EXPLANATORY
HANDBOOK
ON CODES FOR
EARTHQUAKE
ENGINEERING

IS:1893-1975 & IS:4326-1976

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( Incorporating errata and other editorial corrections )

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# FOREWORD

Users of various civil engineering codes have been feeling the need for explanatory handbooks and other compilations based on Indian Standards. The need has been further emphasized in view of the publication of the National Building Code of India 1970 and its implementation. In 1972, the Department of Science and Technology set up an Expert Group on Housing and Construction Technology under the Chairmanship of Maj-Gen Harkirat Singh. This Group carried out in-depth studies in various areas of civil engineering and construction practices. During the preparation of the Fifth Five-Year Plan in 1975, the Group was assigned the task of producing a Science and Technology plan for research, development and extension work in the sector of housing and construction technology. One of the items of this plan was the production of design handbooks, explanatory handbooks and design aids based on the National Building Code and various Indian Standards and other activities in the promotion of the National Building Code. The Expert Group gave high priority to this item and on the recommendation of the Department of Science and Technology the Planning Commission approved the following two projects which were assigned to the Indian Standards Institution.

a) Development programme on Code implementation for building and civil engineering construction, and

b) Typification for industrial buildings.

A special Committee for Implementation of Science and Technology Projects (SCIP) consisting of experts connected with different aspects (*see* page vi) was set up in 1974 to advise the ISI Directorate General in identification and for guiding the development of the work under the Chairmanship of Maj-Gen Harkirat Singh, Retired Engineer-in-Chief, Army Headquarters and formerly Adviser (Construction) Planning Commission, Government of India. The Committee has so far identified subjects for several explanatory handbooks/compilations covering appropriate Indian Standards/Codes/Specifications which include the following:

Design Aids for Reinforced Concrete to IS : 456-1978
Explanatory Handbook on Masonry Code
Explanatory Handbook on Codes for Earthquake Engineering (IS : 1893-1975, IS : 4326-1976)
Concrete Mixes
Summaries of Indian Standards for Building Materials
Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete (IS : 456-1978)
Causes and Prevention of Cracks in Buildings
Foundation of Buildings
Timber Engineering
Functional Requirements of Buildings
Functional Requirments of Industrial Buildings
Concrete Reinforcement
Building Construction Practices
Fire Safety
Tall Buildings
Bulk Storage Structures in Steel
Construction Safety Practices
Steel Code (IS : 800)
Form Work
Prefabrication
Loading Code
Design of Industrial Steel Structures
Inspection of Different Items of Building Work

One of the explanatory handbooks identified is on codes for earthquake engineering. This handbook is in two parts: Part I Explanations on IS : 1893-1975 Criteria

for Earthquake Resistant Design of Structures (*third revision*) and Part II Explanations on IS : 4326-1976 Code of Practice for Earthquake Resistant Design and Construction (*first revision*). This Handbook provides information on the source, interpretation/explanations to certain clauses and worked out examples to illustrate the application of codal provisions wherever required.

Some important points to be kept in view in the use of this Handbook are as follows:

a) In this Handbook wherever the expression "the Code" is used it refers to either IS : 1893-1975 or IS : 4326-1976 depending upon the part in which it is used.

b) This Handbook is to be read along with the relevant codes.

c) The clause numbers in the Explanatory Handbook correspond to the corresponding clause numbers in the relevant code. Only those clauses for which explanations are required find a mention in the Handbook in the same sequence as they occur in the respective codes.

d) Wherever there is any dispute about the interpretation or opinion expressed in this Handbook, the provisions of the code only shall apply; the provisions in this Handbook should be considered as only supplementary and informative.

The Explanatory Handbook is based on the draft prepared by Department of Earthquake Engineering, University of Roorkee, Roorkee. The draft Handbook was circulated for review to Central Public Works Department, New Delhi; India Meteorological Department, New Delhi; Engineer-in-Chief's Branch, Army Headquarters, New Delhi; Tata Consulting Engineers, Bombay; Dr Jai Krishna, Chairman of Earthquake Engineering Sectional Committee, BDC 39; International Airports Authority of India, New Delhi; Geological Survey of India, Calcutta; Central Water Commission, New Delhi and the views received were taken into consideration while finalizing the Handbook.

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# PART I

## EXPLANATIONS ON
## IS : 1893-1975 CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

### *(Third Revision)*

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# 2. TERMINOLOGY

**2.2 Centre of Rigidity** — The point through which the resultant of the restoring forces due to stiffnesses of the various structural elements of a system acts is called the centre of rigidity. In structures which are symmetrical in plan and elevation and having constant stiffness, centre of mass and centre of rigidity are coincident. However, when the structure is unsymmetrical, the centre of rigidity would be closer to the stiffer section of the structure, and the centre of mass could be away from the centre of rigidity causing torsion.

**2.3 Critical Damping** — Friction with air, friction between particles constituting a structure, friction at junctions of structural elements, yielding of the structural material and other processes of dissipation of energy depress the amplitude of motion of a vibrating structure and the vibrations die out in course of time. When such internal and or external friction fully dissipates the energy of the structural system during its motion from a displaced position to its initial position of rest, inhibiting oscillations of the structure, the structure is considered to be critically damped. Thus the damping beyond which the motion will not be oscillatory is described as 'critical damping'.

**2.4 Damping** — The effect of energy dissipation in reducing the successive amplitude of vibrations of a structure displaced from its position of static equilibrium is called damping and is expressed as a percentage of critical damping.

**2.5 Epicentre** — The point on the earth's surface located at the source or vertically above the source of such seismic waves originating from an earthquake is known as epicentre and its location is described by its latitude and longitude.

**2.6 Focus** — The source propagating seismic waves is called focus of the earthquake and is also designated as hypocentre. The depth of the source (focus) below the earth's surface is referred to as focal depth.

**2.7 Intensity of Earthquake** — The intensity of an earthquake at a place is a measure of the degree of shaking caused during the earthquake and thus characterises the effects of the earthquake. Most of the study of earthquakes up to the begining of the twentieth century dealt only with various effects of earthquakes and in order to express these effects in a quantitative way, intensity scales were introduced by various investigators. De Rossi in Italy proposed the first more commonly used intensity scale between 1874 and 1878. In 1881 Forel in Switzerland proposed a similar scale and soon thereafter they jointly developed the Rossi-Forel Scale. This Rossi-Forel intensity scale had ten subdivisions. This scale has undergone several revisions Mercalli in 1888 proposed a scale with twelve subdivisions to permit a clearer distinction

in shocks of extreme intensity or the great catastrophic degree X in Rossi-Forel Scale. In 1904 Cancani suggested that the degrees of the Mercalli scale be correlated with the maximum ground particle acceleration produced by the earthquake. An elaboration of the Mercalli scale incorporating earthquake effects of many kinds was published by Seiberg in 1923, which was subsequently revised by Wood and Newmann in the USA in 1931, and was called the Modified Mercalli Scale or simply the MM scale. Another revision of MM scale was made by Richter in 1956. An abridged version of MM scale of seismic intensities is given in Appendix D of the Code.

# 3. GENERAL PRINCIPLES AND DESIGN CRITERIA

**3.1** The various sub-clauses under this clause are self explanatory. However, for a few clauses additional clarifications are given below:

a) Ground vibrates (moves) in all directions during earthquakes. The horizontal components of the ground motion is generally more intense than that of vertical components during strong earthquakes. The ground motion is random in nature and generally the maximum peaks of various directions may not occur simultaneously. Hence, for design purposes, at any one time. it is assumed that only one horizontal component acts in any one direction.

b) All structures are designed for withstanding their own weight. This could be deemed, as though a vertical acceleration of one gravity is applied to various masses of the system. Since the design vertical forces proposed in the Code are small as compared to the acceleration of one gravity, the same emphasis has not been given to vertical forces as compared to horizontal forces. However, the Code emphasizes that in case of structures where stability is a criterion for design, vertical seismic forces must be considered.

**3.2 Assumptions** — Earthquake causes impulsive ground motion which is complex and irregular in character, changing in period and amplitude each lasting for small duration. Therefore, resonance of the type as visualized under steady state sinusoidal excitations will not occur as it would need time to build up such amplitudes.

a) The first assumption is amply proved by case studies of several strong motion accelerograms. For example, if the damping in an idealized linear single degree freedom system is 5 percent, then for any period the ratio of the peak response acceleration to the peak ground

acceleration is of the order of three, but in the case of steady state excitation this ratio would be ten. It is, therefore, obvious that full resonance is not achieved during earthquakes.

b) It is a fact that earthquakes are a rare phenomena. It is, therefore, very unlikely that the maximum earthquake will coincide with maximum of other occasional forces like wind, floods, etc. Therefore, for design purposes, these are assumed not to occur simultaneously.

## 3.3 Permissible Increase in Stresses and Load Factors

3.3.1 The Code specifies the use of elastic design (working stress method) permitting an increase of $33\frac{1}{3}$ percent in the normal working stresses in materials (concrete, steel, wood, etc.) when the effects of earthquake load are combined with other normal dead and live loads. In prestressed concrete members, the tensile stress in an extreme fibre of the concrete is permitted up to a maximum of $\frac{2}{3}$ of the modulus of rupture of concrete. It is restricted that the stress in steel with a definite yield point should remain below the yield stress, and in steel without a definite yield point, the stress should remain less than 80 percent of the ultimate strength. The increase in stresses is permitted in view of the occasional nature and instantaneous action of the load. Earthquake and wind effects are not to be considered simultaneously.

3.3.2 Since the increase of permissible stresses cuts into the factor of safety, it is natural that load factor in the ultimate load method of analysis should be reduced proportionately as compared with normal dead and live loads. Taking an average load factor of 1·85 for DL and LL for reinforced concrete (IS : 456-1964) and Steel (IS : 800-1962) structures, the load factor for earthquake condition was kept as $\frac{1\cdot85}{1\cdot33} = 1\cdot4$. Similarly taking average load factor of 2 for prestressed concrete (IS : 1343-1964) for DL and LL the load factor for earthquake condition was kept as 1·5.

The load factor (partial safety factor) for concrete structures in limit state method of design is to be taken as 1·2 when dead load, live load and earthquake load are combined in accordance with IS : 456-1978 and IS : 1343-1980.

Since earthquake occurs suddenly and without warning, it is very necessary to avoid construction practices that cause brittle failure (sudden collapse). The current philosophy relies heavily on the action of members to absorb the vibrational energy resulting from strong ground motion by designing members to behave in a ductile manner. In this manner even if earthquake of greater intensity than foreseen occurs, complete collapse of the structure will be avoided.

Ductile coefficients are given in IS : 4326-1976*.

3.3.3 *Permissible Increase in Allowable Bearing Pressure of Soils* — Similar to the increase in stresses in the materials of construction, the allowable bearing pressure in soils also has been increased whenever the earthquake forces are considered alongwith other normal design forces. However, the factor of safety against failure has been reduced for materials of construction; the same is not true for all soils under all conditions. This is because unlike other materials, the allowable bearing pressure of soils is obtained either by adopting a factor of safety against shear failure in the soil or by considering the permissible settlements of the foundations. If the allowable bearing pressure is governed by the former criterion, the permissible increase in its value can be looked upon as a permissible reduction in factor of safety. However, under most of the circumstances, the latter criterion, namely, settlement of foundation, will be the governing factor. Here, the earthquake induced settlement which evidently is a function of the soil-foundation system will govern the permissible increase in the allowable bearing pressure. Where the earthquakes are not expected to cause any significant settlement, it is imperative that the allowable bearing pressure be increased for earthquake loading conditions so as to avoid undesirable differential settlements which can take place prior to earthquake occurrence. This is illustrated below:

Since the earthquake resistant designs are generally performed by pseudo-static analysis, the earthquake loads on the foundations are considered as static loads and thus capable of producing settlements as under the dead loads. Therefore, as the footings are usually designed for equal stresses under them, the footings for exterior columns will have to be made wider. This is because the earthquake forces will cause larger stresses below the exterior columns (*see* Fig. 1). Prior to the earthquake, however, this design will lead to a condition of unequal stresses and hence larger settlements of the column foundations with heavier stress intensities. The differential settlements in the structure is thus increased and to avoid this, the allowable bearing pressure is increased. Consequently, this will necessitate only smaller amount of enlargement of foundations when earthquake forces are also included and will thus be causing only smaller amount of differential settlements. In poor soil foundation systems, ground shaking of even short duration can cause fairly large settlements and so any increase in allowable bearing pressure will lead to unsafe designs. In short, the permissible increase in the allowable bearing pressure will have to depend on the soil-foundation system. Where small settlements are likely to occur during earthquakes

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*Code of practice for earthquake resistant design and construction of buildings (*first revision*).
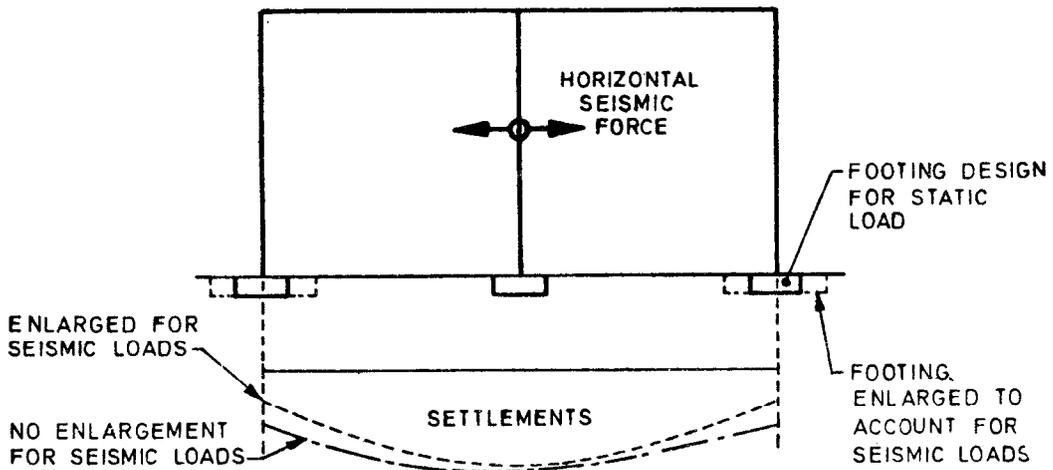
FIG. 1  EFFECT OF SEISMIC LOADS ON THE SETTLEMENT BEHAVIOUR OF A TYPICAL BUILDING

larger increase can be permitted and *vice-versa*. The quantity of increase in the allowable bearing pressure has been arrived at on the basis of experience and engineering judgement. For the present, the permissible increase has been based only on the soil-foundation systems though there appears to be some scope of identifying the influence of the superstructure as well. The present provisions and explanations to Table 1 of the Code are as below:

## PERMISSIBLE INCREASE IN ALLOWABLE BEARING PRESSURE OR RESISTANCE OF SOILS

| TYPE OF SOIL | TYPE OF FOUNDATION | PERMISSIBLE INCREASE IN ALLOWABLE BEARING PRESSURE, PERCENTAGE | EXPLANATIONS |
|---|---|---|---|
| (1) | (2) | (3) | (4) |
| *Type I Rock or Hard Soils* — Well graded gravels and sand gravel mixtures with or without clay binder and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW and SC)* having $N$ above 30, where $N$ is the standard penetration value | All types of foundations | 50 | Only small settlements of foundations are expected during earthquakes though for cohesionless soils short duration loads also can cause deformations, the soils are considered to be stiff enough so as to have sufficient factor of safety against failure under earthquake loading conditions |
| *Type II Medium Soil* — All soils with $N$ between 10 and 30 and poorly graded sands or gravelly sands with little or no fines (SP*) with $N > 15$ | Piles passing through this soil but resting on Type I soil | 50 | Since the piles will act as bearing piles on Type I, the possibility of large settlements is ruled out |
| | Piles not covered under the above | 25 | For friction piles the resisting forces are found to be less during vibration and hence larger settlements. Same will hold good even for piles with enlarged base contained wholly in soil of Type II |
| | Raft foundation | 50 | Differential settlements will be much less and hence larger increase permitted |
| | Other types of foundations | 25 | The soil itself being less dense than Type I, more settlement can be expected |

*See IS : 1498-1970.

(*Continued*)

PERMISSIBLE INCREASE IN ALLOWABLE BEARING PRESSURE OR RESISTANCE OF SOILS — *Contd*

| TYPE OF SOIL | TYPE OF FOUNDATION | PERMISSIBLE INCREASE IN ALLOWABLE BEARING PRESSURE, PERCENTAGE | EXPLANATIONS |
|---|---|---|---|
| (1) | (2) | (3) | (4) |
| *Type III Soft Soils* | Piles passing through this soil but resting on Type I soil | 50 | Same as for Type II |
| | Piles not covered under the above | 25 | Same as for Type II |
| | Raft foundations | 50 | Same as for Type II |
| | Combined or isolated RCC footing with tie beams | 25 | Because of the tie beams, the chances of damage due to differential settlements are considered smaller |
| | Isolated RCC footing without tie beams or unreinforced strip foundations | — | Under these conditions the foundations are considered to be liable to damage when subjected to settlements and hence no increase is permitted |
| | Well foundation | 25 | From the classification of soils it will be noticed that soils likely to liquefy have been excluded where liquefaction does not occur. The well foundations have been found to be satisfactory with little settlements and hence 25 percent increase has been permitted |

TABLE 1, NOTE 3 — It has been observed in many earthquakes that the foundation soil consisting of saturated sand behaved just like a fluid. The associated phenomenon has been termed as liquefaction; liquefaction of loose sand had been solely responsible for the damage to many structures during some of the past earthquakes like Bihar-Nepal 1934, Niigata (Japan) 1964, etc. The structures resting on such soils experienced large settlements and tilts. The soils lose shear strength due to earthquake pressure which is found to be dependent on the earthquake parameters, mainly acceleration, amplitude and duration of ground shaking, and the soil parameters, like the relative density and grain size. If this factor is not taken care of, any amount of safety in the superstructure will not be of any help in the event of an earthquake leading to liquefaction of foundation soil. Therefore, the zones where earthquakes large enough to cause liquefaction of soils falling under soil classification SP have been identified with standard penetration values (*see* Note 5 of Table 1 of the Code).

Methods are available at present to evaluate the liquefaction potential of soils based on the soil data and the design earthquake force for the site. Such procedures have been successfully used to analyse the occurrence of liquefaction in some of the past cases and are being increasingly used to predict the liquefaction potential of sites of important structures. If a site is found susceptible to liquefaction, preventive measures like densification or use of deep foundations to avoid damages during earthquakes are found suitable. If deep foundations are used, it must be borne in mind that it is not a preventive measure of liquefaction itself. Liquefaction usually initiates at some depth below the ground surface and propagates downwards to different depths depending upon the duration of ground shaking. The dissipation of the excess pore pressures also make the top soil to lose its strength. Thus the shear strength of the soil extending from the ground surface to some depth below will be totally lost during liquefaction and hence should not be considered to contribute any resistance to foundation displacements. The lateral resistance of the pile foundations must, therefore, be calculated taking this factor into account as specified in Note 4 of Table 1 of the Code which states 'The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy'. Some of the references for further study on the evaluation of liquefaction potential of soils are given below:

a) Seed (HB) and Idriss (IM). Simplified Procedure for Evaluating Liquefaction Potential. *Journal of the Soil Mechanics and Foundation Division, ASCE*, Vol 97, SM No. 9; P 1249-1973; (1971).

b) Gupta (M K). Liquefaction of Sands During Earthquakes (1977) Ph.D. Thesis, University of Roorkee, Roorkee, India.

c) Arya (A S), Nandakumaran (P), Puri (V K) and Mukerjee (S) 1978. Verification of Liquefaction Potential by Field Blast Tests. Proc. 2nd International Conference on Microzonation, Seattle, U. S. A. Vol II; P 865.

d) Nandakumaran (P) and Mukerjee (S). Evaluation of Liquefaction Potential for Silty Soils (1981). Proc. National Symposium on Earthquake Disaster Mitigation, University of Roorkee, Roorkee, India.

**3.4 Design Seismic Coefficient for Different Zones** — Background leading to the present seismic zoning map of India (*see* Fig. 1 of the Code) is described in Appendix A.

**Philosophy**—The force attracted by a structure during an earthquake is dynamic in nature and is a function of the ground motion and the properties of the structure itself. The dominant effect is equivalent to a horizontal force varying over the height of structure. Therefore, the assumption of a uniform force to be applied along one axis at a time is an over simplification which can be justified for reasons of saving efforts in dynamic analysis. However, a large number of structures designed on this basis have withstood earthquake shocks during the past eighty years or so, which justifies the use of uniform seismic coefficient in a seismic design. In the Code, therefore, it is considered adequate to provide uniform seismic coefficient for ordinary structures. Important and special structures, however, need to be designed on the basis of dynamic analysis.

Seismic coefficients specified in the Code are based on a compromise with regard to degree of desired safety and the cost of earthquake resistant construction. In an effort to economise, it is essential to fully utilise the total energy absorbing capacity of the structure without resulting in a complete collapse. This has resulted in the current philosophy of earthquake resistant design: (a) to adopt lower seismic coefficient and low working stresses, and (b) to have high seismic coefficient and high working stresses. The values of coefficients are fixed arbitrarily on safety and minimum damage criteria. In the Code the maximum value was fixed at 0.08 because the practice in Assam before the Code was originally written in 1960 was to design structures for this value arbitrarily. The structures thus designed withstood the 1950 Assam earthquake (Richter's Magnitude 8.3) which had caused *MM* intensity IX. With this background the basic seismic cofficient for Zone V has been fixed at 0.08. In other zones the values have been reduced as 0.05,

0.04, 0.02 and 0.01 for IV, III, II and I, respectively (*see* Appendix A).

The Code also provides for design to be carried out using response spectrum approach. For this purpose, it is recommended that average spectra be used together with different multiplying factors for each seismic zone. These factors have been determined in such a way that in the short period range (small structures like two or three storeyed buildings) the seismic coefficient derived from spectral considerations would be nearly the same as the basic seismic coefficients mentioned earlier. The *multiplying factors* are interpreted as seismic zone factors and are given as 0.4, 0.25, 0.20, 0.10 and 0.05 for Zone V, IV, III, II and I, respectively. It may be noted that these have the same relative values as the corresponding seismic coefficients.

**3.4.2** (a) — *Seismic Coefficient Method* — In this method, mass of the structure multiplied by design seismic coefficient, acts statically in a horizontal direction. It is also assumed here that the magnitude of the coefficient is uniform for the entire members of the structure. Design shears at different levels in a building shall be computed from the assumption of linear distribution horizontal accelerations, varying from zero at the base of the structure to a maximum at the top. For important and complicated structures this method is not adequate (*see* **4.2** and **5.1.2** of the Code).

b) *Response Spectrum Method* — It is a dynamic method of analysis. In the calculation of structural response (whether modal analysis or otherwise), the structure should be so represented by means of an analytical or computational model that reasonable and rational results can be obtained by its behaviour. Where response spectrum method is used with modal analysis procedure, at least 3 modes of response of the structure should be considered except in those cases where it can be shown qualitatively that either third mode or the second mode produces negligible response. When appropriate, the model maxima should be combined using the square root of the sum of the squares of the individual model values. In this method the building is considered as a flexible structure with lumped masses concentrated at floor levels, with each mass having one degree of freedom that of lateral displacement in the direction under consideration.

**3.4.2.2** In both the above methods, importance of the structure and its soil foundation system shall be considered and also the increase in bearing stress of the foundation soil shall be checked according to Table 1 of the Code.

**3.4.2.3** The value of $\beta$ (coefficient depending on soil-foundation system) shall be obtained from Table 3 of the Code. The value of *I* (coefficient depending upon the importance of the structure) shall be based on Table 4 of the Code.

The value of $\alpha_0$ (basic horizontal seismic coefficient) and $F_0$ (seismic zone factor for average acceleration spectra) shall be obtained from Table 2 of the Code.

$\dfrac{S_a}{g}$ (average acceleration coefficient) shall be read from Fig. 2 of the Code for appropriate natural period of time (see Note below **4.2.1.1** of the Code) and damping of the structure. The damping to be adopted for different types of structures are given in Appendix *F* of the Code. The method of using the spectra for calculating the horizontal seismic force is also indicated in Appendix *F* of the Code.

Natural period $(T)$ of any structure is a function of the structural characteristics and the distribution of the structural masses. It is the time taken by the system in completing one cycle of vibration. In calculating the natural period of vibrations, it is assumed that the structure is fixed at the base. The values given in **4.2.1.1** of the Code may be adopted when exact analysis is not required.

**3.4.3** *Soil-Foundation Factor* — To take into account the soil-foundation systems on which the structure is founded, a factor $\beta$ for various cases is given in Table 3 of the Code.

The effect of the soil-foundation system on the earthquake effect on structures is two-fold: (a) the interaction between the soil foundation system and the super-structure, and (b) the behaviour of the foundation itself under the induced load. It is essentially the latter which is taken care of by the factor specified in the Code. In other words, since the effect of the soil-foundation system on the natural period of the structure (determined on the assumption of infinitely rigid foundation) is to elongate the natural periods, the use of spectra given in Fig. 2 of the Code will show smaller values of $\dfrac{S_a}{g}$ for larger flexibility of the soil foundation system. The effect of flexibility of the soil alone (indicated by hardness and softness of the soil) may, to some extent, be taken care of if soil-dependent spectra were to be used. However, the effect of foundation type on the soil structure interaction can be accounted for, only if the structure is modelled properly and a dynamic analysis is carried out. Because of the large number of variables involved, it would not be possible nor would it be rational to specify coefficients to account for this effect without specifying the type of superstructure as well. Moreover, as already stated, the natural periods of structures are ordinarily computed on the basis of fixed base and since the flexibility of the foundations elongate the periods the spectrum method as given in this Code gives conservative values of seismic coefficient if soil-structure interaction effects are neglected.

The effects of the earthquake ground motion on the damage that can be caused to the structure are indeed dependent on the soil at site as well as the type of foundation. This is obvious because of the fact that though the softness of the soil foundation system will cause only smaller forces to be transmitted through it to the super-structure, the strains in the sub-structure will themselves be quite large enough thereby causing excessive and sometimes objectionable deformations of the super-structure. As can be readily seen, the type of soil, has the greatest influence in this regard; the poorer the soil, larger the chances of damage. A lot of this possible damage can be avoided by engineering better type of foundations, to judiciously transmit the loads to the subsoil.

Table 3 of the Code is self explanatory as far as the type of soil and different types of foundations are concerned, as already explained under **3.4.** Because of the uniform loading on the foundation soil and the associated settlements, the value of $\beta$ is taken as equal to 1·0 for dams.

As can be seen, isolated RCC footings without tie beams of unreinforced strip foundations and well foundations have been considered most vulnerable while in soft soils, only raft foundations, due to its possibility of reducing differential settlements have been considered effective.

It is observed by past experience that foundation of a building should act in an integral manner if damage is to be the minimum. In this context it is recommended that foundation units be tied together.

**3.4.4** Additional factor of safety is required to be provided against earthquake damage for structures whose functioning is of special importance after an earthquake, such as hospitals. And also for structures whose damage is catastrophic to life and property, such as atomic power reactors and dams. The same has been identified and given in Table 4 of the Code. As per the note the importance factors given are for guidance and it is to be based on judgement in every particular case.

**3.4.5** Since the ground moves in all directions in an earthquake and even tilts and rotates; consideration of the combined effect of all these motions must be included in the design of important structures. In most cases, only lateral forces are created by earthquakes, but in actual fact large vertical accelerations can also occur, particularly in epicentral regions. The same must be taken into account particularly where stability is a criterion for design. The vertical seismic coefficient (or the average acceleration coefficient in response spectrum approach) is recommended to be taken as half of horizontal coefficient although it varies considerably with distance from epicentre. It may be about 0·25 to 0·75 times the horizontal component, the higher values being at places close to the epicentre. In the Code therefore, an average value of 0·50 is recommended.

# 4. BUILDINGS

## 4.1 Design Live Loads

**4.1.1** The weight at any floor level of a building would equal the dead load plus live load present. In case of live load, only a fraction of value normally taken for static design is recommended by the Code. This is in view of the probability of its presence during the time of earthquake and also because the live loads provided in the relevant code included impact effect of live load which does not possess mass. For load class 200, 250 and 300 category buildings, only 25 percent of normal design live loads are recommended while for heavier category that is, 400 and above class the recommended values are 50 percent because mostly office buildings and other public buildings fall in this category where quite a good percentage of live load is always present. The Code correctly recommends that if live load at the time of earthquake can be assessed, the same may be used in the seismic design. However the value of design live load assumed shall not be less than the values specified in the Code.

It is to be noted that the same fraction of live loads mentioned above shall also be used for computing stresses due to vertical loads for combining with those due to earthquake forces. The Code recommends that under this condition, the entire building frame may be assumed loaded with the above fraction of live load except the roof.

## 4.2 Design Criteria for Multistoreyed Buildings —
It is recognized that dynamic forces on multi-storeyed buildings are best computed through a detailed vibration analysis. This, however, is a costly preposition for certain buildings and, therefore, it is recommended that detailed dynamic analysis or modal analysis or pseudo static analysis should be carried out depending on the importance of the problem. With this background, it is essential to make detailed dynamic analysis for buildings taller than 90 m in zones III, IV and V while modal analysis is recommended for such buildings in Zones I and II. Buildings having heights between 40 m and 90 m in Zones IV and V must be analysed by modal method while either modal method or pseudo static method is recommended for Zones I to III. Buildings having height less than 40 m may be analysed by pseudo static method.

*Pseudo Static Method*

In all the methods of analysing multistorey buildings recommended in the Code, the structure is treated as discrete system having concentrated masses at floor levels which include half that of columns and walls above and below the floor. In addition, the appropriate amount of live load at this floor is also lumped with it. It is also assumed that the structure is flexible and will deflect with respect to the position of the founda-

tion. The lumped mass system reduces to the solution of a system of second-order differential equation. These equations are formed by distribution of mass and stiffness in a structure, together with its damping characteristics and the dynamic characteristics of the ground motion. In this method, which is also referred to as seismic coefficient method, the design base shear is worked out by the equation given in the Code,

$$V_B = C\alpha_h W \qquad \qquad ...(1)$$

This method though called pseudo static method, does take into account the fact that with increase in period ($T$) of a building the seismic shear must reduce. Factor $C$ has a value $1.0$ up to period equal to about $0.35$ seconds and reduces to about $0.2$ at period of $3.0$ seconds. The method, therefor, requires an estimate of period ($T$) of the building to choose the value of $C$ in equation (1). For this purpose the Code provides use of two empirical formulae*.

In case, the designer is able to get better estimates of $T$, that is, either experimentally or otherwise, the same may be used to obtain the value of $C$ above.

*Distribution of Seismic Force along Height of Building*

Dynamic analysis of buildings has indicated that the seismic forces increase from zero at base to maximum at the top. One type of distribution of this force is an inverted triangle which is used by many designers. This is suitable only for structures in which mass and stiffness in each storey is equal, but since it is usually not so the distribution suggested in the Code gives parabolic distribution of seismic forces such that the seismic shears are higher near top storeys for the same base shear. The distribution of forces along with the height of the building is given by the formula given in the Code,

$$Q = V_B \frac{W_i h_i^2}{\sum\limits_{i=1}^{i=n} W_i h_i^2} \qquad ...(2)$$

The Code restricts the use of pseudo static method to ordinary or normal structures/buildings and excludes all special layouts like Plaza type building or building with flexible first storey or building on hill slopes (see Fig. 4 of the Code). For such buildings, modal method of analysis is recommended.

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*A. W. Anderson, J. A. Blume, H. J. Degenkolb, H. B. Jammill, E. M. Knapik, H. L. Marchand, H. C. Powers, J. E. Rinne, G. A Sedgnick, and H O. Sioberg. Lateral Forces of Earthquake and Wind. *Trans. ASCE*, Vol. 117; P 716–780 (1952).
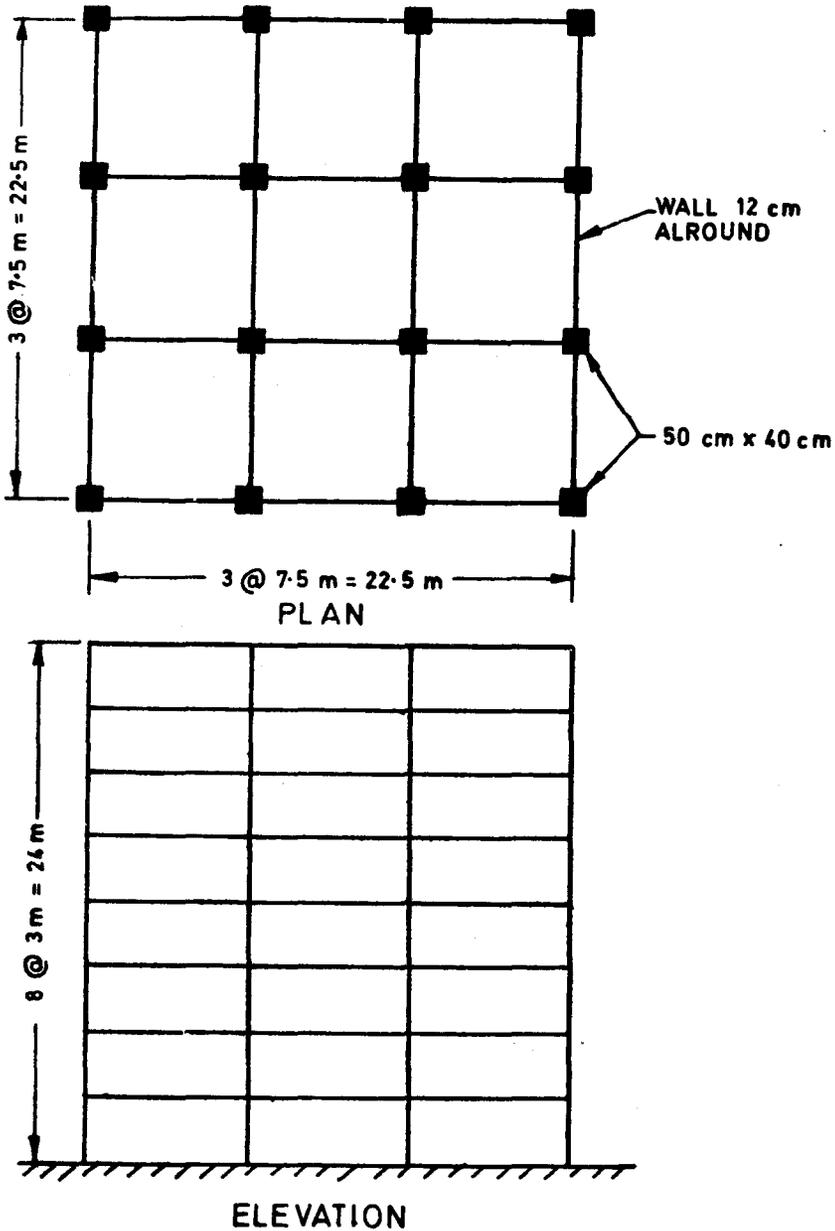
FIG. 2

In calculations of building as regards the influence of seismic forces. they will be considered as static. The static parameters (bending moments, shear and normal forces, moments due to torsion), which are the result of their influence on the building, will be obtained by the usual static methods.

*Example 1*

An eight storeyed RCC framed building with live load of 300 kg/m² (*see* Fig. 2) is to be constructed in Agra (seismic zone III). Work out seismic forces on the structure. All beams and columns may be assumed to be of 25 × 40 cm and 40 × 50 cm respectively. The roof and floor slabs may be assumed as 15 cm thick. The wall alround is 12 cm thick.

a) Dead weights

Weight of beams

$$= 24 \times 7.5 \times 0.4 \times 0.25 \times 2.4$$
$$= 43.2 \text{ t}$$

Weight of columns

$$= 16 \times 3 \times 0.4 \times 0.5 \times 2.4$$
$$= 23.04 \text{ t}$$

Weight of slab

$$= 22 \cdot 5 \times 22 \cdot 5 \times 0 \cdot 15 \times 2 \cdot 4$$
$$= 182 \cdot 25 \text{ t}$$

Weight of walls

$$= 22 \cdot 5 \times 4 \times 3 \times 0 \cdot 12 \times 2 \cdot 0$$
$$= 64 \cdot 8 \text{ t}$$

b) Live load at all floors except roof floor

$$= 22 \cdot 5 \times 22 \cdot 5 \times 0 \cdot 3 \times 0 \cdot 25$$
$$= 37 \cdot 97 \text{ t}$$

c) Lumped mass at floor level 1

$$= W_1 = 43 \cdot 2 + 23 \cdot 04 + 182 \cdot 25$$
$$+ 64 \cdot 8 + 37 \cdot 97 = 351 \cdot 26 \text{ t}$$

Similarly

$$W_1 = W_2 = W_3 = W_4 = W_5$$
$$= W_6 = W_7 = 351 \cdot 26 \text{ t}$$

Lumped mass at roof floor,

$$W_8 = 313 \cdot 29 \text{ t}$$

d) Base shear, $V_B = C \alpha_h W$

$W =$ Total gravity load of the building
$$= 2 \ 772 \cdot 11 \text{ t}$$

The building is without bracing or shear walls.

Therefore, $T = 0 \cdot 1 \ n = 0 \cdot 1 \times 8$
$$= 0 \cdot 8 \text{ seconds}$$

Design seismic coefficient $\alpha_h = \beta I \alpha_0$. For the present case it is assumed that foundation is of pile foundation resting on hard soil which would give $\beta = 1 \cdot 0$; importance factor will be taken as $1 \cdot 0$ (since it is an ordinary office building) and $\alpha_0 = 0 \cdot 04$.

$$\therefore \ \alpha_h = 1 \times 1 \times 0 \cdot 04 = 0 \cdot 04$$

$C$ ( from Fig. 3 of the Code ) $= 0 \cdot 62$

$$V_B = C \ \alpha_h \sum_{i=1}^{i=8} W$$

$$= 0 \cdot 62 \times 0 \cdot 04 \times 2 \ 772 \cdot 11 = 68 \cdot 75 \text{ t}$$

e) Distribution of lateral seismic shear force induced along the height of the building is given by the formula,

$$Q_i = V_B \frac{W_i h_i^2}{\sum\limits_{1=1}^{i=n} W_i h_i^2}$$

in which $h_i$ is the height of $i$th floor measured from the base of the building.

$$\therefore \ h_1 = 3 \text{ m}, \ h_2 = 6 \text{ m}, \ h_3 = 9 \text{ m}, \ h_4 = 12 \text{ m},$$
$$h_5 = 15 \text{ m}, \ h_6 = 18 \text{ m}, \ h_7 = 21 \text{ m and}$$
$$h_8 = 24 \text{ m}.$$

With these forces $Q_1$ to $Q_8$ are worked out and shear forces in the various storeys are also computed as shown in Table 1.

TABLE 1 NODAL FORCES AND SEISMIC SHEAR FORCES AT VARIOUS LEVELS

| Floor | $W_i$ | $h_i$ | $W_i h_i^2$ | $Q_i$ | $V_i$ (Shear Force) |
|---|---|---|---|---|---|
| | (t) | (m) | | (t) | (t) |
| 1 | 351·26 | 3 | 3 161·34 | 0·35 | 68·75 |
| 2 | do | 6 | 12 645·36 | 1·39 | 68·40 |
| 3 | do | 9 | 28 452·06 | 3·14 | 67·01 |
| 4 | do | 12 | 50 581·44 | 5·58 | 63·87 |
| 5 | do | 15 | 79 033·50 | 8·72 | 58·29 |
| 6 | do | 18 | 113 808·24 | 12·56 | 49·57 |
| 7 | do | 21 | 154 905·66 | 17·09 | 37·01 |
| 8 | 313·29 | 24 | 180 455·04 | 19·91 | 19·91 |
| | | | 623 042·64 | | |

The seismic shear force for which the building is to be designed is indicated in the last column of Table 1.

**4.2.2** *Modal Analysis* — This method of analysis is based on the dynamic response of the building idealized as having a lumped mass and stiffnesses in various storeys. It is shown that response of a $N$-degree freedom system (or $N$-storeyed building) can be computed by using the normal mode theory in which the system can be considered as if made of $N$ single degrees of freedom systems whose response is superimposed. For this purpose, the first three modes of the building response are considered to be adequate [*see* commentary on 3.4.2 (b)]. Response in each mode is determined first by using the following relationship:

$$Q_i^{(r)} = W_i \phi_i^{(r)} C_r \alpha_h^{(r)} \qquad \ldots(3)$$

in which $Q_i^{(r)} =$ Force at $i$th floor level when vibrating in its $r$th mode.

$W_i =$ dead load + appropriate percentage of live load at level $h_i$ measured above the ground level,

$\phi_i^{(r)} =$ mode shape coefficient for $i$th floor level when vibrating in $r$th mode, and

$C_r =$ mode participation factor for different floor levels when vibrating in the $r$th mode.

$$C_r = \frac{\sum\limits_{j=1}^{n} W_i \phi_i^{(r)}}{\sum\limits_{j=1}^{n} W_i [\phi_i^{(r)}]^2}$$

where $n$ is total number of storeys in the building

$\alpha_h^{(r)}$ = Design seismic coefficient corresponding to $r$th mode given by,

$$\alpha_h^{(r)} = \beta I \frac{S_a^{(r)}}{g} F_0$$

where $\beta$ and $I$ are soil-foundation factor and importance factor, $\dfrac{S_a^{(r)}}{g}$ is the average acceleration coefficient in the $r$th mode (from Fig. 2 of Code) and $F_0$ is the seismic zone factor.

After obtaining $Q_i^{(r)}$ values, shear forces various storeys are obtained by summing up $Q_i$ values from top and up to the storey in question. These are termed as $V_i^{(r)}$ values, which must be computed for at least the first three modes. The combination or superimposition of various modal values of force is then carried out using the following law:

$$V_i = (1 - \gamma) \Sigma V_i^{(r)} + \gamma \sqrt{\Sigma \{V_i^{(r)}\}^2} \qquad \ldots(4)$$

where $\gamma$ is a function of the height of building as defined in the code; $V_i^{(r)}$ = absolute value of maximum shear at the $i$th storey in the $r$th mode.

*Determination of mode shape coefficient ($\phi_i^{(r)}$*

*values*) — The Code has simply stated that $\phi_i^{(r)}$ values are obtained from free vibration analysis, but has not suggested any method of doing so because such methods are given in detail in books on theory of vibration. However, a brief description of the formulation of problem is included in the following paragraphs:

A popular method for the determination of the fundamental mode is the iterative Stodola method, as this method is usually formulated using matrix notations, it is also called matrix iteration. The equation of motion for a freely vibrating motion of a multi-storeyed lumped mass (undamped) can be written as:

$$[M] [\ddot{X}] + [K] [X] = 0 \qquad \ldots(a)$$

in which $M$ is the diagonal mass matrix, $K$ the stiffness matrix in relation to lateral displacements and, $\ddot{X}$ and $X$ are displacement vector corresponding to storey displacements and acceleration vector corresponding to storey acclerations matrices, respectively. Assuming the free vibration is simple harmonic,

$$[X] = [\phi] \sin pt \qquad \ldots(b)$$

$\phi$ represents the shape of vibrating system which does not change with time '$t$' but varies only with amplitude, $p$ represents circular frequency of the system.

Equation (a) can be rewritten as,

$$-p^2 [M] [\phi] + [K] [\phi] = 0 \qquad \ldots(c)$$

or $\qquad p^2 [M] [\phi] = [K] [\phi] \qquad \ldots(d)$

Premultiplying by $K^{-1}$ on both sides, one obtains

$$[K]^{-1} [M] [\phi] = \frac{1}{p^2} [\phi] \qquad \ldots(e)$$

putting $K^{-1}$ as $G$ (or flexibility matrix), Equation (e) assumes the following form,

$$[G] [M] [\phi] = \frac{1}{p^2} [\phi] \qquad \ldots(f)$$

Equation (f) is of the form

$$AX = \lambda X$$

which represents an eigen value problem whose solution leads to evaluation of natural frequencies and corresponding mode shapes. The methods of solution are generally iterative and are easily amenable to computer programming*. To start with, trial shape $\phi(o)$ shall be assumed. If the assumed shape were a true mode shape $\phi(1)$, then the same frequency would be obtained by taking the coordinate of the structure. However, the desired shape will differ from $\phi(o)$, and a different result will be obtained for each displacement coordinate. Because of this reason a better approximation of frequency can be obtained by an averaging process. The best averaging procedure consists of including the mass distribution a weighting factor. Knowing $p$, the fundamental period for mode one may be computed as:

$$T_1 = \frac{2\pi}{p_1}$$

*Example 2*

Analyse a 15-storeyed RC building as shown in Fig. 3. The live load on all the floors is 200 kg/m² and soil below the building is hard. The site lies in Zone $V$. All the beams are of size 40 × 50 cm and slabs are 15 cm thick. The sizes of columns are 60 × 60 cm in all the storeys and the wall alround is 12 cm thick.

*Analysis of the Building*

a) Calculation of dead load, live load and storey stiffnesses: As in case of seismic coefficient method, dead loads and live loads at each floor are computed and lumped. Stiffness in a storey is lumped assuming all the columns to be acting in parallel with each column contributing stiffness corresponding to $K_c = 12EI/L^3$, where $I$ is the moment of inertia about bending axis, $L$ the column height and $E$ the elastic modulus of column material. The total stiffness of a storey is thus $\Sigma K_c$. The lumped mass at all

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*For details, reference may be made to any book on Theory of Vibration.
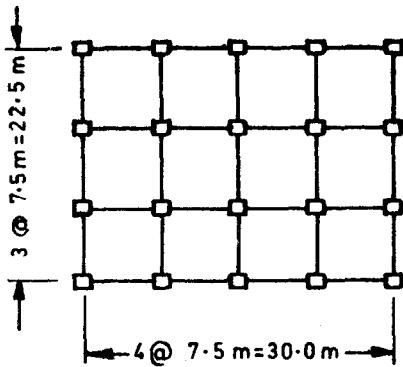
PLAN
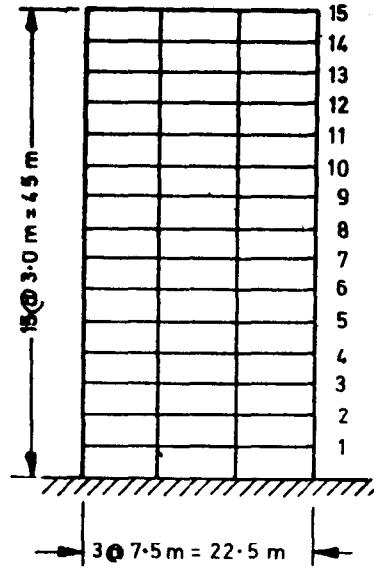
ELEVATION

FIG. 3

floor levels is 52·43 (t.s²/m) and at roof level is 40(t.s²/m). The values of $I$, $K_c$ and $\Sigma K_c$ for all the floors/storeys is $1·08 \times 10^6$ cm⁴, 9 024 t/m and 180 480 t/m, respectively.

b) The first three natural frequencies and the corresponding mode shapes are determined using Stodola Vienello iteration procedure and are given in Table 2.

### TABLE 2 PERIODS AND MODE SHAPE COEFFICIENTS AT VARIOUS LEVELS FOR FIRST THREE MODES

| Mode (r) | 1 | 2 | 3 |
|---|---|---|---|
| Period in Seconds | 1·042 | 0·348 | 0·210 |
| Mode shape coefficients at various floor levels | | | |
| $\phi_{15}^{(r)}$ | 0·356 | −0·355 | 0·353 |
| $\phi_{14}^{(r)}$ | 0·353 | −0·330 | 0·283 |
| $\phi_{13}^{(r)}$ | 0·347 | −0·273 | 0·140 |
| $\phi_{12}^{(r)}$ | 0·336 | −0·190 | −0·039 |
| $\phi_{11}^{(r)}$ | 0·323 | −0·089 | −0·208 |
| $\phi_{10}^{(r)}$ | 0·305 | 0·019 | −0·324 |
| $\phi_{9}^{(r)}$ | 0·285 | 0·127 | −0·355 |
| $\phi_{8}^{(r)}$ | 0·261 | 0·222 | −0·296 |
| $\phi_{7}^{(r)}$ | 0·235 | 0·296 | −0·158 |
| $\phi_{6}^{(r)}$ | 0·206 | 0·342 | 0·019 |
| $\phi_{5}^{(r)}$ | 0·175 | 0·356 | 0·192 |
| $\phi_{4}^{(r)}$ | 0·143 | 0·336 | 0·315 |
| $\phi_{3}^{(r)}$ | 0·108 | 0·285 | 0·356 |
| $\phi_{2}^{(r)}$ | 0·073 | 0·206 | 0·305 |
| $\phi_{1}^{(r)}$ | 0·037 | 0·108 | 0·175 |

c) Horizontal seismic coefficient for design is given by,

$$\alpha_h^{(r)} = \beta I F_0 \frac{S_a^{(r)}}{g}$$

Assuming 5 percent damping in all the three modes, $I = 1·0$ and $\beta = 1·0$, from Table 5 of Code, the design seismic coefficient for zone $V(F_0 = 0·40)$ in the three mode work out as follows:

1st Mode $\dfrac{S_a^{(1)}}{g} = 0·105$; $\alpha_h^{(1)} = 0·042$

2nd Mode $\dfrac{S_a^{(2)}}{g} = 0·184$; $\alpha_h^{(2)} = 0·0737$

3rd Mode $\dfrac{S_a^{(3)}}{g} = 0·200$; $\alpha_h^{(3)} = 0·080$

d) The next step is to obtain seismic forces at each floor level in each individual mode as required in Equation (3). Mode participation factors in each mode is to be obtained. For this, Table 3 would be found convenient wherein the method is explained for computation of $C_1$ (mode participation factor for first mode).

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## TABLE 3 COMPUTATION OF MODE PARTICIPATION FACTOR $C_1$

| Floor No. | Weight $W_i$ | Mode Coefficient $\phi_i$ | $Wi\phi_i$ | $Wi\phi_i^2$ | |
|---|---|---|---|---|---|
| 1 | 514·34 | 0·037 | 19·03 | 0·70 | |
| 2 | 514·34 | 0·073 | 37·55 | 2·74 | |
| 3 | 514·34 | 0·108 | 55·55 | 6·00 | |
| 4 | 514·34 | 0·143 | 73·55 | 10·52 | |
| 5 | 514·34 | 0·175 | 90·01 | 15·75 | |
| 6 | 514·34 | 0·206 | 105·95 | 21·83 | $C_1=\dfrac{1\ 778\cdot88}{498\cdot08}$ |
| 7 | 514·34 | 0·235 | 120·87 | 28·40 | $=3\cdot571$ |
| 8 | 514·34 | 0·261 | 134·24 | 35·03 | |
| 9 | 514·34 | 0·285 | 146·59 | 41·78 | |
| 10 | 514·34 | 0·305 | 156·87 | 47·85 | |
| 11 | 514·34 | 0·323 | 166·13 | 53·66 | |
| 12 | 514·34 | 0·336 | 172·82 | 58·07 | |
| 13 | 514·34 | 0·347 | 178·47 | 61·93 | |
| 14 | 514·34 | 0·353 | 181·56 | 64·09 | |
| 15 | 392·40 | 0·356 | 139·69 | 49·73 | |
| | | | $\Sigma 1\ 778\cdot88$ | $\Sigma 498\cdot08$ | |

Having obtained $C_1 = 3\cdot57$, $C_2$ and $C_3$ are obtained similarly as $1\cdot18$ and $0\cdot698$, respectively.

Seismic force acting at each floor is then computed as per Equation(3) and is given conveniently in tabular form for mode 1 as shown in Table 4. This table also gives the shears in each storey in this mode. Similarly, Tables 5 and 6 tabulate the forces for the second and third mode respectively.

## TABLE 4 COMPUTATION OF LATERAL FORCES AND SHEARS (FIRST MODE)

| Floor No. | $Wi$ | $\phi_i$ | $C_1\alpha_h^{(1)}$ | $Q_i^{(1)}=C_1\alpha_h^{(1)} \times Wi\phi_i$ | $V_i^{(1)}= \Sigma Q_i^{(1)}$ |
|---|---|---|---|---|---|
| 1 | 514·34 | 0·037 | 0·150× 0·042 | 2·81 | 265·29 |
| 2 | 514·34 | 0·073 | do | 5·58 | 262·98 |
| 3 | 514·34 | 0·108 | do | 8·30 | 256·90 |
| 4 | 514·34 | 0·143 | do | 11·03 | 248·60 |
| 5 | 514·34 | 0·175 | do | 13·45 | 237·67 |
| 6 | 514·34 | 0·206 | do | 15·82 | 224·22 |
| 7 | 514·34 | 0·235 | do | 18·03 | 208·40 |
| 8 | 514·34 | 0·261 | do | 20·04 | 190·37 |
| 9 | 514·34 | 0·285 | do | 21·84 | 170·33 |
| 10 | 514·34 | 0·305 | do | 23·42 | 148·49 |
| 11 | 514·34 | 0·323 | do | 24·74 | 125·07 |
| 12 | 514·34 | 0·336 | do | 25·80 | 100·33 |
| 13 | 514·34 | 0·347 | do | 26·59 | 74·53 |
| 14 | 514·34 | 0·353 | do | 27·10 | 47·94 |
| 15 | 392·40 | 0·356 | do | 20·84 | 20·84 |

## TABLE 5 COMPUTATION OF LATERAL FORCES AND SHEAR (MODE 2)

| Floor No. | $Wi$ | $\phi_i$ | $C_2\alpha_h^{(2)}$ | $Q_i^{(2)} = C_2\alpha_h^{(2)} \times Wi\phi_i$ | $V_i^{(2)}= \Sigma Q_i^{(2)}$ |
|---|---|---|---|---|---|
| 1 | 514·34 | 0·108 | 1·18 × 0·073 7 | 4·87 | 51·53 |
| 2 | 514·34 | 0·206 | do | 9·28 | 46·66 |
| 3 | 514·34 | 0·285 | do | 12·82 | 37·38 |
| 4 | 514·34 | 0·336 | do | 15·14 | 24·56 |
| 5 | 514·34 | 0·356 | do | 16·03 | 9·42 |
| 6 | 514·34 | 0·342 | do | 15·40 | —6·61 |
| 7 | 514·34 | 0·296 | do | 13·32 | —22·01 |
| 8 | 514·34 | 0·222 | do | 9·98 | —35·33 |
| 9 | 514·34 | 0·127 | do | 5·70 | —45·31 |
| 10 | 514·34 | 0·019 | do | 0·87 | —51·01 |
| 11 | 514·34 | —0·089 | do | —4·03 | —51·88 |
| 12 | 514·34 | —0·190 | do | —8·56 | —47·85 |
| 13 | 514·34 | —0·275 | do | —12·27 | —39·29 |
| 14 | 514·34 | —0·330 | do | —14·83 | —27·02 |
| 15 | 392·40 | —0·355 | do | —12·19 | —12·19 |

## TABLE 6 COMPUTATION OF LATERAL FORCES AND SHEARS (MODE 3)

| Floor No. | $Wi$ | $\phi_i$ | $C_3\alpha_h^{(3)}$ | $Q_i^{(3)}=C_3\alpha_h^{(3)} Wi\phi_i$ | $V_i^{(3)}= \Sigma Q_i^{(3)}$ |
|---|---|---|---|---|---|
| 1 | 514·34 | 0·175 | 0·698 × 0 080 | 5·01 | 19·37 |
| 2 | 514·34 | 0·305 | do | 8·73 | 14·36 |
| 3 | 514·34 | 0·356 | do | 10·18 | 5·63 |
| 4 | 514·34 | 0·315 | do | 9·00 | —4·55 |
| 5 | 514·34 | 0·192 | do | 5·49 | —13·55 |
| 6 | 514·34 | 0·019 | do | 0·56 | —19·04 |
| 7 | 514·34 | —0·158 | do | —4·52 | —19·60 |
| 8 | 514·34 | —0·295 | do | —8·42 | —15·08 |
| 9 | 514·34 | —0·355 | do | —10·15 | —6·66 |
| 10 | 514·34 | —0·324 | do | —9·25 | 3·49 |
| 11 | 514·34 | —0·208 | do | —5·95 | 12·74 |
| 12 | 514·34 | —0·039 | do | —1·12 | 18·69 |
| 13 | 514·34 | 0·140 | do | 4·01 | 19·81 |
| 14 | 514·34 | 0·283 | do | 8·10 | 15·80 |
| 15 | 392·40 | 0·353 | do | 7·70 | 7·70 |

e) Combination of shears for the three modes:

After getting shear forces in each individual mode, the total shear force in each storey is obtained in accordance with **4.2.2.2** of the Code, total shear force,

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

For height of building equal to 45 m, $\gamma = 0\cdot65$, shear in any storey say top (15th), is given by,
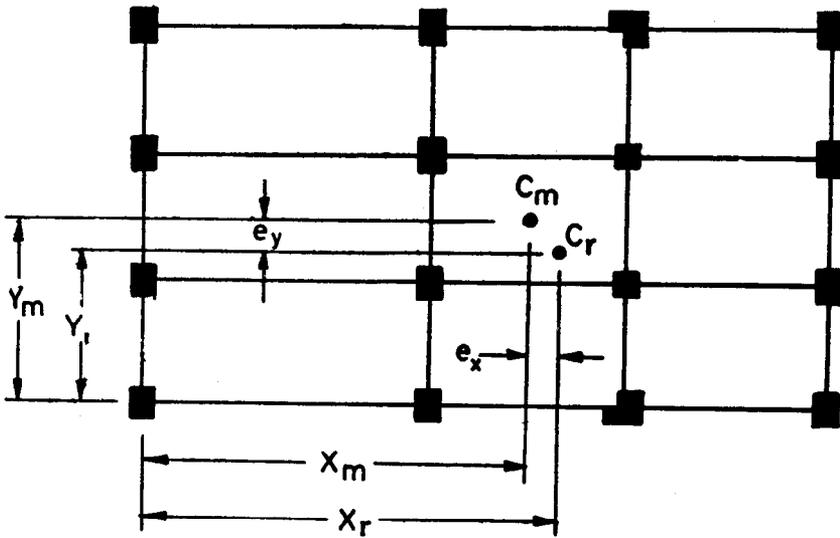
FIG. 4 PLAN OF UNSYMMETRICAL STRUCTURE

$V_{15}' = (1 - 0.65) [(20.84 + 12.19 + 7.70)]$

$+ 0.65\sqrt{(20.84)^2 + (12.19)^2 + (7.7)^2}$

$= 30.73$ t

Similarly, shear in all other storeys (to be taken by all columns) are computed as follows:

$V_1 = 391.37$ t $\qquad V_9 = 217.14$ t

$V_2 = 382.93$ t $\qquad V_{10} = 188.17$ t

$V_3 = 367.19$ t $\qquad V_{11} = 161.08$ t

$V_4 = 346.10$ t $\qquad V_{12} = 132.71$ t

$V_5 = 321.95$ t $\qquad V_{13} = 103.03$ t

$V_6 = 296.95$ t $\qquad V_{14} = 68.98$ t

$V_7 = 272.81$ t $\qquad V_{15} = 30.73$ t

$V_8 = 245.96$ t

The above values of shear forces in various storeys are to be taken for seismic condition in analysis. Table 7 gives maximum interstorey displacement of the building.

NOTE 1 — In building in which the floors and roofs consist of solid or hollow slabs, or of other elements with an equivalent horizontal rigidity, the shearing force $V_i$ acting at level $i$ shall be distributed along the various vertical elements in proportion to their rigidity.

NOTE 2 — The rigidity and the resistance of the slab shall be verified to determine that they are capable of providing such distribution of loading. If the amount of deformation of the slab is not negligible, the effect of their flexibility shall be taken into account in the distribution of the shearing forces.

**4.2.3** *Drift ( Lateral Displacement or Sway ) —* The drift limitation is necessary to save elements (non-structural elements) which form an integral part of the structure and to avoid upsetting of the

TABLE 7 DRIFT OR MAXIMUM INTERSTOREY DISPLACEMENT OF BUILDING (EXAMPLE MODAL ANALYSIS)

| STOREY $i$ | SHEAR, $Vi$ (t) | STIFFNESS, $Ki$(t/cm) | RELATIVE DISPLACEMENT, MAX $Vi/Ki$ (cm) |
|---|---|---|---|
| 1 | 391.37 | 1 804.80 | 0.217 |
| 2 | 382.93 | 1 804.80 | 0.212 |
| 3 | 367.19 | 1 804.80 | 0.203 |
| 4 | 346.10 | 1 804.80 | 0.192 |
| 5 | 321.95 | 1 804.80 | 0.178 |
| 6 | 296.95 | 1 804.80 | 0.165 |
| 7 | 272.81 | 1 804.80 | 0.151 |
| 8 | 245.96 | 1 804.80 | 0.136 |
| 9 | 217.14 | 1 804.80 | 0.120 |
| 10 | 188.17 | 1 804.80 | 0.104 |
| 11 | 161.08 | 1 804.80 | 0.089 |
| 12 | 132.71 | 1 804.80 | 0.074 |
| 13 | 103.03 | 1 804.80 | 0.057 |
| 14 | 68.98 | 1 804.80 | 0.038 |
| 15 | 30.73 | 1 804.80 | 0.017 |

occupants psychologically. It is felt that 0.004 of the interstorey height is a good limit for this purpose. Buildings analysed for seismic forces must be checked for this drift limitation also.

In a 15 storeyed building analysed in Example 2, the drift is worked out in Table 7. It is seen that in a few storeys the drift exceeds $0.004 \times 3 = 0.012$ m and hence the design needs revision from this point of view.

**4.2.4** *Torsion of Buildings —* Horizontal twisting occurs in buildings when centre of mass and centre of rigidity do not coincide. The distance between these two is called eccentricity $(e)$. Lateral force multiplied by this '$e$' cause a torsional moment which must be resisted by the structure
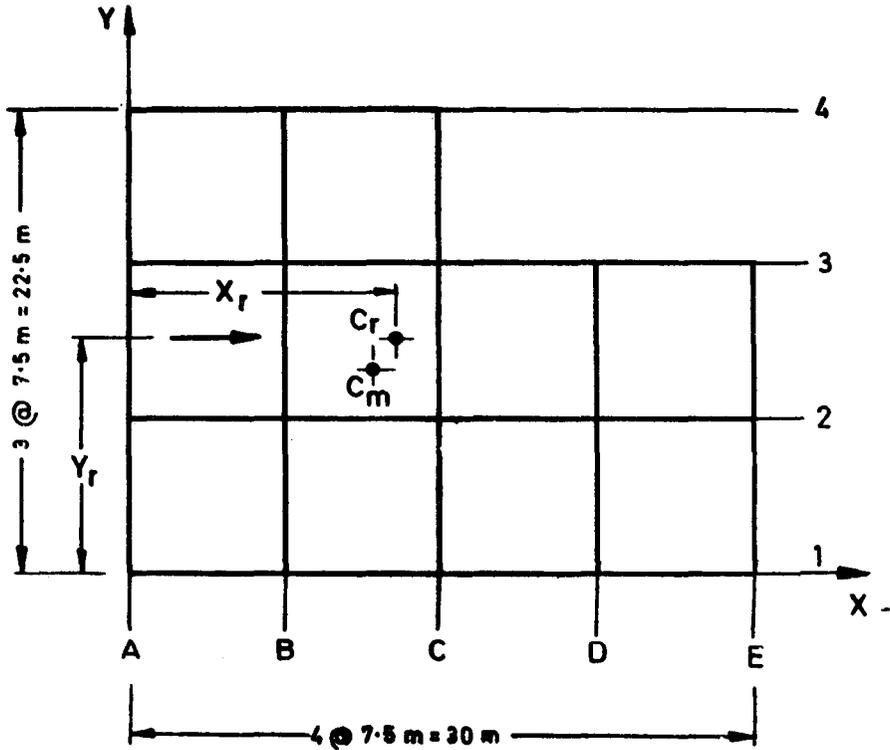
FIG. 5 PLAN OF 4-STOREYED STRUCTURE

in addition to the normal seismic force. Therefore, the Code stipulates that provision shall be made for increase in shear forces acting on particular elements resulting from the horizontal torsion due to an eccentricity between the centre of mass and the centre of rigidity.

It is desirable to plan structural elements of the building in such a way that there is no eccentricity or the building is symmetrically planned with respect to the mass centre. However, it is very difficult to do so in practice and some provision has to be made for it.

Since there could be quite a bit of variation in computed value of $e$, it is recommended by the Code that design eccentricity shall be $1.5\ e$. The net effect of this torsion is to increase shear in certain structural elements and reduction in certain others. The Code recommends that reduction in shear on account of torsion should not be applied and only increased shears in the elements be considered.

The torsional forces shall be distributed to the various vertical components of the seismic resisting system with due consideration given to the relative stiffnesses of the vertical components and the diaphragm. It is then corrected for torsion taking into account the increases produced, but not the decreases as specified in the Code.

In order to understand the method of determining the additional shears due to torsion, the building plan given in Fig. 4 may be examined.

Assuming the origin at point 0, the coordinates of centre of rigidity (geometric centre of stiffnesses of various vertical resistance elements) or point of rotation are computed as follows:

$$X_r = \frac{\Sigma K_y x}{\Sigma K_y}, \qquad Y_r = \frac{\Sigma K_x y}{\Sigma K_x}$$

in which $x$, $y$ are the coordinates of and $K_x$, $K_y$ are stiffness of the various elements in the two directions, respectively. The rotational stiffness $I_{xy}$ of the structure about centre of rotation $C_r$ is given by,

$$I_{xy} = \Sigma [K_x \cdot Y^2 + K_y \cdot X^2]$$

in which $x$ and $y$ are the distances of elements from the centre of rigidity $C_r$. If the torsional moment is $T$ (equal to $Ve$), the torsional shears $V_x$ and $V_y$ on any column line can be computed as:

$$V_x = \frac{T}{I_p} \cdot y \cdot K_{xx}, \text{ and}$$

$$V_y = \frac{T}{I_p} \cdot x \cdot K_{yy}$$

in which $K_{xx}$ and $K_{yy}$ are the total stiffness of the column line under consideration in the $x$ and $y$ directions, respectively.

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*Example 3*

A four storeyed building (with load 300 kg/m²) has the plan as shown in Fig. 5 and is to be designed in seismic Zone III. Work out the seismic shears in the various storeys of the

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proposed building. The factor β may be assumed as 1·0 and importance factor I also as 1·0.

a) Total weight of beams in a storey

$$= 14 \times 7.5 \times 0.4 \times 0.5 \times 2.4$$
$$= 50.4 \text{ t}$$

Total weight of columns in a storey

$$= 18 \times 3 \times 0.4 \times 0.6 \times 2.4$$
$$= 31.10 \text{ t}$$

Total weight of slab in a storey

$$= (22.5 \times 15 + 15 \times 15) \times 0.15$$
$$\times 2.4 = 202.5 \text{ t}$$

Total weight of walls

$$= (22.5 + 15 + 7.5 + 30 + 15 + 15)$$
$$\times 0.2 \times 3 \times 2.0 = 126 \text{ t}$$

Live load in each floor

$$= (22.5 \times 15 + 15 \times 15) \times 0.3 \times 0.25$$
$$= 42.18 \text{ t}$$

Lumped weight at floors 1, 2 and 3

$$= (410.00 + 42.18) = 452.18 \text{ t}$$

Lumped weight at roof floor

$$= 410.00 \text{ t}$$

Total weight of building

$$= 452.18 \times 3 + 410.00$$
$$= 1\,766.5 \text{ t}$$

b) Base shear:

$$T = 0.1 \times 4 = 0.4 \text{ s}$$

C (from Fig. 3 of the Code) $= 0.90$

$$V_B = I\,\alpha_h\,C.W.$$
$$= 1 \times 1 \times 0.04 \times 0.90 \times 1\,766.5$$
$$= 63.59 \text{ t}$$

c) The shear forces in various storeys are computed as shown in Table 8:

d) $e_x$ and $e_y$ are computed as:

$$e_x = 13.75 - 13.51 = 0.24 \text{ m (design value}$$
$$= 0.36 \text{ m)}$$
$$= 0.35 \text{ m (for top floor,}$$
$$\text{design value}$$
$$= 0.52)$$
$$e_y = 10.00 - 9.76 = 0.24 \text{ m (design value}$$
$$= 0.36 \text{ m)}$$
$$= 0.30 \text{ m (for top floor, design value}$$
$$= 0.45 \text{ m)}$$

Total rotational stiffness $I_p$ about centre of rigidity is given by,

$$I_p = (K_x Y^2 + K_y X^2)$$
$$K_x Y^2 = 2\,673.78 \,[5(10^2) + 5(2.5^2)$$
$$+ 5(5^2) \times 3(12.5)^2]$$
$$= 3\,016\,023.8$$
$$K_y X^2 = 6\,016\,[4(13.75^2) + 4(6.25^2) +$$
$$4(1.25^2) + 3(8.75^2) + 3(16.25^2)]$$
$$= 11\,674\,799.0$$
$$I_p = 3\,016\,023.8 + 11\,674\,799.0$$
$$= 14\,690\,822.8$$

e) Torsional moment T at various floors is as follows, considering seismic force in X-direction only,

$$T_1 = 63.59 \times 0.36 = 22.89 \text{ t.m}$$
$$T_2 = 61.36 \times 0.36 = 22.18 \text{ t.m}$$
$$T_3 = 52.44 \times 0.36 = 18.88 \text{ t.m}$$
$$T_4 = 32.36 \times 0.45 = 14.56 \text{ t.m}$$

Torsional shear at each column line is worked out in Table 9 by using equation:

$$V_x = \frac{T}{I_p} \cdot y \cdot K_{xx}$$

**TABLE 8 NODAL FORCES AND SEISMIC SHEAR FORCES AT VARIOUS LEVELS**

| Floor | $W_i$ | $h_i$ | $W_i h_i^2$ | $Q_i$ | $V_i$ (Shear Force) |
|---|---|---|---|---|---|
| | (t) | (m) | | (t) | (t) |
| 1 | 452.18 | 3 | 4 069·62 | 2·23 | 63·59 |
| 2 | ,, | 6 | 16 278·48 | 8·92 | 61·36 |
| 3 | ,, | 9 | 36 626·58 | 20·07 | 52·44 |
| 4 | 410·00 | 12 | 59 040·00 | 32·36 | 32·36 |
| | | | 1 16 014·68 | | |

**TABLE 9 TORSIONAL SHEARS IN VARIOUS STOREYS IN X-DIRECTION (IN TONNES)**

| Column Line | First Storey | Second Storey | Third Storey | Fourth Storey |
|---|---|---|---|---|
| | (1) | (2) | (3) | (4) |
| | $V_x$ | $V_x$ | $V_x$ | $V_x$ |
| 1 | 0·20 | 0·178 | 0·140 | 0·119 |
| 2 | 0·50 | 0·044 | 0·034 | 0·029 |
| 3 | 0·10 | —0·088 | —0·071 | —0·060 |
| 4 | 0·15 | —0·132 | —0·105 | —0·891 |

f) Torsional moment in the various storeys considering seismic force to be acting in Y-direction:

$$T_1 = 63.59 \times 0.36 = 22.89 \text{ t.m}$$
$$T_2 = 61.36 \times 0.36 = 22.18 \text{ t.m}$$
$$T_3 = 52.44 \times 0.36 = 18.88 \text{ t.m}$$
$$T_4 = 32.36 \times 0.52 = 16.82 \text{ t.m}$$

Torsional shears at each column line in this direction are worked out in Table 10 according to expression for

$$V_y = \frac{T \cdot x \cdot K_{yy}}{I_p} .$$

NOTE — It will be noted from Tables 9 and 10 that column lines 1, 2 (in case of seismic force in X-direction-left to right or right to left) and A, B (in Y-direction) will be having increase of shear due to torsion. Other column lines will be relieved of some shear but Code recommends not to reduce the seismic shear on account of such relief.

TABLE 10 TORSIONAL SHEARS IN VARIOUS STOREYS IN Y-DIRECTION (IN TONNES)

| COLUMN LINE | FIRST STOREY | SECOND STOREY | THIRD STOREY | FOURTH STOREY |
|---|---|---|---|---|
| | (1) | (2) | (3) | (4) |
| | $V_y$ | $V_y$ | $V_y$ | $V_y$ |
| A | 0.49 | 0.428 | 0.342 | 0.362 |
| B | 0.277 | 0.194 | 0.159 | 0.162 |
| C | 0.049 | 0.039 | 0.032 | 0.032 |
| D | 0.242 | 0.207 | 0.169 | 0.175 |
| E | 0.444 | 0.380 | 0.309 | 0.318 |

## 4.4 Miscellaneous

**4.4.1 and 4.4.2** Under these clauses the Code advises the designer to be extra careful in design of vertical and horizontal projections like towers, tanks, chimneys, balconies and other cantilever projections or appendages in buildings. Experience from past earthquakes has shown that such appendages get damaged considerably and many lives are lost due to their collapse. Such systems are subject to larger motions than the building to which they are attached. Therefore, the Code recommends that the seismic coefficient for the design of vertical cantilever projections attached to the building should be taken as five times the $\alpha_h$ specified for the main structure. Similarly, the vertical seismic coefficient for all horizontal projections shall be 5 times $\alpha_V$. However, it must be clearly understood that this provision is only for designing the projecting part and their connections with the main structure. This provision is not to be applied to main structure.

*Example 4* — A vertical appendage of size 30 × 30 cm and 1·0 m high is attached to a four storeyed building in Zone III. Work out the design forces for the appendage structure. Take impor-

tance factor as 1·0 and soil foundation factor β as 1·2.

For the Zone III, basic seismic coefficient is 0·04. The design horizontal seismic coefficient for the structure is given by,

$$\alpha_h = 1.0 \times 1.2 \times 0.04 = 0.048$$

For the appendage the seismic coefficient = 5 × 0·048 = 0·24. The seismic force on the appendage is given by,

$$F = 0.24 \times 0.3 \times 0.3 \times 1 \times 2.4 = 0.052 \text{ t}$$

The appendage connection should be designed for a shear force equal to 0·052 t and bending moment equal to 0·052 × 0·5 = 0·026 t.m.

*Example 5* — A horizontal cantilever of size 1 × 1 m having average thickness of 6 cm projects out from the wall of a brick building room (size 5 × 4·5 m). The building has a height of 3 m and has flat roof with 12 cm slab and roof finishing with 240 kg/m². Thickness of wall is 20 cm. The building is located in seismic Zone V (*see* Fig. 6).

The cantilever is to be designed for a vertical seismic coefficient equal to five times the vertical
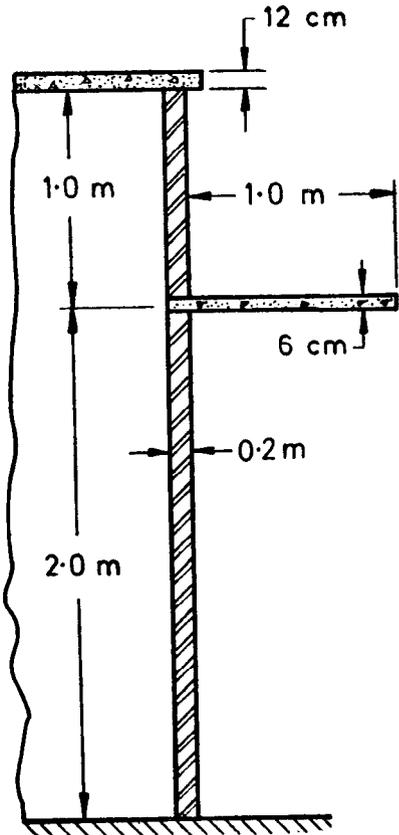


FIG. 6

coefficient for the zone $\therefore \alpha_v = 5 \times \frac{1}{2} \times 0.08 = 0.20$. The cantilever if designed for static forces will be found to be safe because under seismic condition the moments and shears will increase only by 20 percent whereas the allowable stresses are to be increased by $33\frac{1}{3}$ percent as per 3.3 of the Code.

For checking the stability of the cantilever, however, it is important to use $\alpha_v$ as acting downward for the cantilever and upward for the main structure which may be the worst condition in view of difference in characteristics of the two portions (main building and cantilever). This leads to increased overturning moment and reduced stabilizing moment. Referring to Fig. 7 where forces are indicated, the stabilizing moment is given by:

$$M_s = W(1 - \alpha_v)\frac{t}{2}$$

and overturning moment by:

$$M_0 = W_1(1 + \alpha_v)\frac{L}{2}$$

For stability, $M_s$ must be greater than $M_0$.

$$W_1 = 1 \times 1 \times 0.06 \times 2.4 = 0.144\ t$$
$$M_0 = 0.144\ (1 + 0.2) \times 0.5$$
$$= 0.086\ 4\ t.m.$$

Weight of the wall above the cantilever $= 0.2 \times 1.0\ X \times 2.0 = 0.4\ X\ t$

Weight of roof slab per metre run including finishes $= 0.8\ t$

$$\therefore W = 0.4\ X + 0.8\ X = 1.2\ X$$
$$\therefore M_s = 1.2\ X \times 0.1 \times (1 - 0.2)$$
$$= 0.096\ X\ t.m.$$

Equating $M_s$ and $M_0$,

$$0.096\ X = 0.086\ 4$$
$$X = \frac{0.086\ 4}{0.096} = 0.9\ m = 1.0\ m\ (say).$$

The cantilever will have to be embedded under $1.0$ m length of wall. In other words the anchorage length will be $1.0$ m or equal to the width of the cantilever itself.

**5.2 Elevated Tower Supported Tanks** — No empirical formulae are available for prediction of period of water towers, and even though water towers generally have longer period and effective acceleration coefficients are smaller than the peak of the acceleration response spectra, this reduction is not made use of in the seismic coefficient method. It would be desirable that in future revision of the Code, response spectrum method for determination of $\alpha_h$ is recommended for all zones. Alternatively, since a majority of prototype water towers tested have periods longer than $0.6$ s, for preliminary design a reduction factor of $0.75$ be used such that the design seismic coefficient
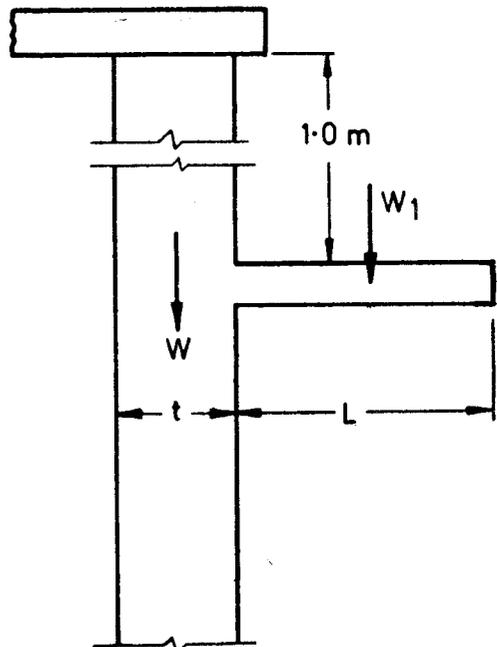
$$\alpha_h = 0.75\ \beta.I.\ \alpha_0$$



FIG. 7

**5.2.4** The analysis based on energy consideration (Reference 1) shows that when the mass $m$ of the tank (including contents) is much greater than the mass of the staging $m_0$, the equivalent mass will be

$$M = m + \frac{1}{3}m_c$$

and when the mass $m$ is smaller than $m_0$, the equivalent mass will be

$$M = m + \frac{1}{3}m_0$$

In general the mass of the staging would be smaller than the mass of the tank (including water) and hence the equivalent mass recommended in the Code is $\frac{1}{3}$ of actual mass.

The reason for considering only $\frac{1}{3}$ of the staging weight at the centroid of the tank is that $\frac{1}{3}$ of the staging weight will act at the bottom of the tank at ground level and hence will not contribute to vibration.

**5.2.7** *Hydrodynamic Pressure in Tanks* — When a tank containing fluid with a free surface is subjected to earthquake ground motion, it experiences dynamic fluid pressures of two types. Firstly, when the walls of tank accelerate, the adjacent fluid also accelerates and exerts on the wall an impulsive pressure which is directly proportional to the acceleration of the wall. Secondly, the effect of the impulsive pressure exerted by the wall on the fluid is to excite the fluid into oscillation and the oscillatory acceleration of the fluid produces convective pressures on the walls and bottom whose amplitudes are proportional to the amplitude of oscillation.
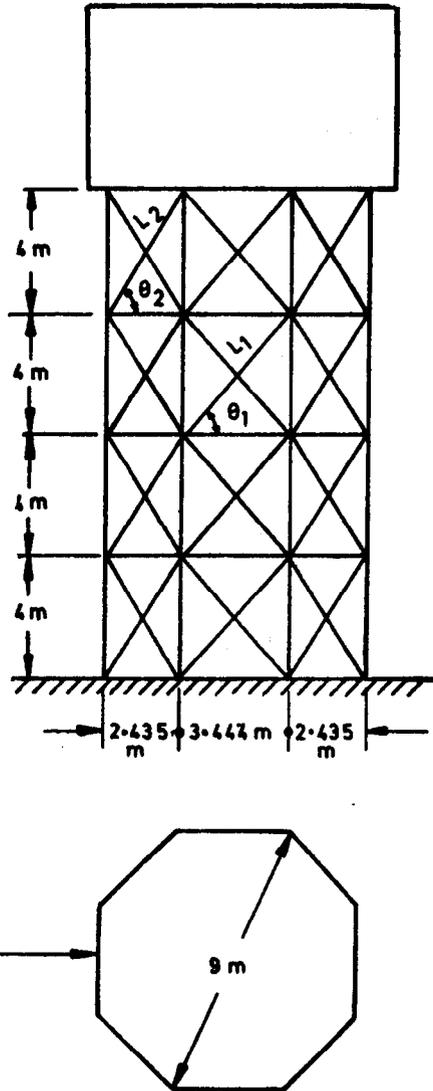
FIG. 8 ELEVATED WATER TANK

Assuming the fluid to be incompressible and fluid displacements small, analytical results have been obtained for rectangular and cylindrical type of tanks (Reference 2). The convective pressures during earthquakes are considerably less in magnitude as compared to impulsive pressures and are therefore usually neglected.

**5.2.7.2 and 5.2.7.3** In case of tanks resting on ground, the acceleration of the fluid in the tank is deemed to be the same as that of ground whereas for tanks supported on staging, the acceleration may correspond to the response acceleration of tank which would be different from that of the ground.

Nowadays, sophisticated methods, such as space frame analysis, are available for evaluation of the stiffness (or its inverse, the flexibility) of the staging. However, the approximate method as illustrated in Example 6 would do.

*Example* 6 — Consider an elevated water tower supported on a concrete staging of 8 columns located on the circumference of a circle of 9 m diameter. The height of the staging is 16 m and horizontal bracing is provided at a vertical spacing 4 m. The circular columns are 52 cm in diameter. Diagonal steel bracing in the form of 18 mm dia bars are provided in all bays (*see* Fig. 8).

*Equivalent Spring Constant*

The staging is assumed to be composed of springs in series connected at the horizontal brace level. The stiffness of the spring in one bay is made up of stiffness of columns and diagonal braces acting as parallel springs.

a) Stiffness of a column in a bay

$$k_c = \frac{12\,E\,I}{L^3}$$

Let

$$E = 2 \times 10^6 \text{ t/m}^2;$$

$$L = 4 \text{ m (centre to centre distance)}$$

The moment of inertia is assumed to be evaluated based on the gross area of concrete.

$$I = \pi d^4/64 = \pi \times (0.52)^4/64$$
$$= 3.589 \times 10^{-3} \text{m}^4$$
$$k_c = 12 \times 2 \times 10^6 \times 3.589 \times 10^{-3}/4^3$$
$$= 1\,345.9 \text{ t/m}$$

Stiffness of 8 columns acting in parallel

$$\Sigma k_c = 8 \times 1\,345.9 = 10\,767.2 \text{ t/m}$$

b) Stiffness of diagonals in a bay (*see* Fig. 8)

It is assumed that only the tension diagonals contribute to stiffness. The stiffness may be taken as:

$$k_b = \frac{AE}{L} \cos^2 \theta$$

where $A$ is the area of cross-section, $E$ the modulus of elasticity, $L$ the length and $\theta$ the angle with the horizontal.

$$A = \pi \times (0.018)^2/4 = 2.545 \times 10^{-4} \text{m}^2;$$
$$E = 2.1 \times 10^7 \text{ t/m}^2$$

The projected lengths of the diagonals on a vertical plane in the direction of earthquake are different. Denoting them by $L_1$ and $L_2$

$$L_1 = 3.444 \sec \theta_1; \quad L_2 = 2.435 \sec \theta_2.$$

$$\tan \theta_1 = 4/3.444 \text{ and } \tan \theta_2 = 4/2.435,$$

$$k_{b_1} = 430.84 \text{ t/m and } k_{b_2} = 308.58 \text{ t/m}$$

As there are 2 braces in the plane parallel to the direction of motion and 4 braces at an inclination to it, 2 braces do not contribute to stiffness being in a plane at right angles to the direction of motion. This gives

$$\Sigma k_b = 2k_{b_1} + 4k_{b_2} = 2\,096 \text{ t/m}$$

c) Stiffness of one bay is given by

$$k = \Sigma k_c + \Sigma k_b = 12\,863{\cdot}2 \text{ t/m}$$

d) There are 4 bays. The equivalent spring of the system $K$ is obtained by summing the stiffnesses of all bays in series.

$$\frac{1}{K} = \sum \frac{1}{k}$$

$$K = k/4 = 12\,863{\cdot}2/4 = 3\,215{\cdot}8 \text{ t/m}$$

A similar procedure may be used if earthquake force is applied in any other direction and the worst situation evaluated for design.

*Example 7* — An elevated water tank has a capacity of 600 m³. The tank is circular with an internal diameter of 12 m and height 6 m. The tank is supported on a concrete staging 16 m in height. The staging consists of columns, and horizontal and diagonal bracings. The structure is located in Zone IV and founded on Type II medium type of soil.

Let the weight of empty tank be 250 t. The weight of water in the tank when it is full is 600 t. Let the weight of staging consisting of columns and bracings be 150 t and the stiffness of the staging in any direction is 3 200 t/m. The value of stiffness $K$ may be obtained by any method of static analysis known to the designer (*see* Example 6). $K$ is the lateral force required to cause unit deflection at the centre of gravity of the tank.

*Lumped Weights*

a) *Tank empty*

Equivalent weight acting at the centre of gravity of tank

$$W_e = 250 + \tfrac{1}{3} \times 150 = 300 \text{ t}$$

b) *Tank full*

Equivalent weight,

$$W_f = 300 + 600 = 900 \text{ t}$$

*Fundamental Period*

$$T = 2\pi \sqrt{\frac{\Delta}{g}}, \quad \Delta = \frac{W}{K}$$

Here $W$ is the lumped weight at centre of gravity of the tank and $K$ is the stiffness of the staging.

Therefore,

$$T = 2\pi \sqrt{\frac{W}{gK}}$$

a) *Tank empty*

$$T = 2\pi \sqrt{\frac{W_e}{gK}} = 2\pi \sqrt{\frac{300}{9{\cdot}81 \times 3\,200}}$$

$$= 0{\cdot}614 \text{ s}.$$

b) *Tank full*

$$T = 2\pi \sqrt{\frac{W_f}{gK}} = 2\pi \sqrt{\frac{900}{9{\cdot}81 \times 3\,200}}$$

$$= 1{\cdot}064 \text{ s}.$$

*Average Acceleration Coefficient*

Assuming a damping of 5 percent of critical, for the above periods, the average acceleration coefficient (from Fig. 2 of the Code) would be:

a) *Tank empty,*

$$S_a/g = 0{\cdot}148$$

b) *Tank full,*

$$S_a/g = 0{\cdot}104$$

$$\alpha_h = \beta\, I.F_0.\ S_a/g$$

$$\beta = 1{\cdot}0,\ I = 1{\cdot}5 \text{ and } F_0 = 0{\cdot}25$$

a) *Tank empty,*

$$\alpha_h = 0{\cdot}055\,5$$

b) *Tank full,*

$$\alpha_h = 0{\cdot}039$$

*Shear Force*

$$V = \alpha_h \cdot W$$

a) *Tank empty,*

$$V = \alpha_h \cdot W_e = 0{\cdot}055\,5 \times 300 = 16{\cdot}65 \text{ t}$$

b) *Tank full,*

$$V = \alpha_h\ W_f = 0{\cdot}039\,0 \times 900 = 35{\cdot}10 \text{ t}$$

Tank full is the severest condition. The force 35·10 t acts horizontally at the centre of gravity of the tank in the plane in which the structure is assumed to oscillate.

*Hydrodynamic Pressure*

The tank wall will be subjected to an acceleration of $(\alpha_h\ g)$ 38·259 cm/s² (corresponding to $\alpha_h = 0{\cdot}039$) with a natural period of 1 064 s, this would correspond to a displacement of

$$a \cdot \frac{T^2}{4\pi^2} = \frac{38 \cdot 259 \times (1 \cdot 064)^2}{4\pi^2} = 1 \cdot 097 \text{ cm}$$

where $a$ is acceleration.

*Impulsive Pressure*

Height of water in tank

$$= \frac{600}{\pi 6^2} = 5 \cdot 305 \text{ m}$$

Pressure on Wall,

$$p_w = \alpha_h \cdot w \cdot h \cdot \sqrt{3} \cos \phi' \left[ \frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2 \right]$$

$$\times \tanh \sqrt{3}\left(\frac{R}{h}\right)$$

$p_w$ is maximum when $\cos \phi'$

$= 1$ (for $\phi' = 0$)

$$p_w = 0 \cdot 039 \times 1\,000 \times 5 \cdot 305 \times \sqrt{3} \left[ \frac{y}{h} - \frac{1}{2} \right.$$

$$\times \left. \left(\frac{y}{h}\right)^2 \right] \tanh \sqrt{3}\left(\frac{6}{5 \cdot 305}\right)$$

$$= 206 \cdot 895 \times \sqrt{3} \left[ \frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2 \right] \frac{3 \cdot 612}{3 \cdot 748}$$

$$= 345 \cdot 35 \left[ \frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2 \right]$$

at $y = 0 \cdot 2 h$

$p_w = 345 \cdot 35 \times 0 \cdot 18 = 62 \cdot 16 \text{ kg/m}^2$

at $y = 0 \cdot 4 h$

$p_w = 345 \cdot 35 \times 0 \cdot 42 = 145 \cdot 06 \text{ kg/m}^2$

at $y = 0 \cdot 6 h$

$p^* = 345 \cdot 35 \times 0 \cdot 42 = 145 \cdot 06 \text{ kg/m}^2$

at $y = 0 \cdot 8 h$

$p_w = 345 \cdot 35 \times 0 \cdot 48 = 165 \cdot 76 \text{ kg/m}^2$

at $y = 1 \cdot 0 h$

$p_w = 345 \cdot 35 \times 0 \cdot 5 = 172 \cdot 20 \text{ kg/m}^2$

Pressure on the bottom of the tank on a strip of width $2l'$ is

$$p_b = \alpha_h \cdot w \cdot h \cdot \sqrt{\frac{3}{2}} \left\{ \frac{\sinh \sqrt{3}\left(\frac{x}{h}\right)}{\cosh \sqrt{3}\left(\frac{l'}{h}\right)} \right\}$$

taking value of $l' = R$, the above expression becomes

$$p_b = \alpha_h \cdot w \cdot h \cdot \sqrt{\frac{3}{2}} \left\{ \frac{\sinh \sqrt{3}\left(\frac{x}{h}\right)}{\cosh \sqrt{3}\left(\frac{R}{h}\right)} \right\}$$

$$= \pm 0 \cdot 039 \times 1\,000 \times 5 \cdot 305 \times \frac{1 \cdot 732}{2}$$

$$\times \left\{ \frac{\sinh \sqrt{3}\left(\frac{x}{h}\right)}{\cosh \sqrt{3}\left(\frac{6}{5 \cdot 305}\right)} \right\}$$

$$= \frac{179 \cdot 171}{3 \cdot 612} \sinh \sqrt{3}\left(\frac{x}{h}\right)$$

$$= 48 \cdot 77 \sinh \sqrt{3}\left(\frac{x}{h}\right)$$

at $x = 0$

$p_b = 0$

at $x = \pm 0 \cdot 2 R$

$p_b = 48 \cdot 77 \times 0 \cdot 401\,9 = \pm 19 \cdot 60 \text{ kg/m}^2$

at $x = \pm 0 \cdot 4 R$

$p_b = 48 \cdot 77 \times 0 \cdot 866\,3 = \pm 42 \cdot 25 \text{ kg/m}^2$

at $x = \pm 0 \cdot 6 R$

$p_b = 48 \cdot 77 \times 1 \cdot 465\,3 = \pm 71 \cdot 46 \text{ kg/m}^2$

at $x = \pm 0 \cdot 8 R$

$p_b = 48 \cdot 77 \times 2 \cdot 292\,1 = \pm 111 \cdot 78 \text{ kg/m}^2$

at $x = \pm 1 \cdot 0 R$

$p_b = 48 \cdot 77 \times 3 \cdot 475\,3 = \pm 169 \cdot 49 \text{ kg/m}^2$

The distribution of pressure along the wall and at the base are as shown in Fig. 9B for $\phi' = 0$ and $l' = R$. Similarly, pressure distributions along the wall and base at various locations can be found out by changing the values of $\phi'$ and $l'$ and studied for design.
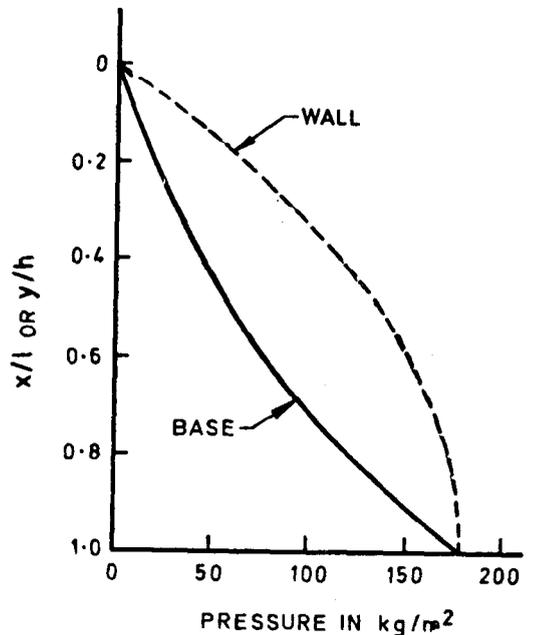


FIG. 9A   RECTANGULAR TANK

*Example* 8 — An elevated water tank has a capacity of 600 m³. The tank is rectangular 12 m × 9 m in size and 6 m in height. The tank is supported on a concrete staging 16 m high. The staging consists of columns, horizontal and diagonal bracings. The structure is located in Zone IV and founded on a Type II medium type of soil.

Let the weight of the empty tank be 250 t. The weight of the water in the tank when it is full is 600 t. Let the weight of the staging, consisting of columns and bracings be 150 t and the stiffness of the staging in any direction is 3 200 t/m.

*Lumped Weights*

a) *Tank empty*

Equivalent weight acting at the centre of gravity of tank

$$W_e = 250 + \tfrac{1}{3} \times 150 = 300 \text{ t}$$

b) *Tank full*

Equivalent Weight

$$W_f = 300 + 600 = 900 \text{ t}$$

*Fundamental Period*

$$T = 2\pi \sqrt{\frac{\Delta}{g}}$$

Here $\Delta$ is the static horizontal deflection at the top of the tank which can be found out by any of the methods from structural analysis.

Using the relation $\Delta = \dfrac{W}{K}$

where $W$ is the lumped weight at centre of gravity of the tank and $K$ the stiffness of the staging the expression for fundamental period becomes

$$T = 2\pi \sqrt{\frac{W}{g K}}$$

a) *Tank empty*

$$T = 2\pi \sqrt{\frac{W_e}{g K}} = 2\pi \sqrt{\frac{300}{9.81 \times 3\,200}}$$
$$= 0.614 \text{ s}$$

b) *Tank full*

$$T = 2\pi \sqrt{\frac{W_f}{g K}} = 2\pi \sqrt{\frac{900}{9.81 \times 3\,200}}$$
$$= 1.064 \text{ s}.$$

*Average Acceleration Coefficient*

Assuming damping as 5 percent of the critical value, for the above periods, the average acceleration coefficients (from Fig. 2 of the Code) would be

a) *Tank empty*

$$\frac{S_a}{g} = 0.148$$

b) *Tank full*

$$\frac{S_a}{g} = 0.104$$

*Seismic Coefficient*

$$\alpha_h = \beta \,.I.F_0\, \frac{S_a}{g}$$

$$\beta = 1.0, \quad F_0 = 0.25$$

a) *Tank empty* $\alpha_h = 0.055\,5$

b) *Tank full* $\alpha_h = 0.039$

*Shear Force* $V = \alpha_h.W$

a) *Tank empty*

$$V = \alpha_h.W_e = 0.055\,5 \times 300 = 16.65 \text{ t}$$

b) *Tank Full*

$$V = \alpha_h.W_f = 0.039 \times 900 = 35.10 \text{ t}$$

Tank full is the severest condition.

The force 35.10 t acts horizontally at the centre of gravity of the tank in the plane in which the structure is assumed to oscillate.

*Hydrodynamic Pressure*

The tank wall will be subjected to an acceleration of $(\alpha_h.g)$ 38.259 cm/s² (corresponding to $\alpha_h = 0.039$) with a natural period of 1.064 s, this would correspond to a displacement of

$$a \times \frac{T^2}{4\pi^2} = \frac{38.259 \times (1.064)^2}{4\pi^2} = 1.097 \text{ cm}$$

where $a$ is acceleration.

*Impulsive Pressure*

$$\text{Height of water in the tank} = \frac{600}{12 \times 9}$$
$$= 5.556 \text{ m}$$

Pressure on the wall,

$$p_w = \alpha_h.w.h \ \sqrt{3} \left[ \frac{y}{h} - \tfrac{1}{2} \left( \frac{y}{h} \right)^2 \right]$$
$$\times \tanh \sqrt{3} \left( \frac{l}{h} \right)$$

$$= 0.039 \times 1\,000 \times 5.556$$

$$\times \sqrt{3} \left[ \frac{y}{h} - \tfrac{1}{2} \left( \frac{y}{h} \right)^2 \right]$$

$$\times \tanh \sqrt{3} \left( \frac{6}{5.556} \right)$$

$$= 375.262\,9 \left[ \frac{y}{h} - \tfrac{1}{2} \left( \frac{y}{h} \right)^2 \right]$$
$$\times\ 0.953\,63$$

$$= 357.862\,4 \left[ \frac{y}{h} - \tfrac{1}{2} \left( \frac{y}{h} \right)^2 \right]$$

at $\dfrac{y}{h} = 0$

$p_w = 0$

at $\dfrac{y}{h} = 0.2$

$p_w = 357.862\,4 \times 0.18 = 64.4 \text{ kg/m}^2$

at $\dfrac{y}{h} = 0.4$

$p_w = 357.862\,4 \times 0.32 = 114.5 \text{ kg/m}^2$

at $\dfrac{y}{h} = 0.6$

$p_w = 357.862\,4 \times 0.42 = 150.3 \text{ kg/m}^2$

at $\dfrac{y}{h} = 0.8$

$p_w = 357.862\,4 \times 0.48 = 171.8 \text{ kg/m}^2$

at $\dfrac{y}{h} = 1.0$

$p_w = 357.862\,4 \times 0.5 = 178.9 \text{ kg/m}^2$

The pressure on the bottom of the tank

$$p_b = \alpha_h . w . h \frac{\sqrt{3}}{2} \left[ \frac{\sinh \sqrt{3} \left( \frac{x}{h} \right)}{\cosh \sqrt{3} \left( \frac{l}{h} \right)} \right]$$

$$= 0.039 \times 1\,000 \times 5.556 \times \frac{\sqrt{3}}{2}$$

$$\left[ \frac{\sinh \sqrt{3} \left( \frac{x}{h} \right)}{\cosh \sqrt{3} \times \frac{6}{5.556}} \right]$$

$$= \frac{0.039 \times 1\,000 \times 5.556 \times \sqrt{3}}{2 \times 3.323}$$

$$\times\ \sinh \sqrt{3} \left( \frac{x}{h} \right)$$

$$= 56.464\,5 \sinh \sqrt{3} \left( \frac{x}{h} \right)$$

at $\dfrac{x}{l} = 0$

$p_b = 0$

at $\dfrac{x}{l} = \pm 0.2$

$p_b = 56.464\,5 \times 0.382\,9 = 21.62 \text{ kg/m}^2$

at $\dfrac{x}{l} = \pm 0.4$

$p_b = 56.464\,5 \times 0.82 = 46.30 \text{ kg/m}^2$

at $\dfrac{x}{l} = \pm 0.6$

$p_b = 56.464\,5 \times 1.373 = 77.54 \text{ kg/m}^2$

at $\dfrac{x}{l} = \pm 0.8$

$p_b = 56.464\,5 \times 2.120\,9 = 119.76 \text{ kg/m}^2$

at $\dfrac{x}{l} = \pm 1.0$

$p_b = 56.464\,5 \times 3.168\,9 = 178.93 \text{ kg/m}^2$

The above type of pressure distribution are plotted and are shown in Figure 9A
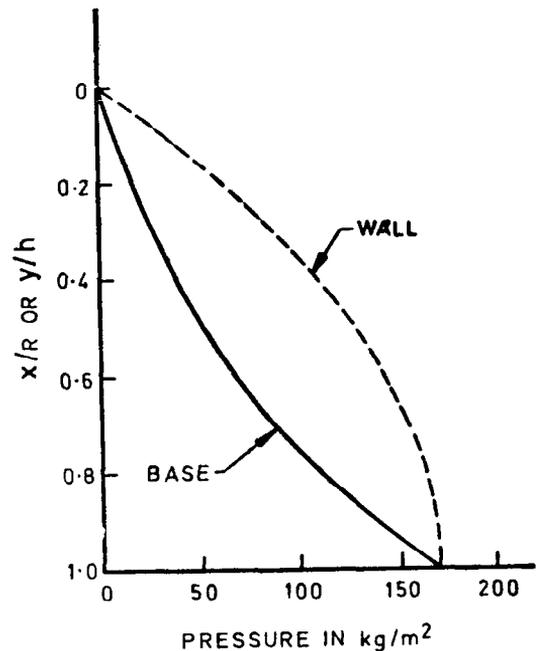


FIG. 9B   CIRCULAR TANK

### 5.3 Stacklike Structure

*General* — Stacklike structures are those in which mass and stiffness is more or less uniformly distributed along the height. For the purpose of analysis these could be treated as cantilever beams.

If the structure is long compared to its cross-sectional dimensions, bending deformations are predominant, but in general shear and rotary inertia deformations should also be considered in the analysis.

The formulae given in the Code concerning period of vibration, base shear and base moments are based on the research work (Reference 3). The following parameters have been varied in the analysis of various types of chimneys:

The slenderness ratio at base, $r_0$, has been varied from 5 to 50 where the lower limit corresponds to short and stout chimneys and the upper limit to long and slender ones. The radius of gyration, $r$, as well as the area of cross-section, $A$, varied linearly along the length $L$. That is, at any section, distance '$x$' from the base,

$$r_a/r_0 = A_a/A_0 = 1 - (1 - \beta) x/L$$

In addition, for a few cases, the variation of $A$, is assumed as,

$$A_a/A_0 = 1 - [(1 - \beta) x/L]^2$$

The taper ratio $\beta$ has been varied from a value of 1·0 to 0·2 which covers a wide range. In all cases, bending moment and shear have been computed both at the base as well as along the length. The term response has been used here to denote such quantities.

Recorded ground motion corresponding to actual earthquakes, namely, NS component of El Centro earthquake of 18 May 1940 and N69W component of Taft earthquake of 21 July 1952 have been used to compute the various response. The undamped fundamental period of vibration had two values, namely, two seconds corresponding to larger values of $r_0$ and one second corresponding to shorter values of $r_0$. Where $r_0$ is slenderness ratio $= L/r$, $r$ being radius of gyration. The first three modes of vibration have been used for mode superposition purposes. Modes of higher order have not been considered as the corresponding periods are so short that ground motion accelerations are not defined for such periods. Two combinations of damping have been considered. In one case, damping has been assumed to be the same in all modes and in the other, damping increased with increase in order of modes thus representing structural damping phenomenon. The first case, in which damping is the same in all modes, would be conservative and gives slightly larger response. For the combination of modes, response has been obtained time-wise and maximum values have also been determined.

In another method of determining response, ground motion has been assumed to be such that the velocity spectra $\frac{T}{2\pi} S_a$ is independent of period. This has been termed as 'flat spectra'. Such an assumption nearly corresponds to that of Housner's average spectra. Flat spectra criterion generally gives larger response for higher modes compared to actual ground motion if the data is so normalized such that first mode response is the same in all cases. Assuming damping to be the same in all modes and using flat spectra, maximum value of response in each mode has been worked out. The total response has been assumed to be equal to the root mean square value of the first three modes.

As far as the distribution of the shear forces and bending moments along the height is concerned, the formulae have been kept unchanged right from the first version of IS : 1893 and is based on Housner's paper (see Reference 4).

The Code provisions should not apply to those cases where some heavy concentrated masses may be located at some levels or to structures supported on stagings like silos.

The Code also assumes that foundation conditions are reasonably good and structure could be assumed to be fixed at the foundation level.

---

*Example 9* — A reinforced concrete chimney of height 28 m is located in seismic Zone V. The outer diameters at base and top are 3·0 m and 1·8 m, respectively. The wall thickness of the structural shell is 0·3 m throughout the height and the asbestos lining is 0·10 m thick. It is required to evaluate earthquake forces on this chimney. The modulus of elasticity of the structural material is $2·1 \times 10^6$ t/m².

a) *Period of Vibration*

Area of cross section A at the base

$$= \frac{\pi}{4} (D^2 - d^2)$$

$$= \frac{\pi}{4} (3^2 - 2·4^2) = 2·54 \text{ m}^2$$

Moment of inertia I at the base $= \frac{\pi}{64}(D^4 - d^4)$

$$= \frac{\pi}{64} (3·0^4 - 2·4^4) = 2·347 \text{ m}^4$$

Radius of gyration $r = \sqrt{\dfrac{I}{A}}$

$$= \sqrt{\dfrac{2·347}{2·54}} = 0·96 \text{ m}$$

Slenderness ratio $= L/r = \dfrac{28}{0·96} = 29·167$

Coefficient $C_T = 54·534$

Weight of Chimney:

Let the density of the shell and lining be 2·4 t/m³ and 2·0 t/m³, respectively.

Mean diameter of the shell $D_m$

$= \frac{1}{2}$ (mean diameter at top + mean diameter at base)

$= \frac{1}{2} (2·7 + 1·5) = 2·10$ m

The volume of the shell $= \pi \cdot D_m \cdot T \cdot h'$

$$= \pi \times 2 \cdot 10 \times 0 \cdot 3 \times 28$$

$$= 55 \cdot 39 \ m^3$$

Mean diameter of lining $= \frac{1}{2} (2 \cdot 3 + 1 \cdot 1) = 1 \cdot 7 \ m$

Volume of the lining $\quad = 1 \cdot 7 \times 0 \cdot 1 \times 28$

$$= 14 \cdot 95 \ m^3$$

Total weight of chimney

$$W_t = 55 \cdot 39 \times 2 \cdot 4 + 14 \cdot 95$$
$$\times 2 \cdot 0$$
$$= 162 \cdot 84 \ t$$

Period of vibration

$$T = C_T \sqrt{\frac{W_t \ h'}{E A \cdot g}}$$

$$= 54 \cdot 534 \sqrt{\frac{162 \cdot 84 \times 28}{2 \cdot 1 \times 10^6 \times 2 \cdot 54 \times 9 \cdot 81}}$$

$$= 0 \cdot 509 \ s$$

b) *Design Seismic Coefficient*

The acceleration coefficient $S_a/g$ corresponding to 5 percent damping is $0 \cdot 159$. The design seismic coefficient $\alpha_h = \beta \cdot I \cdot F_0 \ \dfrac{S_a}{g}$. For hard and medium soils for the type of foundation used $\beta = 1 \cdot 0$. Assuming that the chimney is the part of power house complex, the importance factor $I = 1 \cdot 5$. In Zone $V$, $F_0 = 0 \cdot 4$.

$$\alpha_h = 1 \cdot 0 \times 1 \cdot 5 \times 0 \cdot 4 \times 0 \cdot 159$$
$$= 0 \cdot 095 \ 4$$

c) *Shear Force and Bending Moment*

The design shear force $V$ at a distance $X'$ from the top is calculated by the following formula:

$$V = C_v \alpha_h \ W_t \left[ \frac{5}{3} \left( \frac{X'}{h'} \right) - \frac{2}{3} \left( \frac{X'}{h'} \right)^2 \right]$$

Coefficient $C_v = 1 \cdot 342$ (from **Table 5** of the Code)

$$V = 1 \cdot 342 \times 0 \cdot 095 \ 4 \times 162 \cdot 84 \left[ \frac{5}{3} \left( \frac{X'}{28} \right) \right.$$

$$\left. - \frac{2}{3} \left( \frac{X'}{28} \right)^2 \right]$$

$$= 20 \cdot 848 \ [0 \cdot 059 \ 5 \ X' - 0 \cdot 000 \ 85 \ (X')^2]$$

at $X' = 0$

$$V = 0$$

at $X' = 0 \cdot 2 \ h'$

$$V = 20 \cdot 848 \ (0 \cdot 333 \ 2 - 0 \cdot 026 \ 7) = 6 \cdot 39 \ t$$

at $X' = 0 \cdot 4 \ h'$

$$V = 20 \cdot 848 \ (0 \cdot 666 \ 2 - 0 \cdot 106 \ 62) = 11 \cdot 67 \ t$$

at $X' = 0 \cdot 6 \ h'$

$$V = 20 \cdot 848 \ (0 \cdot 999 \ 6 - 0 \cdot 229 \ 9) = 15 \cdot 84 \ t$$

at $X' = 0 \cdot 8 \ h'$

$$V = 20 \cdot 848 \ (1 \cdot 332 \ 8 - 0 \cdot 426 \ 5) = 18 \cdot 89 \ t$$

at $X' = h'$

$$V = 20 \cdot 848 \ (1 \cdot 666 - 0 \cdot 666 \ 4) = 20 \cdot 85 \ t$$

The design bending moment $M$ at a distance $X'$ from the top is calculated by the following formula:

$$M = \alpha_h \ W_t \ \bar{h} \left[ 0 \cdot 6 \left( \frac{X'}{h'} \right)^{1/2} + 0 \cdot 4 \left( \frac{X'}{h'} \right)^4 \right]$$

$$\bar{h} = 0 \cdot 458 \ h' = 12 \cdot 834 \ m$$

$$M = 0 \cdot 095 \ 4 \times 162 \cdot 84 \times 12 \cdot 834$$

$$\left[ 0 \cdot 6 \left( \frac{X'}{h'} \right)^{1/2} + 0 \cdot 4 \left( \frac{X'}{h'} \right)^4 \right]$$

$$= 199 \cdot 375 \left[ 0 \cdot 6 \left( \frac{X'}{h'} \right)^{1/2} + 0 \cdot 4 \left( \frac{X'}{h'} \right)^4 \right]$$

at $X' = 0$

$$M = 0$$

at $X' = 0 \cdot 2 \ h'$

$$M = 199 \cdot 375 \ [0 \cdot 6 \ (0 \cdot 2)^{1/2} + 0 \cdot 4 \ (0 \cdot 2)^4]$$
$$= 199 \cdot 375 \ (0 \cdot 268 \ 3 + 0 \cdot 000 \ 64)$$
$$= 53 \cdot 62 \ t \cdot m$$

at $X' = 0 \cdot 4 \ h'$

$$M = 199 \cdot 375 \ [0 \cdot 6 \ (0 \cdot 4)^{1/2} + 0 \cdot 4 \ (0 \cdot 4)^4]$$
$$= 199 \cdot 375 \ (0 \cdot 379 \ 5 + 0 \cdot 010 \ 24)$$
$$= 77 \cdot 70 \ t.m$$

at $X' = 0 \cdot 6 \ h'$

$$M = 199 \cdot 375 \ [0 \cdot 6 \ (0 \cdot 6)^{1/2} + 0 \cdot 4 \ (0 \cdot 6)^4]$$
$$= 199 \cdot 375 \ (0 \cdot 464 \ 8 + 0 \cdot 051 \ 84)$$
$$= 103 \cdot 00 \ t.m$$

at $X' = 0 \cdot 8 \ h'$

$$M = 199 \cdot 375 \ [0 \cdot 6 \ (0 \cdot 8)^{1/2} + 0 \cdot 4 \ (0 \cdot 8)^4]$$
$$= 199 \cdot 375 \ (0 \cdot 536 \ 7 + 0 \cdot 163 \ 8)$$
$$= 139 \cdot 67 \ t.m$$

at $X' = 1.0\, h'$

$M = 199.375\, [0.6\, (1.0)^{1/2} + 0.4\, (1.0)^4]$

$= 199.37$ t.m

## 6. BRIDGES

**6.1.1** The safety of bridge as a whole and that of its individual components, such as superstructure, bearings, piers and foundation is, important during an earthquake. The two directions critical for the earthquake resistant design are the axis along the length of the bridge and perpendicular to the axis of the bridge. The horizontal seismic force is assumed to act in one of these directions at a time. For the purpose of design, bridge could be divided into several elements, such as superstructure, bearings, piers, etc. The seismic force equal to weight of the element times the design seismic coefficient should be applied at the centre of gravity of the element. The application of seismic force to various elements in the two perpendicular directions of the bridge is shown in Fig. 10A and 10B.

**6.1.2** The damage study of bridges during past earthquakes has clearly shown that masonry arch bridges are more vulnerable to damage due to brittleness of material. Further, the failure of one of the continuous arch spans causes the failure in other spans. Therefore, construction of masonry arch bridges of span more than 10 m are not recommended in Zones IV and V. Slab, box and pipe culverts are usually not designed for earthquake force. Bridges of length smaller than 60 m and spans less than 15 m are less susceptible to damage due to minor shocks and should, therefore, be designed for earthquake forces only in severe Zones IV and V. Lengths and spans mentioned
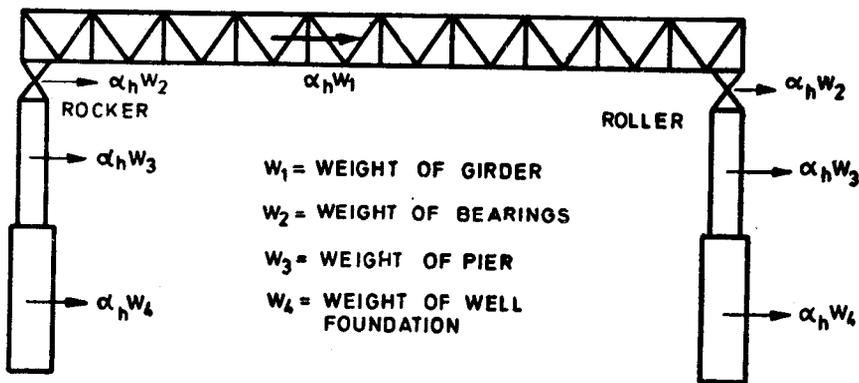


FIG. 10A    SEISMIC FORCE ON BRIDGE IN DIFFERENT COMPONENTS IN LONGITUDINAL DIRECTION
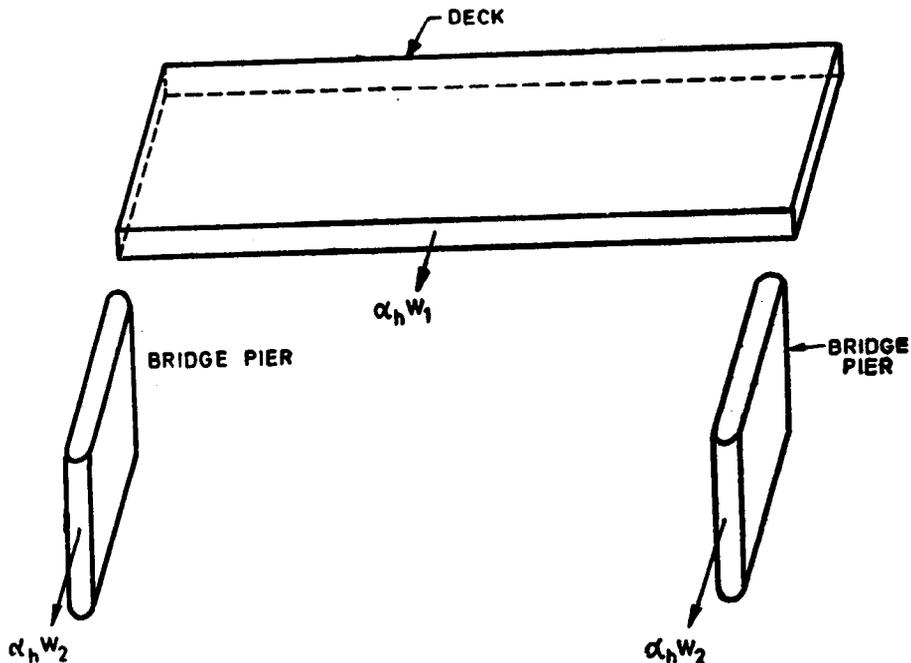


FIG. 10B    SEISMIC FORCE ON BRIDGE COMPONENTS IN TRANSVERSE DIRECTION

above are more or less arbitrary taking into consideration the observations during past earthquakes.

**6.1.3** The special types of bridges, such as suspension bridge, bascule bridge, cable stayed bridge, horizontally curved bridge, reinforced concrete arch bridge or steel bridge and the bridges having piers taller than 30 m and spans more than 120 m, are susceptible to damage due to structural vibrations. It is, therefore, recommended that dynamic behaviour of bridge in such cases must be taken into account. The modal analysis should be carried out particularly for bridges in Zones IV and V. All other bridges should be designed by seismic coefficient method.

**6.1.4** Earthquake force shall be calculated on the basis of depth of scour caused by the discharge corresponding to mean annual flood. The discharge corresponding to mean annual flood should be worked out by taking the average of maximum yearly flood discharges. The design value of earthquake and maximum flood shall be assumed not to occur simultaneously.

**6.2** The significant point to be noted in this Clause is regarding the weight $W_m$. The reductions due to buoyancy or uplift are neglected because buoyancy is a force. The inertia forces are not calculated on a force, these are calculated only on mass.

**6.3.1** The live load here implies vehicular traffic. The seismic force acting on the live load is ignored in the direction of traffic because such a force would cause the wheels to roll and very little or no inertia force would be transmitted to the substructure on this account. Since the bridges are designed for longitudinal force due to braking effects, it is assumed that any inertia force transmitted to the substructure in the traffic direction would be covered by the design for longitudinal braking force. Since the rolling of wheels is not possible in transverse direction, the seismic effects on live load should be included in this direction.

**6.3.2** The amount of design live load to be considered for seismic force evaluation on live load is dependent upon the probability of its presence on the bridge at the time of an earthquake. It is assumed that 100 percent of design live load may be present in railway bridges and 50 percent of design live load may be present on road bridges at the instant of earthquake. These figures are based on probabilistic considerations and should be used for computing stresses due to live loads when combined with earthquake effects. The seismic force on live load is recommended to be considered on 50 percent of the design live load at the time of earthquake. The reduction in design live load for seismic load evaluation is based on the fact that the live load is friction supported on the deck or on the rail and only partial effect

could be transmitted to the supporting structure. In view of this, a factor of 50 percent of design live load (without impact) for railway bridges and 25 percent of design live load (without impact) for road bridges for the purpose of seismic load evaluation on live load in transverse direction (current direction) is recommended.

**6.4.1** Besides horizontal seismic coefficient, the superstructure should also be designed for vertical seismic force. The vertical seismic force has caused jumping of girders in some cases and can cause additional forces and deflection in the girder particularly for longer spans. The vertical seismic force should also be considered on 100 percent design live load (without impact) for railway bridges and 50 percent of design live load (without impact) for road bridges.

**6.4.2** The proper securing of superstructure with the substructure through the bearings is required due to following reasons:

a) The anchor bolts can shear off during an earthquake. In order to prevent such action, anchor bolts should be designed for horizontal seismic force on the girder.

b) The rollers can be dislodged off their bearings during an earthquake. In order to prevent such an action connecting plates or segmental rollers or stoppers should be used. To avoid the jumping of girder, special hooks must be employed which connect the top and the bottom shoe. A typical arrangement for roller bearing is shown in Fig. 11.

**6.4.3** Overturning of girders in the transverse direction is one of the possible modes of failure of superstructure during an earthquake. Therefore, it is recommended that a factor of safety of 1·5 against overturning should be ensured for superstructure. The vertical seismic force should also be considered as explained in **6.4.1** while considering stability. The direction of vertical seismic force should be such that it gives the worst effect.

**6.5.1** a) The transfer of forces from superstructure to substructure through the bearings is worked out in example 10 given below:

*Example* 10 — (*see* Fig. 7A of the Code)

$W' = 800$ t, $e = 6·8$ m, $\mu = 0·3$,

$\alpha_h = 0·05$

$L' = 90$ m, $\alpha_v = 0·0$

Horizontal seismic force on girder
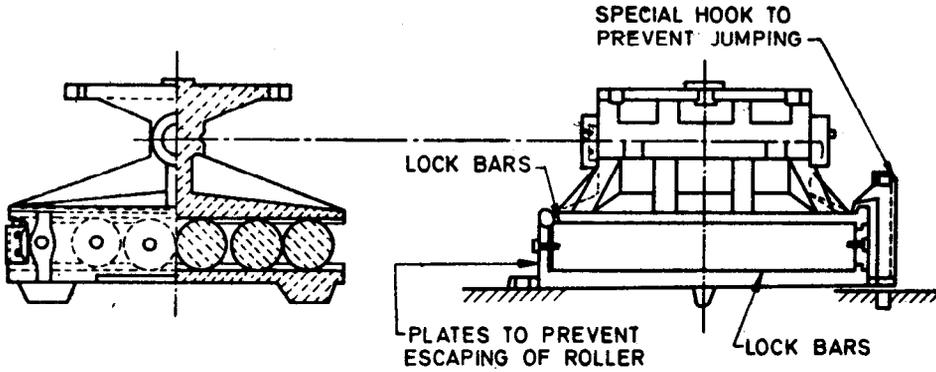
$F' = \alpha_h W' = 0·05 \times 800 = 40$ t

FIG. 11 DESIGN OF ROLLER BEARINGS FOR PREVENTING ESCAPING OF ROLLERS

Change in vertical reactions

$$= \pm \frac{F'e}{L'} = \pm \frac{40 \times 6 \cdot 8}{90} = \pm 3 \cdot 022 \text{ t}$$

$R_1 = 400 \pm 3 \cdot 022 = 403 \cdot 022 \text{ t and } 396 \cdot 978 \text{ t}$

$\mu R_1 = 120 \cdot 90 \text{ t and } 119 \cdot 09 \text{ t}$

$F'/2 = 20 \text{ t}$

$F_1 = F'/2 = 20 \text{ t since } \mu R_1 > F'/2$

$F_2 = F' - F_1 = 40 - 20 = 20 \text{ t}$

If $\alpha_v = 0 \cdot 025$, there will be further change in vertical reaction $= \pm 0 \cdot 025 \times 800 = \pm 20$ t. With this change in vertical reaction included, $\mu R_1$ will be still greater than $F'/2$. Therefore, $F_1 = F_2 = 20$ t in this case also.

b) Horizontal and vertical seismic forces on the piers and wells should be applied at the centre of masses, respectively. The effects of buoyancy and uplift should be neglected as these are reactive forces.

**6.5.2** The additional dynamic force in the form of hydrodynamic force arises on the submerged portion of the piers. This force is primarily due to the vibration of certain mass of water surrounding the pier along with the pier and this is assumed to act in the direction of seismic force. The computation of hydrodynamic force is based on cylinder analogy. For the purpose of simplifying the analysis, this analogy assumes that certain portion of water in the enveloping cylinder formed for the plan of pier acts along with the mass of the pier and gives rise to hydrodynamic force. The circular cross-sectional area to be adopted for enveloping cylinder is dependent upon orientation of pier with respect to earthquake force. Some typical cases of enveloping cylinders are illustrated in Fig. 9 of the Code.

**6.5.2.1** Example 11 below demonstrates the method of calculating hydrodynamic force and hydrodynamic pressure distribution for substructure of a bridge.

*Example* 11 — Calculation of hydrodynamic force and pressure distribution on bridge substructure (*see* Fig. 12).

For pier portion, $\dfrac{h_1}{r_e} = \dfrac{15 \cdot 24}{1 \cdot 2}$

$= 12 \cdot 7 > 4 \cdot 0 \ (C_e = 0 \cdot 73)$

For well portion, $\dfrac{h_2}{r_e} = \dfrac{30 \cdot 55}{2 \cdot 75}$

$= 11 \cdot 09 > 4 \cdot 0 \ (C_e = 0 \cdot 73)$

$\alpha_h = \alpha_0 \beta I, \ \alpha_h = 0 \cdot 08 \times 1 \cdot 5 \times 1 \cdot 5 = 0 \cdot 18$

Weight of water in enveloping cylinder for pier

$$W_{e1} = \frac{\pi}{4} \times 2 \cdot 4^2 \times 15 \cdot 24 \times 1 = 69 \cdot 0 \text{t}$$

Weight of water in enveloping cylinder for well

$$W_{e2} = \frac{\pi}{4} \times 5 \cdot 5^2 \times 30 \cdot 55 \times 1 = 726 \cdot 0 \text{ t}$$

Hydrodynamic force on pier,

$F_1 = 0 \cdot 73 \times 0 \cdot 18 \times 69 \cdot 0 = 9 \cdot 06 \text{ t}$

Hydrodynamic force on well,

$F_2 = 0 \cdot 73 \times 0 \cdot 18 \times 726 = 95 \cdot 50 \text{ t}$

$F = F_1 + F_2 = 9 \cdot 06 + 95 \cdot 5 = 104 \cdot 56 \text{ t}$

For pier,

$$p_{b1} = 1 \cdot 2 \frac{F_1}{h_1} = 1 \cdot 2 \times \frac{9 \cdot 06}{15 \cdot 24} = 0 \cdot 714 \text{ t/m}$$

The pressure distribution for the pier is worked out and is presented in Table 11.

For well, $P_{b2} = 1 \cdot 2 \times \dfrac{F_2}{h_2}$

$$= 1 \cdot 2 \times \frac{95 \cdot 5}{30 \cdot 55} = 3 \cdot 75 \text{ t/m}$$

The pressure distribution for the well portion is worked out and is presented in Table 12.

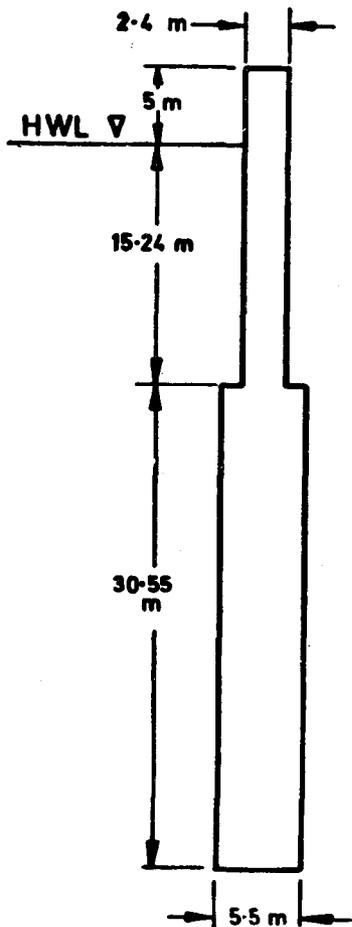The pressure distribution is shown in Fig. 13.

FIG. 12 BRIDGE-SUBSTRUCTURE



FIG. 13 HYDRODYNAMIC PRESSURE DISTRIBUTION ON BRIDGE STRUCTURE

NOTE — In actual practice earthquake forces would be calculated above scour depth in accordance with 6.1.4 of the Code.

TABLE 11 PRESSURE DISTRIBUTION ON PIER

| $C_1$ | $C_1h_1$ | $C_2$ | $C_2P_{b_1}$ (t/m) |
|------|--------|------|--------------|
| 0·1 | 1·524 | 0·410 | 0·292 |
| 0·2 | 3·048 | 0·673 | 0·480 |
| 0·3 | 4·572 | 0·832 | 0·594 |
| 0·4 | 6·096 | 0·922 | 0·658 |
| 0·5 | 7·620 | 0·970 | 0·692 |
| 0·6 | 9·144 | 0·990 | 0·707 |
| 0·8 | 12·192 | 0·999 | 0·713 |
| 1·0 | 15·240 | 1·000 | 0·714 |

TABLE 12 PRESSURE DISTRIBUTION IN WELL PORTION

| $C_1$ | $C_1h_2$ | $C_2$ | $C_2P_{b_2}$ (t/m) |
|------|--------|------|--------------|
| 0·1 | 3·055 | 0·410 | 2·252 |
| 0·2 | 6·110 | 0·673 | 3·234 |
| 0·3 | 9·165 | 0·832 | 3·834 |
| 0·4 | 12·220 | 0·922 | 4·172 |
| 0·5 | 15·275 | 0·970 | 4·352 |
| 0·6 | 18·330 | 0·990 | 4·427 |
| 0·8 | 24·440 | 0·999 | 4·460 |
| 1·0 | 30·550 | 1·000 | 4·464 |

*Example* 12 — The pier well of a submersible bridge is shown in Fig. 14.

Design horizontal seismic coefficient

$$\alpha_h = 0·18$$

Hydrodynamic pressures at $A$, $B$, and $C$ are given below:

$$P_A = 875 \times 0·18 \sqrt{36·09 \times 5·00}$$
$$= 2\,115·7 \text{ kg/m}^2$$

**6.6 Submersible Bridges** — Example 12 demonstrates the method of calculation of hydrodynamic pressure in a submersible bridge.

30
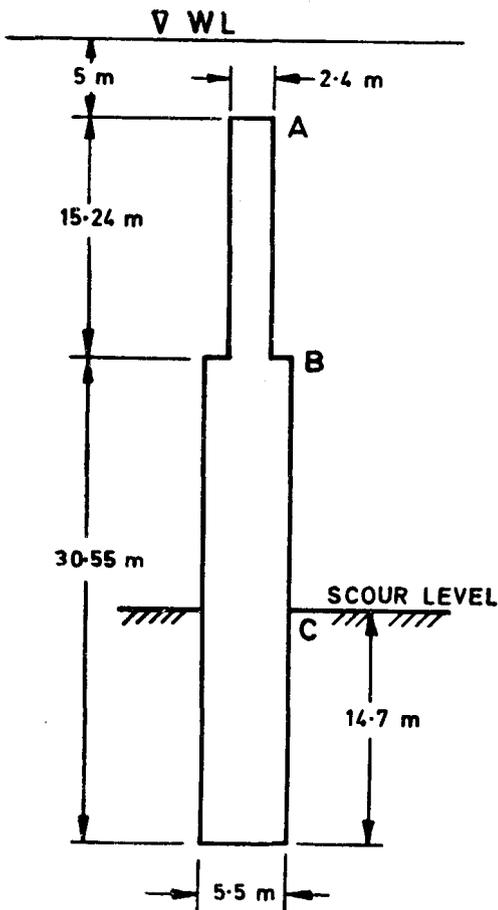
FIG. 14 SUBSTRUCTURE FOR SUBMERSIBLE BRIDGE

$$P_B = 875 \times 0.18 \sqrt{36.09 \times 20.24}$$
$$= 4\ 256.76 \text{ kg/m}^2$$

$$P_C = 875 \times 0.18 \sqrt{36.09 \times 36.09}$$
$$= 5\ 684.18 \text{ kg/m}^2$$

Total horizontal shear and moment per metre width at $A$, $B$ and $C$ due to hydrodynamic pressure are as follows:

$$V_A = \frac{2}{3} \times 2\ 115.72 \times 5 = 7\ 052.40 \text{ kg/m}$$

$$M_A = \frac{4}{15} \times 2\ 115.72 \times 5^2$$
$$= 14\ 104.80 \text{ kgm/m}$$

$$V_B = \frac{2}{3} \times 4\ 256.76 \times 20.24$$
$$= 57\ 437.88 \text{ kg/m}$$

$$M_B = \frac{4}{15} \times 4\ 256.76 \times 20.24^2$$
$$= 465\ 017.06 \text{ kgm/m}$$

$$V_C = \frac{2}{3} \times 5\ 684.18 \times 36.09$$
$$= 136\ 761.35 \text{ kg/m}$$

$$M_C = \frac{4}{15} \times 5\ 684.18 \times 36.09^2$$
$$= 1\ 974\ 287.10 \text{ kgm/m}$$

**7.3 Gravity Dams** — At the time of an earthquake, besides the normal forces, namely, self weight, water pressure and silt or earth pressure, inertia and hydrodynamic forces act on a dam.

The inertia force is the product of mass and acceleration and this force acts in the direction opposite to that of the ground motion. The horizontal inertia force acts from upstream to down stream as well as from down stream to upstream. Similarly the vertical inertia force acts from downwards as well as from upwards to downwards direction. An acceleration downwards decreases the weight. The dam is designed for the worst combination that is for the horizontal and vertical inertia forces.

The interaction effect between the dam and reservoir causes the hydrodynamic forces on the dam. The direction of hydrodynamic force is opposite to the direction of earthquake acceleration but the acceleration changes sign in practice. Hence, the force could be either pressure or suction. Zanger's method (Reference 5) is generally adopted in design practice to find the hydrodynamic forces and it is based on the following assumptions:

a) The fluid is incompressible, and

b) The dam is rigid and hence has the same motion throughout its body as that of the base of the dam.

The dynamic behaviour or dam-reservoir system is usually evaluated treating the dam and reservoir as two uncoupled systems, namely, dynamic response of the dam ignoring the effect of reservoir water and hydrodynamic pressure on the dam to represent the effect of reservoir water. It is invariably assumed that the interaction effects between the dam and reservoir are small so that the solutions of the uncoupled system can be combined to obtain a complete solution for the response of the dam.

For determination of earthquake forces on dams, two methods are suggested, (a) one based on seismic coefficient method, and (b) the other on response spectrum method. The seismic coefficient method has been suggested for dams up to 100 metres in height and the response spectrum method has to be used for dams above this height. The idea of choosing a value of 100 meters is that most of the dams up to this height will have periods shorter than 0.3 s and the spectral acceleration may not be influenced by change of period in

this range. Further, dams beyond 100 metres in height may be thought to be more important requiring more sophisticated analysis.

In the seismic coefficient method, the horizontal acceleration is assumed to have the shape of an inverted triangle. The value at the top has been taken as 1·5 times the seismic coefficient $\alpha_h$. If one takes the shape of the dam as triangular and the distribution of acceleration as an inverted triangle, the base shear would be equal to $0.5\ W\alpha_h$ (see Fig. 15), but since the section of the dam is not a triangle, the base shear tends to be $0.6\ W\alpha_h$.

$$V_y = \int \left(\frac{B}{H}\ y\ \right)(1 - y/H)\ 1.5\ \alpha_h \cdot w_m\ dy$$

$$= 1.5\ \alpha_h \cdot \frac{B}{H}\left(\frac{y^2}{2} - \frac{y^3}{3H}\right)w_m$$

$$V_H = 0.25\ \alpha_h.\ BH\ w_m = 0.5\ \alpha_h \cdot W$$

This compares well with the base shear coefficient given in response spectrum method

$$V_B = 0.6\ W\alpha_h$$

The various formulae given in the response spectrum method are based on research work carried out at Roorkee University. (Reference 6 and 7).

*Vertical Earthquake*

In both the methods, the vertical acceleration has been assumed to have a shape of an inverted triangle with the value at the top of dam being equal to half the horizontal acceleration. This

value of half implies that the vertical intensity of earthquake is equal to half the horizontal intensity. However, in the case of the response spectrum method this value may not be exactly half because of the different definitions of the term $\alpha_h$.

An example illustrating the pertinent clauses of the Code is given below:

*Example* 13 — A 100 metre high concrete gravity dam, as shown in Fig. 16 is located in seismic Zone *V*. Unit weight of the material of dam is 2·5 t/m³ and modulus of elasticity of the material is $2.1 \times 10^6$ t/m². Work out the earthquake forces by seismic coefficient method and as well as by response spectrum method for a unit length of dam.

a) *Seismic Coefficient Method* — Horizontal seismic coefficient $\alpha_h^T$ at the top of the dam, is given by

$$\alpha_h^T = 1.5\beta I\alpha_0\ \text{[from clause \textbf{3.4.2.3} (a)]}$$

$$\beta = 1.0,$$

$$I = 2.0,$$

$$\alpha_0 = 0.08,$$

$$\alpha_h^T = 1.5 \times 1.0 \times 2.0 \times 0.08 = 0.24.$$

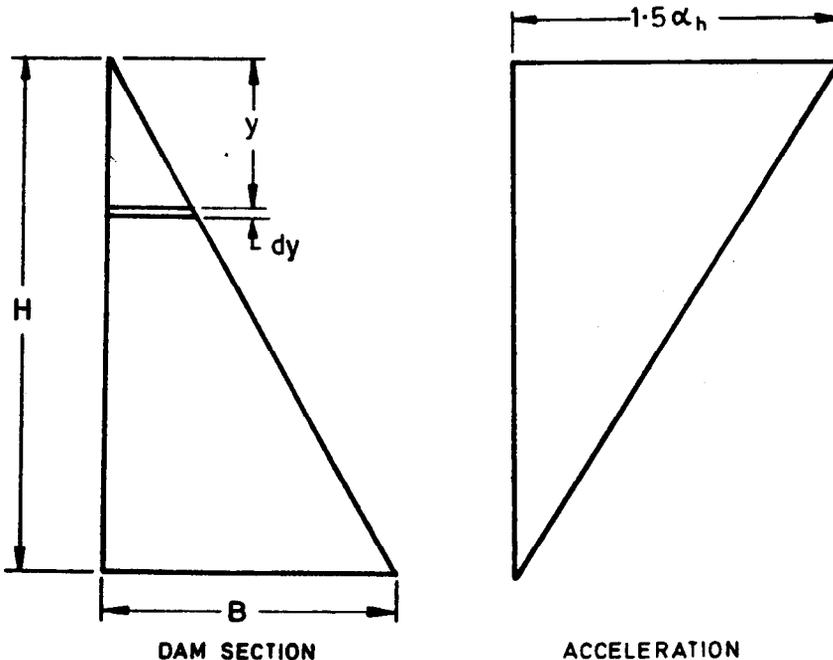It varies linearly from a value of 0·24 at the top to zero at the base.
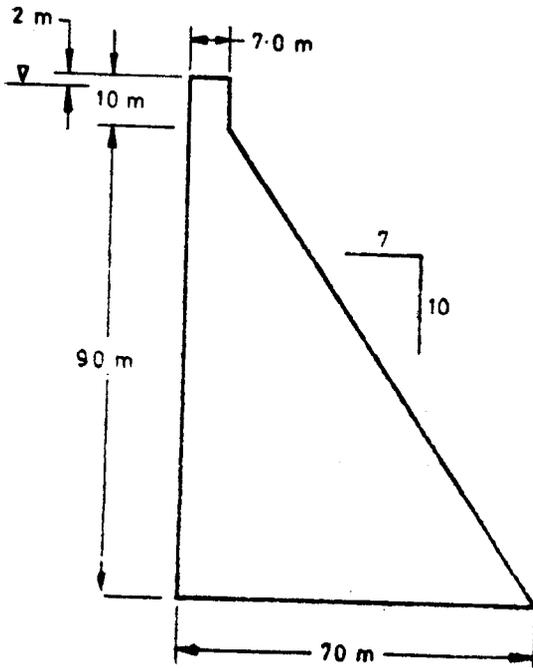


FIG. 15 DAM SECTION WITH ACCELERATION

FIG. 16 SECTION OF A CONCRETE DAM

**At 10 m from top**

$W_s$ = weight above this level for a unit length and height $dx$

$$= 1 \times 7 \times 2.5 \times dx = 17.5\, dx$$

Shear force

$$= \int_0^{10} \alpha_s \cdot W_s \quad \text{where } \alpha_s = (1 - 0.01x)\, 0.24$$

$$V_{10} = \int_0^{10} (1 - 0.01x)\, (0.24)\, (17.5)\, dx = 39.9 \text{ t}$$

Moment $M_{10} = \int_0^{10} \alpha_s \cdot W_s\, (10 - x)$

$$= \int_0^{10} (1 - 0.01x)\, 0.24 \times 17.5\, (10 - x)\, dx$$

$$= 203.0 \text{ t.m}$$

**At the base,**

$W_s$ = weight for height $dx$ (between 10 m and base)

$$= 1\,[7 + 0.7\,(x - 10)]\, dx\, 2.5$$
$$= 0.7\, x \times 2.5\, dx$$
$$= 1.75\, x\, dx$$

$$V_{100} = V_{10} + \int_{10}^{100} (1 - 0.01x)\, (0.24)\, 1.75\, dx$$

$$= 39.9 + 680.4 = 720.3 \text{ t}$$

$$M_B = M_{10} + V_{10} \times 90 + \int_{10}^{100} (1 - 0.01x)$$

$$\times (0.24)\, (1.75x)\, (100 - x)\, dx$$

$$= 602 + 33\,169.5$$

$$= 33\,771.5 \text{ t.m}$$

b) *Response Spectrum Method*

i) The fundamental period of vibration of the dam is given by

$$T = 5.55 \frac{H^2}{B} \sqrt{\frac{W_m}{g E_s}}$$

$$= 5.55 \frac{(100)^2}{70} \sqrt{\frac{2\,500}{9.81 \times 2\,100 \times 10^6}}$$

$$= 0.278 \text{ s}$$

ii) *Design seismic coefficient*

$$\alpha_h = \beta I\, F_0 \frac{S_a}{g}$$

Values of $\beta$ and $I$ are same as taken in previous case.

$F_0 = 0.4$ and $\dfrac{S_a}{g} = 0.19$, for 5 percent damping

$$\alpha_h = 1.0 \times 2.0 \times 0.4 \times 0.19 = 0.152$$

iii) *Base shear*

$$V_B = 0.6\, W\, \alpha_h = 0.6 \times 1 \times 2.5$$

$$\times \left( 7 \times 100 + \frac{63 \times 90}{2} \right) \times 0.152$$

$$= 805.98 \text{ t}$$

iv) *Base moment*

$$M_B = 0.9\, W \cdot \bar{h} \cdot \alpha_h$$

$$\bar{h} = \frac{\left( 7 \times 100 \times \dfrac{100}{2} + \dfrac{1}{2} \times 63 \times 90 \times \dfrac{90}{3} \right)}{7 \times 100 + \frac{1}{2} \times 63 \times 90}$$

$$= 33.96 \text{ m}$$

$$M_B = 0.9 \times 8\,837.5 \times 33.96 \times 0.152$$

$$= 41\,056.62 \text{ t.m}$$

v) Shear at depth $y$ from top is given by

$$V_y = C'_y\, V_B$$

At 10 m from top

$$C'_v = 0.08, \quad V_{10} = 0.08 \times 805.98$$

$$= 64.48 \text{ t}$$

vi) Moment at depth $y$ from top is given by

$$M_y = C'_m M_B$$

At 10 m from top

$$C'_m = 0.02, M_{10} = 0.02 \times 41\ 056.62$$

$$= 821.13 \text{ t.m}$$

It is seen that seismic coefficient method gives somewhat lower values at the base but considerably lower values near the top. This is due to the load diagram which is specified in first method, whereas in the second method shears and moments are specified.

*Hydrodynamic Pressure (or Suction)*

a) The hydrodynamic pressure at depth $y$ below the reservoir surface is given by,

$$p = C_s \cdot \alpha_h \cdot w.h$$

$$C_s = \frac{C_m}{2}\left\{\frac{y}{h}\left(2 - \frac{y}{h}\right) + \sqrt{\frac{y}{h}\left(2 - \frac{y}{h}\right)}\right\}$$

$C_m = 0.735$ (from Fig. 10 for $\theta = 0°$)

$$C_s = \frac{0.735}{2}\left\{\frac{y}{98}\left(2 - \frac{y}{98}\right)\right.$$

$$\left. + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right\}$$

$$p = \frac{0.735}{2}\left\{\frac{y}{98}\left(2 - \frac{y}{98}\right)\right.$$

$$\left. + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right\} \times 0.152 \times 1 \times 98$$

$$= 5.474\ 3\left\{\frac{y}{98}\left(2 - \frac{y}{98}\right)\right.$$

$$\left. + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right\}$$

At depth 8 m from reservoir level or 10 m from top dam

$$p_{10} = 3.023 \text{ t/m}^2$$

At depth 18 m from reservoir level

$$p_{20} = 4.99 \text{ t/m}^2$$

At depth 38 m from reservoir level

$$p_{40} = 7.75 \text{ t/m}^2$$

At depth 58 m from reservior level

$$p_{60} = 9.56 \text{ t/m}^2$$

At depth 78 m from reservoir level

$$p_{80} = 10.60 \text{ t/m}^2$$

At depth 98 m from reservoir level

$$p_{100} = 10.95 \text{ t/m}^2$$

b) Horizontal shear at any depth $y$ from reservoir level due to hydrodynamic pressure is given by,

$$V_h = 0.726\ py$$

At depth 8 m from reservoir level or 10 m from top of dam

$$V_{10} = 0.726 \times 3.023 \times 8 = 17.56 \text{ t/m}$$

At depth 18 m from reservoir level

$$V_{20} = 0.726 \times 4.99 \times 18 = 65.21 \text{ t/m}$$

At depth 38 m from reservoir level

$$V_{40} = 0.726 \times 7.751 \times 38$$

$$= 213.84 \text{ t/m}$$

At depth 58 m from reservoir level

$$V_{60} = 0.726 \times 9.56 \times 58 = 402.55 \text{ t/m}$$

At depth 78 m from reservoir level

$$V_{80} = 0.726 \times 10.606 \times 78$$

$$= 600.60 \text{ t/m}$$

At depth 98 m from reservoir level

$$V_{100} = 0.726 \times 10.949 \times 98$$

$$= 778.99 \text{ t/m}$$

c) Moment at any depth $y$ from reservoir level due to hydrodynamic pressure is given by,

$$M_h = 0.299\ py^2$$

At depth 8 m from reservoir level or 10 m from the top of the dam

$$M_{10} = 0.299 \times 3.023 \times (8)^2$$

$$= 57.85 \text{ t.m/m}$$

At depth 18 m from reservoir level

$$M_{20} = 0.299 \times 4.99 \times (18)^2$$

$$= 483.41 \text{ t.m/m}$$

At depth 38 m from reservoir level

$$M_{40} = 0.299 \times 7.751 \times (38)^2$$

$$= 3\ 346.54 \text{ t.m/m}$$

At depth 58 m from reservoir level

$$M_{60} = 0.299 \times 9.56 \times (58)^2$$

$$= 9\ 615.79 \text{ t.m/m}$$

At depth 78 m from reservoir level

$$M_{80} = 0.299 \times 10.606 \times (78)^2$$
$$= 19\ 293.54 \text{ t.m/m}$$

At depth 98 m from reservoir level

$$M_{100} = 0.299 \times 10.949 \times (98)^2$$
$$= 31\ 441.10 \text{ t.m/m}$$

*Example* 14 — A 100 m high concrete dam as shown in Fig. 17 is located in seismic Zone V, has to resist pressure from 98 m depth of water. Find the hydrodynamic pressure at base and at depth 10 m from top of the dam. Take value of $\alpha_h$ as 0.152.

a) The hydrodynamic pressure at any depth $y$ below the reservoir surface is given by,

$$p = C_s.\alpha_h.w \cdot h$$

here $C_s = \dfrac{C_m}{2}\left\{\dfrac{y}{h}\left(2 - \dfrac{y}{h}\right) + \sqrt{\dfrac{y}{h}\left(2 - \dfrac{y}{h}\right)}\right\}$

The height of the vertical portion of the upstream face of the dam is less than one half the total height of dam. Therefore, modified slope for design purpose is obtained by joining the point of intersection of the upstream face of the dam and reservoir surface with the point of intersection of upstream face of the dam and the foundation

$$\theta = \tan^{-1}\left(\frac{4.5}{98}\right) = 2.63°$$

$C_m = 0.725$ (from Fig. 10 of the Code)

$$C_s = \frac{0.725}{2}\left\{\frac{y}{98}\left(2 - \frac{y}{98}\right) + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right\}$$

$$p = \frac{0.725}{2}\left\{\frac{y}{98}\left(2 - \frac{y}{98}\right) + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right\} \times 0.152 \times 1 \times 98$$

$$= 5.399\ 8\left\{\frac{y}{98}\left(2 - \frac{y}{98}\right) + \sqrt{\frac{y}{98}\left(2 - \frac{y}{98}\right)}\right\}$$

At depth 8 m from reservoir level or 10 m from top of dam

$$p_{10} = 2.982 \text{ t/m}^2$$

At depth 48 m from reservoir level

$$p_{50} = 8.638 \text{ t/m}^2$$

At depth 98 m from reservoir level

$$p_{100} = 10.8 \text{ t/m}^2$$

b) Horizontal shear at any depth $y$ from reservoir level due to hydrodynamic pressure is given by
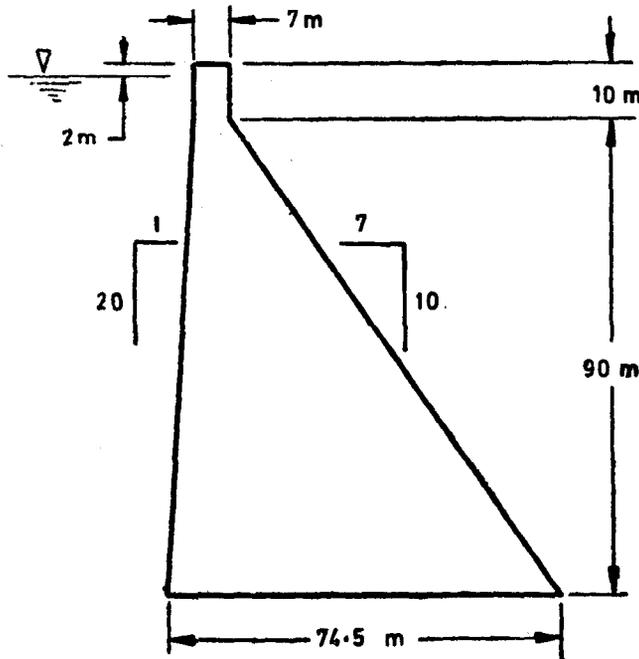
$$V_h = 0.726\ py$$



FIG. 17 A SECTION OF A CONCRETE DAM

At depth 8 m from reservoir level or 10 m from top of dam

$$V_{10} = 0.726 \times 2.98 \times 8 = 17.32 \text{ t/m}$$

At depth 48 m from reservoir level

$$V_{50} = 0.726 \times 8.638 \times 48$$
$$= 301.02 \text{ t/m}$$

At depth 98 m from reservoir level

$$V_{100} = 0.726 \times 10.8 \times 98$$
$$= 768.40 \text{ t/m}$$

c) Moment at any depth $y$ from reservoir level due to hydrodynamic pressure is given by,

$$M_h = 0.299 \, py^2$$

At depth 8 m from reservoir level or 10 m from top of dam

$$M_{10} = 0.299 \times 2.982 \times (8)^2 = 57.06 \text{ t.m/m}$$

At depth 48 m from reservoir level

$$M_{50} = 0.299 \times 8.638 \times (48)^2 = 5\,950.68 \text{ t.m/m}$$

At depth 98 m from reservoir level

$$M_{100} = 0.299 \times 10.8 \times (98)^2 = 31\,013.26 \text{ t.m/m}$$

*Effect of Horizontal Earthquake Coefficient on the Vertical Component of Reservoir*

Since the hydrodynamic pressure acts normal to the face of the dam, there will be a vertical component of this force, if the face of the dam against which it is acting is sloping. The magnitude of this load at any horizontal section is given by

$$W_h = (V_2 - V_1) \tan \theta$$

where

$W_h$ = increase or decrease in vertical component of load due to hydrodynamic force,

$V_2$ = total shear due to horizontal component of hydrodynamic force at the elevation of the section being considered,

$V_1$ = total shear due to horizontal component of hydrodynamic force at the elevation at which the slope of the dam face commences, and

$\theta$ = angle between the face of the dam and vertical.

At the depth 10 m from top of dam from where the upstream slope commences, $W_{10} = 0.0$ t

At depth 50 m from top of dam

$$V_2 = 301.02 \text{ t}$$
$$V_1 = 17.32 \text{ t}$$
$$\tan\theta = 0.05$$
$$W_{50} = (301.017 - 17.319) \, 0.05 = 14.18 \text{ t}$$

At depth 100 m from top of dam

$$V_2 = 768.40$$

$$V_1 = 17.32 \text{ t}$$
$$\tan\theta = 0.05$$
$$W_{100} = (768.398 - 17.319) \, 0.05 = 37.55 \text{ t}$$

*Effect of Vertical Earthquake Acceleration*

a) *Seismic Coefficient Method*

Vertical Seismic Coefficient $\alpha_v^T$ at the top of the dam is taken as,

$$\alpha_v^T = 0.75 \, \alpha_h^T$$

where

$$\alpha_h^T = \beta I \alpha_0$$

Here $\beta$, $I$ and $\alpha_0$ have the same values as used earlier.

$$\alpha_h^T = 1.0 \times 2.0 \times 0.08 = 0.16$$

$$\alpha_v^T = 0.75 \times 0.16 = 0.12$$

It varies linearly from a value of 0.12 at the top to zero at the base.

At 10 m from top

$W_s$ = weight above this level for a unit length, and height $dx = 1 \times 7 \times 2.5 \times dx = 17.5 \, dx$

Increase or decrease in weight

$$= \int_0^{10} \alpha_s \cdot W_s \cdot dx \quad \text{(where } \alpha_s = (1 - 0.01x) \, 0.12)$$

$$= \int_0^{10} (1 - 0.01 \, x) \, (0.12) \, (17.5) \, dx = 19.95 \text{ t}$$

At the base,

$W_s$, weight for height $dx$ (between 10 m and base)

$$= 1 \times \{7 + 0.7 \, (x - 10)\} \, dx \, 2.5$$
$$= 1.75 \, x \times dx$$

Increase or decrease in weight

$$= 19.95 \int_{10}^{100} (1 - 0.1x) \, (0.12) \, 1.75x. \, dx$$

$$= 19.95 + 340.2 = 360.15 \text{ t}$$

b) *Response Spectrum Method*

$$\alpha_h = 0.152 \text{ (as taken earlier)}$$

$$\alpha_v^T = 0.75 \times 0.152 = 0.114$$

It varies from a value of 0.114 at the top to zero at base.

At 10 m from top

Increase or decrease in weight

$$= \int_0^{10} \alpha_x \cdot W_x \cdot dx$$

(where $\alpha_x = (1 - 0.01x)\, 0.114$)

$$= \int_0^{10} (1 - 0.01x)\, 0.114 \times 17.5\; dx$$

$$= 18.95\; t$$

At the base

Increase or decrease in weight

$$= 18.95 + \int_{10}^{100} (1 - 0.01x)\, 0.114 \times 1.75\; dx$$

$$= 18.95 + 323.19$$

$$= 342.14\; t$$

**7.4 Earth and Rockfill Dams and Embankments** — The dynamic analysis of a dam should be directed towards determining the deformations that an earthquake will cause. Once this is known, a design to prevent catastrophic failure is possible. The concept of a factor of safety is sufficient and reliable for design when earthquake forces are absent because factor of safety less than unity will mean failure along the critical slip surface. However, under the dynamic loads, as during an earthquake, the factor of safety greater than unity does not guarantee safety against failure while a value less than unity does not necessarily mean failure. The reasons for this are as follows:

a) The factor of safety is determined from statics where an additional horizontal force is included to account for the dynamic loads.

b) While the horizontal load as given above is taken to act permanently, the earthquake forces are acting only for short periods and reverse in direction many times during these periods.

c) The strength characteristics of the materials forming the dam does not figure in these types of analysis. While in certain conditions the strength of earth under the dynamic loads are greater than its static strength, in some others there may be considerable reduction in strength due to increase in pore pressures, etc. Therefore, a blanket increase or decrease in strength to account for dynamic loads will furnish erroneous information on the safety of dams from the concept of a 'factor of safety'.

Therefore, a proper dynamic analysis should consist of the following steps:

a) Determination of stress conditions and displacement in the dam at various points due to earthquake.

b) Determination of the strength properties of the materials under the computed stress conditions.

c) Determination of the displacement at all points in the dam section from the knowledge of stresses and stress strain characteristics.

From the above, it can be seen that all the quantities are time dependent and so, very much complicated and difficult to be made use of in design.

New analytical techniques, such as the finite element method, have made it possible to determine the stress conditions in the dam section considering even the non-linear behaviour of soils. However, the non-linear behaviour of soils can be determined only after step (b) has been performed. Thus step (b) and (c) are complementary to each other. The lack of proper apparatus for determining the strength characteristics of earth materials under dynamic condition, therefore, is a serious draw back in conducting a proper dynamic analysis of earth and rockfill dams.

In the absence of a proper dynamic analysis it will be worthwhile to consider a safe analysis. Unless excess pore pressures are allowed to develop within the dam section thereby causing undesirable consequences, the concept of 'factor of safety' can be seen to be a safe form of design. However, it is possible to extend this concept nearer to the actual behaviour of dams by considering realistic forces for design.

**7.4.2** *Seismic Force on Soil Mass* — The fundamental period of an embankment dam can be expressed as $T_1 = \dfrac{K.H_t}{V_s}$

where

$K =$ a value dependent on the assumption of the shape of the dam,

$= 2.4048$ for triangular sections,

$= 2.38$ for trapezoidal sections. (This value becomes 2.9 when the value of shear modulus is changed to account for non-uniformity of stresses along horizontal anes.),

$H_t =$ height of the dam, and

$V_s =$ velocity of shear waves through the dam.

The above formula is based on the assumption of shear deformation only and has been adopted in the Code.

It will be seen that the value of the fundamental natural period is dependent on the velocity of shear waves propagating through the dam. This quantity cannot be a constant one as it depends on the value of the shear modulus of the material which increases with increase in confinement pressure in granular materials. Therefore, it is desirable

to consider the value of $G$ in the equation in the form $G = (K_2)_{max} (\sigma_m)^{0.5}$. The value of $(K_2)_{max}$ may be suitably determined by field or laboratory tests and $\sigma_m$ may be considered at mid-height of the dam.

From the elastic analysis of a typical dam section, it can be found that the crest of the dam is subjected to much larger forces than the toe, the increase in acceleration from the toe to the crest being almost parabolic in nature. This behaviour has been borne out even by actual measurements in different dams in Japan. Therefore, design of upper portions of the dam to resist larger forces become essential. This can be achieved by assuming probable slip surfaces entirely in the upper ranges only, say extending to the upper 7/8 to 3/4 range from the top. Because of the larger horizontal forces, the stability will be least for such slip circles especially in the absence of cohesion. Therefore, in clauses **7.4.2.1** and **7.4.2.2** of the Code the method of determining the seismic coefficients have been given for any slip circle whether extending for the full height from top or contained only for a limited portion from top. The procedure can be summarized as below:

After obtaining the fundamental period, the spectral acceleration ($S_a$) can be read from the average acceleration spectra as given in Fig. 2 of the Code. Damping values of 10 percent of critical would be appropriate as it is not desirable to permit large deformations.

From the above value of spectral acceleration, the design seismic coefficient $\alpha_h$ can be worked out as $2 F_0(S_a/g)$ where 2 is the importance factor and $F_0$ is the zone factor. If the lowest point of the rupture wedge considered extends to a distance of only $y$ below the crest of the dam, instead of the full height of the dam, then the equivalent uniform seismic coefficient for design will be given as,

$$\alpha_y = \left( 4.0 - 2.4 \frac{y}{H} \right) \alpha_h \qquad \ldots\ldots\ldots(1)$$

The inertia force acting at the centre of gravity of the rupture wedge will be equal to $\alpha_y$ times the weight of the wedge. Where $H$ is the height of the dam. This relationship is shown diagramatically in Fig. 18.

$W_1$ and $W_2$ are the weight of the rupture wedges acting at their centre of gravity.

If, however, the design spectra for the dam site is available after rigorous seismological studies, the equation (1) can be rewritten as:

$$\alpha_y = \left( 2.5 - 1.5 \frac{y}{H} \right) \frac{S_a}{g}$$

where $S_a$ is the spectral acceleration for the computed fundamental period of the dam from the spectra for the site.

**7.4.3.3** As hydrodynamic suction due to the acceleration on water has already been taken into account as per **7.4.3.2** of the Code, it is considered not necessary to consider any increase in pore pressure also because it is unlikely that in the well compacted embankments larger pore pressures than the hydrodynamic suction will develop.

All the above provisions are for the stability of slips alone, assuming that a transverse motion of the ground takes place during an earthquake. However, there are many more complex problems connected with the stability of a dam for which code provisions are impossible with the present state of knowledge. The important factors to be considered in this regard are:

a) Spillways, their connections with the main dam, spillway lining, etc;

b) Drainage provisions, in case failure of the dam takes place and emptying of the reservoir becomes necessary to facilitate repairs;

c) Abutments of the dam; and

d) Other appurtenant structures which may have any connections with the main dam.

For embankment dams, the Code specifies only a pseudo static stability analysis wherein a factor



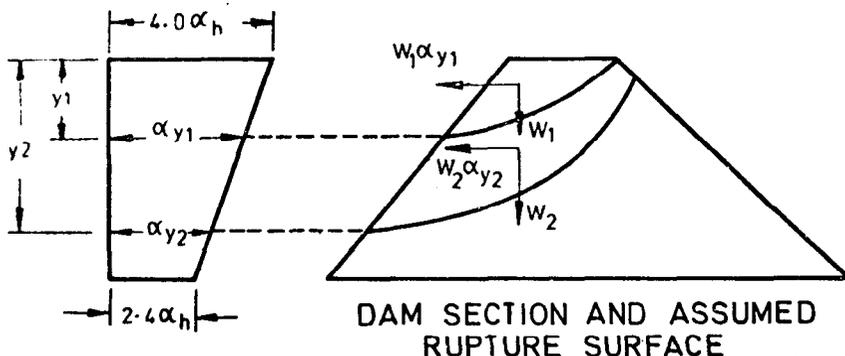DAM SECTION AND ASSUMED RUPTURE SURFACE

FIG. 18   SEISMIC CO-EFFICIENT DISTRIBUTION WITH HEIGHT-DAM SECTION AND ASSUMED RUPTURE SURFACE

of safety is ascertained for the upstream and downstream slopes. The inertia force considered in the above analysis is fairly rational since the height of the dam, the average material properties and also the response characteristics are accounted for to some extent. The main drawback of this method however, is that the averaging technique used for obtaining the seismic coefficient considers only the shear forces and not the duration for which these forces are operative. In other words, the damage potential of these forces have not been considered in this method at all. Obviously, the damage potential of the forces depends on the magnitude of the forces, the duration for which the forces act and also the strength (or stress-strain) characteristics of the dam material. Thus even if the factor of safety as calculated by the method specified is less than unity, it may not mean 'failure' by any means if the total duration of the earthquake is short or if the dynamic strength of the dam material considered is that corresponding to a larger number of cycles of loading than that during the actual earthquake. Conversely, if the dynamic strength considered is on the unsafe side (when few number of cycles of loading than the earthquake has been considered for fixing the dynamic strength characteristics) even a factor of safety of unity and more cannot guarantee safety of the dam since undesirably large deformation could result. The deformation of the dam will manifest itself on the dam in the form of settlement or slumping of the crest, flattening of the slopes, minor or major longitudinal cracks or minor or major transverse cracks. If these deformations had been anticipated and measures adopted to avoid undesirable consequences, these deformations could be termed as permissible and repairs undertaken after the event. For example, a slumping of the crest of the dam if anticipated, can be taken care of by provision of additional free board; cracks in the dam made less harmful by engineering the structure to prevent concentrated leaks, etc, are some of the more commonly adopted techniques.

It thus becomes clear that a proper and rational method of arriving at the seismic stability of embankment dams is to ascertain the deformation of the dam when subjected to the earthquake forces. The methods that can be used are:

a) Determining the yield acceleration of the structure considering the effective stress parameters and computing the displacements on the assumption that the ruptured mass is similar to a rigid mass resting on an inclined plane (Reference 8).

b) Determining the average stress conditions along potential slip circles prior to and during the earthquake and ascertaining the strains these stresses will cause on representative soil samples tested in the laboratory. Summing up the strains to obtain loss of free board has also been attempted.

c) Use of averages from a parametric study to obtain the order of deformations.

## 8. Retaining Walls

8.1 Retaining walls are designed to resist the lateral earth pressure caused by the backfill. The earth pressure is dependent on the geometry, the backfill properties as well as the wall movements. The stability of retaining walls, therefore, is calculated in terms of factors of safety against sliding, against overturning and against failure of foundation. Soil-survey of damage during past earthquakes shows that earth retaining structures have moved out away from the backfill. In the absence of a theory to predict the displacement of retaining walls under earthquake loading conditions, the philosophy of design adopted is the one in which the active earth pressures have been considered to increase and the lateral earth resistance to decrease during earthquakes.

The earliest approach to account for the above was to consider a reduction in the value of the angle of internal friction of the backfill material. The reduced value was recommended as $\phi_{dyn}$, the angle of repose of a cohesionless material under the influence of a horizontal acceleration in addition to the vertical acceleration due to gravity. The value of $\phi_{dyn}$ to be used for design would thus be,

$$\phi_{dyn} = \phi - \tan^{-1} \alpha_h$$

where $\phi$ is the angle of internal friction of the backfill material and $\alpha_h$ is the horizontal seismic coefficient.

When the vertical earthquakes accelerations are also considered,

$$\phi_{dyn} = \phi - \tan^{-1} \frac{\alpha_h}{\alpha_v}$$

where $\alpha_v$ is the vertical seismic coefficient.

According to the practice, the modified value of $\phi_{dyn}$ was used in conjunction with either Rankine's theory or Coulomb's theory, the two classical earth pressure theories widely used all over the world.

8.1.1 The formula given is the result of the modification of the wedge theory of Coulomb. All the assumptions of the Coulomb's theory are, therefore, made in this as well. In addition, it is assumed that the earthquake forces can be represented by inertia forces acting as static forces. The equilibrium of the wedges are ascertained with two additional forces besides the forces considered in Coulomb's theory. These forces are:

a) A horizontal force equal to the horizontal seismic coefficient times the weight of the wedge, and

b) A vertical force equal to the vertical seismic coefficient times the weight of the wedge. The horizontal force has been considered to act away from the backfill.

The vertical force, however, has to be considered in conjunction with the stability of the retaining wall. Here, depending upon the properties of the wall, a vertical downward inertia force may mean larger pressure as well as larger resistance. Therefore, both the directions of the force will have to be considered to ascertain the critical one for design. Accordingly, the sign preceding $\alpha_v$ has been specified as either positive or negative.

When inertia forces are considered in addition to the forces in Coulomb's theory, the resultant of the weight and the inertia force is weight component (say $W$) of the wedge becomes inclined at an angle $\lambda$ equal to $\tan^{-1} \dfrac{\alpha_h}{1 \pm \alpha_v}$ and also its value is increased to $\dfrac{W(1 \pm \alpha_v)}{\cos \lambda}$. Now, if the plane of the problem is given a rotation through an angle $\lambda$ so as to make the resistant weight component vertical, Coulomb's theory can be used as such. However, the rotation effected as described above causes the inclination of the face of the wall with vertical to increase from $\alpha$ to $(\alpha + \lambda)$ and the slope of the backfill with horizontal to increase from $i$ to $(i + \lambda)$. But, the length of the face of the wall remains equal to $\dfrac{h}{\cos \alpha}$ where $h$ is the height of the wall. Thus, substituting $(\alpha + \lambda)$ for $\alpha$ and $(i + \lambda)$ for $i$ in the Coulomb's theory, the active earth pressure on the retaining walls is obtained as per the formula given.

It will be noticed that the term $\dfrac{(1 \pm \alpha_v)}{\cos \lambda}$ is associated with the increase in the weight component while the terms $\cos^2 \alpha$ in the denominator which remains unaltered as compared to Coulomb's theory is indicative of the length of the face of the wall remaining unaltered due to the rotation.

**8.1.1.1** The graphical method due to Culmann is a procedure of graphical differentiation of the problem of the equilibrium of the rupture wedge with a plane rupture surface that is Coulomb's theory itself. The procedure given in Appendix H of the Code is a modification of the original Culmann's procedure wherein the inertia forces are also considered. For simple geometry as shown in the method, the rupture wedges are taken to be of weight as simple multiples of the first wedge. This will reduce the calculations to the minimum.

NOTE — If the method is to be used for the cases where the backfill has a broken surface or carries a surcharge, suitable modifications will be necessary. It needs to be remembered that the resultant of the weight and inertia forces are marked on the BB' line (*see* Fig. 19 of the Code).

**8.1.1.2** As has been amply described in literature, Coulomb theory does not give any idea of the distribution of earth pressure on the retaining wall. The magnitude of pressure alone is given by the theory. However, the form of the final equation and the use of Rankine's theory had led designers to adopt a hydrostatic distribution of Coulomb pressures. Also, a large number of laboratory tests appear to support this assumption.

When Coulomb's theory was modified by Japanese investigators Monomobe and Okabe to derive the equation given under **8.1.1**, the distribution of pressures was not considered differently from the static case.

Different laboratory tests have led to the distribution of dynamic pressures being assumed to be of different types. All the experimental studies have demonstrated that the magnitude of the dynamic component of the earth pressure does not increase with depth but has the maximum value somewhere in between the top and the base of the wall. If a simple form of distribution of the dynamic component of pressures is desired, a parabola is found to be quite accurate and sufficient. This will mean that the dynamic component will have a point of application at mid-height of the wall. Thus, the earth-pressure on the wall, computed in accordance with the formula given in **8.1.1** of the Code and which consists of the effects of the static forces as well as the inertia forces, is split into two; the static component by neglecting the inertia forces and the dynamic component being the difference between the dynamic pressure and the static component. While the static component is assumed to be distributed hydrostatically, a parabolic pressure distribution with the maximum value at mid-height of the wall is considered for the dynamic component. As already described, since the point of application is immaterial in Coulomb's theory, none of the conditions of the derivation of the formula is vitiated by such assumptions of the pressure distribution.

**8.1.2 to 8.1.2.1** The passive pressure which in reality is equal to the lateral earth resistance, is also calculated on the same principles as for the active earth pressure, that is, as a modification of Coulomb's theory. The only difference in this case is that the inertia forces are considered to act towards the backfill as this condition gives the minimum value of passive pressures.

**8.1.2.2** In the absence of any laboratory test data a very conservative assumption regarding the point of application of the passive pressures is made. Accordingly, the dynamic component (reduction over the static passive pressures) is arbitrarily assumed to have a point of application at $\frac{2}{3}$ the height of the wall above its base.

**8.1.3** The concept is the same as Coulomb's theory.

**8.1.3.1** As described earlier, the static earth pressure due to the gravity effects is distributed hydrostatically thereby indicating a constant value of earth pressure coefficient with depth. (The earth pressure coefficient is the ratio between the lateral pressure and the effective vertical pressure at any point). But the dynamic component of earth pressure is distributed in a parabolic fashion which indicates that the coefficient of earth pressure is distributed as an inverted triangle with the largest value at the top and decreasing to zero at the base of the wall.

Since the uniform surcharge on the surface of the backfill leads to a constant increase in the vertical pressure at all depths, the lateral pressure due to surcharge will have a distribution similar to the distribution of the earth pressure coefficient with depth. Accordingly, the dynamic increment in active pressures due to uniform surcharge shall be applied at an elevation of $0.66\ h$ above the base of the wall while the static component shall be applied at mid-height of the wall.

**8.1.4** This forms modification of Coulomb's theory.

**8.1.4.1** In the absence of data, the same provision as for active pressures will apply.

**8.2.2** The value of tan $\lambda$ is increased by a factor $\dfrac{w_s}{w_s - 1}$ where $w_s$ is the saturated unit weight of the backfill material. This is based on the consideration that while the reaction along the rupture plane is a function of the effective pressures and hence the submerged unit weight, the inertia force is the product of the design acceleration and the mass of the solid particles as well as the water in the pores. Thus, the inertia force on the wedge is equal to (Volume) $\times (w_s) \times (\alpha_h)$ while the effective weight is equal to (Volume) $\times (w_s - 1) \times (1 \pm \alpha_v)$ thus,

$$\tan \lambda = \frac{w_s \cdot \alpha_h}{(w_s - 1)(1 \pm \alpha_v)}.$$

**8.2.3** Due to the fact that the inertia force due to the mass of water contained in the pores has already been taken into account, as stated above, hydrodynamic pressure is not taken separately.

**8.3 Partially Submerged Backfill** — It has already been described that the earth pressure coefficients are different for the submerged and dry backfills because of the changes in the values of $\delta$, the angle of wall friction and $\lambda$. Thus, the distribution of the coefficient of dynamic increment with height will be as shown in Fig. 13 of the Code. The value of the earth pressure coefficient at different elevations has been worked out on the basis of the total pressure which has been kept equal to the one computed by the formula given in **8.1.1** of the Code.

**8.3.2** If the ratio of dynamic decrement of passive pressures with the vertical effective stress is

derived from a stepped triangular distribution similar to that of Fig. 13 of the Code, the point of application of dynamic decrement of passive pressure will be below $0.66\ h$ from the base, which would be in violation of **8.1.2.2** and **8.1.4.1** of the Code. The procedure to be utilized in this clause for evaluation of distribution of dynamic decrements of passive pressures will have suitable inverted triangular distribution of ratio of lateral dynamic decrement in passive pressures to the vertical effective pressures so that the point of application of dynamic decrement of passive pressures is at a height of $0.66\ h$ from the base in accordance with **8.1.2.2** and **8.1.4.1** of the Code.

**8.4** Concrete or masonry inertia force will have to be considered while ascertaining the stability of retaining walls. Also in view of the shorter duration of the earthquake vibrations, the factors of safety against sliding, overturning and failure of the foundation soil have been kept at lower levels than under a static loading conditions as can be seen in the Note under **8.4** of the Code.

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*Example* 15

Height $= 12$ m

Slope of earthfill ($i$) $= 5°$

Inclination of wall with vertical $\alpha = 15°$

Angle of internal friction $\phi = 30°$

Angle of friction between wall and earthfill $\delta = 20°$

Density of saturated soil $w_s = 2$ t/m³

Density of dry/moist soil $= 1.6$ t/m³

Retaining wall is located in seismic Zone V

Height of water table $= 6$ m

Horizontal seismic coefficient $\alpha_h = \beta . I . \alpha_0$
$= 1 \times 1 \times 0.08 = 0.08$

Vertical seismic coefficient may be taken as:
$\alpha_v = \frac{1}{2}\alpha_h = 0.04$

$$\lambda = \tan^{-1}\left[\frac{w_s}{w_s - 1} \cdot \frac{\alpha_h}{1 \pm \alpha_v}\right]$$

$\lambda =$ (Corresponding to $+ \alpha_v$) $= 8.746$

$\lambda =$ (Corresponding to $- \alpha_v$) $= 9.462$

*Active Earth Pressure*

[*for Submerged Soil* $\delta = \frac{1}{2} \delta$ (*dry*)]

*For* $+ \alpha_v$ :

Cos $(\phi - \lambda - \alpha) =$ Cos $(30 - 8.746 - 15)$
$= 0.994$

Cos $(\delta + \alpha + \lambda) =$ Cos $(10 + 15 + 8.746)$
$= 0.831$

Sin $(\phi - i - \lambda) =$ Sin $(30 - 5 - 8.746)$
$= 0.279$

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$$\text{Cos } \lambda = \text{Cos } (8\cdot746) = 0\cdot988$$

$$\text{Cos } \alpha = \text{Cos } (15) = 0\cdot965$$

$$\text{Cos } (\alpha - i) = \text{Cos } (15 - 5) = 0\cdot984$$

$$\text{Sin } (\phi + \delta) = \text{Sin } (30 + 10) = 0\cdot642$$

$$C'_a = \frac{(1 + \alpha_v)\,\text{Cos}^2\,(\phi - \alpha - \lambda)}{\text{Cos } \lambda\,\text{Cos}^2\,\alpha\,\text{Cos } (\delta + \alpha + \lambda)}$$

$$\times \left[ \frac{1}{1 + \left\{ \dfrac{\text{Sin } (\phi + \delta)\,\text{Sin } (\phi - i - \lambda)}{\text{Cos } (\alpha - i)\,\text{Cos } (\delta + \alpha + \lambda)} \right\}^{1/2}} \right]^2$$

$$C'_a = \frac{1\cdot04 \times (0\cdot994)^2}{0\cdot988 \times (0\cdot965)^2 \times 0\cdot831}$$

$$\times \left[ \frac{1}{1 + \left\{ \dfrac{0\cdot642 \times 0\cdot279}{0\cdot984 \times 0\cdot831} \right\}^{1/2}} \right]^2$$

$$= 1\cdot344 \times 0\cdot464 = 0\cdot623$$

*For* $-\alpha_v$ :

$$\text{Cos } (\phi - \lambda - \alpha) = \text{Cos } (30 - 9\cdot462 - 15)$$
$$= 0\cdot995$$

$$\text{Cos } (\delta + \alpha + \lambda) = \text{Cos } (10 + 15 + 9\cdot462)$$
$$= 0\cdot824$$

$$\text{Sin } (\phi - i - \lambda) = \text{Sin } (30 - 5 - 9\cdot462)$$
$$= 0\cdot267$$

$$\text{Cos } (\lambda) = \text{Cos } (9\cdot462) = 0\cdot986$$

$$\text{Cos } (\alpha) = \text{Cos } (15) = 0\cdot965$$

$$\text{Cos } (\alpha - i) = \text{Cos } (15 - 5) = 0\cdot984$$

$$\text{Sin } (\phi + \delta) = \text{Sin } (30 + 10) = 0\cdot642$$

$$C'_a = \frac{0\cdot96 \times (0\cdot995)^2}{0\cdot986 \times (0\cdot965)^2 \times 0\cdot824}$$

$$\times \left[ \frac{1}{1 + \left\{ \dfrac{0\cdot642 \times 0\cdot267}{0\cdot984 \times 0\cdot824} \right\}^{1/2}} \right]^2$$

$$= 1\cdot256 \times 0\cdot469 = 0\cdot589$$

when $\alpha_h = \alpha_v = \lambda = 0$

$$\text{Cos } (\phi - \lambda - \alpha) = \text{Cos } 15 = 0\cdot965$$

$$\text{Cos } (\delta + \alpha + \lambda) = \text{Cos } 25 = 0\cdot906$$

$$\text{Sin } (\phi - i - \lambda) = \text{Sin } 25 = 0\cdot422$$

$$\text{Cos } \lambda = \text{Cos } 0 = 1$$

$$\text{Cos } \alpha = \text{Cos } 15 = 0\cdot965$$

$$\text{Cos } (\alpha - i) = \text{Cos } 10 = 0\cdot984$$

$$\text{Sin } (\phi + \delta) = \text{Sin } 40 = 0\cdot642$$

$$K'_a = \frac{1 \times (0\cdot965)^2}{1 \times (0\cdot965)^2 \times 0\cdot906}$$

$$\times \left[ \frac{1}{1 + \left\{ \dfrac{0\cdot642 \times 0\cdot422}{0\cdot984 \times 0\cdot906} \right\}^{1/2}} \right]^2$$

$$= 1\cdot103 \times 0\cdot415 = 0\cdot457$$

*For Dry/Moist Back Fills,* $\delta = 20°$:

$$\lambda = \tan^{-1} \frac{\alpha_h}{1 \pm \alpha_v} = \tan^{-1} \frac{0\cdot08}{1 \pm 0\cdot04}$$

$$\lambda \text{ (corresponding to } + \alpha_v) = 4\cdot398$$

$$\lambda \text{ (corresponding to } - \alpha_v) = 4\cdot763$$

*For* $+ \alpha_v$ :

$$\text{Cos } (\phi - \lambda - \alpha) = \text{Cos } (30 - 4\cdot398 - 15)$$
$$= 0\cdot982$$

$$\text{Cos } (\delta + \alpha + \lambda) = \text{Cos } (20 + 15 + 4\cdot398)$$
$$= 0\cdot772$$

$$\text{Sin } (\phi - i - \lambda) = \text{Sin } (30 - 5 - 4\cdot398)$$
$$= 0\cdot351$$

$$\text{Cos } \lambda = \text{Cos } 4\cdot398 = 0\cdot997$$

$$\text{Cos } \alpha = \text{Cos } 15 = 0\cdot965$$

$$\text{Cos } (\alpha - i) = \text{Cos } 10 = 0\cdot984$$

$$\text{Sin } (\phi + \delta) = \text{Sin } (30 + 20) = 0\cdot766$$

$$C_a = \frac{1\cdot04 \times (0\cdot982)^2}{0\cdot997 \times (0\cdot965)^2 \times 0\cdot772}$$

$$\times \left[ \frac{1}{1 + \left\{ \dfrac{0\cdot766 \times 0\cdot351}{0\cdot984 \times 0\cdot772} \right\}^{1/2}} \right]^2$$

$$= 1\cdot399 \times 0\cdot393 = 0\cdot549$$

*For* $- \alpha_v$ :

$$\text{Cos } (\phi - \lambda - \alpha) = \text{Cos } (30 - 4\cdot763 - 15)$$
$$= 0\cdot984$$

$$\text{Cos } (\delta + \alpha + \lambda) = \text{Cos } (20 + 15 + 4\cdot763)$$
$$= 0\cdot768$$

$$\text{Sin } (\phi - i - \lambda) = \text{Sin } (30 - 5 - 4\cdot763)$$
$$= 0\cdot346$$

$$\text{Cos } \lambda = \text{Cos } (4\cdot763) = 0\cdot996$$

$$\text{Cos } \alpha = \text{Cos } (15) = 0\cdot965$$

$$\text{Cos } (\alpha - i) = \text{Cos } (15 - 5) = 0\cdot984$$

$$\text{Sin } (\phi + \delta) = \text{Sin } (30 + 20) = 0\cdot766$$

$$C_a = \frac{0.96 \times (0.984)^2}{0.996 \times (0.965)^2 \times 0.768}$$

$$\times \left[ \frac{1}{1 + \left\{ \frac{0.766 \times 0.346}{0.984 \times 0.768} \right\}^{1/2}} \right]^2$$

$$= 1.305 \times 0.394 = 0.514$$

When $\alpha_h = \alpha_v = \lambda = 0$

$$\text{Cos} (\phi - \lambda - \alpha) = \text{Cos} (30 - 15) = 0.965$$

$$\text{Cos} (\phi + \alpha + \lambda) = \text{Cos} (20 + 5) = 0.819$$

$$\text{Sin} (\phi - i - \lambda) = \text{Sin} (30 - 5) = 0.422$$

$$\text{Cos} \lambda = \text{Cos} 0 = 1$$

$$\text{Cos} \alpha = \text{Cos} 15 = 0.965$$

$$\text{Cos} (\alpha - i) = \text{Cos} (15 - 5) = 0.984$$

$$\text{Sin} (\phi + \delta) = \text{Sin} (30 + 20) = 0.766$$

$$K_a = \frac{1 \times (0.965)^2}{1 \times (0.965)^2 \times 0.819}$$

$$\times \left[ \frac{1}{1 + \left\{ \frac{0.766 \times 0.422}{0.984 \times 0.819} \right\}^{1/2}} \right]^2$$

$$= 1.221 \times 0.374 = 0.456$$

*Passive Earth Pressure for Submerged Soil:*

*For + $\alpha_v$ :*

$$\text{Cos} (\phi + \alpha - \lambda) = \text{Cos} (30 + 15 - 8.746)$$
$$= 0.806$$

$$\text{Cos} (\delta - \alpha + \lambda) = \text{Cos} (10 - 15 + 8.746)$$
$$= 0.997$$

$$\text{Sin} (\phi + i - \lambda) = \text{Sin} (30 + 5 - 8.746)$$
$$= 0.442$$

$$\text{Cos} \lambda = \text{Cos} (8.746) = 0.988$$

$$\text{Cos} \alpha = \text{Cos} (15) = 0.965$$

$$\text{Sin} (\phi + \delta) = \text{Sin} (30 + 10) = 0.624$$

$$\text{Cos} (\alpha - i) = \text{Cos} (15 - 5) = 0.984$$

$$C_a = \frac{1.04 \times (0.806)^2}{0.988 \times (0.965)^2 \times 0.997}$$

$$\times \left[ \frac{1}{1 - \left\{ \frac{0.624 \times 0.442}{0.984 \times 0.997} \right\}^{1/2}} \right]^2$$

$$= 0.736 \times 4.531 = 3.335$$

*For $- \alpha_v$ :*

$$\text{Cos} (\phi + \alpha - \lambda) = \text{Cos} (30 + 15 - 9.462)$$
$$= 0.813$$

$$\text{Cos} (\delta - \alpha + \lambda) = \text{Cos} (10 - 15 + 9.462)$$
$$= 0.997$$

$$\text{Sin} (\phi + i - \lambda) = \text{Sin} (30 + 5 - 9.462)$$
$$= 0.431$$

$$\text{Cos} \lambda = \text{Cos} 9.462 = 0.986$$

$$\text{Cos} \alpha = \text{Cos} 15 = 0.965$$

$$\text{Sin} (\phi + \delta) = \text{Sin} (30 + 10) = 0.624$$

$$\text{Cos} (\alpha - i) = \text{Cos} (15 - 5) = 0.984$$

$$C_a' = \frac{0.96 \times (0.813)^2}{0.986 \times (0.965)^2 \times 0.997}$$

$$\times \left[ \frac{1}{1 - \left\{ \frac{0.624 \times 0.431}{0.984 \times 0.997} \right\}^{1/2}} \right]^2$$

$$= 0.693 \times 4.406 = 3.053$$

when $\alpha_h = \alpha_v = \lambda = 0$

$$\text{Cos} (\phi + \alpha - \lambda) = \text{Cos} (30 + 15) = 0.707$$

$$\text{Cos} (\delta - \alpha + \lambda) = \text{Cos} (10 - 15) = 0.996$$

$$\text{Sin} (\phi + i - \lambda) = \text{Sin} (30 + 5) = 0.573$$

$$\text{Cos} \lambda = \text{Cos} 0 = 1$$

$$\text{Cos} \alpha = \text{Cos} 15 = 0.965$$

$$\text{Sin} (\phi + \delta) = \text{Sin} (30 + 10) = 0.642$$

$$\text{Cos} (\alpha - i) = \text{Cos} (15 - 5) = 0.984$$

$$K_a' = \frac{1 \times (0.707)^2}{1 \times (0.965)^2 \times (0.996)}$$

$$\times \left[ \frac{1}{1 - \left\{ \frac{0.642 \times 0.573}{0.984 \times 0.996} \right\}^{1/2}} \right]^2$$

$$= 0.539 \times 6.665 = 3.592$$

*For Dry/Moist Backfill, $\delta = 20°$*

*For + $\alpha_v$ :*

$$\text{Cos} (\phi + \alpha - \lambda) = \text{Cos} (30 + 15 - 4.398)$$
$$= 0.759$$

$$\text{Cos} (\delta - \alpha + \lambda) = \text{Cos} (20 - 15 + 4.398)$$
$$= 0.986$$

$$\text{Sin} (\phi + i - \lambda) = \text{Sin} (30 + 5 - 4.398)$$
$$= 0.509$$

$$\text{Cos} \lambda = \text{Cos} (4.398) = 0.997$$

$\text{Cos } \alpha \qquad\qquad = \text{Cos } 15 \qquad\qquad = 0.967$

$\text{Cos } (\alpha - i) \qquad = \text{Cos } (15 - 5) \qquad = 0.984$

$\text{Sin } (\phi + \delta) \qquad = \text{Sin } (30 + 20) \qquad = 0.766$

$$C_a = \frac{1.04 \times (0.759)^2}{0.997 \times (0.967)^2 \times 0.986}$$

$$\times \left[ \cfrac{1}{1 - \left\{ \cfrac{0.766 \times 0.509}{0.984 \times 0.986} \right\}^{1/2}} \right]^2$$

$$= 0.651 \times 7.462 = 4.857$$

*For* $-\alpha_v$:

$\text{Cos } (\phi + \alpha - \lambda) = \text{Cos } (30 + 15 - 4.763)$
$$= 0.763$$

$\text{Cos } (\delta - \alpha + \lambda) = \text{Cos } (20 - 15 + 4.763)$
$$= 0.985$$

$\text{Sin } (\phi + i - \lambda) = \text{Sin } (30 + 5 - 4.763)$
$$= 0.503$$

$\text{Cos } \lambda \qquad\qquad = \text{Cos } (4.763) \quad = 0.996$

$\text{Cos } \alpha \qquad\qquad = \text{Cos } (15) \qquad = 0.965$

$\text{Cos } (\alpha - i) \qquad = \text{Cos } (15 - 5) = 0.984$

$\text{Sin } (\phi + \delta) \qquad = \text{Sin } (30 + 20) = 0.766$

$$C_a = \frac{0.96 \times (0.763)^2}{0.996 \times (0.965)^2 \times 0.985}$$

$$\times \left[ \cfrac{1}{1 - \left\{ \cfrac{0.766 \times 0.503}{0.984 \times 0.985} \right\}^{1/2}} \right]^2$$

$$= 0.611 \times 7.324 = 4.475$$

*For* $\alpha_h = \alpha_v = \lambda = 0$ :

$\text{Cos } (\phi + \alpha - \lambda) = \text{Cos } (30 + 15) = 0.707$

$\text{Cos } (\delta - \alpha + \lambda) = \text{Cos } (20 - 15) = 0.996$

$\text{Sin } (\phi + i - \lambda) = \text{Sin } (30 + 5) = 0.573$

$\text{Cos } \lambda \qquad\qquad = \text{Cos } 0 \qquad = 1$

$\text{Cos } \alpha \qquad\qquad = \text{Cos } 15 \qquad = 0.965$

$\text{Cos } (\alpha - i) \qquad = \text{Cos } (15 - 5) = 0.984$

$\text{Sin } (\phi + \delta) \qquad = \text{Sin } (30 + 20) = 0.766$

$$K_a = \frac{1 \times (0.707)^2}{1 \times (0.965)^2 \times 0.996}$$

$$\times \left[ \cfrac{1}{1 - \left\{ \cfrac{0.766 \times 0.573}{0.984 \times 0.996} \right\}^{1/2}} \right]^2$$

$$= 0.539 \times 9.139 = 4.926$$

*Active Earth Pressure (Dry or Moist Case)*

$P_a$ (for $+\alpha_v$) $= \frac{1}{2} wh^2 (C_a)$
$$= \frac{1}{2} \times 1.6 \times 12 \times 12 \times 0.549$$
$$= 63.244 \text{ t}$$

$P_a$ (for $-\alpha_v$) $= \frac{1}{2} wh^2 (C_a)$
$$= \frac{1}{2} \times 1.6 \times 12 \times 12 \times 0.514$$
$$= 59.212 \text{ t}$$

$P_a$ (*static active pressure*)
$$= \frac{1}{2} \times 1.6 \times 12 \times 12 \times 0.456$$
$$= 52.531 \text{ t}$$

This will be acting at a height of 4 m from the base of wall (*see* Fig. 19).
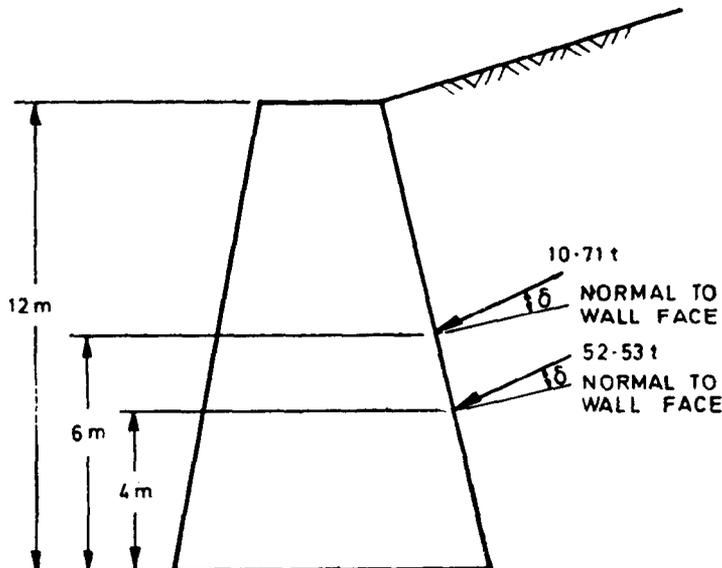


FIG. 19   DRY OR MOIST CASE FOR $+\alpha_v$
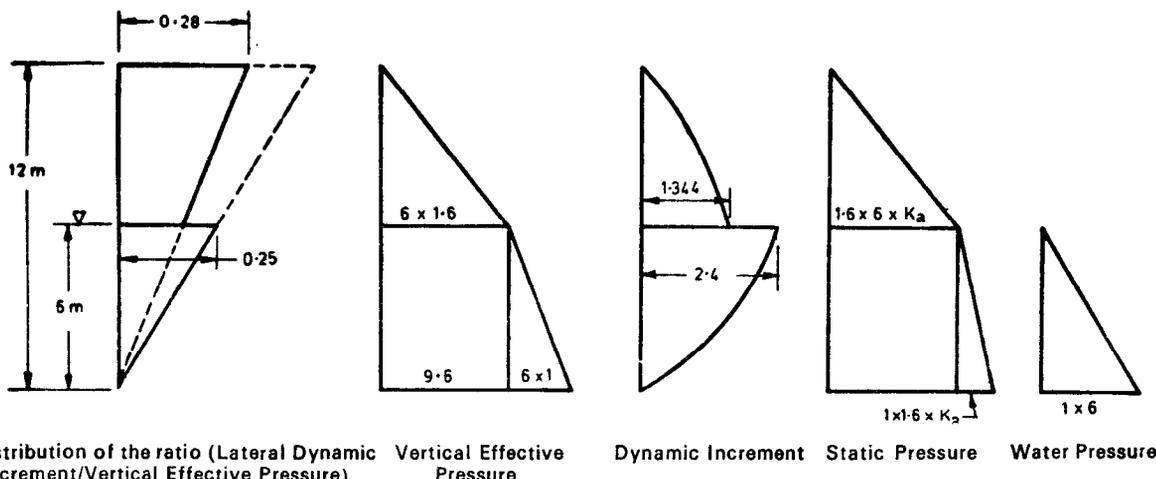
FIG. 20   DISTRIBUTION OF PRESSURE WITH HEIGHT OF WALL

*Dynamic Increment for Active Pressure*

(for $+ \alpha_v$) = $63 \cdot 244 - 52 \cdot 531 = 10 \cdot 713$ t

This will be acting at a height of 6 m from the base of wall (*see* Fig. 19 and 20).

$$3 (C_a - K_a) = 3 (0 \cdot 549 - 0 \cdot 456) = 0 \cdot 28$$

$$3 (C'_a - K'_a) \; \frac{h'}{h} = 3 (0 \cdot 623 - 0 \cdot 457) \frac{6}{12}$$

$$= 0 \cdot 25$$

*Dynamic Increment for Active Pressure*
(For $-\alpha_v$)

$$= 59 \cdot 212 - 52 \cdot 531 = 6 \cdot 681$$

This will be acting at a height of 6 m from the base of wall (Fig. 21).

$$3 (C_a - K_a) = 3 (0 \cdot 514 - 0 \cdot 456) = 0 \cdot 174$$

$$3 (C'_a - K'_a) \; \frac{h'}{h} = \{3 (0 \cdot 589 - 0 \cdot 457)\} \frac{6}{12}$$

$$= 0 \cdot 198$$

*Passive Earth Pressure (Dry or Moist Case)*

$$P_p \text{ (for } +\alpha_v) = \tfrac{1}{2} wh^2 \; C_a$$
$$= \tfrac{1}{2} \times 1 \cdot 6 \times 12 \times 12 \times 3 \cdot 335$$
$$= 384 \cdot 192$$

$$P_p \text{ (for } -\alpha_v) = \tfrac{1}{2} wh^2 \; C_a$$
$$= \tfrac{1}{2} \times 1 \cdot 6 \times 12 \times 12 \times 3 \cdot 053$$
$$= 351 \cdot 705 \text{ t}$$
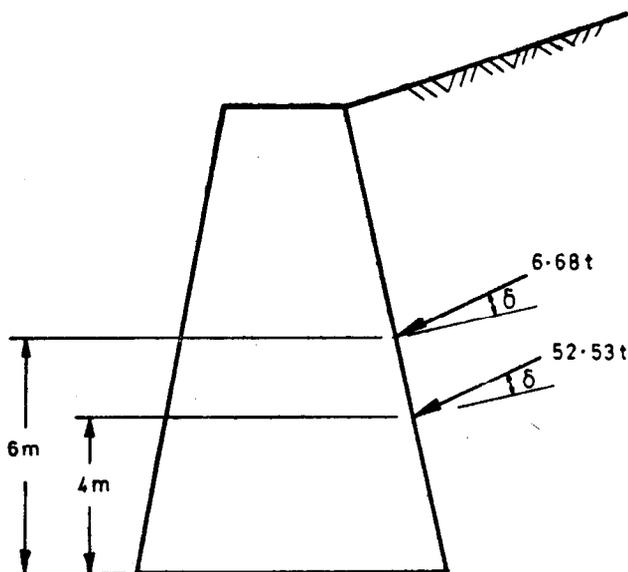


FIG. 21   DRY OR MOIST CASE FOR $-\alpha$
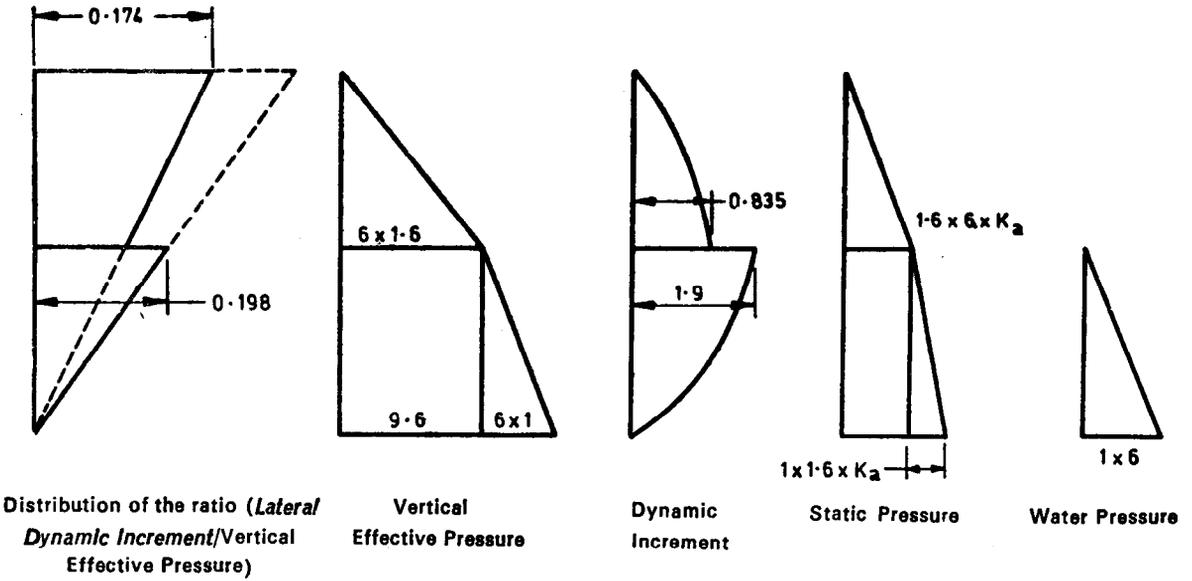
FIG. 22 DISTRIBUTION OF PRESSURE WITH HEIGHT OF WALL
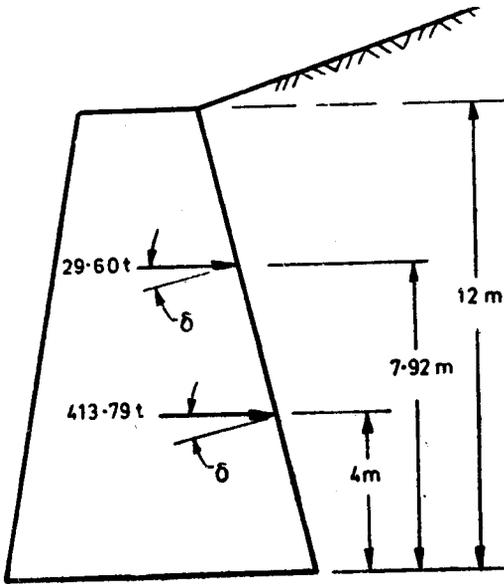


FIG. 23 DRY OR MOIST CASE FOR $+\alpha_v$
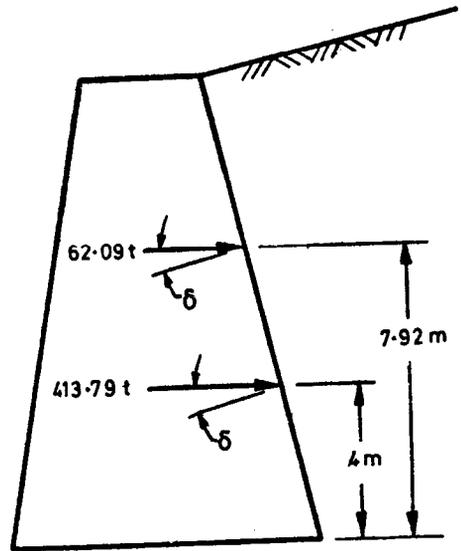


FIG. 24 DRY OR MOIST CASE FOR $-\alpha_v$

$P_a$ (*static passive pressure*)

$$= \tfrac{1}{2} \times 1 \cdot 6 \times 12 \times 12 \times 3 \cdot 592 = 413 \cdot 798 \text{ t}$$

This will act at a height of 4 m from the base of the wall (*see* Fig. 22).

*Dynamic Decrement for Passive Pressure* for $+\alpha_v$

$$= 413 \cdot 798 - 384 \cdot 192 = 29 \cdot 606 \text{ t}$$

This will be acting at a height of $(0 \cdot 66 \, h) = 7 \cdot 92$ m from the base of wall (*see* Fig. 23).

*Dynamic Decrement for Passive Pressure* (for $-\alpha_v$) (*see* Fig. 24)

$$= 413 \cdot 798 - 351 \cdot 705 = 62 \cdot 09 \text{ t}$$

This will act at a height of $(7 \cdot 92 \text{ m})$ from the base of wall.

# APPENDIX A

## (*Clause* 3.4)

## BACKGROUND TO SEISMIC ZONING MAP OF INDIA

Various attempts have been made in the past for seismic zoning of India based on available data on earthquake occurrence. Zoning maps were prepared in which the country was classified into different zones indicating intensity of damage or frequency of earthquake occurrence of probable accelerations or factor of safety to be adopted in the seismic design of structures. These maps provided a qualitative schematization of earthquake effects and thus served limited purpose. In 1960, the Indian Standards Institution initiated action for the formulation of unified recommendations for earthquake resistant design of structures which resulted in publication of IS : 1893-1962 Recommendations for earthquake resistant design of structures. This standard incorporated a seismic zoning map of India to provide a unified picture of seismicity taking into consideration the available data on earthquake occurrence and seismotectonic frame-work of the country. This portrayed the probable maximum intensity of earthquake for evaluation of strong ground motion characteristics to evaluate the basic design seismic coefficients for earthquake resistant design of structures in different parts of the country. A brief description of the general approach followed in the preparation of seismic zoning maps in 1962 and its subsequent revisions in 1966 and 1970 is given in the following paragraphs.

## SEISMIC ZONING MAPS

Seismic zoning was undertaken to demarcate the characteristics of earthquake occurrence in future so that the resulting map of the region can be utilized to provide information of expected earthquake effects. Maximum intensity of earthquake (M. M. Intensity), or peak ground motion (acceleration, velocity, displacement, etc) to evaluate design response spectra or seismic coefficient (expressed as a fraction of *g*) are generally portrayed in seismic zoning maps, and associated background information in the form of epicentral map, tectonic map, etc, helps in making various decisions for earthquake resistant design of structures.

## SEISMIC ZONING MAP OF INDIA IN IS : 1893-1962

With the development of systematic study of earthquake engineering problems in the country, it was considered necessary to have a seismic zoning map to indicate broadly the seismic coefficients that could generally be adopted for design in different parts of the country. The data avail-

able in the preparation of the seismic zoning map included in IS : 1893-1962 consisted of epicentral distribution of earthquakes of Richters magnitude 5 and above, and known tectonic and geological feather of the country. The following procedure has been followed in preparing the seismic zoning map shown in Fig. 25 :

a) The epicentres of all known earthquakes of magnitude 5 and above, and maximum Modified Mercalli intensities at various points were plotted from the isoseismals of major eathquakes for which records were available. Since the 1819 Kutch earthquake, 1897 and 1950 Assam earthquakes, 1905 Kangra earthquake, and 1934 Bihar-Nepal earthquake were of very high magnitudes; they adequately encompassed the effects of the lesser magnitude earthquakes occurring in different regions. In addition to these severe earthquakes, a number of other earthquakes taken into consideration including a few smaller earthquakes in the Himalayas, Delhi earthquakes, Satpura and Rewa earthquakes, Bellary earthquake and the Zone of minor tremors from Trivandrum to Madras and East Coast regions.

b) After plotting the idealized isoseismals for the earthquakes, enveloping lines for different M. M. intensities were drawn. These lines were modified where necessary, taking into account the magnitude of intervening earthquakes, local ground conditions, principal tectonic trends as portrayed in the preliminary Tectonic Map of India (Prepared by the Geological Survey of India in 1962), basement configuration of the Indo-Gangetic Plains and other alluvial basins as revealed by geophysical surveys of the Oil and Natural Gas Commission and the likely trend of the Ganga-Brahmaputra rift postulated by Mithal and Srivastava (1959).

c) After drawing the modified isoseismals, the zones with M. M. intensity, V, VI, VII, VIII, IX and X and above were designated as seismic zones I, II, III, IV, V and VI and the region with M. M. intensity less than V was designated as seismic Zone 0. This zero Zone was not a Zone of zero earthquakes, but the designation was given to suggest that no earthquake problem of any significance may occur in this region.

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d) Suitable seismic coefficients were assigned based on an engineering judgement of the likely intensity in each zone and variation of these coefficients according to ground conditions and type of structure were specified.

It is important to note that the seismic coefficient, used in the design of any structure is dependent on many variable factors including the influence of local soil conditions and soil-foundation-structure interaction. It is, therefore, necessary to indicate broadly the seismic coefficients that could generally be adopted in different parts of Zones of the country though of course a rigorous analysis considering all the factors involved has got to be made in the case of all important projects in order to arrive at suitable time-history of ground motion for evaluation of design forces. The object of seismic zoning map is to classify the area of the country into a number of zones in which one can reasonably forecast the characteristics of the strong ground motion shocks which will occur in the event of a future earthquake which need not necessarily be always the severest that would occur anywhere within the given zone. It is possible in some cases that earthquake resulting in more severe strong ground motion may occur at any particular place which is unpredictable. The probabilities, however, are that a structure designed on the assumption that strong ground motion characteristics indicated for each Zone are about the maximum that are likely to occur, would ensure a reasonable amount of safety.

It has to be appreciated that in view of scanty data available, the statistical approach is not likely to give realistic estimate of earthquake parameters or ground motion characteristics, and zoning is also not possible based entirely on scientific basis. Though the magnitudes of different earthquakes which have occurred in the past are known to a reasonable amount of accuracy, the strong ground motion caused by these earthquakes at various distances have so far been mostly estimated from ratings of intensity of earthquakes based on damage surveys and there is little instrumental evidence to corroborate the conclusions arrived therefrom. Maximum intensity of earthquake at different places is rated on the basis of the observations made and recorded after the earthquake and thus a zoning map which is based on such ratings is likely to lead in some cases to an incorrect conclusion in view of : (a) incorrectness in the assessment of earthquake intensity rating; (b) human error in judgement during the damage survey; and (c) variation in quality and design of structures causing variation in type and extent of damage to the structures for the same intensity of earthquake. However, in the absence of strong ground motion data, a rational approach to the problem would be to arrive at intensity map based on available data on earthquake occurrence in terms of intensity rating magnitudes and

epicentral locations assuming all other conditions as being average, and to modify such an average idealized isoseismal map in the light of tectonics, lithology and the maximum intensities as recorded from damage surveys, etc.

The seismic zoning map of India in IS : 1893-1962 utilized intensity of earthquake based on data on earthquake occurrence as the main parameter for zoning with the basic assumption that the geology of the area is the dominant controlling factor, both for intensities and frequencies of major earthquake occurrence. Thus, seismic zone based on intensity evaluation would be in conformity with earthquake occurrence and geology (including tectonics) of the region. However, such an assumption will be valid if data on earthquake occurrence are available for sufficiently long duration. Epicentral distribution could then be considered to demonstrate the trend and extent of the seismotectonic belts conforming to the geology (and tectonics) of the region and earthquake intensities can be utilized as quantitative measures of seismicity in different parts of the region for preparation of seismic zoning maps. However, as data on earthquake occurrence in India is not available for a long period, seismic zoning maps prepared on statistical evaluation of long term seismicity from limited and or short data on earthquake occurrence due to lack of historical records and suitable network of seismological observatories, presents a picture with greater influence of known earthquakes and does not necessarily reflect the seismotectonic set up of the region, in which major earthquakes could occur due to the occurrence of geotectonic processes. With this in view when additional data on geology, tectonics and earthquake occurrence became available for the revision of the seismic Zoning map 1966 (IS : 1893-1966) shown in Fig. 26, in 1970 (IS : 1893-1970) and in 1975 (IS : 1893-1975) shown in Fig. 27 and the seismotectonic set up of the country was given greater significance.

## SEISMIC ZONING MAP OF INDIA IN IS : 1893-1966 AND IS 1893-1970

The seismic zoning map of India in IS : 1893 – 1966 (Fig. 26) took into account additional data on earthquake occurrence and detailed information on the tectonic framework of the country by (Geological Survey of India Tectonic Map of India published at the time of the International Geological Congress held at New Delhi in 1964) and followed the same general approach as for the 1962 map except that a greater recognition was given to the tectonic features.

In both seismic zoning maps of India in IS : 1893-1962 and IS : 1893-1966 outlined mostly data on past earthquake occurrence and it was assumed that the pattern of earthquake occurrence in the past will repeat itself in future. However, as data on earthquake occurrence and

its effects was not still available for a considerably long duration, these maps did not provide realistic and reasonable assessment of intensity of earthquake in future and the intensity estimates for many parts of the country were conjectural or arbitrary in character. In 1968 after the Koyna earthquake of 11 December 1967, it was considered essential to give greater emphasis to the geotectonic set up so that the seismotectonic features as source regions of earthquake occurrence in future were identified with genetic significance and estimates of earthquake intensity were consistent with the geological cause, seismotectonic framework and the pattern of earthquake occurrence in each seismic zone. Thus taking into consideration the tectogenesis and geological history of the country and the understanding of the operative processes responsible for the development of the various structural and related physiographical features which could lead to earthquake occurrence in future, the seismic zones were demarcated to broadly follow the areal extent and distribution of five principal seismotectonic units of India (*see* Table 13) :

1) the orogenic unit of Cainozonic folding and faulting;

2) the unit of Himalayan fore deep and marginal depression;

3) the unit of Peninsular Shield segmented by territory;

4) quaternary fault movements, including the Gondawana graben and adjacent parts of the Shield as well as marginal parts of the Peninsular Shield with platform cover of Mesozoic-Cainozoic sediments; and

5) the unit of generally stable Peninsular Shield with locally partitioned areas bearing relatively ancient faults and localized seismic activity (*see* Table 13).

Though a reasonable estimate of the probable maximum magnitude in each of the tectonic units could be obtained, little information was available on depth of focus. It was difficult to establish definite associations of earthquake occurrence with tectonic features in these tectonic units. Thus probable maximum intensities around the tectonic features, based on known earthquake data, were mostly tentative, till evidence of movements along these feature during earthquakes could be confirmed. It was also considered desirable to reduce the number of zones from seven zones in IS : 1893-1966 map to five zones in IS : 1893-1970 map, as the earthquake effects below Modi-

fied Mercalli intensity VI (seismic zones 0 and I of IS : 1893-1962 and IS : 1893-1966 seismic zoning maps of India) are insignificant in terms of design to call for a separation, Likewise the seismic zone VI of IS : 1893-1962 and IS : 1893-1966 seismic zoning maps for M.M. intensity and above was omitted as recorded strong ground motion characteristics in regions with Modified Mercalli intensity IX and above show comparable ground accelerations, though with longer durations of strong ground motion with increase in magnitude. Thus the separation between seismic zone V and VI was considered to be of little practical utility.

From the foregoing description it will be noted that seismic zoning based on data on past earthquake occurrence had greater acceptability by the users in the initial stages of the implementation of the Indian Standard recommendations for earthquake resistant design of structures. The seismic zoning maps in IS : 1893-1962 and IS : 1893-1966 were thus mostly based on data on past earthquake occurrence and probable correlations with known tectonic features. With the general understanding of the earthquake engineering problems, greater recognition to seismotectonic framework of the various parts of the country to predict new potential earthquake source regions and their effects could be given in the preparation of the 1970 seismic zoning map included in IS : 1893-1970. This made it possible to take into consideration the probable seismic activity in potential earthquake source regions where there had been a gap in activity during the period for which data is available, and serious economic losses could be reduced due to seismological uprise in such gaps in future. This has been incorporated in the third revision of the Code (IS : 1893-1975) without any modification. All these maps are based on subjective estimates of intensity from available information on earthquake occurrence, geology and tectonics of the country. However, as more data on earthquake occurrence in India and their associated seismotectonic elements become available for seismic risk analysis to provide adequate safety during the desired life of the structures these zones will need revision. It is emphasized that seismic zoning of a country is a continuous process depending on its gradual acceptance by users, and the guiding philosophy and methodology of analysis, evaluation of seismic risk for seismic zoning depend on the available data and state of knowledge of the fundamental seismological, geological and tectonic principles for the physical understanding and forecasting earthquake occurrence in future.

**TABLE 13 GENERALISED TECTONIC UNITS OF INDIA WITH DECREASING MAGNITUDE AND FREQUENCY OF EARTHQUAKE OCCURRENCE AFTER KRISHNASWAMY[10][11]**

| TECTONIC UNIT | | EARTHQUAKE OCCURRENCE | SEISMIC ZONES |
|---|---|---|---|
| Name | Description | | |
| Orogenic unit | Orogenic unit of Cainozoic folding and faulting. The Shillong massif, which has been greatly affected by this faulting has been included in this zone | Common shocks of magnitude 5-6·5 with a number of shocks of magnitude 6·5-7·5, a few shocks of magnitude 7·5-8 and occasional shocks greater than 8 originating on some of the major Himalayan thrust and faults (Satlitta thrust, Panjal thrust, Central Himalayan thrust, Dauki fault, etc.) | V and IV |
| Foredeep and marginal depression unit | Unit of Himalayan foredeep and marginal depression (where the boundary is not positively established, some of the Shield may really be included in this Zone. The tectonic map provisionally defines the boundary at 200 to 1 000 m contour of the basement at margin of the Shield) | Common shocks of magnitude 5-6 with a few shocks of magnitude 6-7 and occasional shocks of magnitude 7·5-8 originating along active faults in the basement (Patna fault of other basement faults, Kutch faults) | IV and III with islet of V |
| West Coast and Narmada Tapti unit | Unit of Shield with tertiary quarternary fault movement including the West Coast seismogenic Zone, the Narmada-Son rift Zone, the Tapti rift Zone and their postulated extensions | Common shocks of magnitude 5-6 with few shocks of magnitude 6-7 in the Narmada and Tapti rifts, past epicentres can be related to extensions of partly mapped faults. Maximum recorded magnitude on West Coast Zone 6·6-7, on Narmda rift 6·5, Tapti rift 6·25 | III with islets of IV\ |
| Gondwana Rifts unit | Unit of Shield with Mesozoic fault movements and later adjustments, includes the Gondwana rift zone and adjacent parts of the Shield, marginal parts of the Peninsular Shield to the east and north with platform cover of Mesozoic-Cainozoic sediments | Occasional shocks of magnitude 5-6 with few centres which may have magnitude 6-6·5 and may be related to boundary faults of the Gondwana basin and faults of limited extent in the Mesozoic-Cainozoic cover on the platform | III |
| Shield unit | Generally a seismogenic and partitioned areas of the Peninsular Shield with ancient faults and with localized faults and with localized seismogenic features | Occasional shocks of magnitude 5-6 with exceptional activity along local faults in the Archaeans with magnitude 6-6·5 | I and II with islets of III |

Fig. 25   Map of India Showing Seismic Zones (IS : 1893-1962)

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FIG. 26  MAP OF INDIA SHOWING SEISMIC ZONES  (IS : 1893-1966)

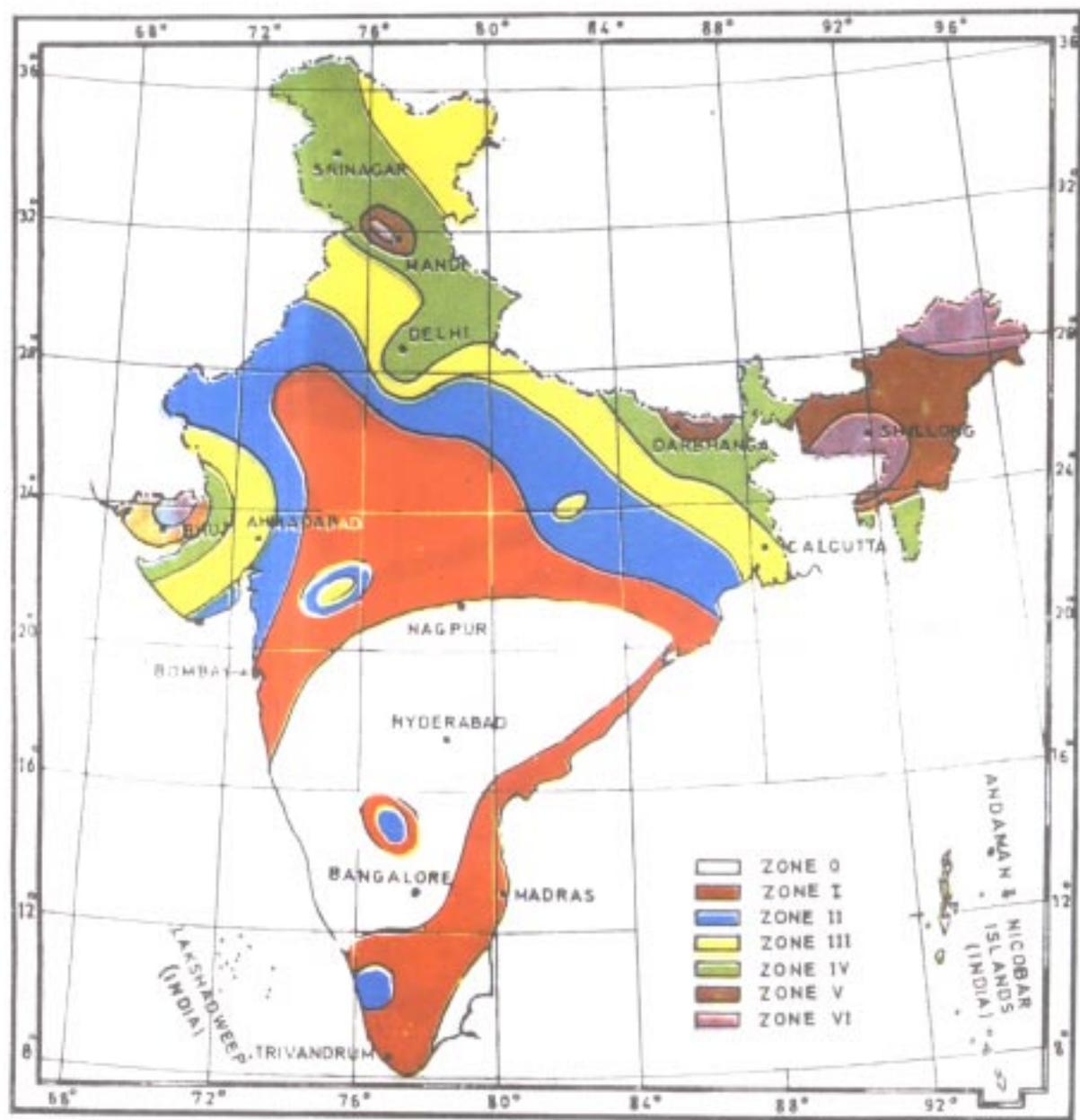As in the Original Standard, this Page is Intentionally Left Blank
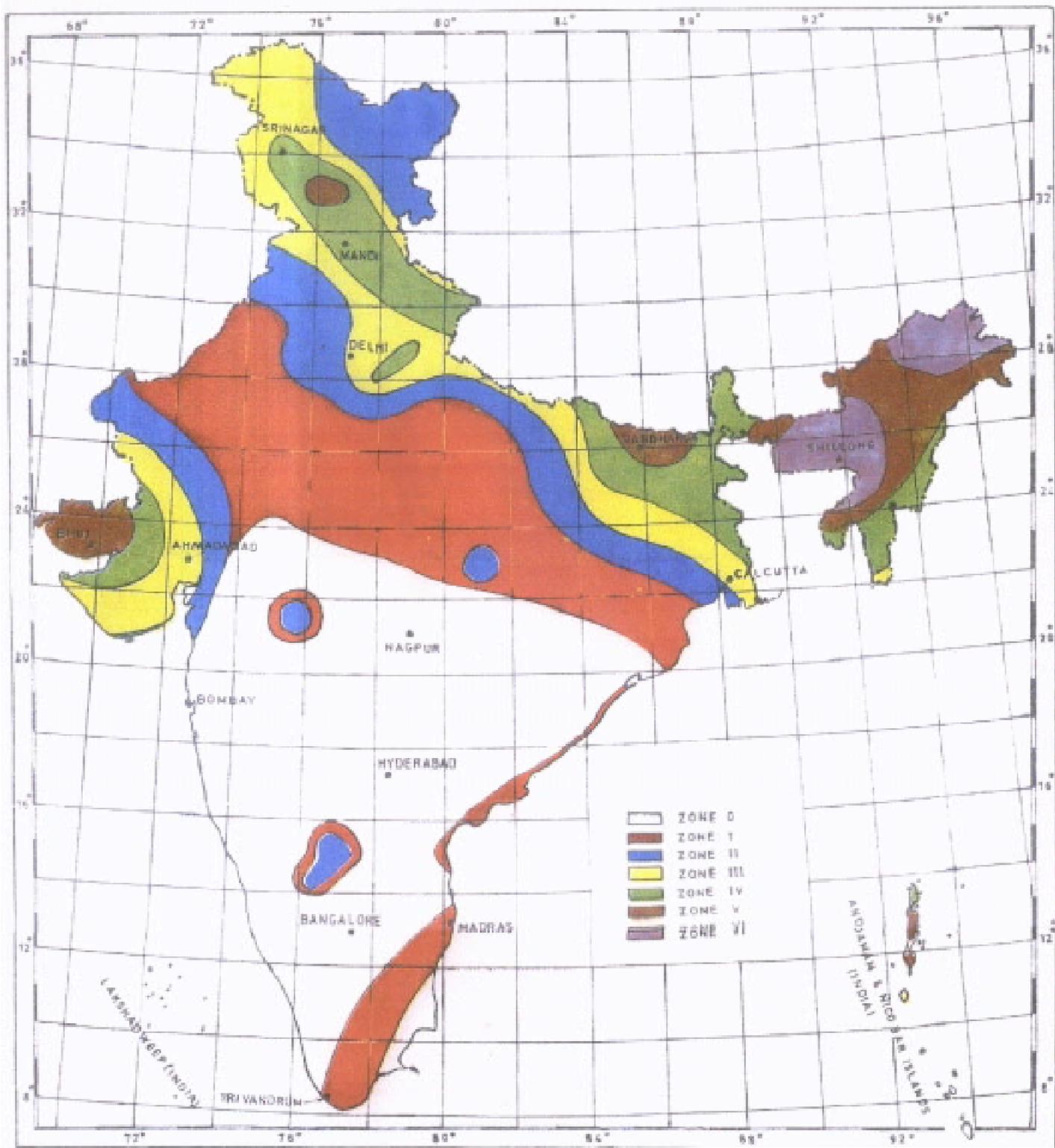
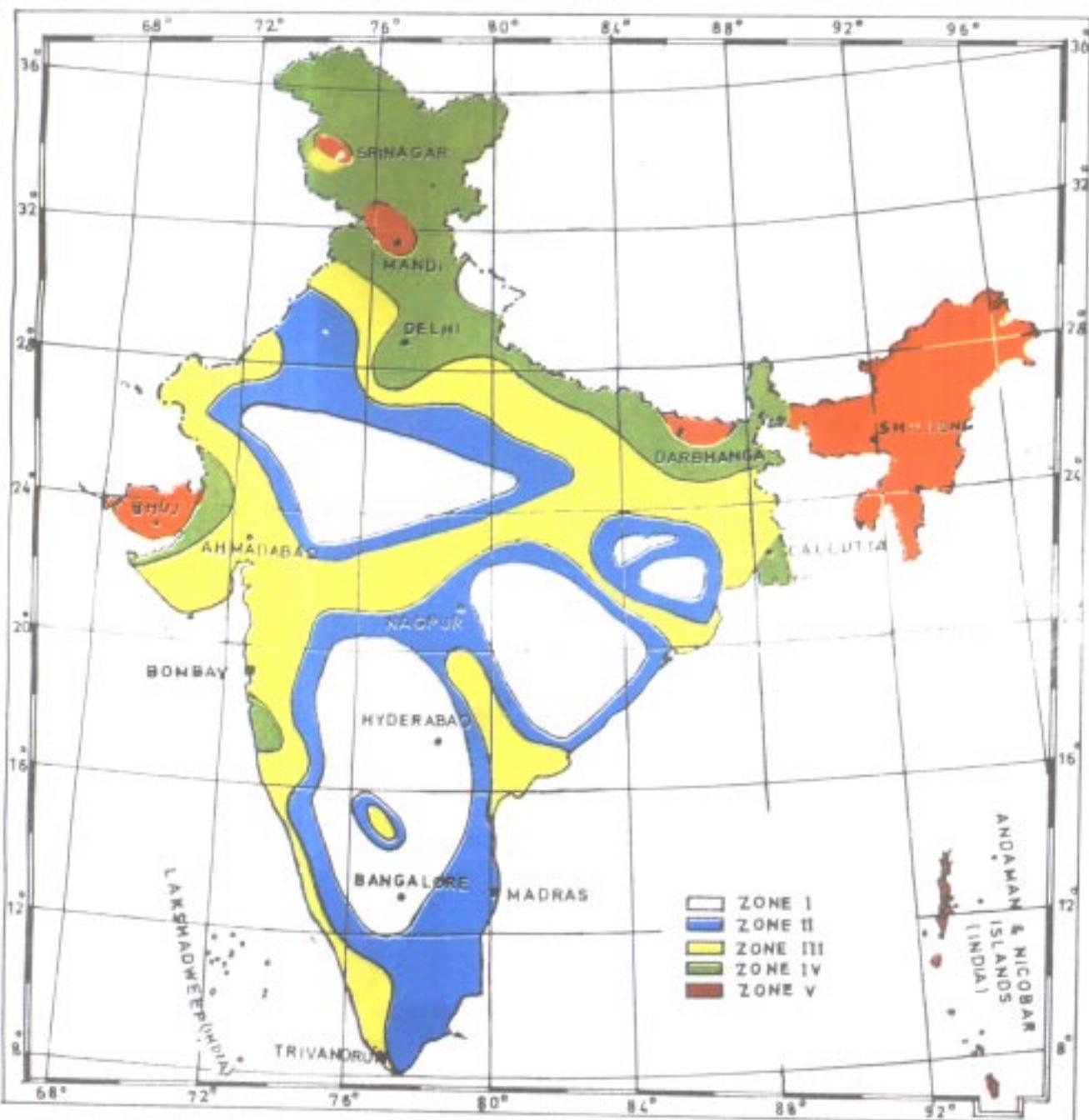Based upon Survey of India map with the permission of the Surveyor General of India.

© Government of India Copyright, 1983,

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured
from the appropriate base line,

The responsibility for correctness of internal details rests with the publisher,

Fig. 27   MAP OF INDIA SHOWING SEISMIC ZONES (IS : 1893-1970)

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# PART II

## EXPLANATIONS ON
## IS : 4326-1976 CODE OF PRACTICE
## FOR EARTHQUAKE RESISTANT
## DESIGN AND CONSTRUCTION

### ( First Revision )

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# 0. FOREWORD

The Code of practice for earthquake resistant design of structures was originally brought out in 1962 and has been revised three times and the current version of the Code in use being IS : 1893–1975. This Code deals with the seismic forces to be taken into account while designing different structures in various parts of the country. However, a need was felt to lay down specific provisions indicating various construction features to be incorporated in buildings to be constructed in various seismic regions. IS : 4326 was, therefore, brought out for this purpose originally in 1967 and has been revised in 1976. This Code takes into account the prevailing construction practices in the country and lays down guidelines for construction of steel, reinforced concrete, masonry and timber buildings.

# 2. TERMINOLOGY

**2.1.1** *Crumple Section* — A separation section filled with appropriate material like sheet metal flashing, particle board, cork, etc, which can crumple or fracture in an earthquake. The material filled will not obstruct the free movement of the building when it vibrates during an earthquake.

**2.2 Centre of Rigidity** — The centre of rigidity and centre of mass at which the lateral load acts are two different terms. Centre of rigidity is calculated considering the stiffnesses of resisting elements like columns and walls in the respective directions. Centre of rigidity is the point in a structure where a lateral force shall be applied to produce equal deflections of its components at any one level in any particular direction.

**2.3 Shear Wall** — *See* Fig. 1.

**2.4 Space Frame** — *See* Fig. 2.

**2.4.2** *Moment Resistant Frame* — *See* Fig. 3.

**2.4.3.** *Moment Resistant Frame with Shear Walls* — *See* Fig. 4.

**2.5 Box System** — The box system utilizes walls which support vertical loads and also resist seismic forces. There may be other elements, such as a partial simple frame system, which support a portion of the vertical loads. This system also has horizontal resisting elements, such as diaphragms and trusses, which transmit the seismic forces to the walls.

Box system functions in the same manner as a rectangular box in resisting the internal forces in an earthquake. As illustrated in Fig. 5, the principal elements of the system are as follows:

a) *Vertical Beams* — The exterior walls and interior partitions act as vertical beams delivering reactions to the floors.

b) *Horizontal Diaphragms* — The floor and roof act as horizontal girders which resist the reactions from the vertical elements,
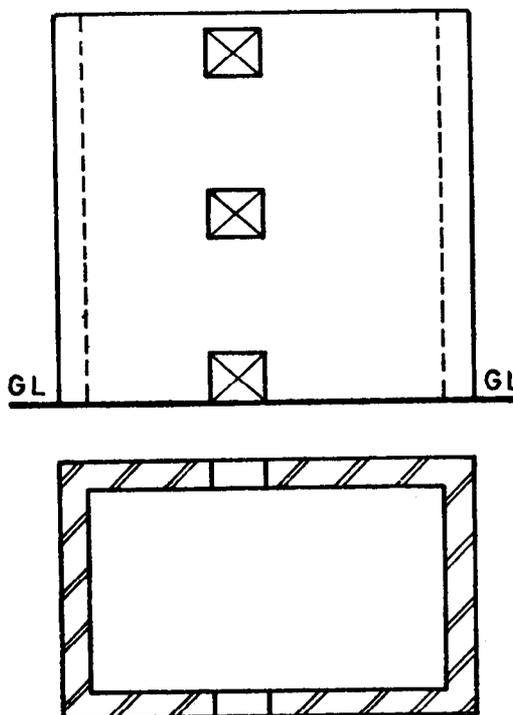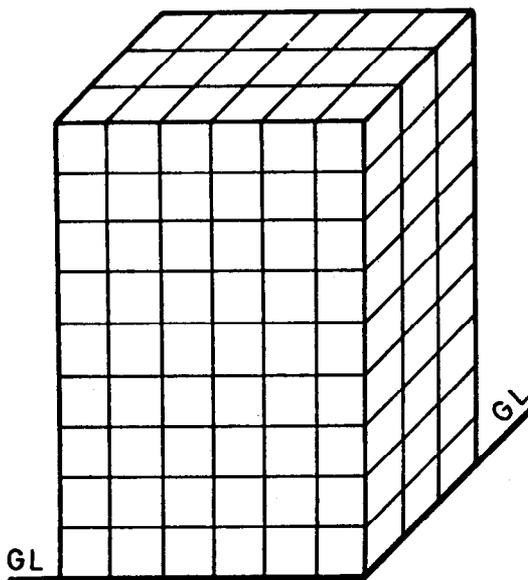


FIG. 1 SHEAR WALL



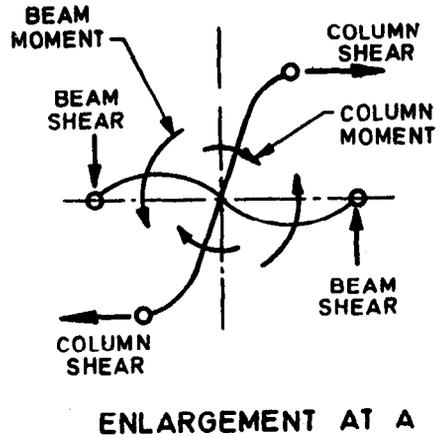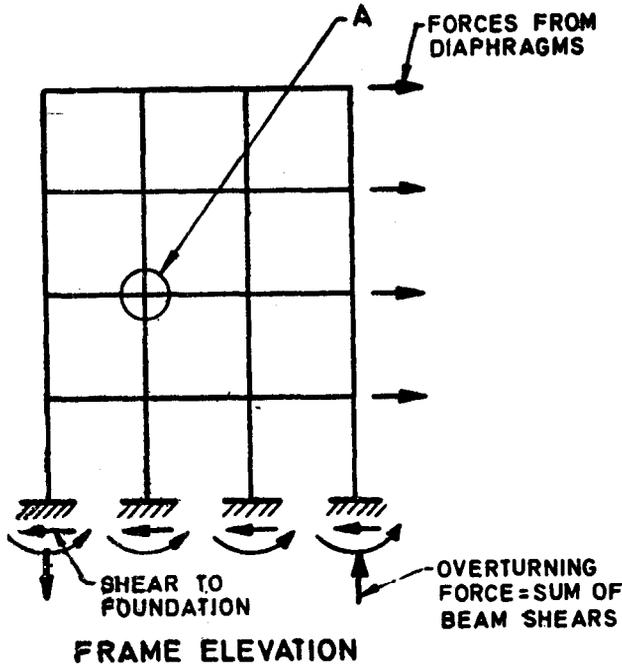FIG. 2 SPACE FRAME

ENLARGEMENT AT A
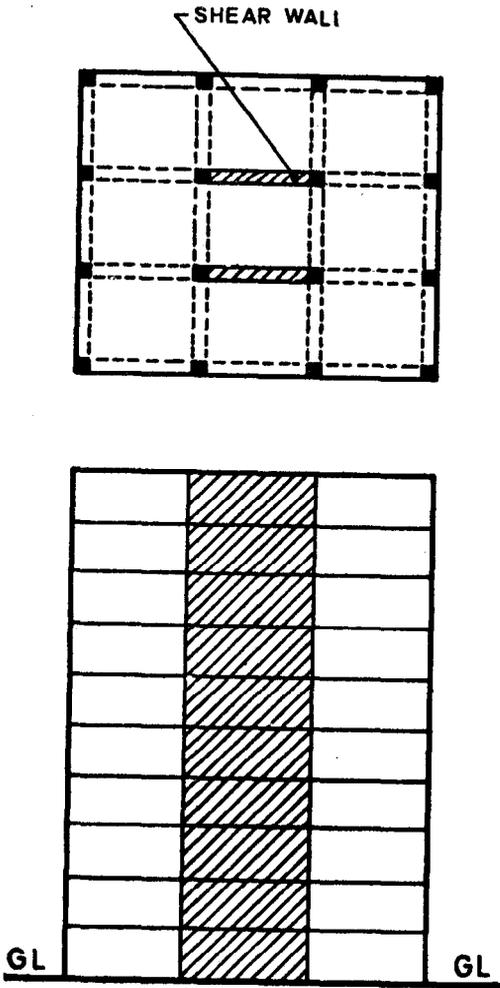
FRAME ELEVATION

FIG. 3 MOMENT RESISTING FRAME



FIG. 4 MOMENT RESISTANT FRAME WITH SHEAR WALLS

as well as inertia forces generated from the mass of the floor or roof itself.

c) *Shear Walls* — The structural walls oriented parallel to the direction of the load resist the reactions from the diaphragm as well as inertia forces from the mass of the wall itself. The shear wall then acts as a vertical cantilevered beam supported at the base of its foundation.

The basic elements of the box system listed above can take any one of a number of forms, but the basic function will remain same. The diaphragm can consist of almost any structural material or could be replaced by horizontal trusses or bracing systems consisting of steel, wood or concrete members without departing from the basic system. Similarly, the shear walls could consist of any of many different materials or even replaced by a system of bracing or vertical trusses.

**2.6 Band** — A reinforced concrete or reinforced brick runner provided in the walls to tie them together, so that the structure acts as one unit to resist the earthquake effects. These bands are generally provided at plinth, lintel or roof level. The most common among these is the lintel band.

### 3.2 Continuity of Construction

**3.2.1** Generally all parts of the building should be tied together in such a manner that the building acts as one unit. Plinth band, lintel band or roof band are used for this in brick masonry construction.

**3.2.3** Continuous rows of openings in a load bearing wall makes it weak and hence it should be preferably avoided or otherwise the openings should be staggered. The openings should also be reinforced all round.
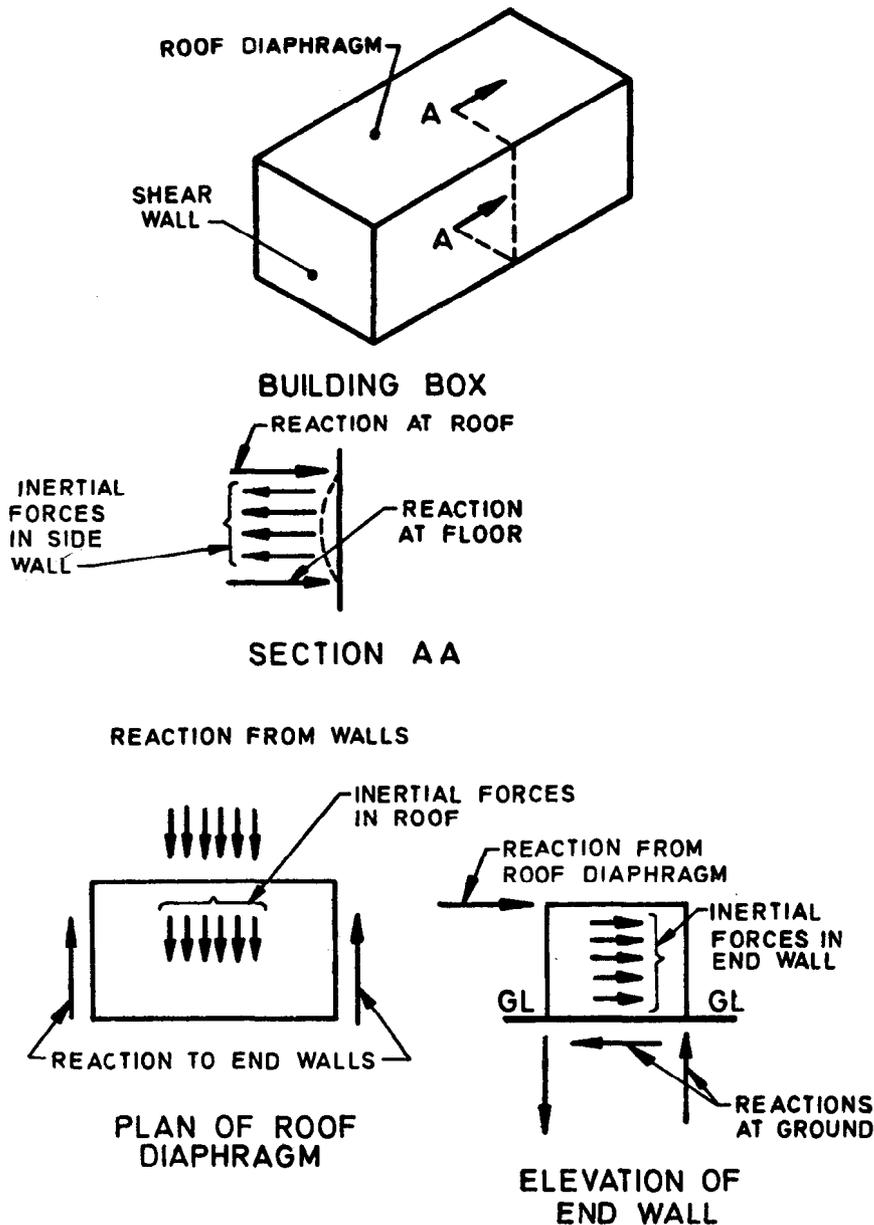
ROOF DIAPHRAGM

A

SHEAR
WALL

A

BUILDING BOX

REACTION AT ROOF

INERTIAL
FORCES
IN SIDE
WALL

REACTION
AT FLOOR

SECTION AA

REACTION FROM WALLS

INERTIAL FORCES
IN ROOF

REACTION TO END WALLS

PLAN OF ROOF
DIAPHRAGM

REACTION FROM
ROOF DIAPHRAGM

INERTIAL
FORCES IN
END WALL

GL        GL

REACTIONS
AT GROUND

ELEVATION OF
END WALL

FIG. 5   BOX SYSTEM

**3.2.4** If a new structure is built by the side of an existing structure adequate precaution should be taken to provide separation between them to avoid hammering during seismic vibration.

## 3.3 Projecting Parts

**3.3.1** Overhanging parts, such as projecting cornices, balconies, parapets and chimneys are the first to fall during an esrthquake. Not only that there is damage to the building but such parts, when they fall, injure the people who may be running out of the houses or moving on the streets. Such projecting and overhanging parts should be avoided as far as possible or enough care should be taken to reinforce them and anchor them to the main structure adequately.

**3.3.2** Ceiling plaster should be as thin as possible and the suspended ceiling should be avoided as far as possible. Light weight suspended ceiling in any building should be adequately fastened. Such ceilings often used for aesthetic reasons are usually brittle and weak and incapable of resisting horizontal vibrations. Thus special care is required in the design of suspended ceilings if they cannot be avoided. They should be strong and rigidly tied to the roof truss or be ductile enough to withstand the strains during ground motion.

## 3.4 Shape of Building

**3.4.1** Unsymmetrical buildings usually develop torsion due to seismic forces. Hence the building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity, so that the centre of mass and centre of rigidity of the building coincide with each other and there is no eccentricity. Since zero eccentricity is very difficult to achieve in design, efforts should be made to have minimum eccentricity in the building with suitable provisions for torsional effect. Irregular shape buildings may be designed as a combination of few regular shapes with suitable construction joints. The design requirements for torsion are covered in IS : 1893-1975*.

**3.4.2** If symmetry of the structure is not possible in plan and elevation, provisions should be made for torsional effect due to earthquake forces in the structural design by increasing the lateral shear. Also parts of different rigidities may be separated. Between the separation sections the length to width ratio of the buildings should not normally exceed three. Simple rules for plan layouts are illustrated in Fig. 6.

**3.5 Strength in Various Directions** — The structure should be designed to have adequate strength against earthquake effects along both the horizontal axes. In providing resisting elements care should be taken to see that the structure has more or less equal stiffness in both directions. Large differences in stiffness in two directions may lead to large eccentricity causing torsion in the structure.

**3.6 Foundations** — The foundations founded on soils liable to liquefy need design considerations to avoid damage to the structure due to foundation failure. However, such designs will be much expensive.

**3.7 Ductility** — To avoid sudden collapse of the structures during an earthquake and to enable them to absorb energy by deformation beyond yield point, the main structural elements and their connections should be so designed that the failure
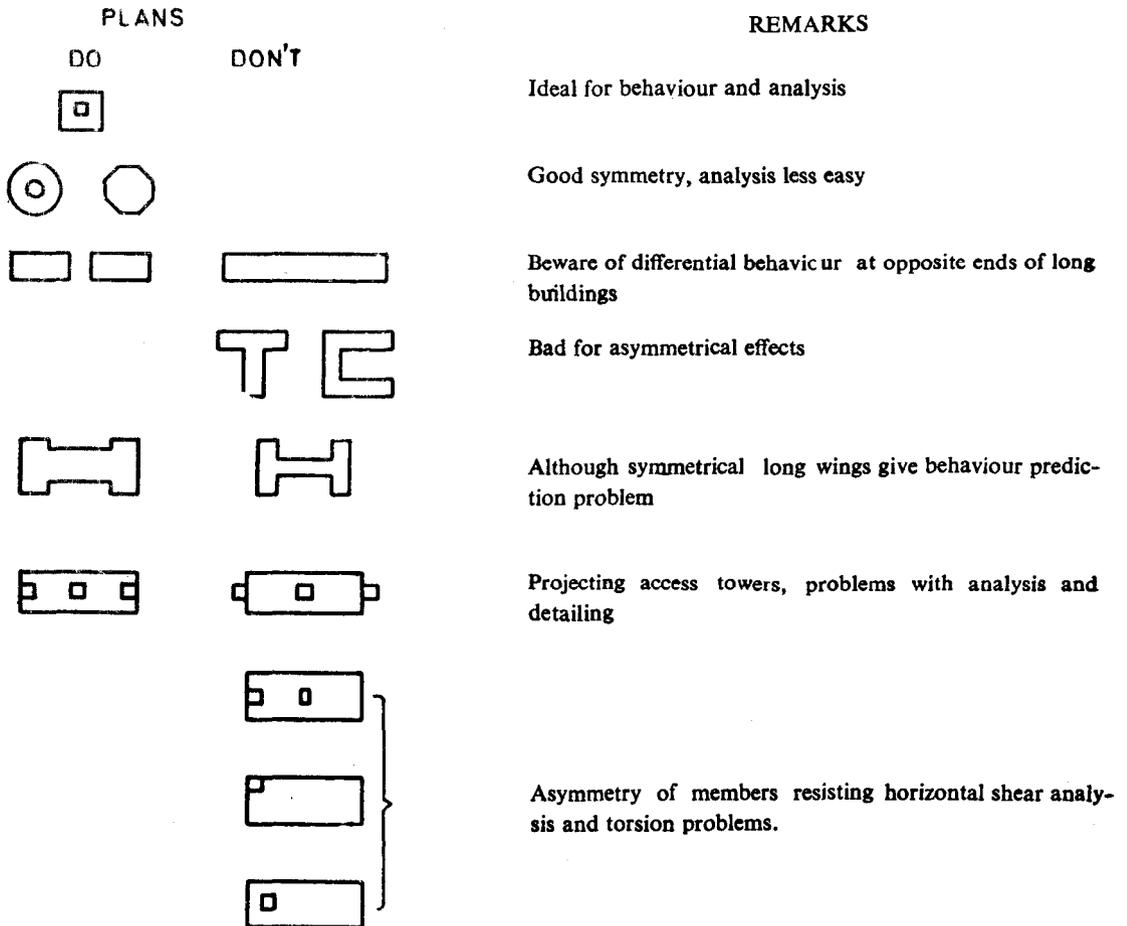
PLANS                  REMARKS

DO     DON'T

Ideal for behaviour and analysis

Good symmetry, analysis less easy

Beware of differential behaviour at opposite ends of long buildings

Bad for asymmetrical effects

Although symmetrical long wings give behaviour prediction problem

Projecting access towers, problems with analysis and detailing

Asymmetry of members resisting horizontal shear analysis and torsion problems.

FIG. 6   GUIDELINES FOR PLANNING OF SEISMIC BUILDINGS

*Criteria for earthquake resistant design of structures (*third revision*).

is of a ductile nature. From earthquake consider-
ations a ductile structure is preferable as it has
enough energy to absorb the shocks. Therefore,
detailing of the member (including minimum
reinforcement) should be such that the sudden
collapse of structure is avoided in the event of any
failure.

Ductile structure yields and absorbs and
dissipates very large amounts of energy by moving
out of the elastic range well into the inelastic
range. In favourable cases, the displacements of
the inelastic structure are similar to those of elastic
structure and, therefore, the forces in the elastic
yielding structure are much less.

**3.8 Damage to Non-Structural Parts** — Non-
structural parts, such as filler walls, partitions, etc,
should be designed and connected to the main
structure in such a manner so as to minimize
their damage during an earthquake. The reason
for this are that in addition to being hazardous,
repairs or replacement of such parts is quite
costly.

**3.9 Fire Safety** — It has been observed that
fire frequently follows an earthquake and, there-
fore, the buildings should be constructed to make
them fire resistant in accordance with the provi-
sions of relevant Indian standards for fire safety.
If fire breaks out, the components of the building
should be able to retard the fire for a few hours,
that is the material used in construction should
have fire resistance ratings specified in the National
Building Code.

# 4. SPECIAL CONSTRUCTION FEATURES

**4.1** The Code specifies the gap widths for the
various types of buildings and for different seismic
coefficients.

The separation of the joints should be complete
except below the plinth level, since the structure
is assumed fixed at that level and only the structure
above vibrates.

**4.2.1** The clearance between adjacent structures
or parts of the same structure which are dissimilar
in mass or stiffness should provide for maximum
amplitudes of the motion of each structure or
part thereof. Fragile crumple joints continuous
through walls and roof, are often used to cover
this gap. The Code suggests details of such crumple
sections (*see* Fig. 2 of the Code).

**4.3 Foundations**

**4.3.1** The following Indian standards are useful
in the design of building foundations which should
take into account seismic forces on the structure
specified in IS : 1893-1975*:

---

*Criteria for earthquake resistant design of structures
(*third revision*).

| | | |
|---|---|---|
| 1) | IS : 1080-1980 | Code of practice for design and construction of simple spread foundations (*first revision*) |
| 2) | IS : 1904-1978 | Code of practice for structural safety of buildings : Shallow foundations (*second revision*) |
| 3) | IS : 2950 (Part I)-1982 | Code of practice for design and construction of raft foundations : Part I Design (*second revision*) |
| 4) | IS : 2911 (Part I/Sec 1)-1979 | Code of practice for design and construction of pile foundations : Part I Concrete piles, Section 1 Driven cast *in-situ* concrete piles (*first revision*) |
| 5) | IS : 2911 (Part I/Sec 2)-1979 | Code of practice for design and construction of pile foundations: Part I Concrete piles, Section 2 Bored cast *in-situ* piles (*first revision*) |
| 6) | IS : 2911 (Part I/Sec 3)-1979 | Code of practice for design and construction of pile foundations : Part I Concrete piles, Section 3 Driven precast concrete piles (*first revision*) |
| 7) | IS : 2911 (Part III)-1980 | Code of practice for design and construction of pile foundations : Part III Under-reamed piles (*first revision*) |

**4.3.2** The Code recommends that as far as
possible entire building should be founded on the
same type of soil in order to avoid differential
settlement.

**4.3.3** Loose fine sand, soft silt and expansive
clays may give rise to large differential settlements
and should generally be avoided. However, a raft
foundation in such soils is less vulnerable and may
be used. Alternatively, a pile foundation would
also obviate such a problem. If the structure is
light (where a pile/raft foundation is not required),
the Code suggests some methods for improving
the foundation soil.

**4.3.4** To avoid large differential settlements,
all the individual footings or pile caps in Type III
soft soils shall be connected by ties or slab. The
ties or the slab should be designed to take up the
forces resulting from the seismic forces. The
magnitude of these forces is difficult to determine.

A conservative approach for design of these members is indicated in the Code, recognizing the buckling aspect of the members.

**4.4.1 & 4.4.1.1** The floor and roof construction should be made as rigid as possible in the horizontal plane and shall be built into the walls along their perimeter. There have been evidences in 1934 earthquake of Bihar that the verandah and porches with series of independent pillars with roof merely resting on top had failed badly; jack arched roofs and roofs consisting of beams and rafters supporting brick tiles were damaged and ill maintained tiled roof proved very hazardous. Due to shaking, the gap between two joists widened, and tiles and bricks fell. Such roofing and flooring units, where used instead of monolithic slabs, are to be tied together and fixed to the supporting members so as to prevent their dislodging due to shaking. For this reason, corrugated iron or asbestos sheets would be found better than earthern tiles, slates, etc. Joists, timber or reinforced concrete, if used for supporting flooring units, should be blocked at ends and tied together so as to prevent any relative displacement between them.

### 4.4.2 *Pent Roofs*

**4.4.2.1** Due to the horizontal force acting on the truss at the time of an earthquake, if the ends are not held down by bolts, the truss will be displaced horizontally and may fall off its bearing and may be hazardous. Hence, it is recommended that ends of the truss should be bolted down to take up the required seismic force.

**4.4.2.2** Diagonal bracing should also be provided at the tie level of the trusses so as to make them stiffer and distribute the horizontal forces due to the earthquakes.

**4.4.3** *Jack Arches* — The behaviour of arches has been found to be unsatisfactory in resisting the horizontal loads due to earthquake as they have a tendency to separate out and collapse. To prevent this, the Code recommends that Jack arch roofs or floors where used should be provided with mild steel ties in the end spans and where the number of spans is large, such ties should also be provided in every fourth span.

### 4.5 Staircases

**4.5.1** Concrete stairways often suffer seismic damage due to differential displacement of the connected floors. This can be avoided by providing open joints in each floor at the stairway to eliminate the bracing effects or by providing an adequate load path for the forces by using stairwalls extending the full height of stairs. Three types of staircases are recommended in the Code.

## 5. TYPES OF CONSTRUCTION

**5.2 Framed Construction** — This construction is suitable where heavy loads are expected such as in multistoreyed buildings and also in industrial structures.

**5.2.2.1** In the case of design of moment resistant frames with shear walls the total lateral force due to earthquake should be distributed to frame and the shear wall. Although the shear wall would take up most of the lateral load in view of its large in plane stiffness, the frame by itself should also be capable of resisting some part of load. A minimum value equivalent to 25 percent of total earthquake force is recommended by the Code for this purpose.

**5.2.2.3** On the basis of studies carried out, it is observed that if the height of shear walls is about 85 percent of the total height of building, the efficiency is optimum.

**5.3 Box Type Construction** — *See* Fig. 5.

## 6. STEEL CONSTRUCTION

Steel frames consist of four main parts which play their role in lateral load resistance : beams, columns, connections and diaphragms. All parts must be so designed that they have adequate stiffness, strength and plastic deformation capacity. The significant features are:

a) *Beams* — The factors to be checked besides bending strength are:

1) Local buckling of plate elements in compression,

2) Lateral buckling of compressionflange, and

3) Ultimate shear strength.

Since the moments due to earthquake forces are reversible in nature, both flanges must be checked for local and lateral buckling. It may be pointed out that since the criterion of design now is deformation capacity, the limits on minimum dimensions will be more than in the elastic range particularly in the plastic hinge zones. The moment gradient along the length of beam should be taken into account in determining more economic limits of buckling parameters.

b) *Columns* — Under earthquake condition, the columns are usually bent into double curvature and carry axial loading which may be from 15 to 50 percent of the yield load $P_y$ of the section. The plastic hinge's, if any, occur at the end of columns. The buckling of column in the plane of

bending or in the perpendicular plane decreases the rotation capacity at the ends. Therefore, it is important to check the column section for slenderness ratio in both the principal planes, local buckling of plate elements, twist buckling of the section and combined axial load and bending. Again, the restriction will be much more severe than in the elastic range. Since the collapse of a column will lead to the collapse of a part of the building, this type of damage is too serious to be left to chance. The best course perhaps would be to avoid this problem by preparing a weak-girder strong-column design of the steel frame forcing the plastic hinges to develop in the beams rather than in the columns.

c) *Connections* — In steel frame buildings, usually the columns are made continuous through the joints and the beams are connected to them. If the plastic hinges under earthquake or wind loading are forced to form at the beam ends, the connections are automatically required to have adequate strength and ductility to meet the demand. Where connections are made by riveting or bolting, the ductility is provided by connecting angles and tees. Where connections are made by welding, such details should be adopted that the welds are not required to yield but the connecting plates should provide the yielding. In each case, stiffners must be included on column webs to prevent web crippling and shear buckling and all plate elements must satisfy the local buckling design criteria for plastic deformations.

d) *Diaphragms* — In all tier buildings, the floors act as the horizontal diaphragms and transmit the lateral load to the stiffer elements, that is, vertical diaphragms like shear walls. The connecting elements between the two will have to carry heavy axial and lateral forces and must be designed adequately. Diagonal bracing elements including tension members should also have such slenderness ratios that elastic buckling does not occur.

# 7. REINFORCED CONCRETE CONSTRUCTION

**7.1.2** The primary members of structure, such as beams and columns, are subject to stress reversals from earthquake loads. The reinforcement must be provided considering the reversal of sign of moments in the beams and columns.

**7.1.3** The monolithic beam-column connections are desirable so to accommodate reversible lateral deformations after the formation of plastic hinges. The maximum moments caused by earthquake occur near the ends of the beams and columns, the plastic hinges are likely to occur there and most of the ductility requirement apply to sections near the junctions.

**7.1.4** The earthquake shear forces can change sign in the beam. The vertical closed stirrups are preferable to inclined bars because the vertical closed stirrups can resist shears of any sign. The Code recommends to restrict the shear capacity of inclined bars to 50 percent because of its partial effectiveness in resisting reversible shears. The close spacing of steel stirrups is desirable because it increases the ultimate strain in concrete and improves the behaviour of beam in diagonal tension.

**7.1.5** The earthquake motion often induces forces large enough to cause inelastic deformations in the structure. If the structure is brittle, sudden failure could occur. But if the structure had ductile behaviour, the structure will be able to sustain the earthquake effect with some deflection $\Delta m$ larger than the yield deflection $\Delta y$ by absorption of energy. Therefore, besides the design for strength of the frame, ductility is also required as an essential element for safety from sudden collapse during severe shocks. The past experience on the behaviour of buildings during earthquakes has clearly shown that structures designed for low seismic coefficients of the Code survived the severe earthquake because of energy absorption in plastic deformations. The plastic deformations are, therefore, considered to bring down the effective values of design seismic coefficients. The decision on the extent of admissible plastic deformation should rest with the designer. A ductility factor of about 5 is considered adequate for reinforced concrete. To withstand the severe shocks, the Code recommends that ductility provisions be carefully adopted. This recommendation is generally applicable to all seismic zones but its importance is greater where the severe earthquake loading will become much more significant than other concurrent loads, as may be the case where the design seismic coefficient (including the effects of soil-foundation and importance factors) will be 0·05 or more. Therefore, the Code specifies these requirements especially in these cases. It will certainly be safer to adopt them for smaller design coefficient cases as well.

## 7.2 Flexural Members

a) *Objective of Design* — The purpose of earthquake resistant design in reinforced concrete is to avoid sudden collapse due to diagonal tension, buckling or bond failure. The possibility of such failures should not arise at all before the members become fully plastic and achieve their ultimate strength in flexure.

b) *Flexural Capacity at Ultimate Strength* — Members designed to resist earthquake effects should be such that the upper limit of strain of compressed concrete reaches after yielding of tension steel. This may be achieved by keeping tensile steel ratio sufficiently less than the balanced value. Such sections are classified as moderately reinforced sections, these may contain 0·75 to 2 percent steel for M 150 concrete. These sections will attain their ultimate capacity after development of considerable inelastic strain in steel. The over reinforced sections in which the upper limit of strain of compressed concrete reaches before yielding of tensile steel are undesirable in earthquake resistant design because of their restricted energy absorption capacity.

c) *Ductility Ratio* — The ductility ratio of a member in the limit state design is often defined as the ratio of deformation at ultimate to that at yield. A convenient measure of ductility is thus a ratio $\phi_u/\phi_y$, the ratio of curvature at ultimate to that at yield.

**7.2.1** For moderately reinforced section with minimum tensile steel ratio $(K_u)$, the ratio of depth of neutral axis at ultimate to effective depth will lie in the following limits:

$$\frac{\epsilon_{cu}}{\epsilon_y + \epsilon_{cu}} > K_u > \frac{\epsilon_{cu}}{\epsilon_h + \epsilon_{cu}} \qquad ...(1)$$

where

$\epsilon_{cu}$ = Concrete strain at ultimate

$\epsilon_y$ = yield strain of steel

$\epsilon_h$ = strain at strain hardening in steel

$K_u = \dfrac{\rho F_y}{f_{cu}}$

$\rho$ = $A_s/bd$ tensile steel ratio

$F_y$ = yield stress in steel

$f_{cu}$ = average stress in concrete in the compression zone at ultimate

= $0·7 f'_c$

= $0·55 F_c$

$f'_c$ = 28 day cylinder strength

$F_c$ = 28 day cube strength

The minimum and maximum steel ratio can be obtained from equation (1), that is,

$$\rho_{min} \frac{F_y}{f_{cu}} > \frac{\epsilon_{cu}}{\epsilon_h + \epsilon_{cu}} \qquad ...(2)$$

$$\rho_{max} \frac{F_y}{f_{cu}} < \frac{\epsilon_{cu}}{5\epsilon_y + \epsilon_{cu}} \qquad ...(3)$$

Here a ductility factor 5 is included in denominator of equation (3). This means that steel strain is required to be 5 times more than $\epsilon_y$. For members with compression steel ratio, $\rho$, the expression for $K_u$ takes the following form*:

$$K_u = \frac{(\rho - \rho_c) F_y}{f_{cu}} \qquad ...(4)$$

where $\rho$ = tensile steel ratio.

Making use of (3) and (4), the maximum tensile steel ratio can be obtained from,

$$(\rho_{max} - \rho_c) \frac{F_y}{f_{cu}} < \frac{\epsilon_{cu}}{5\epsilon_y + \epsilon_{cu}} \qquad ...(5)$$

Using $\epsilon_{cu} = 0·033$, $\epsilon_h = 0·024$,

$f_{cu} = 0·55 F_c$ in Equation (2)

$$\rho_{min} = 0·0611 \frac{F_c}{F_y}$$

For M 150 concrete, $F_c = 150$ kg/cm²,

and $F_y = 2\,600$ kg/cm²

$\rho_{min} = 0·003\,52$

Under the action of an actual earthquake, flexural members will probably be subjected to a number of reversals of bending moment. To meet this situation, a minimum amount of compression steel equal to about 0·35 percent of the section must be provided which should be anchored adequately so as to be able to act as tension reinforcement also. Similarly, the same minimum amount of reinforcement should also be provided at the top of beam throughout the length so as to take care of the shifting of the points of contraflexure towards the centre of span due to increase of negative moment at one end and decrease at the other. Not less than 2 bars each should be provided at top as well as bottom of any section.

**7.2.2** The maximum tensile steel ratio in a doubly reinforced member specified in this clause can be obtained from equation (5):

Calculation of amount of maximum tensile steel, $\rho_{max}$ : substituting $f_{cu} = 0·55 F_c$ in equation (5),

$$\rho_{max} = \rho_c + 0·55 \frac{\epsilon_{cu}}{(5\epsilon_y + \epsilon_{cu})} \frac{F_c}{F_y} \qquad ...(6)$$

Using, $\epsilon_{cu} = 0·003$, $\epsilon_y = 0·001\,2$
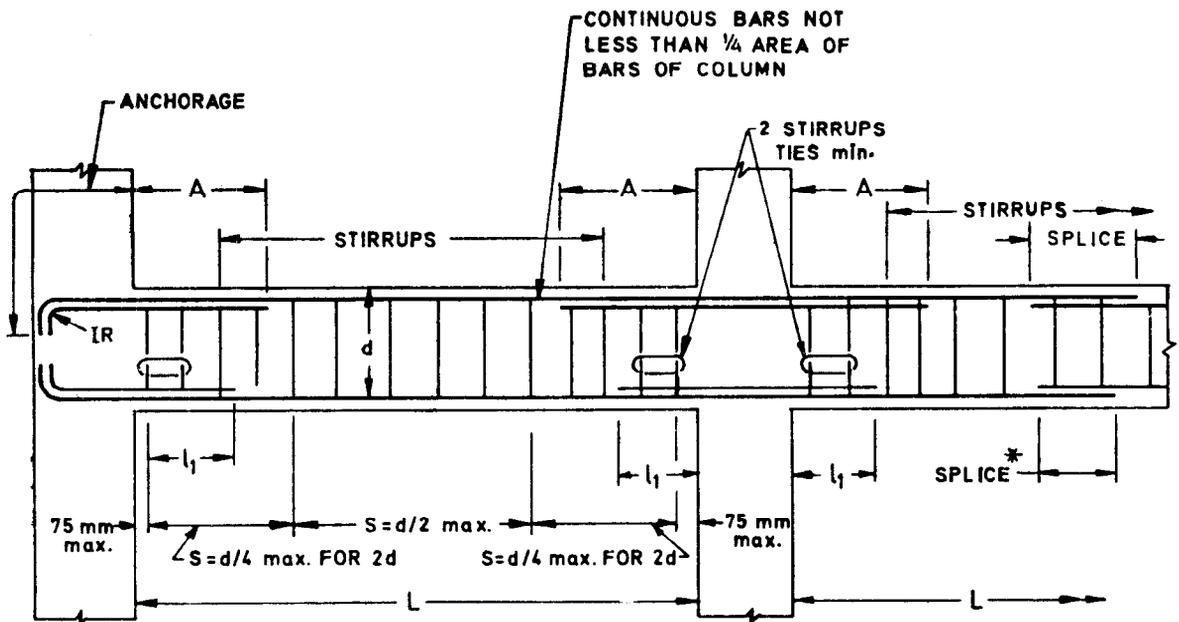
$\rho_{max} = \rho_c + 0·19\, F_c/F_y$

For M 150 concrete, $F_c = 150$ kg/cm²,

and $F_y = 2\,600$ kg/cm²

$$\rho_{max} = \rho_c + 0·011 \qquad ...(7)$$

For concrete reinforced with cold worked deformed bars,

*Blume J. A., Newmark N. M. and Corning L. H. Design of Multistorey Reinforced Concrete Building for Earthquake Motions. Portland Cement Association, USA.

FIG. 7 EXAMPLE OF TYPICAL BAR DETAILS FOR SPECIAL DUCTILE MOMENT RESISTING FRAMES

Designer should pro-
vide dimension $A$.
$s$, $d$, $l_1$ anchorage
length, cutoff points
of discontinuous bars,
etc

$A$ = distance to point of inflection plus anchorage length but
not less than $L/4$. Designer may cut some bars shorter
than this but at least one third the area of bars of
the column must extend this distance
$d$ = beam design depth
$IR$ = 4D minimum, 6D preferable
$l_1$ = distance required by design for moment plus anchorage
length
$D$ = diameter of bar

*Provide not less than
two stirrup ties throu-
ghout splice length,
spaced no further
apart than 16 bar dia-
meters or 30 cm

$\epsilon_y = 0.00167$, $\epsilon_{cu} = 0.003$

$\rho_{max} = \rho_c + 0.15 F_c/F_y$

**7.2.3** The required amount of longitudinal
reinforcement shall be continuous through inter-
mediate supports. When framing into external
columns, it shall be extended to the far face of the
confined region and anchored by bond, hooks, or
mechanical anchors to develop the yield strength
of the reinforcement at the support. Figure 7
shows the typical detail for a beam framing into
column from one side or two sides. Such an
arrangement will ensure a ductile junction and
provide adequate anchorage of beam reinforcement
into columns. Top and bottom longitudinal steel
for beams framing into both sides of column
should extend through the column without
splicing.

**7.2.4** The tensile steel bars should not be spliced
at sections of maximum tension. Continuous top
bars should be spliced near the centre of a span
in regular frames where earthquake moments are
usually minimum and gravity load moments do
not usually produce tensile stress. For splices in
the reinforcement, full bond length (development
length) preferably with standard hooks at the ends
should be used and the splice should be contained
within at least two stirrup ties so that the splice
does not open out. The stirrup ties should be of
closed type (see Fig. 8).

**7.2.5** *Shear Reinforcement* — Let a beam of
span ($l$) carry dead and load and live load ($D$ and $L$)
and let the plastic moment capacities at ends of
the beam be $M_{pa}$ and $M_{pb}$ for hogging, and $M'_{pa}$
and $M'_{pb}$ for sagging moments respectively. The
maximum and minimum shears that can be
caused at the two ends of the beam will be as
follows:

$$(V_a)_{max} = V_a^{D+L} + \frac{M_{pa} + M_{pb}}{l}$$

$$(V_a)_{min} = V_a^{D+L} + \frac{M'_{pa} + M'_{pb}}{l}$$

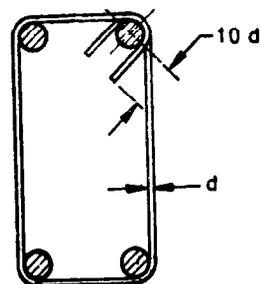$$(V_b)_{max} = V_b^{D+L} + \frac{M_{pb} + M_{pa}}{l}$$



FIG. 8 CLOSED STIRRUP

$$(V_b)_{min} = V_b^{D+L} - \frac{M'_{pb} + M'_{pa}}{l}$$

NOTE — In the above stated equations 'a' is for left support and 'b' is for right support of the beam. $V_a$ is the shear force taken upward to the left of the section. Moments are taken positive if $M_{pa}$ and $M_{pb}$ are hogging and $M'_{pa}$ and $M'_{pb}$ are sagging.

It can be seen from the above that the minimum sheare forces may change their sign and shear failure is particularly undesirable. Therefore, vertical closed stirrups will be preferable to inclined bars because the vertical stirrups will be able to resist shears of any sign. The sectional area as of all vertical legs of a stirrup and spacing of the stirrups may be designed for the maximum shear force using yield stress in steel $F_y$ as usual. For good behaviour of beam in diagonal tension, their spacing should not exceed $d/4$ in a length of the beam equal to $2d$ measured from the face of column. The first stirrup may be located 5 cm from the face of the column. In the remaining length of the beam, the maximum spacing should be limited to $d/2$.

### 7.3 Columns Subjected to Axial Load and Bending

Ductility requirement is more difficult to achieve in columns because of buckling effects. The ductility in the column section can be increased considerably by the confinement of concrete by means of special lateral reinforcement in the form of spirals or closely spaced lateral hoops particularly near the junctions of beam. In addition to this, the dimensions of the column section should be chosen so that it does not become a long column.

While designing columns, it is to be kept in view that columns should not fail before beams, as loss of the columns support may allow collapse of the floor system.

**7.3.2** *Special Reinforcement in Columns* — For working out the amount of lateral hoops for necessary confinement, the criterion may be adopted that the load carrying capacity of the cross-section of the column may be made equal to that of the column without the shell concrete. Load deformation behaviour of such a column is superior to ordinarily tied columns. Using this criterion, the cross-sectional area of the bar forming circular hoops or a spiral used for confinement of concrete is given in the Code.

**7.3.3** As with flexural members the most critical situation for column is adjacent to the beam column joint. This clause defines the range within which the required transverse reinforcement is to be provided.

**7.3.4** In addition to the above, transverse reinforcement should be provided throughout the length of the column to resist shear force, the spacing of which shall not be less than $d/2$.

### 7.4 Beam Column Connections

Joints between exterior columns and adjoining flexural members shall be confined by transverse column reinforcement through the joint. Such reinforcement shall consist of helical reinforcement or ties as required at the column ends. This is required because on exterior and corner columns the joint core is not confined by flexural members on all sides. To provide some measure of confinement in these situations giving some strength against brittle failure in the joint core, transverse reinforcement as required at the column ends is continued through the joint core (*see* Fig. 9).

The transverse reinforcement as required at the end of the column applies to connection as well. If the connection is confined by beams from all four sides, the amount of transverse reinforcement may be reduced to half this value. The tie reinforcement may be reduced to half this value. The tie reinforcement at beam column joints may be provided by U-shaped ties, the length of the legs beyond the columns being kept is dictated by bond requirements so as to develop full strength of the ties.
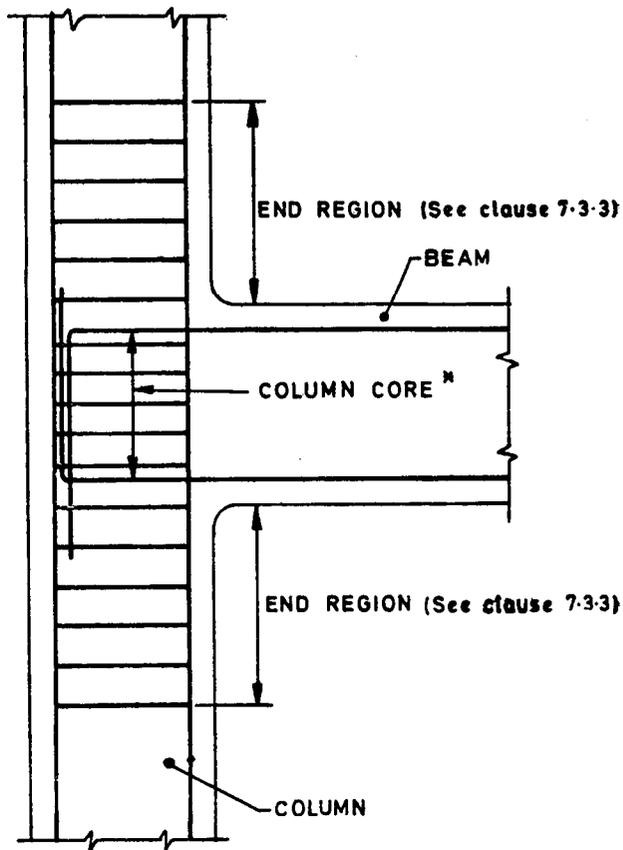


*Column core has to be confined by hoop or helical reinforcement in accordance with end region spacing

FIG. 9 BEAM-COLUMN JOINT AT EXTERNAL COLUMNS

# 8. MASONRY CONSTRUCTION

**8.1 General** — From the numerous observations of damage, it appears that unreinforced brick, composite constructions, and *adobe* houses are not the suitable forms of construction in seismic areas since these have large weight and almost no lateral strength or ductility. Besides, workmanship is yet another factor which effects its performance and hence emphasis should be given on good quality of workmanship in order to achieve best results in this material. The basic advantage of this construction lies in the fact that it is possible to use the same elements to perform a variety of functions which in a framed building have to be provided for separately with consequent complication in detailed construction. This form thus simultaneously provides the structure, subdivision of space, thermal and acoustic insulation, fire and other weather protection. Therefore, finding effective methods of improving their earthquake resistance is of utmost importance for a vast majority of people throughout the world.

For closer examination, building components could be studied separately also to establish where each one lacks strength and should be strengthened. The following paragraphs discuss the behaviour of such buildings in detail and make a review of the methods of strengthening such buildings against seismic forces.

*Structural Action of Building Elements* — Buildings respond to ground motion like all other structures and attract inertia forces depending on their stiffness and damping characteristics. Following this, the roof tends to separate from supports, the roof covering tends to be dislodged and walls tend to tear apart. If unable to do so, the walls tend to shear off diagonally in the direction of motion. In case of filler walls in reinforced concrete or timber frame, these may fall out of the frame unless properly tied to the frame. In this section, however, only load bearing wall type construction are discussed. For this, some of the basic ideas must be understood first. The walls which resist forces in their own planes will be referred to as shear walls and those in perpendi-

cular direction as cross walls. It is thus clear that in a building the same walls could act as shear walls or cross walls depending on direction of earthquake motion. Referring to Fig. 10, for the X-direction of motion, walls B act as shear walls while offering resistance against the collapse of wall A as well. Wall A acts as vertical slab supported on two vertical sides and bottom and subjected to inertia force of its own mass. Near the edges, the wall will have bending moments in the horizontal plane for which brickwork has little strength. This may result in cracking and separation of the walls. If, however, a horizontal bending member is introduced at a suitable level in wall A and continued in wall B, the tension in horizontal plane may be taken care of. Tension on account of vertical bending may generally get relieved due to self weight and can be made to take care of bending tensions. The same will be true for wall B when ground motion is in Y-direction. Thus the horizontal bending member is a very desirable provision in such buildings. Such a member is called a runner or a band and depending on its location it may be called a roof band, lintel band, or gable band.

The roof slab transfers its inertia force at top of the walls causing shearing and overturning forces in them. Major portion of this load is taken up by the shear walls on account of their large inplane stiffness compared to the cross walls. However, the slab must have enough strength in bending in horizontal plane to be able to transfer the force in the aforesaid manner. Reinforced concrete or reinforced brick slabs would normally possess this but other types of roof/floor, such as brick-tile coverings or timber planks-joists floors, must be connected together and fixed to walls suitably to achieve this purpose. Shear walls should of course be able to take the shear of the slab in addition to its own inertia forces and should be designed safe for the bending and normal stresses resulting from such forces.

In the barrack type construction, the roof trusses rest on walls 'A' while the walls 'B' are
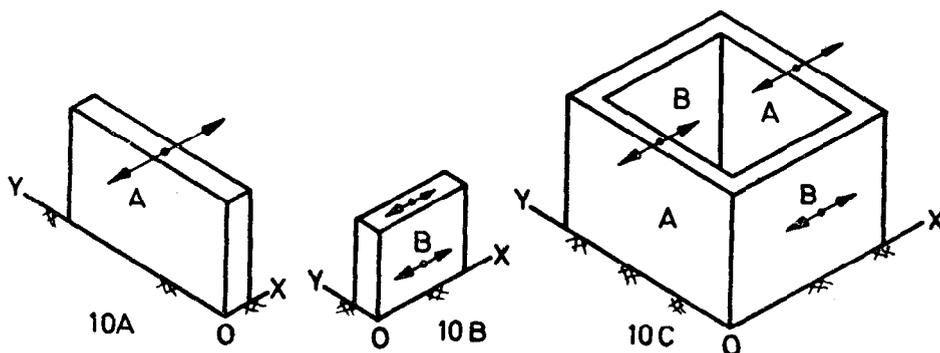

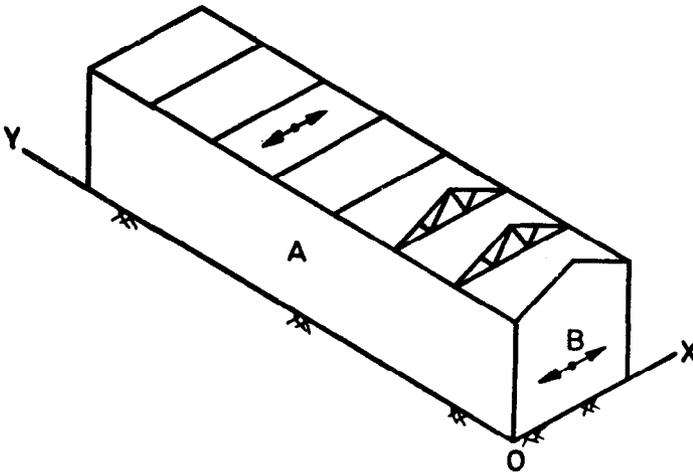
FIG. 10 SHEAR WALLS AND CROSS WALLS

FIG. 11 BARRACK TYPE STRUCTURE

gabled to receive the purlins (*see* Fig. 11). In such cases, the trusses must be anchored into the walls by 'holding down bolts' and the walls 'A' must be treated as vertical cantilevers. Also a band may be necessary to transfer the horizontal forces. Alternately, diagonal bracing may be provided at the main tie level extending from one gable end to the other.

The above structural behaviour leads to the following requirements of structural safety of brick building against earthquake forces:

a) A free wall must be designed as vertical cantilever;

b) Shear walls must resist forces transferred to it by its deformation;

c) Roof/floor elements must be tied together and be capable of transferring their inertia forces to the walls; and

d) Walls must be effectively tied together to avoid separation at joints due to shaking. Horizontal bands may be provided for this purpose at suitable places.

*Behaviour of Brick Shear Walls* — Shear walls are the main elements resisting the lateral forces in a building. In fact, the strength of such walls determines the lateral load carrying capacity of the building. However, their strength depends on a number of factors which are sometimes very difficult to take into account in a theoretical analysis. The main source of error and uncertainty is the workmanship. With this difficult parameter playing an important part in the strength of a shear wall, any effort to use the more sophisticated methods may appear to be an exercise in futility. Simpler approaches were, therefore, suggested in which a masonry wall was treated as a series of piers formed by the opening in a shear wall (*see* Fig. 12). For calculating stresses in such piers, it is

assumed that the rotational component of deformations of the portions above and below the openings are small compared to those of the piers between the openings and are neglected. The points of contraflexure are assumed at the mid height of such piers and the forces (the lateral shear) are shared among the piers such that their tops deflect by equal amount. The deflection of each pier is calculated by assuming the ends to be restrained against rotation. Thus the deflection is given by:

$$\Delta = \frac{Vh^3}{12EI} + \frac{1 \cdot 2Vh}{AG} \qquad ...(1)$$

in which $V$ is shear in the pier, $h$ its height, $A$ the area of cross-section, $I$ the moment of inertia, $E$ the modulus of elasticity and $G$ the modulus of rigidity of the material. The shear stiffness $S$, of the pier is given by,

$$S = V/\Delta \qquad ...(2)$$

The total horizontal shear in a wall will, therefore, be distributed in the various piers in proportion to their shear stiffnesses. This shear causes bending moment equal to $Vh/2$ at the top and bottom sections of the pier. Also, there would be overturning forces in these piers on account of this horizontal force in addition to the direct stresses due to the dead load of the building above the lintel level. The total stress $\sigma$ is given by,

$$\sigma = \sigma_b + \sigma_d + \sigma_0 \qquad ...(3)$$

in which $\sigma_b$ is the stress due to bending, $\sigma_d$ the stress due to the dead load and $\sigma_0$ the stress due to overturning forces. Such computations should be made along each axis of the building for reversible earthquake force. If the combined stress $\sigma$ exceeds the ultimate strength of brickwork in tension, cracking would occur. The sections of the piers
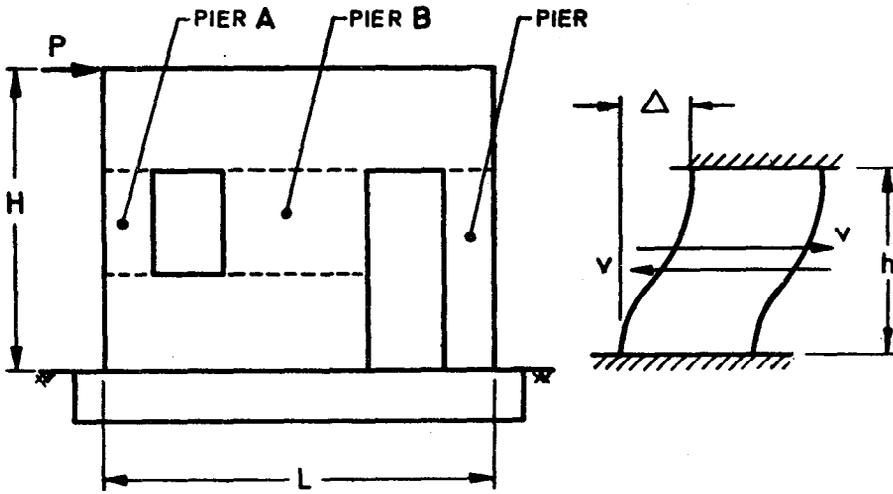
12A Wall with door and window      12B Deflection of a typical pier

FIG. 12   A TYPICAL SHEAR WALL WITH PIERS AND THE BEHAVIOUR OF PIER
UNDER LATERAL FORCES

should either be designed such that tension does not appear or else provision should be made for reinforcing the section at such crucial points.

Sections along the jambs of openings and the corners in a shear wall have been identified as the vulnerable points in the walls. Since the ordinary brickwork has very little strength in tension, it may seem essential that such wall sections be strengthened by reinforcing steel in vertical direction, particularly in active seismic zones. Taking a clue from here, the efficacy of such strengthening measures was examined and experimental study of house models was carried out. The various strengthening measures studied included vertical steel at corners, vertical steel at jambs and their combinations with the lintel band. It was found that the strength of models increased many folds with the introduction of these methods. Further, it became very clear that such measures would not allow the entire building to collapse during an earthquake. The quantity of such reinforcing steel would naturally depend on the number of storeys in a building and on the severity of the zone. With these methods, it is possible to go up to four storeyed construction. The Code provides for such special measures in detail. Special features of design and construction for earthquake resistance masonry buildings, in Zones III, IV and V are also covered in IS : 1905-1980*.

**8.1.1** *Materials* — In view of the explanation given under **8.1** the Code recommends good quality bricks, and prohibits the use of sun dried bricks. It also permits use of squared stone masonry or random rubble masonry brought to courses at regular intervals of 60 cm and also hollow concrete block masonry which possesses adequate strength as laid down in the relevant standards.

**8.1.2** *Mortar* — Since strength of masonry building is largely dependent upon the strength of mortar used, it is recommended in the Code that only those mortars which possess adequate strength characteristics be used in construction. On this basis, Table 3 in the Code is formulated.

If a reinforcing bar is used in masonry, it is desirable to have richer material around it to ensure proper bond. For this purpose, either 1 : 4 cement-sand mortar or M 10 or M 15 grade concrete is recommended for use.

## 8.2 Walls

**8.2.1** The Code recommends that load bearing masonry walls should not be more than 15 m total height or four storeys. Moinfar (1972) from his experience on observations of earthquake damage to brick masonry buildings in Iran has reported that for ordinary workmanship and quality of building materials the height of a dwelling should not exceed three storeys, preferably two, and under no circumstances should the total height of the dwelling exceed 11 m including the height of the parapet. However, this experience is based on observations of unreinforced buildings. Here up to 4 storeys are allowed in view of proposed reinforcing.

Random rubble masonry (brought to courses at 60 cm vertical intervals) is recommended only up to 2 storeys or (8 m in height) in view of its poor performance observed during the past earthquakes.

**8.2.2** The load bearing walls must be straight and symmetrical in plan so that torsional shears are avoided or minimized.

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*Code of practice for structural safety of buildings: Masonry walls (*second revision*).

**8.2.3** As explained in **8.1** (*see* Fig. 10) wall *A* acts as a vertical slab supported on two shear walls, the bottom and the roof slab. It is subjected to inertia force on its own mass and thus deforms as a plate under the action of such forces. For simplicity of computations, the Code advises that a check should be made on the strength of a unit width of such walls assuming them to be vertical beams subjected to inertia forces of their own mass. This would be a conservative check as far as safety is concerned.

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*Example* 1 — Check a wall panel of a room 6·4 m × 4·4 m in plan having 20 cm thick, 3·5 m high brick masonry walls in 1 : 6 cement sand mortar. The roof is in RB weighing 300 kg/m². The design seismic coefficient is 0·04 (*see* Fig. 13).

Considering 1 m wide strip of the wall, the bending moment at the point *A* (and *B*) will be given by

$$M = \frac{wh^2}{12}$$

in which *w* is inertia force per unit length of vertical strip and *h* is the height of room. At section *A* the compressive stress will be the minimum because the self weight of wall is nil and only the RB slab gives some compression. From the safety point of view tension should not exceed the safe limit for the 1 : 6 cement-sand mortar.

$$w = 0·04 \times 1·0 \times 1·0 \times 0·20 \times 1·92$$
$$= 1·54 \times 10^{-2} \text{ t/m}$$

Bending moment at *A*,

$$M_A = \frac{1·54 \times 10^{-2} \times 3·5^2}{12}$$
$$= 1·57 \times 10^{-2} \text{ kg.m}$$

$$Z = \frac{bd^2}{6} = 0·006\ 67 \text{ m}^3$$

∴ Bending stress

$$\sigma_b = \frac{0·016}{0·006\ 67} = 2·4 \text{ t/m}^2 \text{ (0·24 kg/cm}^2\text{)}$$

Weight of RB slab per metre

$$= \frac{0·300 \times 6·4 \times 4·4}{2(6·2+4·2)} = 0·406 \text{ t}$$

∴ Direct stress

$$\sigma_d = \frac{0·406}{0·2 \times 1} = 2·03 \text{ t/m}^2 \text{ (0·203 kg/cm}^2\text{)}$$

Therefore, the net tensile stress in the brick element works out as $(1·2 - 0·203) = 0·997$ kg/cm² (comp) which is well within the permissible value of tension in brick masonry with 1 : 6 cement mortar.

It turns out that panel walls having storey heights of the order of 3·5 m or so and laterally supported at top will be found to be safe and, therefore, the Code mentions this in the Note following this clause.

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**8.2.4** A free standing wall is recommended to have a factor of safety of 1·5 in overturning in view of the importance of such walls.

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*Example* 2 — Check overturning factor of safety for a 3 m high 20 cm thick uniform brick wall forming boundary of a factory, located in seismic Zone IV (*see* Fig. 14).

Considering unit length of wall the overturning moment (due to inertia forces) *Mo* is given by,

$$Mo = 0·04 \times 1·0 \times 3·0 \times 0·2 \times 1·92 \cdot \times 1·5$$
$$= 0·089 \text{ t.m}$$

Stabilizing moment (due to dead weight) about point *A* is given by

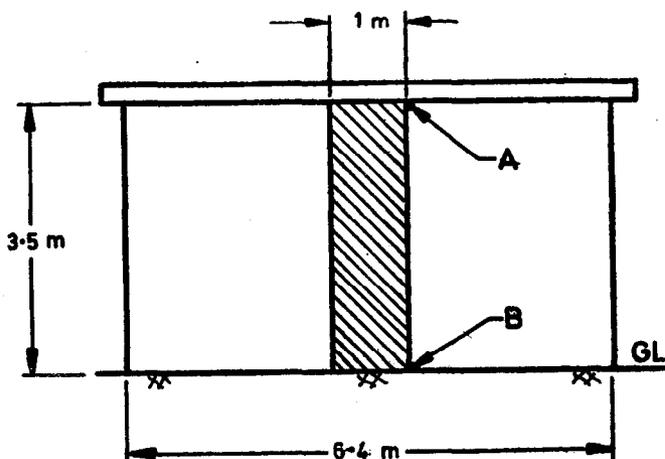$$M_s = 3 \times 1·0 \times 0·20 \times 1·92 \times 0·10$$
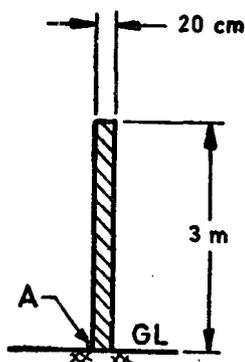$$= 0·115 \text{ t.m}$$



FIG. 13

FIG. 14

The wall, therefore, has a factor of safety against overturning

$$= \frac{0.115}{0.069} = 6 > 1.5 \ (O.K.)$$

**8.2.5** During an earthquake shaking, infills in framed building will behave according to its physical connection with the frames. The infills usually add to the stiffness and strength of a building if bonded properly with the frame. It is, therefore, recommended in the Code that the bonding should be done properly either by suitable mortar or dowels. If this is not achieved, the infills would behave just like a free standing wall. In either situation, the infill should be checked for safety against inertia force acting on its own mass as provided in **8.2.3** or **8.2.4** of the Code.

**8.3 Openings in Bearing Walls**

**8.3.1** to **8.3.6** Openings divide a bearing wall into a series of piers whose strength determines the strength of the wall element. Analysis has shown that sections around jambs of openings are the vulnerable sections and must be safeguarded. It is also seen that the larger the opening, smaller is the strength of wall. Also the strength of wall depends upon the placing of opening in the wall. The more central location of opening leads to a higher strength and higher the opening higher is the strength. In view of these, the Code recommends certain specifications as regards the size and placing of openings in load bearing walls.

If openings do not comply with the recommendations made in the various subclauses, the Code recommends strengthening of openings by providing steel at jambs or by framing the opening in reinforced concrete as shown in Fig. 11 of the Code.

It also suggests that as far as possible all openings in a storey should have their top at the same level to facilitate provision of continuous reinforced concrete band alround the building.

**8.3.7** Projecting parts are always potential hazards during an earthquake and, therefore, these must be well anchored in reinforced masonry or concrete.

**8.3.8** Observations in past earthquakes have shown that use of arches to span across openings is a source of weakness and must be avoided unless steel ties are provided.

**8.4 Strengthening Arrangements**

**8.4.1** This is the most important operative clause of the Code. It specifies the strengthening measures to be adopted in case of masonry buildings for which design seismic coefficient is worked out first in accordance with provisions of IS : 1893-1975*. It may be recalled that design seismic coefficient is worked out on the basis of seismic zone, soil-foundation factor ($\beta$) and importance factor ($I$).

Depending upon the value of design seismic coefficient, the strengthening arrangements vary — higher seismic coefficient inviting more elaborate provisions compared to those for lower coefficients. Basically the following six provisions are specified:

a) Masonry mortar (*see* explanatory **8.1.2**);

b) Lintel band alround to tie up the building;

c) Roof band (and gable band in case of sloping roofs);

d) Vertical steel at corners and junctions of walls to take up tension which occurs at these points;

e) Vertical steel in jambs of openings (*see* explanatory **8.3.6**); and

f) Bracing in plan at tie level (*see* explanatory **4.4.2.2**).

(b), (c) and (d) are explained in the following subclauses.

**8.4.2** to **8.4.5** *Bands or Runners* — Lack of proper connection between various elements of the buildings, like walls or between walls and roof (or floor), has often resulted in damage to masonry buildings during earthquakes. As explained in 8, runners or bands of reinforced concrete or reinforced brickwork provided in all the load bearing walls at different levels together with vertical reinforced concrete elements which are not necessarily load bearing, provide excellent connections for the building to act as one unit under earthquake motion, thus increasing considerably its resistance and minimizing damage. Locating the runners at lintel and/or roof level is recommended depending upon the seismic intensity, type of construction and soil strength.

The various bands may be made in M15 or richer concrete on reinforced brickwork in cement sand mortar not leaner than 1 : 3. A minimum thickness of 7.5 cm and width equal to thickness of the wall is recommended.

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*Criteria for earthquake resistant design of structures (*third revision*).

One steel bar near each face of the wall is recommended in the reinforced concrete band. In reinforced brickwork, the reinforcement may be provided in two consecutive courses with two bars near each face of the wall so that the total area of steel is the same as in a reinforced concrete band. That is, with the number of bars doubled, the diameter of bars may be reduced to 70 percent for obtaining the same total steel area. The joints in reinforced brickwork containing the steel bars should be increased so as to have a minimum mortar cover of 6 mm around the bar. Figure 15 shows typical sections of such bands. Example 3 illustrates the design of such bands.

*Example* 3 :

Figure 16 shows a single room building with brick masonry walls (20 cm thick) 3·5 m high. The reinforced brickwork roof slab weighs 300 kg/m². The design seismic coefficient is, 0·08. Design the lintel band and the roof band.

a) *Design of Lintel Band*

Neglecting openings, horizontal seismic load coming on lintel band (perpendicular to plane of wall)

$$q_h = 0·08 \times \frac{2·1 + 1·4}{2} \times 0·20 \times 1\,920$$

$$= 53·76 \text{ kg/m}$$

Assuming continuity of band at corners of walls, maximum bending moment in the band (horizontal bending) is given by

$$M = 53·76 \times \frac{6·2^2}{10}$$

$$= 206·66 \text{ kg. m}$$

Also maximum shear force is given by

$$F = 53·66 \times \frac{6·0}{2}$$

$$= 160·98 \text{ kg/m}$$

Taking the normal allowable bending compression in concrete as 50 kg/cm² and shear as 5 kg/cm² with tensile stress in steel as 1 400 kg/cm² and allowing an increase of 33⅓ percent in these values for seismic condition, the area of steel reinforcement (on either face) is worked out as follows:

$$A_t = \frac{206·66 \times 100}{1·33 \times 1\,400 \times 15} = 0·738 \text{ cm}^2$$

in which 15 cm is the distance between the steel reinforcement at the inner and the outer faces.

The thickness of band from consideration of diagonal tension works out to

$$t = \frac{160·98}{1·33 \times 5 \times 0·86 \times 17·5} = 1·61 \text{ cm}$$

However a minimum thickness of 7·5 cm is recommended in view of the proper cover for steel on both the sides. The requirement of steel is also less but a minimum of one bar of 12 mm diameter on each face is recommended together with links of 6 mm dia spaced at 15 cm apart for tying the two bars together. However, this arrangement is valid for spans say up to 8 m or so beyond which the adequacy of band must be checked as above.
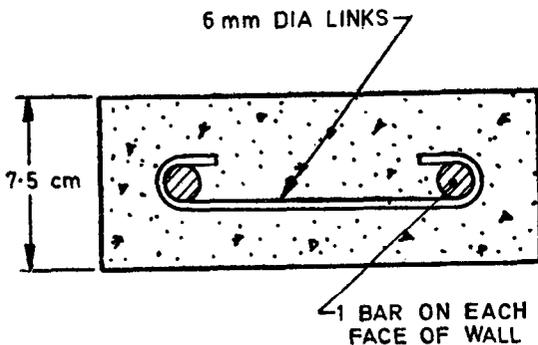


15A Concrete band

FIG. 15 CROSS SECTION OF RUNNERS
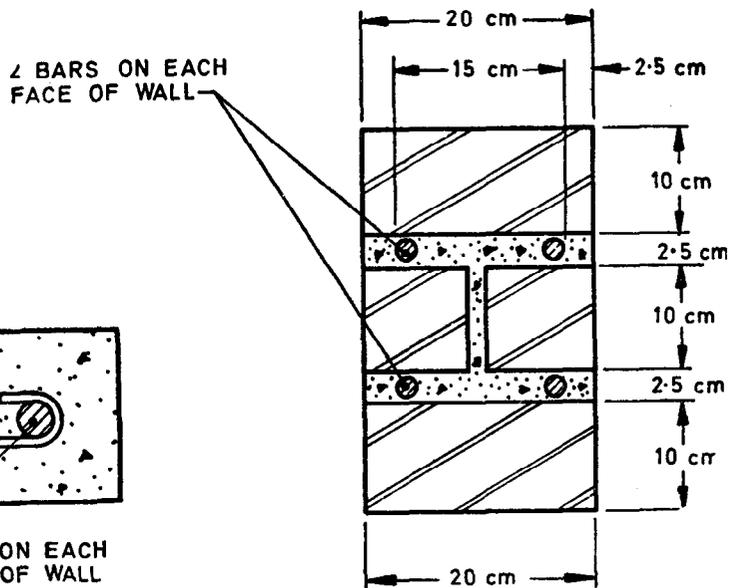


15B Reinforced brick work band
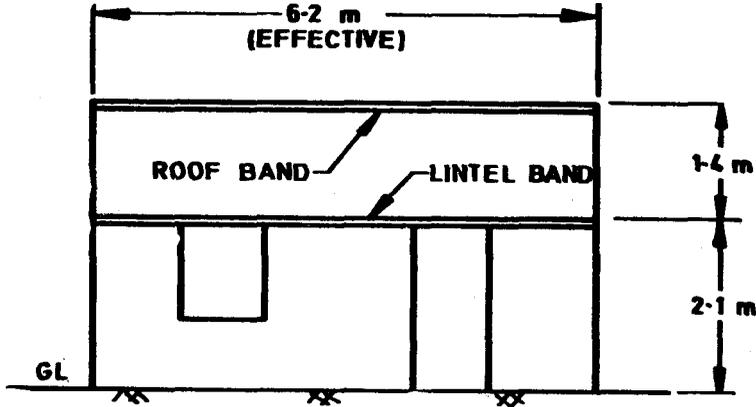
FIG. 15 CROSS SECTION OF RUNNERS

FIG. 16  BUILDING WITH ROOF BAND AND LINTEL BAND

b) *Design of Roof Band*

The inertia force due to load coming on roof band depends upon the arrangement of roofing material. Assuming that the roofing elements are tied to longitudinal walls, the inertia force on roof band is given by

$$q_h = 0.08 \times 300 \times \frac{4.4}{2} + 0.20 \times 1\,920$$

$$\times \frac{1.4}{2} = 74.30 \text{ kg/m}$$

Bending moment is then obtained as

$$M = 74.30 \times \frac{6.2^2}{10} = 285.62 \text{ kg.m}$$

and shear force

$$F = 74.30 \times \frac{6.0}{2} = 222.90 \text{ kg}$$

Assuming same working stresses as in the earlier example,

$$A_t = \frac{285.62 \times 100}{1.33 \times 1\,400 \times 15} = 1.02 \text{ cm}^2$$

Thickness of band

$$t = \frac{222.90}{1.33 \times 5 \times 0.86 \times 17.5} = 2.23 \text{ cm}$$

Minimum reinforcement and thickness as in case of lintel band are to be used.

In case of sloping roofs, the roof band is made continuous over the gable ends of the end wall below the purlins. Obviously, the specifications for the gable band, roof band and lintel band are, therefore, the same.

In case of structures where design seismic coefficient works out more than 0.08 (due to importance of the structure or soil foundation system), these bands are provided with more reinforcement as given in Table 4 of the Code.

Roof band is not required in case of reinforced concrete or reinforced brick slabs which are continuous over the whole building or between crumpled sections, if any, and cover the width of walls fully as they have a binding action on the walls and are also capable of transmitting their inertia force to the shear walls.

Runners/bands at any level should be made continuous at the corners and junctions of walls. The gable band should also be continuous with the roof band at tie level. Typical details for continuity are shown in Fig. 17.

8.4.6 Horizontal runners may not be much effective by themselves in increasing the lateral resistance of buildings. Vertical steel at corners and jambs of openings provides much greater strengthening, particularly in combination with the horizontal steel reinforcement bars in runners.

By reinforcing these critical sections alone, brick buildings up to four storeys are strengthened sufficiently at a small extra cost and without additional skill required in construction to escape collapse even under most severe earthquake forces. The recommendations of the Code are based on this cost consideration rather than on no damage criterion.

Vertical steel need not be provided in buildings up to 2 storeys in places where design seismic coefficient works out as $0.06 < \alpha_h < 0.08$. However, if in this situation, 3 or 4 storeyed buildings must have vertical steel in view of possible ampli-
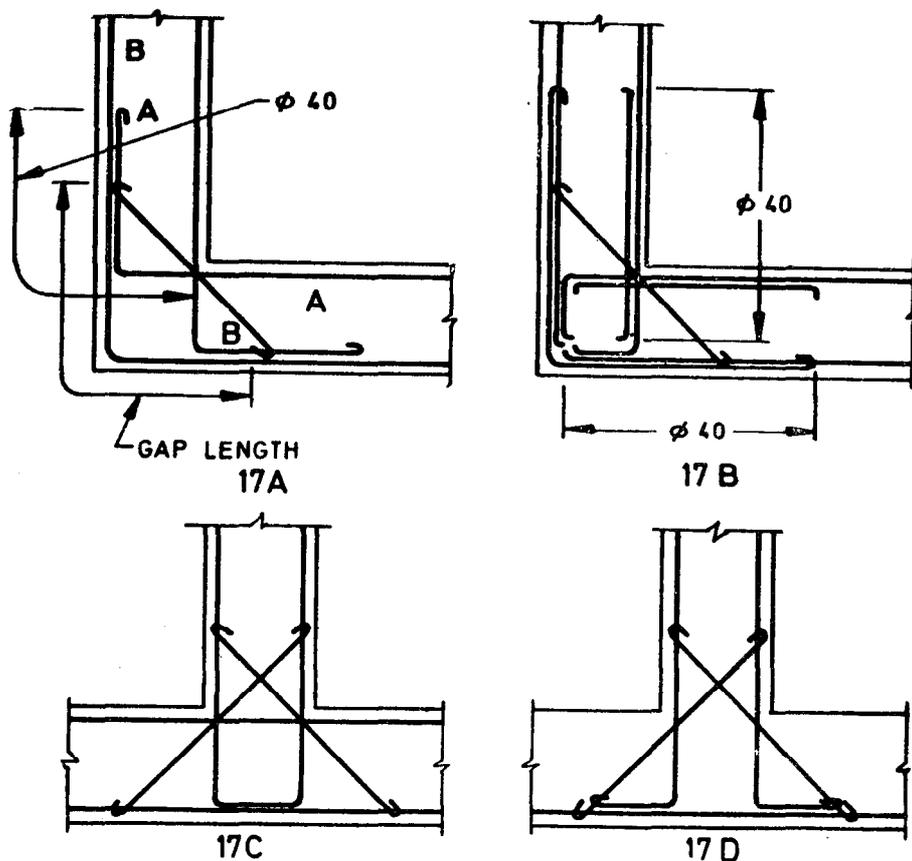
FIG. 17   BAND REINFORCEMENT DETAIL AT CORNER AND JUNCTION OF WALLS

fication of acceleration in such structures. Of course, if the design seismic coefficient is greater than 0·08, all buildings (1 to 4 storeys) must be reinforced by such steel. The quantity of steel in each case is specified in Table 5 of the Code. The details of providing the vertical steel in brickwork, hollow block and stone masonry at corners, T-sections and jambs of openings are also illustrated in Fig. 8, 9 and 10 of the Code.

# 9. TIMBER CONSTRUCTION

**9.0** Timber is often used for structural work particularly in hilly regions as also for temporary construction. It is well known that timber has poor weather resistance and is liable to seasonal changes, cracks and warping.

**9.1** Earthquake force attracted by a structure is proportional to its weight. Also it is well known that failure occurs on account of tension in structures as was explained in 8 on masonry. A suitable material for earthquake resistance would, therefore, be one in which the strength per unit weight is higher. Timber has a high strength to unit weight ratio and is, therefore, very suitable for earthquake resistant construction.

**9.2** Timber is not as strong as other materials of construction, namely, masonry or reinforced concrete and although it has a high strength per unit weight its construction is generally advised to be restricted to two storeys in seismic areas.

**9.3** Quite often fire breaks out following an earthquake on account of electric short circuiting, kitchen fire, etc, and for this reason the Code recommends that attention should also be paid to fire safety in timber construction.

**9.4** In order to ensure an integrated action by the structure during an earthquake, it is necessary to make the superstructure rigid by appropriate techniques so that it behaves as one unit during vibration. Therefore, attention must be paid to suitable construction detailing of junctions of the members and the wall panels since the rigidity is very intimately associated with such detailing.

**9.5 Foundations**

**9.5.1 to 9.5.2.2** For timber buildings, it must be ensured that the structure remains intact all the time for which the Code recommends that the portion of the building below the plinth level must

be constructed in masonry or concrete. This is desirable in view of the fact that timber may deteriorate or rot if taken underground. The superstructure may or may not be rigidly connected to the plinth masonry. Experience from past earthquakes has shown that buildings not fixed with the foundation escape collapse although they could move side ways. The Code also suggests appropriate details of connection of columns with the foundation masonry in case it is desired to have the superstructure rigidly fixed into the plinth masonry/concrete foundation. In case of small buildings, however, the Code permits the vertical poles to be embedded into the ground.

**9.6** Stud wall construction and brick nogged timber frame construction are generally adopted in practice while constructing buildings in timber. The Code recommends appropriate sizes of structural members for use in these two types of construction. These dimensions are based on an estimate of earthquake forces expected in the various seismic zones and the minimum sizes indicated in the Code correspond to the expected earthquake forces in Zone V.

## 9.7 Stud Wall Construction

**9.7.1 to 9.7.10** In this form of construction, the timber studs and corner posts are framed into sills, top plates and wall plates. Horizontal studs and diagonal members are used to stiffen the frame against lateral forces. The joints must be covered by suitable steel strap. The wall cladding can be either timber boards or the conventional *EKRA* construction in some parts of the country. The Code recommends the minimum size of the studs and diagonal bracings depending on the seismic

coefficient for two categories of timber, namely. Class I and Class II as defined in IS : 883-1970*.

## 9.8 Brick Nogged Timber Frame

This form of construction consists of intermediate verticals, columns, sills, wall plates, horizontal nogging members and diagonal members framed into each other and the space between the remaining members is filled with tight fitting masonry. The minimum size of various timber elements to be used in this construction is specified in the Code. The joints of wall plate and sill plate with studs must be covered with suitable steel straps.

**9.9 Connections Between Timber Members —** Cutting and notching are very important and significantly affect the strength of a member. For this reason, the Code recommends that notching or cutting should be limited to about 20 mm in depth unless steel strips are provided to strengthen the notched face of the member to compensate for the loss of material. Where it is necessary to cut or notch a member by about 40 mm in depth, the steel reinforcing strip should be placed along the notch only. However, where the cut is more than 40 mm in depth or a member is completely cut through steel strips should be placed on both edges of the member. Figure 15 of the Code indicates the detail of such notching and reinforcing.

**9.10 Bridging and Blocking —** In order to provide rigidity to the timber frame, the Code recommends cross bridging of wooden joists at every 3·5 m length. The Code also recommends providing blocks at all bearings of such joists to block the space between joists.

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*Code of practice for design of structural timber in buildings (*third revision*).