As discussed earlier, a dynamic analysis may yield fundamental period which may be unusually high and that the empirical equations usually give a more reliable estimate of the fundamental period. Hence, this clause requires that if the dynamic analysis (either a time history or a response spectrum analysis, either based on code-specified design spectrum or based on site-specific studies) gives an overall design base shear which is less than what one obtains by using fundamental period (C_a T_a), then the response quantities are to be scaled up. The intention is to rely upon the dynamic analysis primarily for distribution of earthquake force with building height, and to different vertical elements of the building, and not for assessing the magnitude of design seismic forces.

C4.6.4.2: As against specifying the number of modes of vibration that should be considered in design, the draft code requires that as many modes be included in analysis as are necessary to ensure that at least 90% of the total seismic mass is excited by these modes. To illustrate this clause, consider a six-storey R.C. frame building with stiff shear walls in only the lower two storeys. In the first several modes of vibration, the mass at the first and the second floor will not be much excited. Hence, consideration of only the first three modes will underestimate the design force, since it will not account for the inertia force caused by the mass at the first two floors. Therefore, either more modes should be considered or a "missing mass correction" may be appropriate.

C4.6.4.3: In the response spectrum method of analysis, the response in different modes of vibration may be combined at different levels. For instance, one could calculate the final response quantities (member forces, deflections, displacements) by carrying out a static analysis due to imposed seismic force for each mode of vibration, and the response quantities for different modes could then be combined (as per 4.6.4.4). However, this requires a separate static analysis for each mode of vibration. Often, it may be more convenient to first obtain the design earthquake force at each floor level due to the effect of all modes being considered, and then carry out one analysis to get the member forces and deflections. In such as case, if one were to combine the earthquake force at a given floor due to different modes of vibration, the result is an overestimation in design earthquake force since the information about some forces in higher modes cancelling each other is not accounted. Hence, this clause requires that the overall design force be obtained through the storey shears; i.e., for a given storey combine the storey shear for different modes and then calculate backwards the external forces at the floor levels which give the same storey shears (Example 2).

C4.6.4.4: The response spectrum method of dynamic analysis gives the maximum response in different modes of vibration. However, the maximum response of different modes occur at different time instances. Hence, to obtain the overall maximum response due to the combined effect of all the modes, approximate procedures have been developed based on probability theory. The expression suggested in 4.6.4.4(a) is the well known "square root of the sum of the squares (SRSS)" method which is considered appropriate when natural periods of different modes are well separated. However, when the natural periods closely spaced, response due to such modes is to be combined by the absolute sum and this resultant is to be combined with the response of remaining well-separated modes. The condition of closely-spaced modes has been defined as those modes for which the natural period lies within 15% of each other.

C4.6.4.6: This clause allows a designer to use the "stick model" for analysis of buildings which are regular (or nominally irregular) in plan. In the stick model, the building is modelled as a number of lumped masses (each representing a floor) connected in series through springs with each spring representing the storey stiffness. In such a case, the building is analyzed by considering a separate stick model in each of the principal directions. Note that the accuracy of the stick model depends on how accurately the storey stiffness is modelled. The expressions given in 4.6.4.6(a) to (f) are the standard expressions that one obtains from dynamic analysis of a stick-type model.

Deformations (C4.7)

Storey Drift Limitation (C4.7.1): Storey drift is the maximum lateral displacement of one floor relative to the floor below caused by the seismic loads. Lateral displacement or deflection is the absolute displacement of any point of the structure with respect to its base; this is clearly different from storey drift. The draft code requires that in each of the storeys the storey drift caused by the design seismic force should not exceed 0.004 times the height of concerned storey; i.e., in no storey should the drift exceed this limit even though in all other storeys the drift is below this limit. Since reliance is placed on overstrength and ductility of the building in the event of strong shaking, the actual deformations during such a motion will be much larger than those calculated for design force; the value of 0.004 has been arrived at taking due consideration of this fact.

In calculation of drift, the stiffness contribution of non-structural elements and structural non-seismic elements (i.e., elements not designed to share the seismic loads) should not be included; this is because such elements cannot be relied upon to provide lateral stiffness at large displacements. In calculating lateral displacements, all possible flexibility contributions should be considered, e.g., effect of joint rotation (flexibility of beams), bending and axial deformations of columns, and shear and flexural deformations in shear walls.

The draft code puts an upper limit (clause 4.4.3) on the fundamental period to be used for calculating design seismic force or a lower limit (clause 4.6.2) on the design seismic force obtained by dynamic analysis. These conditions are applicable for calculating design force for evaluation of member forces. However, for drift calculations, the code relaxes these conditions. That is, one may satisfy the drift criteria for seismic forces obtained as per calculated value of the fundamental period, without restrictions of clauses 4.4.3 and 4.6.2. However, to do so, the conditions to be satisfied are that (a) the analysis model of the structure for calculation of drift is the same as that for determining T, and (b) in case of dynamic analysis, the design spectrum used is not lower than that specified in this code.

Deformation Compatibility of Non-Seismic Members(C4.7.2):

This clause is particularly important when not all the structural elements are expected to participate in seismic load resistance. Common examples are those of the flat-plate type building or a building of prefabricated elements where the seismic load is resisted by the shear walls. The flat-plate or the prefabricated frame system is used to carry the gravity loads, and have low stiffness in the lateral direction. Such a frame system, even though may not share any significant lateral loads, does undergo lateral deformations alongwith the shear walls. There is a distinct possibility that the moments and shears induced by the lateral deformations may make these frames lose their capacity to carry the gravity loads, leading to collapse. To safeguard against such damage, this clause requires that the gravity system be deformed in the lateral direction by an amount equal to R times the calculated deflections; under this deformed configuration (with the resulting moments and shears), the capacity of the frame to carry the gravity loads be examined and ensured. Since the deflections calculated are only for the design forces which are significantly reduced from the maximum expected, the actual deflections during the design shaking will be about R times those calculated.

Separation Between Adjacent Units (C4.7.3): There have been numerous situations of destructive hammering ("pounding") during strong shaking between the adjacent buildings, or adjacent units of the same building. This clause intends to prevent such a damage. The hammering effect is significantly more serious when the floors of one unit hit at the mid height of columns in the other unit. Hence, when the two units have floors at the same elevations this condition is relaxed by replacing R by $R/2$.

Torsion (C4.8) :

C4.8.1: The seismic force is caused by inertia of the building mass; hence, the resultant of seismic forces on any floor acts at centre of mass of that floor. If the building is not symmetrical about the two principal axes, the centre of mass does not coincide with centre of resistance. In this case, the lateral force at centre of mass, which can be thought of as lateral force at centre of stiffness plus a twisting moment (Fig. C14), causes torsion, i.e., it also tries to rotate the floors about the centre of stiffness. This induces additional lateral force on some elements and reduces the lateral force on other elements. The code requires that while the detrimental effect of torsion, i.e., increase in force, will be accounted for, the reduction in element force due to torsion will not be considered. Example 4 illustrates the calculation of location of stiffness and lateral load distribution in case of torsion.

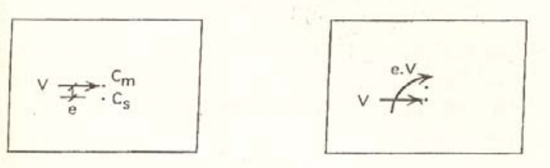


FIG. C14 SEISMIC FORCE ACTS AT CENTRE OF MASS; IT IS SAME AS A FORCE PLUS A TWISTING MOMENT AT CENTRE OF STIFFNESS

C4.8.2: Dynamic analysis shows that often the actual torque on the building exceeds the shear times the calculated eccentricity, i.e., the dynamic eccentricity is higher than the calculated eccentricity. This phenomenon is accounted for in the code by specifying that the design eccentricity to be used should be 1.5 times the calculated eccentricity (Example 4). When using computer programs which themselves calculate the centre of stiffness, this clause can be implemented by specifying the centre of mass at a distance of 0.5 times the approximately calculated eccentricity such that the eccentricity between the centre of stiffness and the point at which load is applied is 1.5 times the calculated eccentricity (Fig. C15). However, such programs do not usually have provision for not incorporating reduction in element force due to torsion; special care is required in this regard.

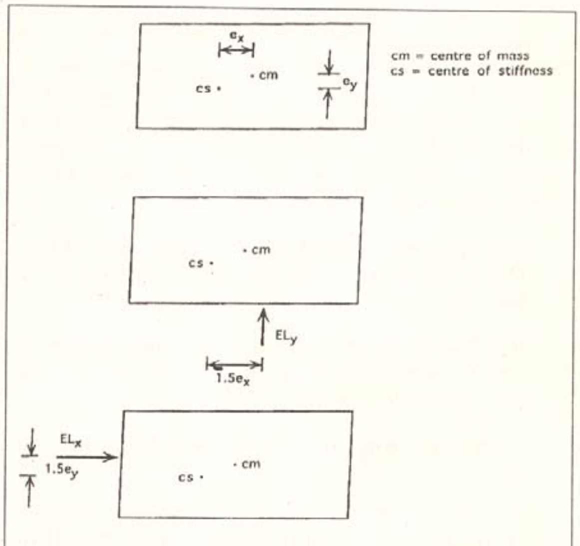


FIG. C15 REQUIREMENT ON DESIGN ECCENTRICITY CAN BE FULFILLED BY APPLYING EARTHQUAKE FORCE AWAY FROM CENTRE OF MASS AT A DISTANCE 0.5 TIMES THE CALCULATED ECCENTRICITY, SUCH THAT ECCENTRICITY BETWEEN CENTRE OF STIFFNESS AND THE LOAD IS 1.5 TIMES THE CALCULATED ECCENTRICITY.

C4.8.3: Since the calculation of the location of centre of mass and centre of stiffness, and therefore of eccentricity, is only approximate, the code requires that a minimum eccentricity of at least 5% of the base dimension perpendicular to the direction of applied force (Fig. C16) be considered to account for "accidental eccentricity".

Irregular Buildings (C4.9):

C4.9.1: As discussed earlier, the building configuration has a very significant effect on the seismic performance of a building. The seismic provisions are basically applicable to buildings having regular configuration. Buildings with irregular configurations are more prone to damage in

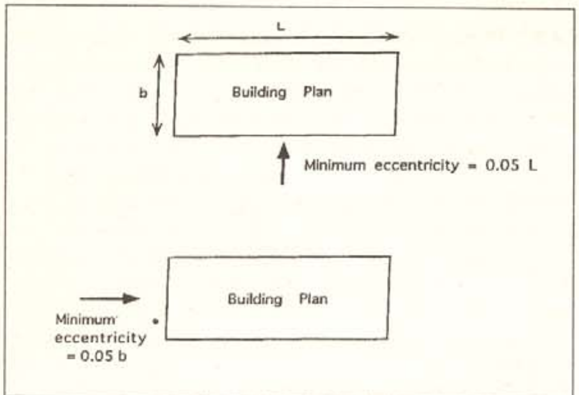


FIG. C16 MINIMUM ECCENTRICITY PROVISION OF THE CODE

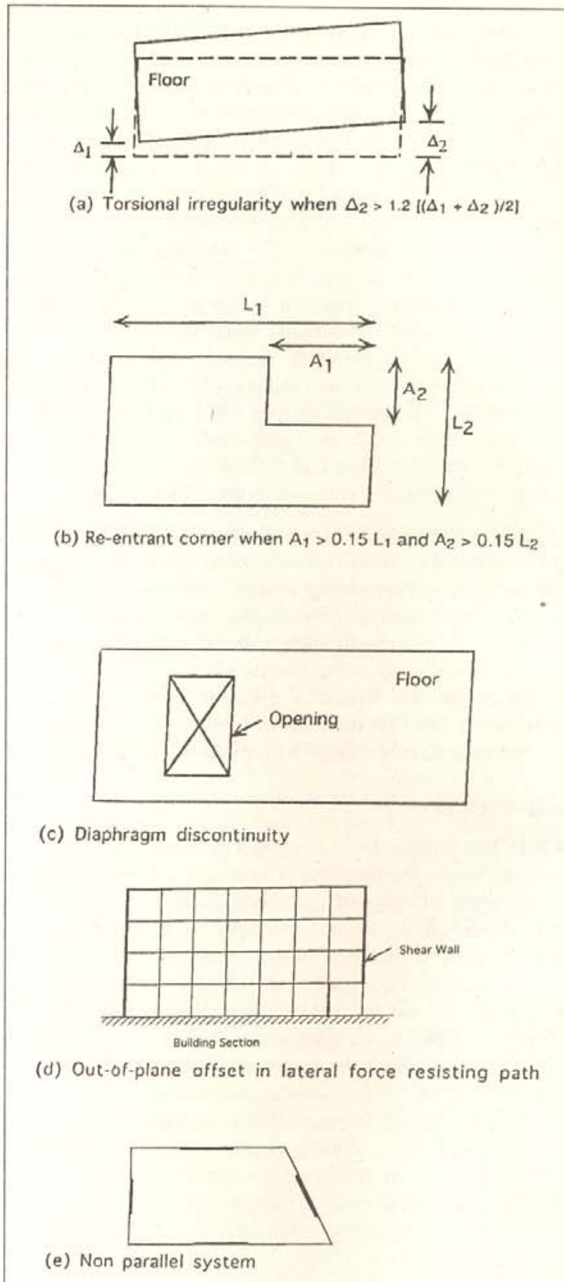


FIG. C17 PLAN IRREGULARITIES (TABLE 8)

case of strong earthquake shaking as compared to those with regular configuration, and of comparable quality of design and construction. Buildings with irregular features should be avoided as far as possible; however, when it is necessary special attention needs to be paid to the analysis and design of such buildings. The term "irregular buildings" is used when the usual codal procedures are no longer adequate; such buildings include:

a. Buildings with mass or stiffness properties not uniformly distributed in the plan and the elevation of the building,

b. Buildings in which there is a strong coupling between the lateral motions in the two translational and one rotational directions, and

c. Buildings with irregular distribution of lateral strength of the storeys.

This clause attempts to define "irregular buildings". Broadly, the irregularity can be of one or both of the two types: plan irregularity (Table 8) and vertical irregularity (Table 9). The conditions of irregularity are illustrated in Figs. C17 and C18.

The irregular configuration affects the seismic response in a number of ways:

a. In buildings with vertical irregularity, the load distribution with height cannot be approximated by simple expressions, given in (clause 4.5.1) the draft code; dynamic analysis is required to assess a reasonable load distribution in such buildings.

b. In buildings with plan irregularity, the load distribution to different vertical elements becomes complex and requires a three-dimensional dynamic analysis with due regard to member stiffness and the floor diaphragm action; such as analysis should have at least three degrees of freedom per floor - two translational and one rotational. One should also consider the fact that a given mode may be excited by both horizontal components of the ground motion, and that torsional modes may be excited by the translational component of ground motion.

c. In buildings with irregular configuration, there may be concentration of ductility demand at a few locations, i.e., some sections may require unusually large

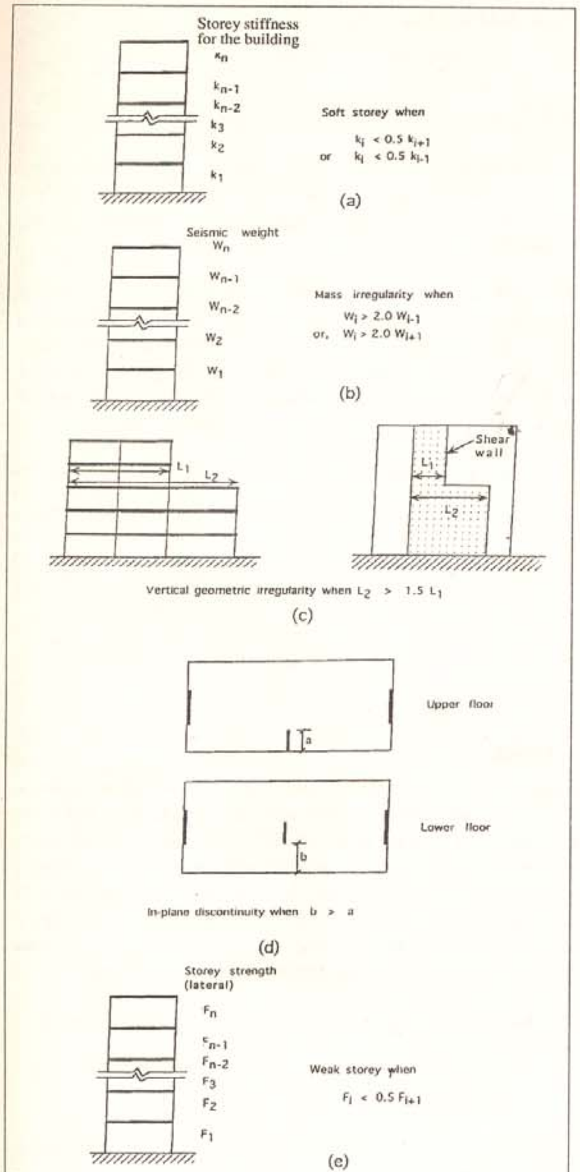


FIG. C18 VERTICAL IRREGULARITIES (TABLE 9)

ductility to enable the structure to possess a give amount of overall displacement ductility. Since such members may not have been detailed to provide this much section ductility, such buildings usually perform poorly as compared to regular-configuration buildings with same quality of design and construction. Hence, special attention is needed for detailing such buildings. The static or dynamic analysis will be unconservative if the lateral strength of building is distributed irregularly with respect to height; this gives rise to concentration of ductility demand in a few storeys of the building. In such a case, one should consider using a lower value of R[12,13].

EXAMPLES

Example 1: Consider a four-storey reinforced concrete office building shown in Fig. C19, located in Shillong (seismic zone V). The soil conditions are medium stiff and the entire building is supported on a raft foundation. The R.C. frames are infilled with brick masonry. The lumped weight due to dead loads is 12 kN/sq.m on floors and 10 kN/sq.m on the roof. The floors are to cater for a live load of 4 kN/sq.m roof and 1.5 kN/sq.m. Determine design seismic load on the structure by the equivalent static method.

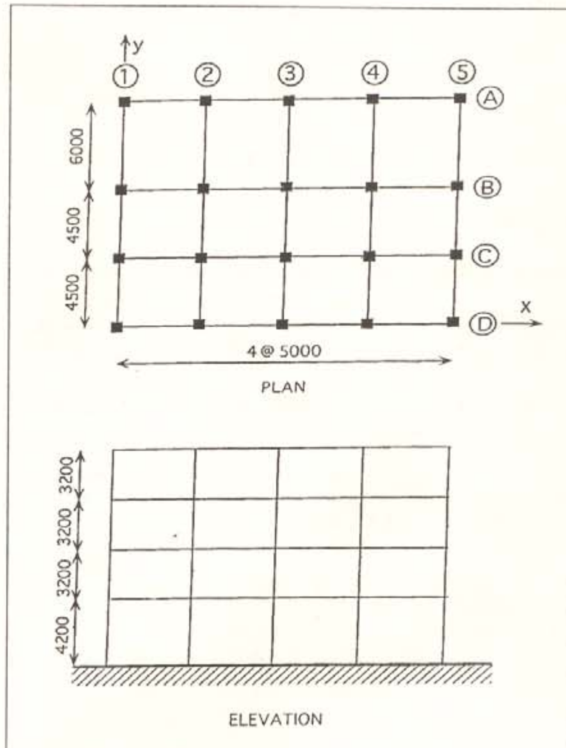


FIG. C19 BUILDING CONFIGURATION (EXAMPLE 1)

Solution:

Design Parameters: For seismic zone V, the zone factor Z is 0.50 (Table 2). Since the building is supported on medium stiff soil, soil profile factor S is 1.2 (Table 5). Being an office building, the importance factor, I, is 1.0 (Table 3). It is assumed that the building will be provided with moment resisting frames detailed as per IS:13920-1993 and hence the response reduction factor, R, is 10 (Table 6).

Seismic Weights: The floor area is 300 sq. m. Since the live load class is 400 kg/sq.m, only 50% of the live load is lumped at the floors (clause 4.1.1, Table 6). At roof, no live load is to be lumped. Hence, the total seismic weight on the floors and the roof is:

Floors: $W_1 = W_2 = W_3 = 4{,}200$ kN
Roof: $W_4 = 3{,}000$ kN

Total seismic weight of the structure, $W = \sum W_i = 15{,}600$ kN

Fundamental Period: Lateral load resistance is provided by moment resisting frames infilled with brick masonry panels. Hence, approximate fundamental natural period obtained by clause 4.4.2 is:

EL in *X*-Direction:

$T = 0.09\, h/\sqrt{d} = 0.28$ sec.
$C = 1/T^{2/3} = 2.35$

But, $C\,S = 2.35 \times 1.2 = 2.82 > 2.0$; Hence, $C\,S = 2.0$;

$$A = \frac{ZICS}{R} = 0.10 \text{ (clause 3.4.2)}$$

Design base shear $V_B = A\, W = 0.10 \times 15{,}600 = 1{,}560$ kN

Force Distribution with Building Height: The design base shear is to be distributed with height as per clause 4.5.1. Table C1 gives the calculations. Fig. C20(a) shows the design seismic force in *X*-direction for the entire building.

TABLE C1: LATERAL LOAD DISTRIBUTION WITH HEIGHT BY THE STATIC METHOD

Storey Level	W_i (kN)	h_i (m)	$W_i h_i^2$ (x 1000)	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Lateral Force at *i* th Level for *EL* in direction (kN)	
					X	*Y*
4	3,000	13.8	571.3	0.424	661	661
3	4,200	10.6	471.9	0.350	546	546
2	4,200	7.4	230.0	0.171	267	267
1	4,200	4.2	74.1	0.055	86	86
$\sum$			1,347.3	1.000	1,560	1,560

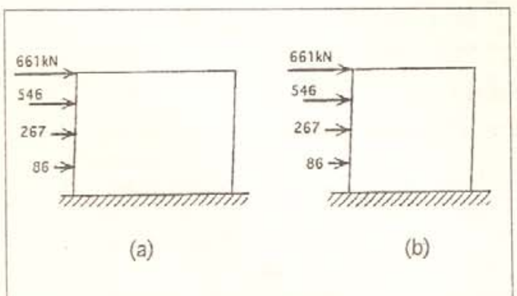


FIG. C20 DESIGN SEISMIC FORCE ON THE BUILDING FOR (a) *X*-DIRECTION, AND (b) *Y*-DIRECTION, BY STATIC PROCEDURE (EXAMPLE 1)

EL in *Y*-Direction:

$T = 0.09\, h/\sqrt{d} = 0.32$ sec.
$C = 1/T^{2/3} = 2.13$

But, $CS = 2.13 \times 1.2 = 2.56 > 2.0$; Hence, $C\,S = 2.0$

Therefore, for this building the design seismic force in *Y*-direction is same as that in the *X*-direction. Fig.C20(b) shows the design seismic force on the building in the *Y*-direction.

Example 2: For the building of Example 1, the dynamic properties (natural periods, and mode shapes) for vibration in the *X*-direction have been obtained by carrying out a free vibration analysis (Table C2). Obtain the design seismic force in the *X*-direction by the dynamic analysis method (modal analysis method) and distribute it with building height.

TABLE C2: FREE VIBRATION PROPERTIES OF THE BUILDING FOR VIBRATION IN THE *X*-DIRECTION

	Mode 1	Mode 2	Mode 3
Natural Period (sec)	0.860	0.265	0.145
	Mode Shape		
Roof	1.000	1.000	1.000
3rd Floor	0.904	0.216	-0.831
2nd Floor	0.716	-0.701	-0.574
1st Floor	0.441	-0.921	1.016

Solution:

Table C3 illustrates the calculation of modal mass (clause 4.6.4.6 a) and modal participation factor (clause 4.6.4.6 b). It is seen that the first mode excites 92.6% of the total mass. Hence, in this case, codal requirements on number of modes to be considered such that at least 90% of the total mass is excited, will be satisfied by considering the first mode of vibration only. However, for illustration, solution to this example considers the first three modes of vibration.

The lateral load Q_{ik} acting at i^{th} floor in the k^{th} mode is (clause 4.6.4.6 c)

$$Q_{ik} = A_k\, \phi_{ik}\, P_k\, W_i$$

The value of A for different modes is obtained from clause 3.4.2.

Mode 1

$T_1 = 0.860$ sec; $C = 1.106$; $CS = 1.106 \times 1.2 = 1.327 < 2.0$

$A_1 = 0.0796$
$Q_{i1} = 0.0796 \times 1.240 \times \phi_{i1}\ W_i$

Mode 2

$T_2 = 0.265$ sec; $C = 2.424$; $CS = 2.424 \times 1.2 > 2.0$;

Hence $CS = 2.0$
$A_2 = 0.10$
$Q_{i2} = 0.10 \times (-0.329) \times \phi_{i2}\ W_i$

TABLE C3 : CALCULATION OF MODAL MASS AND MODAL PARTICIPATION FACTOR										
Storey Level i	Weight W_i (kN)	Mode 1			Mode 2			Mode 3		
		ϕ_i	$W_i\,\phi_i$	$W_i\,\phi_i^2$	ϕ_i	$W_i\,\phi_i$	$W_i\,\phi_i^2$	ϕ_i	$W_i\,\phi_i$	$W_i\,\phi_i^2$
4	3,000	1.000	3,000	3,000	1.000	3,000	3,000	1.000	3,000	3,000
3	4,200	0.904	3,797	3,432	0.216	907	196	−0.831	−3,490	2,900
2	4,200	0.716	3,007	2,153	−0.701	−2,944	2,064	−0.574	−2,411	1,384
1	4,200	0.441	1,852	817	−0.921	−3,868	3,563	1.016	4,267	4,335
Σ	15,600		11,656	9,402		−2,905	8,822		1,366	11,620
$M_k = \dfrac{\left[\sum W_i\ \phi_{ik}\right]^2}{g\,\Sigma\,W_i\ \phi_{ik}^2}$		14,450 kN			957 kN			161 kN		
% of Total weight		92.6%			6.1%			1.0%		
$P_k = \dfrac{\sum W_i\ \phi_{ik}}{\Sigma\,W_i\ \phi_{ik}^2}$		1.240			−0.329			0.118		

Mode 3

$T_3 = 0.145$ sec; $C = 3.623$; $CS = 3.623 \times 1.2 > 2.0$;

Hence $CS = 2.0$

$A_3 = 0.10$

$Q_{i3} = 0.10 \times (0.118) \times \phi_{i3}\ W_i$

Table C4 summarises the calculation of lateral load at different floors in each mode. Since all of the modes are well-separated (clause 4.6.4.4 a), the contribution of different modes is combined by the SRSS (square root of the sum of the square) method (clause 4.6.4.6 *f*).

TABLE C4: LATERAL LOAD CALCULATION BY MODAL ANALYSIS METHOD (EARTHQUAKE IN X-DIRECTION)										
Floor Level i	Weight W_i (kN)	Mode 1			Mode 2			Mode 3		
		ϕ_{i1}	Q_{i1}	V_{i1}	ϕ_{i2}	Q_{i2}	V_{i2}	ϕ_{i3}	Q_{i3}	V_{i3}
4	3,000	1.000	296.1	296.1	1.000	- 98.7	-98.7	1.000	35.4	35.4
3	4,200	0.904	374.8	670.9	0.216	- 29.8	-128.5	-0.831	-41.2	-5.8
2	4,200	0.716	296.8	967.7	-0.701	96.9	-31.6	-0.574	-28.4	-34.2
1	4,200	0.441	182.8	1150.0	-0.921	127.3	95.7	1.016	50.4	16.2

$V_1 = 314.1$ kN

$V_2 = 683.1$ kN

$V_3 = 968.8$ kN

$V_4 = 1{,}154.6$ kN

The externally applied design loads are then obtained as (clause 4.6.4.6 f)

$Q_4 = V_4 = 314.1$ kN

$Q_4 = V_3 - V_4 = 369.0$ kN

$Q_2 = V_2 - V_3 = 285.7$ kN

$Q_1 = V_1 - V_2 = 185.8$ kN

Clause 4.6.2 requires that the base shear by dynamic analysis (= 1,154.6 kN) be compared with that obtained using fundamental period as $T_1 = C_a\,T_a$.

For earthquake in X- direction, $T_a = 0.28$ sec

For seismic Zone V, $C_a = 1.2$ (Table 7)

Hence,

$T_1 = 0.336$ sec; $C = 2.069$; $CS = 2.0$.

$A_1 = 0.10$

For modes 2 and 3, $A_2 = 0.10$ and $A_3 = 0.10$ as per earlier calculations. Base shear in the i^{th} mode is given by $(M_i)\ g\ (A_i)$

Mode 1: $V_{B1} =$ 1,445 kN

Mode 2: $V_{B2} =$ 96 kN

Mode 3: $V_{B3} =$ 16 kN

Overall base shear by SRSS = 1,448 kN

Thus, seismic force obtained by dynamic analysis earlier is to be scaled up to obtain the design forces. Hence,

$Q_4 = 394$ kN

$Q_3 = 463$ kN

$Q_2 = 358$ kN

$Q_1 = 233$ kN

Fig. C21 shows the design seismic force as per dynamic analysis. Fig. C22 shows a comparison of the results by the two methods. It is clear that the dynamic analysis significantly affects the load distribution with height; this is because the expression for load distribution with height by

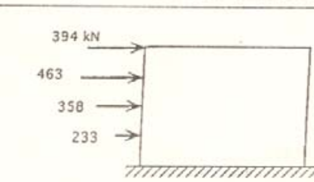


FIG. C21 DESIGN SEISMIC FORCE *X*-DIRECTION BY DYNAMIC ANALYSIS (EXAMPLE 2)

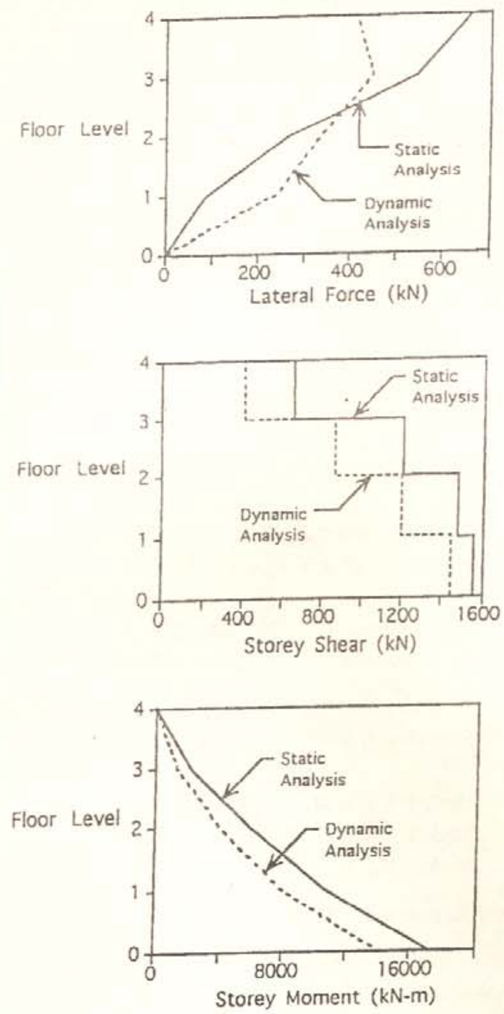


FIG. C22 COMPARISON OF DESIGN SEISMIC FORCE IN *X*-DIRECTION BY STATIC AND DYNAMIC PROCEDURES (EXAMPLES 1 AND 2)

the static method is quite conservative and leads to higher force at upper storeys and lower force at the lower storeys. Clause 4.6.2 protects the design base shear by ensuring that the large value of natural period from dynamic analysis would not lead to low design shear. In the absence of this protection, the dynamic method would give a design base shear of only 1,155 kN which is about 74% of that by the static method. The real advantage of dynamic analysis in this case is in a more realistic lateral load distribution with height which leads to reduced design storey moments and hence reduced axial force in the columns.

Example 3: For the first six modes of vibration, the maximum response (say base shear) and the natural periods, in appropriate units, are as follows. Estimate the maximum response quantity (base shear).

Mode:	1,	2,	3,	4,	5,	6
Natural periods:	0.94,	0.78,	0.74,	0.34,	0.26	0.25
Maximum response:	850	230	190	200	90	80

Solution:

In this case, the closely-spaced modes are: modes 2 and 3; and modes 5 and 6. Hence,

Combined response of modes 2 and 3	=	420
Combined response of modes 5 and 6	=	170
Combined response of all the modes by SRSS	=	984

Example 4 : The outer and inner beams in building of Example 1 (Fig. C19) are of different sizes; this results in different lateral stiffness of the interior and the exterior frames. The relative lateral stiffness of the frames is as follows:

Frames 1, 5	1.5 k
Frames 2, 3, 4	1.0 k
Frames A, D	2.0 k
Frames B, C	1.2 k

The floors are of cast-in-situ reinforced concrete and provide rigid floor diaphragm action. Distribute the design lateral force obtained in Example 1 to different frames.

Solution:

Design Eccentricity : The building is symmetrical for earthquake load (*EL*) in the *y*-direction, but asymmetric in the *x*-direction. Let the location of the centre of stiffness be (x', y'). Then,

$$x' = 10.0 \text{ m}$$

$$y' = 7.22 \text{ m}$$

Hence, calculated eccentricity:

For *EL* in *X*-direction, $e_y = 0.28$ m
For *EL* in *Y*-direction, $e_x = 0.0$ m

Design eccentricity = 1.5 times the calculated eccentricity (clause 4.8.2)

$$e_y = 0.42 \text{ m}$$
$$e_x = 0.0 \text{ m}$$

But, design eccentricity should not be less than 5% of the plan dimension of the building perpendicular to the direction of force under consideration (clause 4.8.3):

$$e_y = 0.75 \text{ m}$$
$$e_x = 1.00 \text{ m}$$

Hence, in this case the accidental eccentricity is more than 1.5 times the calculated eccentricity and hence governs the design. Note that the accidental eccentricity may be on either side of the centre of mass, i.e., we need to account for $e_y = \pm\ 0.75$ m, $e_x = \pm\ 1.0$ m.

The above implies that we need to assume the design lateral force acting at the calculated location of the centre of stiffness (10.0 m, 7.22m) plus a twisting moment equal to the design lateral force times the design eccentricity (Fig. C23).

Force Distribution due to Lateral Loads Applied at the Centre of Stiffness : Let us denote the design force in *X*-or in *Y*-direction by *F*. This force will be shared by frames in the concerned direction in proportion to their lateral stiffness as per Fig. C9.

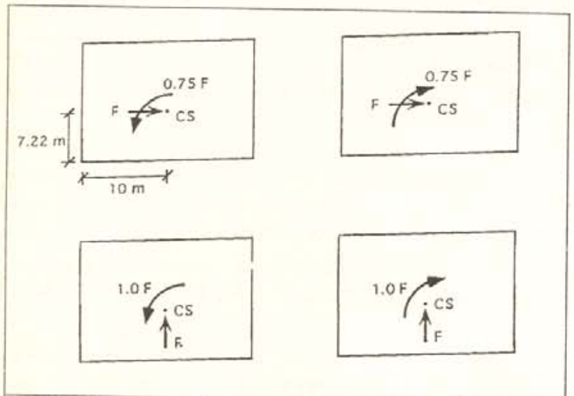


FIG. C23 BUILDING IS TO BE DESIGNED FOR DESIGN LATERAL LOAD PLUS TWISTING MOMENT ACTING AT CENTRE OF STIFFNESS (EXAMPLE 4)

Force in Frames A, D	=	0.31 F
Force in Frames B, C	=	0.19 F
Force in Frames 1, 5	=	0.25 F
Force in Frames 2, 3, 4	=	0.17 F

Force Distribution due to Torsional Moment: When the lateral force acts in the *X*-direction, torsional moment

$$M_t = F \times e_y = 0.75\,F$$

When the lateral force acts in the *Y*-direction, torsional moment

$$M_t = F \times e_x = 1.0\,F$$

All frames, irrespective of orientation, resist the twisting moment (as per Fig. C10). The calculations are shown in Table C5. The final design force for different frames are obtained by substituting the overall design force on the building (Fig. C20(a)) in place of *F*; these are shown in Fig. C24.

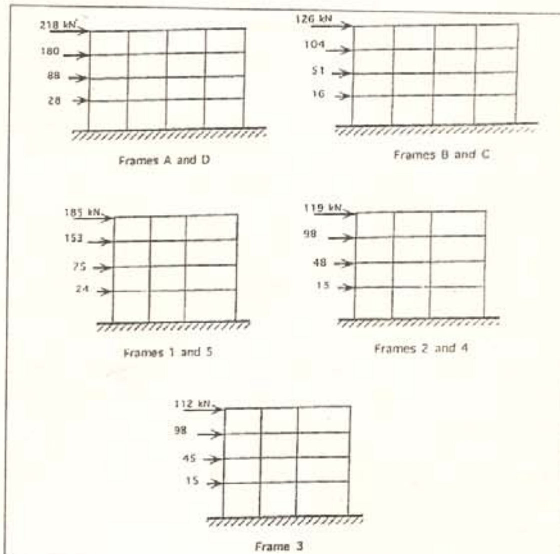


FIG. C24 DESIGN SEISMIC FORCE FOR DIFFERENT FRAMES (EXAMPLE 4)

DISCUSSION AND CONCLUSION

The proposed draft code includes significant improvements over the 1984 version. However, there are a number of areas where the code needs to be further improved; these include:

TABLE C5: LATERAL LOAD DISTRIBUTION DUE TO TORSION

Frame	k_i	r_i (m)	$k_i r_i$	$k_i r_i^2$	$\frac{k_i r_i}{\Sigma k_i r_i^2}$	Frame forces when *EL* is in *X*-direction			Frame forces when *EL* is in *Y*-direction		
						Due to torsion	Due to direct force	Total force	Due to torsion	Due to direct force	Total force
A	2.0	7.78	15.56	121.1	0.0265	0.020F	0.31F	0.33F	0.027F	-	0.03F
B	1.2	1.78	2.14	3.8	0.0036	0.003F	0.19F	0.19F	0.004F	-	0.00F
C	1.2	-2.72	-3.26	8.88	-0.0055	0.004F	0.19F	0.19F	0.006F	-	0.01F
D	2.0	-7.22	-14.44	104.3	-0.0246	0.018F	0.31F	0.33F	0.025F	-	0.03F
1	1.5	-10.0	-15.0	150.0	-0.0255	0.019F	-	0.02F	0.026F	0.25F	0.28F
2	1.0	-5.0	-5.0	25.0	-0.0085	0.006F	-	0.01F	0.008F	0.17F	0.18F
3	1.0	0.0	0.0	0.0	0.0	0.000F	-	0.00F	0.000F	0.17F	0.17F
4	1.0	5.0	5.0	25.0	0.0085	0.006F	-	0.01F	0.008F	0.17F	0.18F
5	1.5	10.0	15.0	150.0	0.0255	0.019F	-	0.02F	0.026F	0.25F	0.28F
Σ				588.0							

a) Seismic design provisions for architectural, mechanical, and electrical components in the building. These are integral part of a building, and damage to these may constitute a significant loss.

b) Seismic design provisions for different types of foundations for buildings. Foundations are indeed very important component of the building and need to be protected during strong ground shaking. Foundations require additional conservatism in design as compared to that for the superstructure because (i) the foundations support the entire superstructure and hence loss of foundation support can be disastrous, and (ii) the damage to foundations is difficult to inspect or repair after the earthquake events.

c) Requirements on explicit consideration of P-Δ effect for high-rise buildings.

d) Quality assurance in both design and construction. Eventually, in the event of earthquake the building behaviour depends on how it was actually built and not on what were the intentions in the design and construction. A very large percentage of failures during earthquakes are due to poor quality of construction. Hence, quality assurance is foremost amongst the most important elements in the seismic behaviour of buildings.

Many of the advanced codes do incorporate the above features (e.g., NEHRP[12, 13], UBC [19]); however, these cannot be adopted in the Indian practice without due consideration to the Indian design and construction practices. It is expected that provisions on these aspects will be developed, and that these features will be incorporated in future revision of the code.

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