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CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

(Fourth Revision)

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CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

(*Fourth Revision*)

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Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

(Fourth Revision)

0. FOREWORD

0.1 This Indian Standard (Fourth Revision) was adopted by the Indian Standards Institution on 16 November 1984, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Himalayan-Nagalushai region, Indo-Gangetic plain, Western India, Kutch and Kathiawar regions are geologically unstable parts of the country and some devastating earthquakes of the world have occurred there. A major part of the peninsular India has also been visited by strong earthquakes, but these were relatively few in number and had considerably lesser intensity. The earthquake resistant design of structures taking into account seismic data from studies of these Indian earthquakes has become very essential, particularly in view of the heavy construction programme at present all over the country. It is to serve this purpose that IS : 1893-1962 'Recommendations for earthquake resistant design of structures' was published and subsequently revised in 1966.

0.2.1 As a result of additional seismic data collected in India and further knowledge and experience gained since the publication of the first revision of this standard, the Sectional Committee felt the need to revise the standard again incorporating many changes, such as revision of maps showing seismic zones and epicentres, adding a more rational approach for design of buildings and substructure of bridges, etc. These were covered in the second revision of IS : 1893 brought out in 1970.

0.2.2 As a result of the increased use of the standard, considerable amount of suggestions were received for modifying some of the provisions of the standard and, therefore, third revision of the standard was brought out in 1975. The following changes were incorporated in the third revision:

a) The standard incorporated seismic zone factors (previously given as multiplying factors in the second revision) on a more rational basis.

b) Importance factors were introduced to account for the varying degrees of importance for various structures.

c) In the clauses for design of multi-storeyed building the coefficient of flexibility was given in the form of a curve with respect to period of buildings.

d) A more rational formula was used to combine modal shears.

e) New clauses were introduced for determination of hydrodynamic pressures in elevated tanks.

f) Clauses on concrete and masonry dams were modified, taking into account their dynamic behaviour during earthquakes. Simplified formulae for design forces were introduced based on results of extensive studies carried out since second revision of the standard was published.

0.3 The fourth revision has been prepared to modify some of the provisions of the standard as a result of experience gained with the use of this standard. In this revision a number of important basic modifications with respect to load factors, field values of N, base shear and modal analysis have been introduced. A new concept of performance factor depending on the structural framing system and brittleness or ductility of construction has been incorporated. Figure 2 for average acceleration spectra has also been modified and a curve for zero percent damping has been incorporated.

0.4 It is not intended in this standard to lay down regulations so that no structure shall suffer any damage during earthquake of all magnitudes. It has been endeavoured to ensure that, as far as possible, structures are able to respond, without structural damage to shocks of moderate intensities and without total collapse to shocks of heavy intensities. While this standard is intended for earthquake resistant design of normal structures, it has to be emphasized that in the case of special structures detailed investigation should be undertaken, unless otherwise specified in the relevant clauses.

0.4.1 Though the basis for the design of different types of structures is covered in this standard, it is not implied that detailed dynamic analysis should be made in every case. There might be cases of less importance and relatively small structures for which no analysis need be made, provided certain simple precautions are taken in the construction. For example, suitably proportioned diagonal bracings in the vertical panels of steel and concrete structures add to the resistance of frames to withstand earthquake forces. Similarly in highly seismic areas, construction of a type which entails heavy debris and consequent loss of life and property, such as masonry, particularly mud masonry and rubble masonry, should be avoided in preference to construction of a type which is known to withstand seismic effects better, such as construction in light weight materials and well braced timber-framed structures. For guidance on precautions to be observed in the construction of buildings, reference may be made to IS : 4326-1976*.

0.5 Attention is particularly drawn to the fact that the intensity of shock due to an earthquake could greatly vary locally at any given place due to variation in the soil conditions. Earthquake forces would be affected by different types of foundation system in addition to variation of ground motion due to various types of soils. Considering the effects in a gross manner, the standard gives guidelines for arriving at design seismic coefficients based on type of soil and foundation system.

0.6 Earthquakes can cause damage not only on account of the shaking which results from them but also due to other chain effects like landslides, floods, fires and disruption to communication. It is, therefore, important to take necessary precautions in the design of structures so that they are safe against such secondary effects also.

0.7 It is important to note that the seismic coefficient, used in the design of any structure, is dependent on many variable factors and it is an extremely difficult task to determine the exact seismic coefficient in each given case. It is, therefore, necessary to indicate broadly the seismic coefficients that could generally be adopted in different parts or zones or 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 seismic coefficients for design. The Sectional Committee responsible for the formulation of this standard has attempted to include a seismic zoning map (*see* Fig. 1) for this purpose. The object of this map is to classify the area of the country into a number of zones in which one may reasonably expect earthquake shock of more or less same intensity in future. The Modified Mercalli Intensity (*see* **2.7**) broadly associated with the various zones is V or less, VI, VII, VIII and IX and above for zones I, II, III, IV and V respectively. The maximum seismic ground acceleration in each zone cannot be presently predicted with accuracy either on a deterministic or on a probabilistic basis. The design value chosen for a particular structure is obtained by multiplying the basic horizontal seismic coefficient for that zone, given in Table 2, by an appropriate Importance Factor as suggested in Table 4. Higher value of importance factor is usually adopted for those structures, consequences of failure of which, are serious. However, even with an importance factor of unity, the probability is that

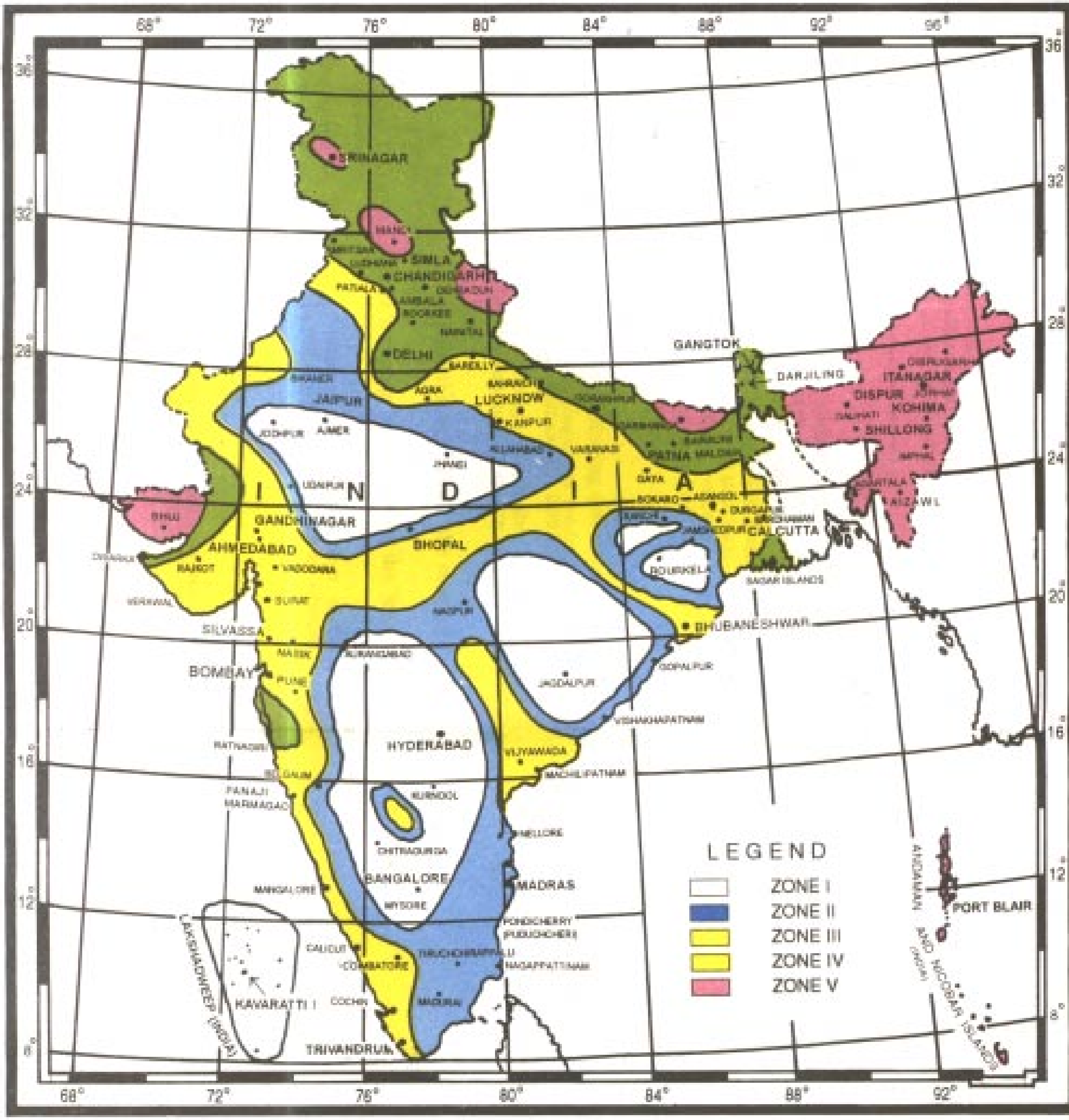
*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

a structure which is properly designed and detailed according to good construction practice, will not suffer serious damage.

It is pointed out that structures will normally experience more severe ground motion than the one envisaged in the seismic coefficient specified in this standard. However, in view of the energy absorbing capacity available in inelastic range, ductile structures will be able to resist such shocks without much damage. It is, therefore, necessary that ductility must be built into the structures since brittle structures will be damaged more extensively.

0.7.1 The Sectional Committee has appreciated that there cannot be an entirely scientific basis for zoning in view of the scanty data available. Though the magnitudes of different earthquakes which have occurred in the past are known to a reasonable amount of accuracy, the intensities of the shocks caused by these earthquakes have so far been mostly estimated by damage surveys and there is little instrumental evidence to corroborate the conclusions arrived at. Maximum intensity at different places can be fixed on a scale only on the basis of the observations made and recorded after the earthquake and thus a zoning map which is based on the maximum intensities arrived at, is likely to lead in some cases to an incorrect conclusion in the view of (a) incorrectness in the assessment of intensities, (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 shock. The Sectional Committee has, therefore, considered that a rational approach to the problem would be to arrive at a zoning map based on known magnitudes and the known epicentres (*see* Appendix A) assuming all other conditions as being average, and to modify such an average idealized isoseismal map in the light of tectonics (*see* Appendix B), lithology (*see* Appendix C) and the maximum intensities as recorded from damage surveys, etc. The Committee has also reviewed such a map in the light of past history and future possibilities and also attempted to draw the lines demarcating the different zones so as to be clear of important towns, cities and industrial areas, after making special examination of such cases, as a little modification in the zonal demarcations may mean considerable difference to the economics of a project in that area. Maps shown in Fig. 1 and Appendices A, B and C are prepared based on information available up to 1986.

0.8 In the formulation of this standard due weightage has been given to international coordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country.



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

Responsibility for the correctness of internal details shown on the maps rests with the publisher.

Based upon Survey of India map with the permission of the Surveyor General of India.

FIG. 1

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0.8.1 In the preparation of this standard considerable help has been given by the School of Research and Training in Earthquake Engineering, University of Roorkee; Geological Survey of India; India Meteorological Department and several other organizations.

0.9 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standrd.

1. SCOPE

1.1 This standard deals with earthquake resistant design of structures and is applicable to buildings; elevated structures; bridges, concrete, masonry and earth dams; embankments and retaining walls.

1.2 This standard does not deal with the construction features relating to earthquake resistant design in buildings and other structures. For guidance on earthquake resistant construction of buildings, reference may be made to IS : 4326-1976†. Further, provisions of this standard shall be used along with IS : 4326-1976†.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions shall apply.

NOTE — For the definition of terms pertaining to soil mechanics and soil dynamics, reference may be made to IS : 2809-1972‡ and IS : 2810-1979§.

2.1 Centre of Mass — The point through which the resultant of the masses of a system acts. This corresponds to centre of gravity of the system.

2.2 Centre of Rigidity — The point through which the resultant of the restoring forces of a system acts.

2.3 Critical Damping — The damping beyond which the motion will not be oscillatory.

2.4 Damping — The effect of internal friction, imperfect elasticity of material, slipping, sliding, etc, in reducing the amplitude of vibration and is expressed as a percentage of critical damping.

*Rules for rounding off numerical values (*revised*).

†Code of practice for earthquake resistant design and construction of buildings (*first revision*).

‡Glossary of terms and symbols relating to soil engineering (*first revision*).

§Glossary of terms relating to soil dynamics (*first revision*).

2.5 Epicentre — The geographical point on the surface of earth vertically above the focus of the earthquake.

2.6 Focus — The originating source of the elastic waves which cause shaking of ground.

2.7 Intensity of Earthquake — The intensity of an earthquake at a place is a measure of the effects of the earthquake, and is indicated by a number according to the Modified Mercalli Scale of Seismic Intensities (*see* Appendix D).

2.8 Liquefaction — Liquefaction is a state in saturated cohesionless soil wherein the effective shear strength is reduced to negligible value for all engineering purposes due to pore pressures caused by vibrations during an earthquake when they approach the total confining pressure. In this condition the soil tends to behave like a fluid mass.

2.9 Lithological Features — The nature of the geological formation of the earth's crust above bed rock on the basis of such characteristics as colour, structure, mineralogic composition and grain size.

2.10 Magnitude of Earthquake (Richter's Magnitude) — The magnitude of an earthquake is the logarithm to the base 10 of the maximum trace amplitude, expressed in microns, with which the standard short period torsion seismometer (with a period of 0·8 second, magnification 2 800 and damping nearly critical) would register the earthquake at an epicentral distance of 100 km. The magnitude M is thus a number which is a measure of energy released in an earthquake.

2.11 Mode Shape Coefficient — When a system is vibrating in a normal mode, the amplitude of the masses at any particular instant of time expressed as a ratio of the amplitude of one of the masses is known as mode shape coefficient.

2.12 Normal Mode — A system is said to be vibrating in a normal mode or principal mode when all its masses attain maximum values of displacements simultaneously and also they pass through equilibrium positions simultaneously.

2.13 Response Spectrum — The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during that earthquake. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity or maximum relative displacement.

2.14 Seismic Coefficients and Seismic Zone Factors

2.14.1 *Basic Seismic Coefficient* (α_o) — A coefficient assigned to each seismic zone to give the basic design acceleration as a fraction of the acceleration due to gravity.

2.14.2 *Seismic Zone Factor* (F_o) — A factor to be used for different seismic zone along with the average acceleration spectra.

2.14.3 *Importance Factor* (I) — A factor to modify the basic seismic coefficient and seismic zone factor, depending on the importance of a structure.

2.14.4 *Soil-Foundation System Factor* (β) — A factor to modify the basic seismic coefficient and seismic zone factor, depending upon the soil foundation system.

2.14.5 *Average Acceleration Coefficient* — Average specturm acceleration expressed as a fraction of acceleration due to gravity.

2.14.6 *Design Horizontal Seismic Coefficient* (α_h) — The seismic coefficient taken for design. It is expressed as a function of the basic seismic coefficient (α_o) or the seismic zone factor together with the average acceleration coefficient, the importance factor (I) and the soil-foundation system factor (β).

2.15 Tectonic Feature — The nature of geological formation of the bed rock in the earth's crust revealing regions characterized by structural features, such as dislocation, distortion, faults, folding, thrusts, volcanoes with their age of formation which are directly involved in the earth movement or quakes resulting in the above consequences.

3. GENERAL PRINCIPLES AND DESIGN CRITERIA

3.1 General Principles

3.1.1 Earthquakes cause random motion of ground which can be resolved in any three mutually perpendicular directions. This motion causes the structure to vibrate. The vibration intensity of ground expected at any location depends upon the magnitude of earthquake, the depth of focus, distance from the epicentre and the strata on which the structure stands. The predominant direction of vibration is horizontal. Relevant combinations of forces applicable for design of a particular structure have been specified in the relevant clauses.

3.1.2 The response of the structure to the ground vibration is a function of the nature of foundation soil; materials, form, size and mode of construction of the struture; and the duration and the intensity of ground motion. This standard specifies design seismic coefficient for structures standing on soils or rocks which will not settle or slide due to loss of strength during vibrations.

3.1.3 The seismic coefficients recommended in this standard are based on design practice conventionally followed and performance of structures in past earthquakes. It is well understood that the forces which structures would be subjected to in actual earthquakes, would be very much larger than specified in this standard as basic seismic coefficient. In order to take care of this gap, for special cases importance factor and performance factor (where necessary) are specified in this standard elsewhere.

3.1.4 In the case of structures designed for horizontal seismic force only, it shall be considered to act in any one direction at a time. Where both horizontal and vertical seismic forces are taken into account, horizontal force in any one direction at a time may be considered simultaneously with the vertical force as specified in **3.4.5**.

3.1.5 The vertical seismic coefficient shall be considered in the case of structures in which stability is a criterion of design or, for overall stability, analysis of structures except as otherwise stated in the relevant clauses.

3.1.6 Equipment and systems supported at various floor levels of structures will be subjected to motions corresponding to vibrations at their support points. In important cases, it may be necessary to obtain floor response spectra for design.

3.2 Assumptions — The following assumptions shall be made in the earthquake resistant design of structures:

a) 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.

b) Earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves.

c) The value of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in such condition.

3.3 Permissible Increase in Stresses and Load Factors

3.3.1 *Permissible Increase in Material Stresses* — Whenever earthquake forces are considered along with other normal design forces, the permissible stresses in materials, in the elastic method of design, may be increased by one-third. However, for steels having a definite yield stress, the stress be limited to the yield stress; for steels without a definite yield point, the will stress will be limited to 80 percent of the ultimate strength or 0·2 percent proof stress whichever is smaller and that in prestressed concrete members, the tensile stress in the extreme fibres of the concrete may be permitted so as not to exceed 2/3 of the modulus of rupture of concrete.

3.3.2 *Load Factors* — Whenever earthquake forces are considered along with other normal design forces, the following factors may be adopted:

a) For ultimate load design of steel structures:

$$UL = 1{\cdot}4\ (DL + LL + EL)$$

where

UL = the ultimate load for which the structure or its elements should be designed according to the relevant Indian Standards for steel structures;

DL = the dead load of the structure;

LL = the superimposed load on the structure considering its modified values as given in the relevant clauses of this standard; and

EL = the value of the earthquake load adopted for design.

b) For limit state design of reinforced and prestressed concrete structures.

The partial safety factors for limit states of serviceability and collapse and the procedure for design as given in relevant Indian Standards (*see* IS : 456-1978* and IS : 1343-1980†) may be used for earthquake loads combined with other normal loads. The live load values to be used shall be as given in the relevant clauses of this standard.

NOTE 1 — The members of reinforced or prestressed concrete shall be under reinforced so as to cause a tensile failure. Further, it should be suitably designed so that premature failure due to shear or bond may not occur subject to the provisions of IS : 456-1978* and IS : 1343-1980†.

NOTE 2 — The members and their connections in steel structures should be so proportioned that high ductility is obtained avoiding premature failure due to elastic or inelastic buckling of any type.

NOTE 3 — Appropriate details to achieve ductility are given in IS : 4326-1976‡.

3.3.3 *Permissible Increase in Allowable Bearing Pressure of Soils* — When earthquake forces are included, the permissible increase in allowable bearing pressure of soil shall be as given in Table 1, depending upon the type of foundation of the structure.

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*Code of practice for plain and reinforced concrete (*third revision*).

†Code of practice for prestressed concrete (*first revision*).

‡Code of practice for earthquake resistant design and construction of buildings (*first revision*).

TABLE 1 PERMISSIBLE INCREASE IN ALLOWABLE BEARING PRESSURE OR RESISTANCE OF SOILS

(*Clause* 3.3.3)

Sl No.	Type of Soil Mainly Constituting the Foundation	Permissible Increase in Allowable Bearing Pressure, Percent					
		Piles Passing Through Any soil But Resting on Soil Type I	Piles Not Covered Under Col 3	Raft Foundations	Combined or Isolated RCC Footing with Tie Beams	Isolated RCC Footing Without Tie Beams or Unreinforced Strip Foundations	Well Foundations
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	*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	50	—	50	50	50	50
ii)	*Type II Medium Soils* — All soils with N between 10 and 30 and poorly graded sands or gravelly sands with little or no fines (SP*) with $N > 15$	50	25	50	25	25	25
iii)	*Type III Soft Soils* — All soils other than SP* with $N<10$	50	25	50	25	—	25

NOTE 1 — The allowable bearing pressure shall be determined in accordance with IS : 6403-1981‡ or IS: 1888-1982§.

NOTE 2 — If any increase in bearing pressure has already been permitted for forces other than seismic forces, the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.

NOTE 3 — Submerged loose sands and soils falling under classification SP with standard penetration values less than the values specified in Note 5 below, the vibrations caused by earthquake may cause liquefaction or excessive total and differential settlements. In important projects this aspect of the problem need be investigated and appropriate methods of compaction or stabilization adopted to achieve suitable *N*. Alternatively, deep pile foundation may be provided and taken to depths well into the layer which are not likely to liquefy. Marine clays and other sensitive clays are also known to liquefy due to collapse of soil structure and will need special treatment according to site conditions.

NOTE 4 — The piles should be designed for lateral loads neglecting laterel resistance of soil layers liable to liquefy.

NOTE 5 — Desirable field values of *N* are as follows:

Zone	*Depth below ground level in metres*	*N Values*	*Remarks*
III, IV and V	Up to 5	15	For values of depth between 5 to 10 m
	10	25	linear interpolation is recommended
I and II (for important	Up to 5	10	
structures only)	10	20	

*See IS : 1498-1970 Classification and identification of soils for general engineering purposes (*first revision*).
†See IS : 2131-1981 Method of standard penetration test for soils (*first revision*).
‡Code of practice for determination of bearing capacity of shallow foundations (*first revision*).
§Method of load tests on soils (*second revision*).

3.4 Design Seismic Coefficient for Different Zones

3.4.1 For the purpose of determining the seismic forces, the country is classified into five zones as shown in Fig. 1.

3.4.2 The earthquake force experienced by a structure depends on its own dynamic characteristics in addition to those of the ground motion. Response spectrum method takes into account these characteristics and is recommended for use in case where it is desired to take such effects into account. For design of other structures an equivalent static approach employing use of a seismic coefficient may be adopted.

3.4.2.1 Unless otherwise stated, the basic seismic coefficients (α_0) and seismic zone factors (F_0) in different zones shall be taken as given in Table 2 and Appendices E and F.

TABLE 2 VALUES OF BASIC SEISMIC COEFFICIENTS AND SEISMIC ZONE FACTORS IN DIFFERENT ZONES

(*Clauses* 3.4.2.1, 3.4.2.3 *and* 3.4.5)

SL No.	ZONE No.	METHOD	
		Seismic Coefficient Method	Response Spectrum Method (*see* Appendix F)
		Basic horizontal seismic coefficient, α_0	Seismic zone factor for average acceleration spectra to be used with Fig. 2, F_0
(1)	(2)	(3)	(4)
i)	V	0·08	0·40
ii)	IV	0·05	0·25
iii)	III	0·04	0·20
iv)	II	0·02	0·10
v)	I	0·01	0·05

NOTE — For under ground structures and foundations at 30 m depth or below, the basic seismic coefficient may be taken as $0.5\,\alpha_0$; for structures placed between ground level and 30 m depth, the basic seismic coefficient may be linearly interpolated between α_0 and $0.5\,\alpha_0$.

The seismic coefficients according to **3.4.2.1** for some important towns and cities are given in Appendix E.

3.4.2.2 The design seismic forces shall be computed on the basis of importance of the structure and its soil-foundation system.

3.4.2.3 The design values of horizontal seismic coefficient, α_h in the *Seismic Coefficient* and *Response Spectrum* methods shall be computed as given by the following expressions:

a) *In Seismic Coefficient Method* the design value of horizontal seismic coefficient α_h shall be computed as given by the following expression:

$$\alpha_h = \beta\, I\, \alpha_0$$

where

β = a coefficient depending upon the soil-foundation system (*see* Table 3),

I = a factor depending upon the importance of the structure (*see* Table 4), and

α_0 = basic horizontal seismic coefficient as given in Table 2.

b) *In Response Spectrum Method* the response acceleration coefficient is first obtained for the natural period and damping of the structure and the design value of horizontal seismic coefficient is computed using the following expression:

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

where

β = a coefficient depending upon the soil-foundation system (*see* Table 3),

I = a factor dependant upon the importance of the structure (*see* Table 4),

F_o = seismic zone factor for average acceleration spectra as given in Table 2, and

$\frac{S_a}{g}$ = average acceleration coefficient as read from Fig. 2 for appropriate natural period and damping of the structure.

NOTE 1 — Where a number of modes are to be considered for seismic analysis, α_h shall be worked out corresponding to the various mode periods and dampings and then design forces computed as specified in relevant clauses (*see* **4.2.2**).

NOTE 2 — In case design response spectra is specifically prepared for a structure at a particular site, the same may be used for design directly, and the factor β, I and F_o given in this code which are meant to be used with spectra given in Fig. 2 should not be used in such cases.

3.4.3 To take into account the soil-foundation systems on which the structure is founded, a factor β for various cases is given in Table 3.

3.4.4 The importance factor (I) for various categories of structures shall be as given in Table 4.

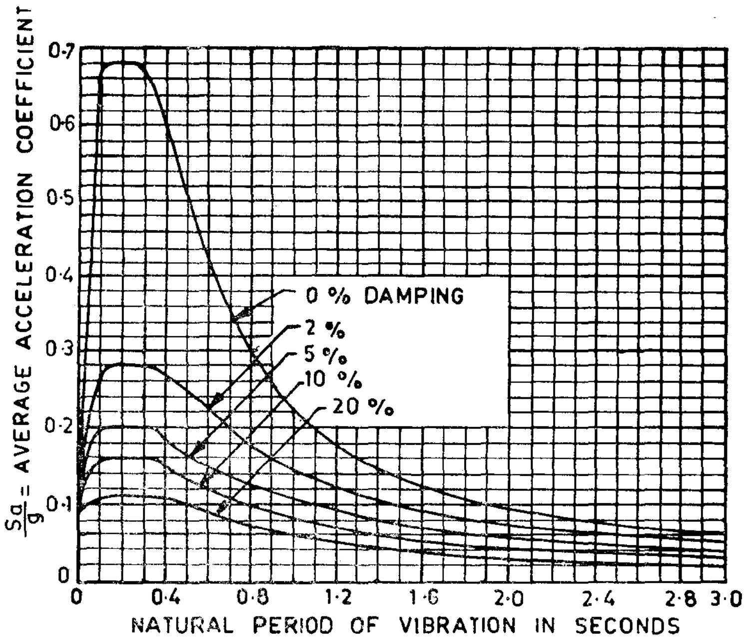


FIG. 2 AVERAGE ACCELERATION SPECTRA

3.4.5 The vertical seismic coefficient where applicable (*see* **3.1.5**) may be taken as half of the horizontal seismic coefficient as indicated in **3.4.2.** In important structures where there is a possibility of amplification of vertical seismic coefficient, dynamic analysis is preferable. In that case F_0 values in Table 2 should be multiplied by 0·5.

4. BUILDINGS

4.1 Design Live Loads

4.1.1 For various loading classes as specified in IS : 875-1960*, the horizontal earthquake force shall be calculated for the full dead load and the percentage of live loads as given below:

Load Class	*Percentage of Design Live Load*
200, 250 and 300	25
400, 500, 750 and 1 000	50

*Code of practice for structural safety of buildings : Loading standards (*revised*).

TABLE 3 VALUES OF β FOR DIFFERENT SOIL-FOUNDATION SYSTEMS

(*Clause* 3.4.3)

Sl No.	Type of Soil Mainly Constituting the Foundation	Values of β for					
		Piles Passing Through Any Soil, but Resting on Soil Type I	Piles Not Covered Under Col 3	Raft Foundations	Combined or Isolated RCC Footings with Tie Beams	Isolated RCC Footings Without Tie Beams or Unreinforced Strip Foundations	Well Foundations
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Type I Rock or hard soils	1·0	—	1·0	1·0	1·0	1·0
ii)	Type II Medium soils	1·0	1·0	1·0	1·0	1·2	1·2
iii)	Type III Soft soils	1·0	1·2	1·0	1·2	1·5	1·5

NOTE — The value of β for dams shall be taken as 1·0.

TABLE 4 VALUES OF IMPORTANCE FACTOR, *I*

(*Clauses* 3.4.2.3 *and* 3.4.4)

Sl No.	Structure	Value of Importance Factor, *I* (*see* Note)
(1)	(2)	(3)
i)	Dams (all types)	3·0
ii)	Containers of inflammable or poisonous gases or liquids	2·0
iii)	Important service and community structures, such as hospitals; water towers and tanks; schools; important bridges; important power houses; monumental structures; emergency buildings like telephone exchange and fire bridge; large assembly structures like cinemas, assembly halls and subway stations	1·5
iv)	All others	1·0

NOTE — The values of importance factor, *I* given in this table are for guidance. A designer may choose suitable values depending on the importance based on economy, strategy and other considerations.

NOTE 1 — The percentage of live loads given above shall also be used for calculating stresses due to vertical loads for combining with those due to earthquake forces. Under the earthquake condition the whole frame except the roof may be assumed loaded with live load proportions specified above, without further reductions in live load as envisaged in IS : 875-1964*.

NOTE 2 — The proportions of the live load indicated above for calculating the horizontal seismic forces are applicable to average conditions. Where the probable loads at the time of an earthquake are more accurately assessed, the designer may alter the proportions indicated or even replace the entire live load proportions by the actual assessed load.

NOTE 3 — If the live load is assessed instead of taking the above proportions for calculating horizontal earthquake force, only that part of the live load shall be considered which possesses mass. Earthquake force shall not be applied on impact effects.

4.1.2 For calculating the earthquake force on roofs, the live load may not be considered.

4.2 Design Criteria for Multi-storeyed Buildings

4.2.1 The criteria for design of multi-storeyed buildings shall be as follows:

a) In case of buildings with floors capable of providing rigid horizontal diaphragm action, a separate building or any block of a building between two separation sections shall be analyzed as a whole for seismic forces as per **3.1.4.** The total shear in any horizontal plane shall be distributed to various elements of lateral forces resisting system assuming the floors to be infinitely rigid in the horizontal plane. In buildings having shear walls together with frames, the frames shall be designed for at least 25 percent of the seismic shear.

b) In case of buildings where floors are not able to provide the diaphragm action as in (a) above the building frames behave independently; and may be analyzed frame by frame with tributory masses for seismic forces as per **3.1.4.**

c) The following methods are recommended for various categories of buildings in various zones:

Building Height	*Seismic Zones*	*Recommended Method*
Greater than 40 m	III, IV and V	Detailed dynamic analysis (either modal anaylsis or time history analysis based on expected ground motion for which special studies are required). For preli-

*Code of practice for structural safety of buildings: Loading standards (*revised*).

Building Height	*Seismic Zones*	*Recommended Method*
		minary design, modal analysis using response spectrum method may be employed
Greater than 90 m	I and II	Modal analysis using response spectrum method
Greater than 40 m and up to 90 m	All zones	Modal analysis using response spectrum method. Use of seismic coefficient method permitted for zones I, II and III
Less than 40 m	All zones	Modal analysis using response spectrum method. Use of seismic coefficient method permitted in all zones

d) Check for drift and torsion according to **4.2.3** and **4.2.4** is desirable for all buildings, being particularly necessary in cases of buildings greater in height than 40 m.

NOTE 1 — For buildings having irregular shape and/or irregular distribution of mass and stiffeners in horizontal and/or vertical plane it is desirable to carry out modal analysis using response spectrum method (*see also* Note 2 below **4.2.1.1**).

NOTE 2 — For multi-storeyed buildings, it is assumed that the storey heights are more or less uniform ranging between 2·7 and 3·6 m. In exceptional cases where one or two-storey heights have to be up to 5 m, the applicability of the clause is not vitiated.

4.2.1.1 The base shear V_B is given by the following formula:

$$V_B = KC\alpha_h W$$

where

K = performance factor depending on the structural framing system and brittleness or ductility of construction (*see* Table 5),

C = a coefficient defining the flexibility of structure with the increase in number of storeys depending upon fundamental time period T (*see* Fig. 3),

α_h = design seismic coefficient as defined in **3.4.2.3** (a),

W = total dead load + appropriate amount of live load as defined in **4.1**, and

T = fundamental time period of the building in seconds (*see* Note 1).

NOTE 1 — The fundamental time period may either be established by experimental observations on similar buildings or calculated by any rational method of analysis. In the absence of such data T may be determined as follows for multi-storeyed buildings:

a) For moment resisting frames without bracing or shear walls for resisting the lateral loads

$$T = 0{\cdot}1\, n$$

where

n = number of storeys including basement storeys.

b) For all others

$$T = \frac{0{\cdot}09\, H}{\sqrt{d}}$$

where

H = total height of the main structure of the building in metres, and

d = maximum base dimension of building in metres in a direction parallel to the applied seismic force.

NOTE 2 — The above clause shall not apply to buildings having irregular shape and/or irregular distribution of mass and stiffness in horizontal and/or vertical plane. A few buildings of this type are shown in Fig. 4. For such buildings modal analysis shall be carried out.

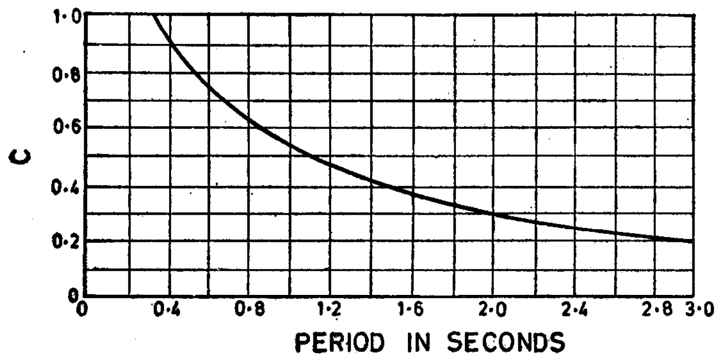


FIG. 3 *C Versus* PERIOD

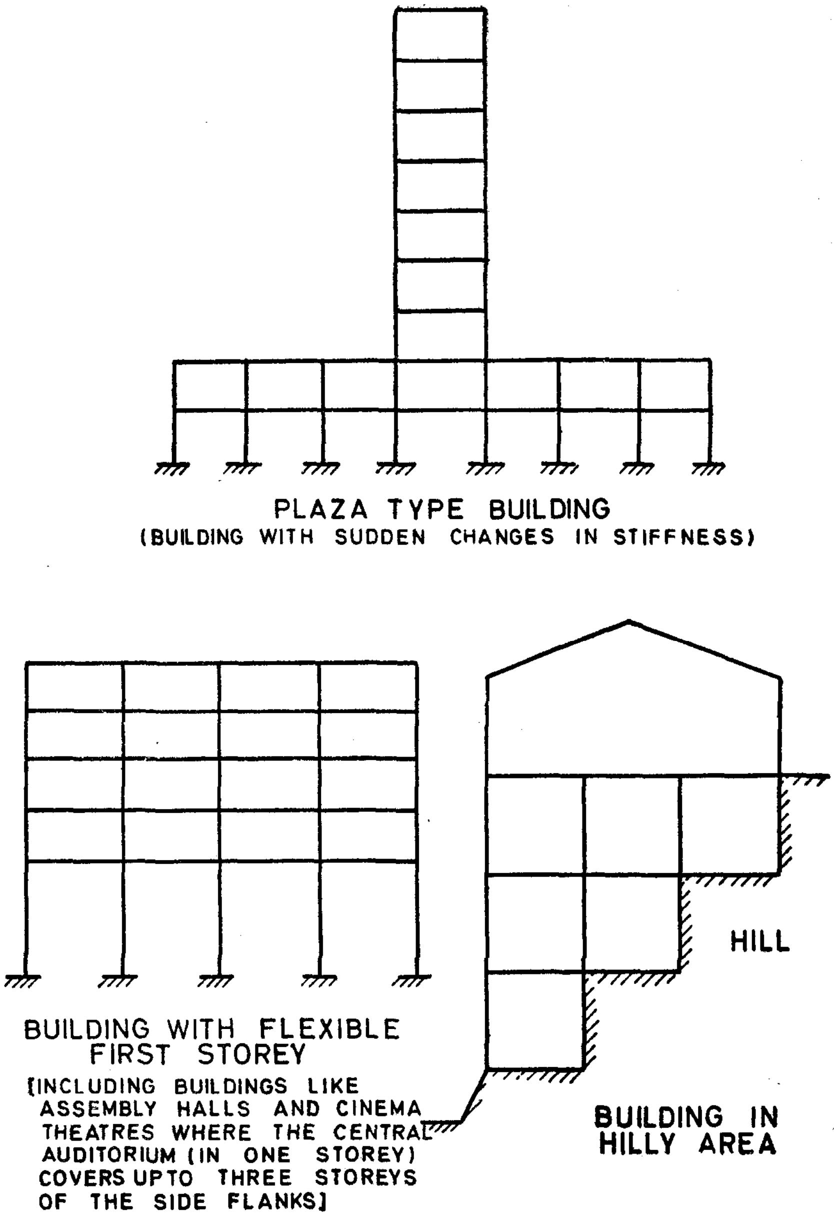


FIG. 4 BUILDINGS IN WHICH CLAUSE 4.2.1.1 SHALL NOT BE APPLICABLE

TABLE 5 VALUES OF PERFORMANCE FACTOR, K

(*Clause* 4.2.1.1)

Sl No.		Structural Framing System	Values of Performance Factor, K	Remarks
(1)		(2)	(3)	(4)
i)	a)	Moment resistant frame with appropriate ductility details as given in IS : 4326-1976* in reinforced concrete or steel	1·0	—
	b)	Frame as above with R. C. shear walls or steel bracing members designed for ductility	1·0	These factors will apply only if the steel bracing members and the infill panels are taken into consideration in stiffness as well lateral strength calculations provided that the frame acting alone will be able to resist at least 25 percent of the design seismic forces
ii)	a)	Frame as in (i) (a) with either steel bracing members or plain or nominally reinforced concrete infill panels	1·3	
	b)	Frame as in (i) (a) in combination with masonry infills	1·6	
iii)		Reinforced concrete framed buildings [Not covered by (i) or (ii) above]	1·6	

*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

4.2.1.2 Distribution of forces along with the height of the building is given by the following formula:

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

where

Q_i = lateral forces at roof of floor i,

V_B = base shear as worked out in **4.2.1.1**,

W_i = load (dead load + appropriate amount of live load) of the roof or any floor i (*see* Note below),

h_i = height measured from the base of building to the roof or any floor i; and

n = number of storeys including the basement floors, where the basement walls are not connected with ground floor deck or the basement walls are not fitted between building columns, but excluding the basement floors where they are so connected.

NOTE — In calculating, W_i, the weight of walls and columns in any storey is assumed to be shared half and half between the roof or floor at top and the floor or ground at bottom, and all weights are assumed to be lumped at the level of the roof or any floor i.

4.2.1.3 The force and shear distributions for at en-storeyed building are illustrated in Fig. 5.

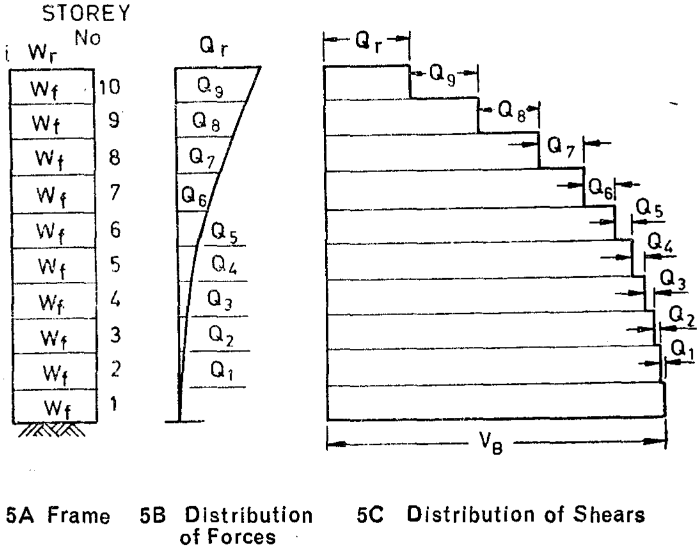


FIG. 5 FORCE AND SHEAR DISTRIBUTION FOR TEN-STOREYED BUILDING

Example:

For a ten-storeyed building in Fig. 5:

$V_B = C\alpha_h K (W_r + 9 W_f)$

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

$$V_j = \sum_{i=j}^{i=n} Q_i$$

where

V_j = shear in jth storey.

NOTE — For other notations, *see* **4.2.1.1** and **4.2.1.2**.

4.2.2 *Modal Analysis* — The lateral load $Q_i^{(r)}$ acting at any floor level i due to rth mode of vibration is given by the following equation:

$$Q_i^{(r)} = K W_i \phi_i^{(r)} C_r \alpha_h^{(r)}$$

where

W_i = weight of the floor i as given in **4.2.1.2**,

K = performance factor depending upon the type of buildings as given in Table 5,

$\phi_i^{(r)}$ = mode shape coefficient at floor i in rth mode vibration obtained from free vibration analysis,

C_r = mode participation factor, and

$\alpha_h^{(r)}$ = design horizontal seismic coefficient as defined in **3.4.2.3** (b) corresponding to appropriate period and damping in the rth mode.

4.2.2.1 The mode participation factor C_r may be given by the following equation:

$$C_r = \frac{\sum_{i=1}^{i=n} W_i \phi_i^{(r)}}{\sum_{i=1}^{i=n} W_i \left[\phi_i^{(r)} \right]^2}$$

where

i, $W_i, \phi_i^{(r)}$ are same as defined in **4.2.2**, and

n = total number of storeys as defined in **4.2.1.1**.

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

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

where

$V_i^{(r)}$ = absolute value of maximum shear at the ith storey in the rth mode; the value of γ shall be as given below:

Height, H m	γ
Up to 20	0·40
40	0·60
60	0·80
90	1·00

NOTE — For intermediate heights of buildings, value of γ may be obtained by linear interpolation.

4.2.2.3 The total load at Q_n and Q_i acting at roof level n and floor level i will be computed from the following equations respectively:

$$Q_n = V_n$$

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

The overturning moments at various levels of the building may be computed by using the above roof and floor level forces.

4.2.3 *Drift* — The maximum horizontal relative displacement due to earthquake forces between two successive floors shall not exceed 0·004 times the difference in levels between these floors.

4.2.4 *Torsion of Buildings* — Provision shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the centre of mass and the centre of rigidity. The design eccentricity shall be taken as 1·5 times the computed eccentricity between the centre of mass and the centre of rigidity. Negative torsional shears shall be neglected.

4.3 Type of Construction — For different types of construction adopted the constructional details and the appropriate design criteria to be adopted shall be according to **5** of IS : 4326-1976*.

4.4 Miscellaneous

4.4.1 Towers, tanks, parapets, smoke stacks (chimneys) and other vertical cantilever projections attached to buildings and projecting above the roofs shall be designed for five times the horizontal seismic coefficient specified in **3.4.2.1**. However, compound walls need not be designed for increased seismic coefficient except where the environmental circumstances indicate that their collapse may lead to serious consequences.

4.4.2 All horizontal projections like cornices and balconies shall be designed to resist a vertical force equal to five times the vertical seismic coefficient specified in **3.4.5** multiplied by the weight of the projection.

NOTE — The increased seismic coefficients specified in **4.4.1** and **4.4.2** are for designing the projecting part and its connection with the main structure. For the design of the main structure such increase need not be considered.

4.4.3 For industrial structures and frame structures of large spans and heights, modal analysis using response spectrum method is recommended.

5. ELEVATED STRUCTURES

5.1 General

5.1.1 The elevated structures covered by these provisions include elevated tanks, refinery vessels and stacklike structures, such as chimneys of normal proportions. In the case of the elevated structures of unusual proportions, more detailed studies shall be made.

5.2 Elevated Tower-Supported Tanks

5.2.1 For the purpose of this analysis, elevated tanks shall be regarded as systems with a single degree of freedom with their mass concentrated at their centres of gravity.

5.2.2 The damping in the system may be assumed as 2 percent of the critical for steel structures and 5 percent of the critical for concrete (including masonry) structures.

5.2.3 The free period T, in seconds, of such structures shall be calculated from the following formula:

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

*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

where

Δ = the static horizontal deflection at the top of the tank under a static horizontal force equal to a weight W acting at the centre of gravity of tank. In calculating the period of steel tanks, the members may be assumed to be pin-joined with only the tensile members of the bracing regarded as active in carrying the loads. No pre-tension shall be assumed in the bracing rods; and

g = acceleration due to gravity.

5.2.4 The design shall be worked out both when the tank is full and when empty. When empty, the weight W used in the design (*see* **5.2.3**) shall consist of the dead load of the tank and one-third the weight of the staging. When full, the weight of contents is to be added to the weight under empty condition.

5.2.5 Using the period T as calculated in **5.2.3** and appropriate damping, the spectral acceleration shall be read off from the average acceleration spectra given in Fig. 2. The design horizontal seismic coefficient, α_h shall be calculated as in **3.4.2.3** (b).

5.2.6 The lateral force shall be taken equal to:

$$\alpha_h W$$

where

α_h = design horizontal seismic coefficient as given in **5.2.5**, and

W = weight as defined in **5.2.4**.

This force shall be assumed to be applied at the centre of gravity of the tank horizontally in the plane in which the structure is assumed to oscillate for purposes of carrying out the lateral load analysis.

5.2.7 *Hydrodynamic Pressure in Tanks*

5.2.7.1 When a tank containing fluid vibrates the fluid exerts impulsive and convective pressures on the tank. The convective pressures during earthquakes are considerably less in magnitude as compared to impulsive pressures and its effect is a sloshing of the water surface. For the purpose of design only the impulsive pressure may be considered.

5.2.7.2 *Rectangular container*

The pressure at any location x (*see* Fig. 6) is given by:

$$p = \alpha_h \, wh \sqrt{3} \left[\frac{y}{h} - \frac{1}{2} \left(\frac{y}{h} \right)^2 \right] \times \frac{\sinh \sqrt{3} \left(\frac{x}{h} \right)}{\cosh \sqrt{3} \left(\frac{l}{h} \right)}$$

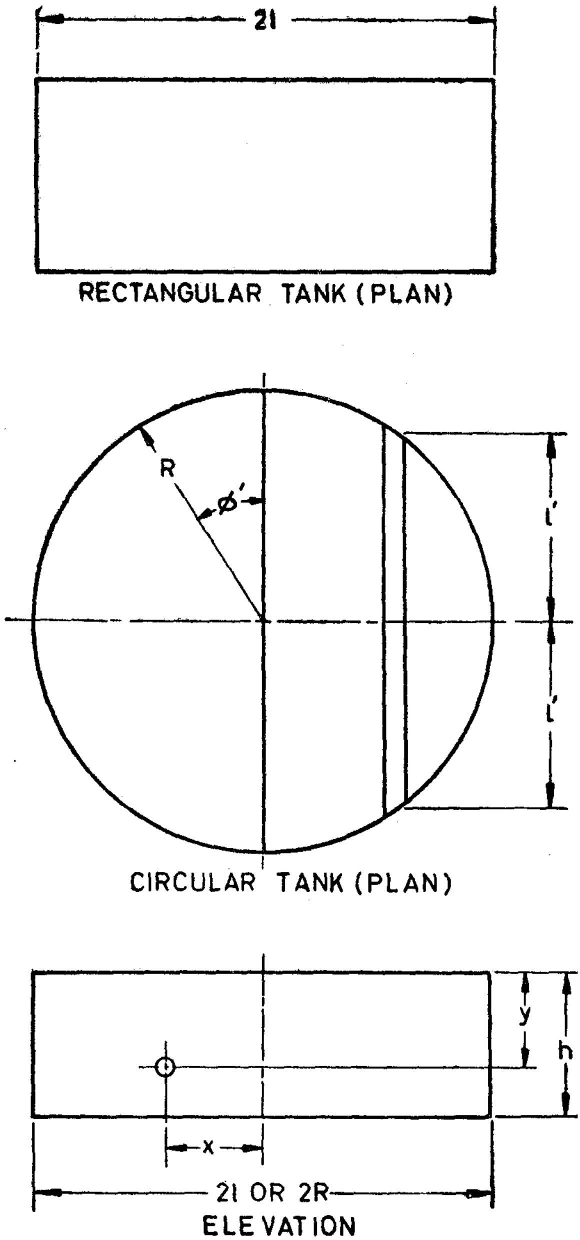


Fig. 6 Rectangular and Circular Water Tanks

The pressure on the wall would be:

$$p_w = \alpha_h wh \sqrt{3}\left[\frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2\right] \tanh \sqrt{3}\left(\frac{l}{h}\right), \text{ and}$$

The pressure on the bottom of the tank would be:

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

where

x, y, l and h are as defined in Fig 6 and w is the unit weight of water, and α_h for tanks located on towers is to be taken as per response spectrum method and for those located on ground corresponding to seismic coefficient method [*see* **3.4.2.3** (a)].

5.2.7.3 *Circular container* — The pressure on the wall would be :

$$p_w = \alpha_h wh \sqrt{3}\cos\phi'\left[\frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2\right]\tanh \sqrt{3}\left(\frac{R}{h}\right), \text{ and}$$

The pressure on the bottom of the tank on a strip of width $2\ l'$ (*see* Fig. 6), would be:

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

where

x, y, l', R and h are as defined in Fig. 6 and w and α_h are as defined in **5.2.7.2**.

5.3 Stacklike Structures

5.3.1 Stacklike structures are those in which the mass and stiffness is more or less uniformly distributed along the height. Cantilever structures like chimneys and refinery vessels are examples of such structures (*see* Note).

NOTE — Such structures will not include structures like bins, hyperbolic cooling towers, refinery columns resting on frames or skirts. Modal analysis will be necessary in such cases.

5.3.2 Period of free vibration, T, of such structures when fixed at base, shall be calculated from the following formula:

$$T = C_T \sqrt{\frac{W_t h'}{E_s A_g}}$$

where

C_T = coefficient depending upon the slenderness ratio of the structure given in Table 6,

W_t = total weight of structure including weight of lining and contents above the base,

h' = height of structures above the base,

E_s = modulus of elasticity of material of the structural shell,

A = area of cross-section at the base of the structural shell, and

g = acceleration due to gravity.

5.3.2.1 For circular structures, $A = 2\pi rt$ where r is the mean radius of structural shell and t its thickness.

5.3.3 Using the period T, as indicated in **5.3.2**, the horizontal seismic coefficient α_h shall be obtained from the spectrum given in Fig. 2 and as in **3.4.2.3** (b).

TABLE 6 VALUES OF C_T AND C_V

(*Clauses* 5.3.2 *and* 5.3.4)

Ratio k	Coefficient C_T	Coefficient C_V
5	14·4	1·02
10	21·2	1·12
15	29·6	1·19
20	38·4	1·25
25	47·2	1·30
30	56·0	1·35
35	65·0	1·39
40	73·8	1·43
45	82·8	1·47
50 or more	1·8k	1·50

where

k = ratio, h'/r_e; and

r_e = radius of gyration of the structural shell at the base section.

5.3.4 The design shear force V, for such structures at a distance x' from the top, shall be calculated by the following formula:

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

where

C_v = coefficient depending on slenderness ratio k given in Table 6,

α_h = design horizontal seismic coefficient determined in accordance with **5.3.3**, and

W_t and h' are same as defined in **5.3.2**.

5.3.5 The design bending moment M at a distance x' from top shall be calculated by the following formula:

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

where

$\bar{h}$ = height of centre of gravity of structure above base. Other notations are the same as given in **5.3.2** and **5.3.4**.

6. BRIDGES

6.1 General

6.1.1 Bridge as a whole and every part of it shall be designed and constructed to resist stresses produced by lateral forces as provided in the standard. The stresses shall be calculated as the effect of a force applied horizontally at the centres of mass of the elements of the structure into which it is conveniently divided for the purpose of design. The forces shall be assumed to come from any horizontal direction.

6.1.2 Masonry and plain concrete arch bridges with spans more than 10 m shall not be built in zones IV and V.

6.1.3 Slab, box and pipe culverts need not be designed for earthquake forces.

6.1.4 Bridges of length not more than 60 m and spans not more than 15 m need not be designed for earthquake forces other than in zones IV and V.

6.1.5 Modal analysis shall be necessary, in the following case, in zones IV and V:

a) in the design of bridges of type, such as, suspension bridge, bascule bridge, cable stayed bridge, horizontally curved girder bridge and reinforced concrete arch or steel arch bridge; and

b) when the height of substructure from base of foundations to the top of pier is more than 30 m or when the bridge span is more than 120 m.

6.1.6 Earthquake force shall be calculated on the basis of depth of scour caused by the discharge corresponding to the average annual flood [*see* IS : 4410 (Part 2/Sec 5)-1977]*. Earthquake and maximum flood shall be assumed not to occur simultaneously.

6.2 Seismic Force — In seismic coefficient method, the seismic force to be resisted shall be computed as follows:

a) $F_h = \alpha_h W_m$

where

F_h = horizontal seismic force to be resisted,

α_h = design horizontal seismic coefficient as specified in **3.4.2.3** (a), and

W_m = weight of the mass under consideration ignoring reduction due to buoyancy or uplift.

b) $F_v = \alpha_v W_m$

where

F_v = vertical seismic force to be resisted, and

α_v = design vertical seismic coefficient.

6.3 Live Load on Bridges

6.3.1 The seismic force due to live load shall be ignored when acting in the direction of the traffic but shall be taken into consideration when acting in the direction perpendicular to traffic as specified in **6.3.2**.

6.3.2 The seismic force due to live load shall be calculated for 50 percent of the design live load excluding impact for railway bridges and 25 percent of the design live load excluding impact for road bridges specified in the relevant Indian Standards. These percentages are only for working out the magnitude of seismic force. For calculating the stresses due to live load, 100 percent of the design live load for railway bridges and 50 percent of the design live load for road bridges specified in the relevant Indian Standards shall be considered at the time of earthquake.

*Glossary of terms relating to river valley projects: Part 2 Hydrology, Section 5 Floods.

6.4 Superstructure

6.4.1 The superstructure shall be designed for horizontal seismic coefficient specified in **3.4.2.3** and vertical seismic coefficient according to **3.4.5** due to the dead load and the live load as specified in **6.3**.

6.4.2 The superstructure of the bridge shall be properly secured to the piers (particularly in zones IV and V) to prevent it from being dislodged off its bearings during an earthquake by suitable methods.

6.4.3 The superstructure shall have a minimum factor of safety of 1·5 against overturning in the transverse direction due to simultaneous action of the horizontal and vertical accelerations.

6.5 Substructure

6.5.1 The seismic forces on the substructure above the normal scour depth (*see* **6.1.6**) shall be as follows:

a) Horizontal and vertical forces due to dead, live and seismic loads as specified in **6.4** transferred from superstructure to the substructure through the bearings as shown in Fig. 7.

b) Horizontal and vertical seismic forces according to **3.4.2.3** and **3.4.5** due to self-weight applied at the centre of mass ignoring reduction due to buoyancy or uplift.

c) Hydrodynamic force as specified in **6.5.2** acting on piers and modification in earth pressure due to earthquake given in **8.1.1** to **8.1.4** acting on abutments.

6.5.1.1 Piers shall be designed for the seismic forces given in **6.5.1** assuming them to act parallel to the current and traffic directions taken separately.

6.5.1.2 In the case of piers, oriented skew either to the direction of current or traffic, they shall be checked for seismic forces acting parallel and perpendicular to pier direction.

6.5.1.3 The substructure shall have a minimum factor of safety of 1·5 due to simultaneous action of the horizontal and vertical accelerations.

6.5.2 For submerged portions of the pier, hydrodynamic force (in addition to earthquake force calculated on the mass of the pier) shall be assumed to act in a horizontal direction corresponding to that of earthquake motion. The total horizontal force F shall be given by the following formula:

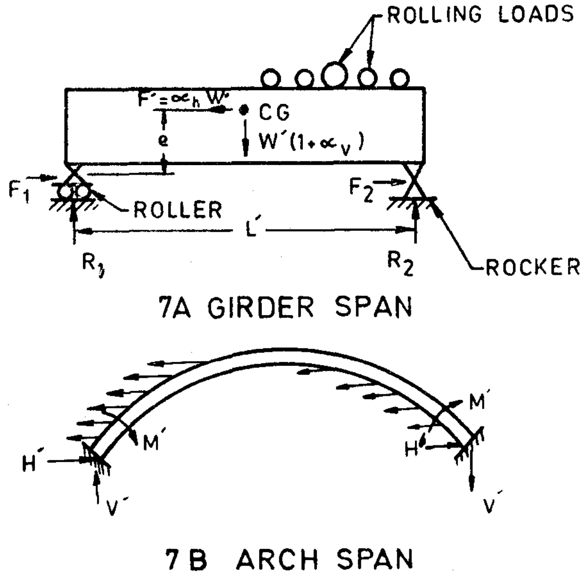
$$F = C_e \, \alpha_h \, W_e$$

where

C_e = a coefficient (*see* Table 7),

α_h = design horizontal seismic coefficient as given in **3.4.2.3** (a), and

W_e = weight of the water of the enveloping cylinder (*see* **6.5.2.2**).



R_1 and R_2 are reactions at the two supports after being modified due to movement (Fe).

Change in vertical reactions = $\pm\ Fe/L'$

$F_1 = \mu R_1$ (if $\mu R_1 < F'/2$)

$F_1 = F'/2$ (if $\mu R_1 > F'/2$)

$F_2 = F' - F_1$

FIG. 7 TRANSFER OF FORCES FROM SUPERSTRUCTURE TO SUBSTRUCTURE

TABLE 7 VALUES OF C_e

$\dfrac{\text{Height of Submerged Portion of Pier } (H)}{\text{Radius of Enveloping Cylinder}}$	C_e
1·0	0·390
2·0	0·575
3·0	0·675
4·0	0·730

6.5.2.1 The pressure distribution will be as shown in Fig. 8. Values of coefficients C_1, C_2, C_3 and C_4 for use in Fig. 8 are given below:

C_1	C_2	C_3	C_4
0·1	0·410	0·026	0·934 5
0·2	0·673	0·093	0·871 2
0·3	0·832	0·184	0·810 3
0·4	0·922	0·289	0·751 5
0·5	0·970	0·403	0·694 5
0·6	0·990	0·521	0·639 0
0·8	0·999	0·760	0·532 0
1·0	1·000	1·000	0·428 6

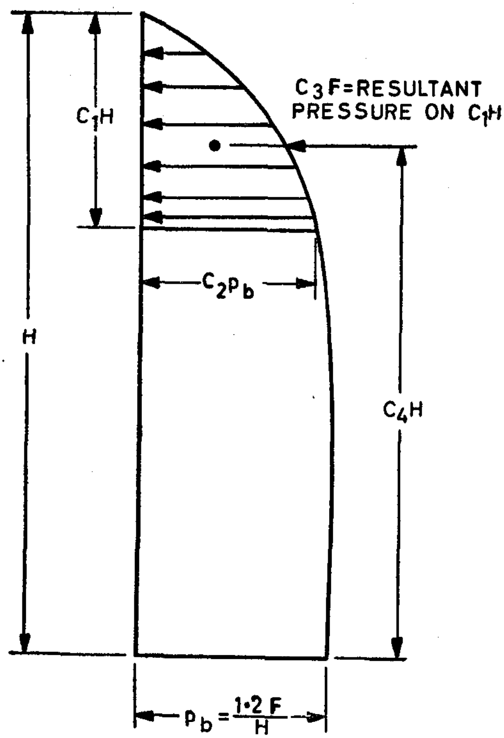


FIG. 8 DIAGRAM SHOWING PRESSURE DISTRIBUTION

6.5.2.2 Some typical cases of submerged portions of piers and the enveloping cylinders are illustrated in Fig. 9.

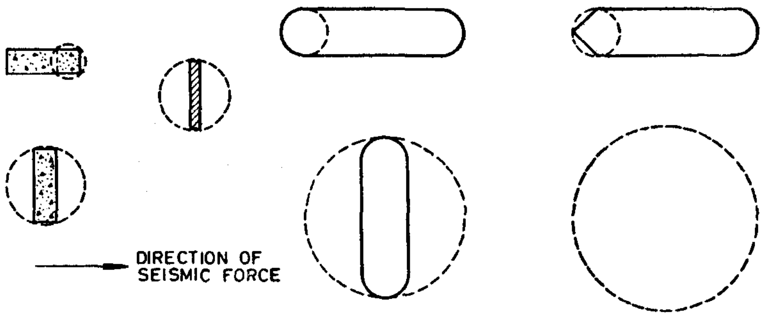


FIG. 9 CASES OF ENVELOPING CYLINDER

6.5.2.3 Thc earth pressure on the back of abutments of bridge shall be calculated as in **8** (*see* Note).

NOTE — The hydrodynamic suction from the water side and dynamic increment in earth pressures from the earth side shall not be considered simultaneously. The water level on earth side may be treated as the same as on the river side.

6.6 Submersible Bridges — For submerged superstructure of submersible bridges, the hydrodynamic pressure shall be determined by the following equation:

$$p = 875\, \alpha_h \sqrt{Hy}$$

where

p = hydrodynamic pressure in kg/m^2;

α_h = design horizontal seismic coefficient as given in **3.4.2.3** (a);

H = height of water surface from the level of deepest scour (*see* **6.1.6**) in m; and

y = depth of the section below the water surface in m.

6.6.1 The total horizontal shear and moment per metre width about the centre of gravity of the base at any depth y, due to hydrodynamic pressure are given by the following relations:

$$V_h = 2/3\, py$$
$$M_h = 4/15\, py^2$$

where

V_h = hydrodynamic shear in kg/m, and

M_h = hydrodynamic moment in kg.m/m.

7. DAMS AND EMBANKMENTS

7.1 General — In the case of important dams it is recommended that detailed investigations are made in accordance with IS : 4967-1968* for estimating the design seismic parameters. However, where such data are not available and in the case of minor works and for preliminary design of major works, the seismic forces specified in **7.2** and **7.3** or **7.4**, as the case may be, shall be considered.

7.2 Hydrodynamic Effects Due to Reservior

7.2.1 *Effects of Horizontal Earthquake Acceleration* — Due to horizontal acceleration of the foundation and dam there is an instantaneous hydrodynamic pressure (or suction) exerted against the dam in addition to hydrostatic forces. The direction of hydrodynamic force is opposite to the direction of earthquake acceleration. Based on the assumption that water is incompressible, the hydrodynamic pressure at depth y below the reservoir surface shall be determined as follows:

$$p = C_s \alpha_h w h$$

where

p = hydrodynamic pressure in kg/m² at depth y,

C_s = coefficient which varies with shape and depth (*see* **7.2.1.1**),

α_h = design horizontal seismic coefficient [*see* **3.4.2.3** (b) and **7.3.1**],

w = unit weight of water in kg/m³, and

h = depth of reservior in m.

7.2.1.1 The variation of the coefficient C_s, with shapes and depths, is illustrated in Appendix G. For accurate determination, these values may be made use of. However, approximate values of C_s for dams with vertical or constant upstream slopes may be obtained as follows :

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

where

C_m = maximum value of C_s obtained from Fig. 10,

y = depth below surface, and

h = depth of reservoir.

*Recommendations for seismic instrumentation for river valley projects.

For dams with combination of vertical and sloping faces, an equivalent slope may be used for obtaining the approximate value of C_s. The equivalent slope may be obtained as given in **7.2.1.2**.

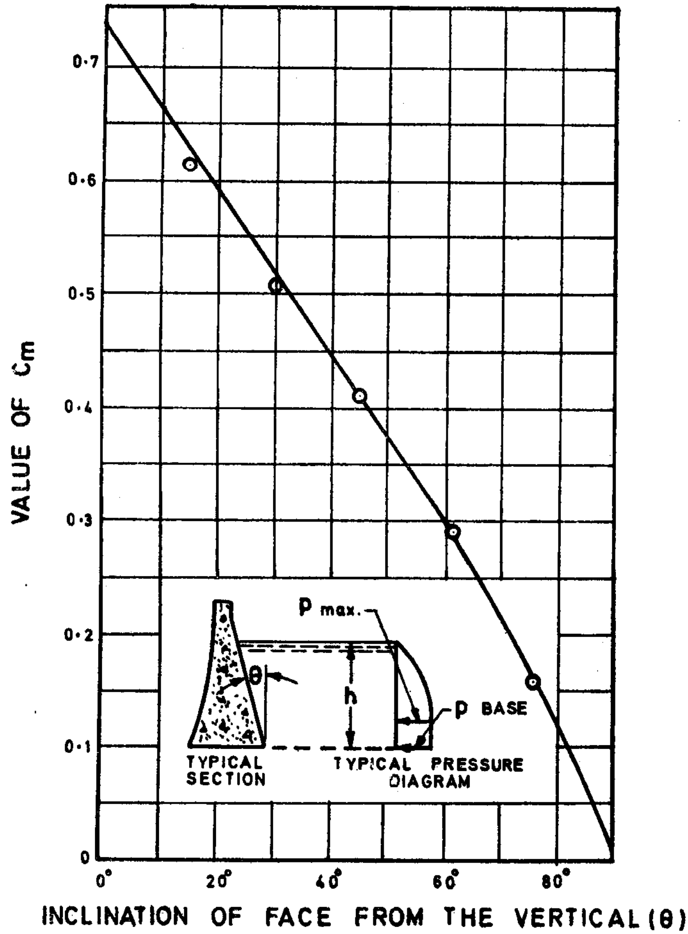


FIG. 10 MAXIMUM VALUES OF PRESSURE COEFFICIENT (C_m) FOR CONSTANT SLOPING FACES

7.2.1.2 If the height of the vertical portion of the upstream face of the dam is equal to or greater than one-half the total height of the dam, analyze it as if vertical throughout. If the height of the vertical portion of the upstream face of the dam is less than one-half the total height of the dam, use the pressure on the sloping line connecting the point of intersection of the upstream face of the dam and the reservoir surface with the point of intersection of the upstream face of the dam with the foundation.

7.2.1.3 The approximate values of total horizontal shear and moment about the centre of gravity of a section due to hydrodynamic pressure are given by the relations:

$$V_h = 0 \cdot 726\ py$$
$$M_h = 0 \cdot 299\ py^2$$

where

V_h = hydrodynamic shear in kg/m at any depth, and

M_h = moment in kg.m/m due to hydrodynamic force at any depth y.

7.2.2 *Effect of Horizontal Earthquake Acceleration on the Vertical Component of Reservoir and Tail Water Load* — Since the hydrodynamic pressure (or suction) acts normal to the face of the dam, there shall, therefore, be a vertical component of this force if the face of the dam against which it is acting is sloping, the magnitude at any horizontal section being:

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

where

W_h = increase (or decrease) in vertical component of load in kg due to hydrodynamic force,

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

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

θ = angle between the face of the dam and the vertical.

The moment due to the vertical component of reservoir and tail water load may be obtained by determining the lever arm from the centroid of the pressure diagram.

7.3 Concrete or Masonry Gravity and Buttress Dams

7.3.1 *Earthquake Forces* — In the design of concrete and masonry dams, the earthquake forces specified in **7.3.1.1** to **7.3.1.4** shall be considered in addition to the hydrodynamic pressures specified in **7.2**. For dams up to 100 m height the horizontal seismic coefficient shall be taken as 1·5 times seismic coefficient, α_h in **3.4.2.3** (a) at the top of the dam reducing linearly to zero at the base. Vertical seismic coefficient shall be taken as 0·75 times the value of α_h at the top of the dam reducing linearly to zero at the base. For dams over 100 m height the response spectrum method shall be used for the design of the dams. Both the seismic coefficient method (for dams up to 100 m height) and response spectrum method (for dams greater than 100 m height) are meant only for preliminary design of dams. For final design dynamic analysis is desirable. For design of dam using the approach of linear variation of normal stresses across the cross-section, tensile stresses may be permitted in the upstream face up to 2 percent of the ultimate crushing strength of concrete.

7.3.1.1 *Concrete or masonry inertia force due to horizontal earthquake acceleration*

a) *Seismic coefficient method* (*dams up to* 100 m *height*) — The horizontal inertia force for concrete or masonry weight due to horizontal earthquake acceleration shall be determined corresponding to the horizontal seismic coefficient specified in **7.3.1**. This inertia force shall be assumed to act from upstream to downstream or downstream to upstream to get the worst combination for design. It causes an overturning moment about the horizontal section adding to that caused by hydrodynamic force.

b) *Response spectrum method* (*dams greater than 100* m *height*)

1) The fundamental period of vibration of the dam may be assumed as:

$$T = 5{\cdot}55 \frac{H^2}{B} \sqrt{\frac{w_m}{gE_s}}$$

where

H = height of the dam in m,

B = base width of the dam in m,

w_m = unit weight of the material of dam in kg/m³,

g = acceleration due to gravity in m/s², and

E_s = modulus of elasticity of the material in kg/m².

2) Using the period in (1) and for a damping of 5 percent, the design horizontal seismic coefficient α_h shall be obtained from **3.4.2.3** (b).

3) The base shear, V_B and base moment M_B may be obtained by the following formulae:

$$V_B = 0{\cdot}6\ W\alpha_h$$

$$M_B = 0{\cdot}9\ W\bar{h}\ \alpha_h$$

where

W = total weight of the masonry or concrete in the dam in kg,

$\bar{h}$ = height of the centre of gravity of the dam above the base in m, and

α_h = design seismic coefficient as obtained in **7.3.1.1** (b) (2).

4) For any horizontal section at a depth y below top of the dam shear force, V_y and bending moment M_y may be obtained as follows:

$$V_y = C'_v\ V_B$$
$$M_y = C'_m\ M_B$$

where C'_v and C'_m are given in Fig. 11.

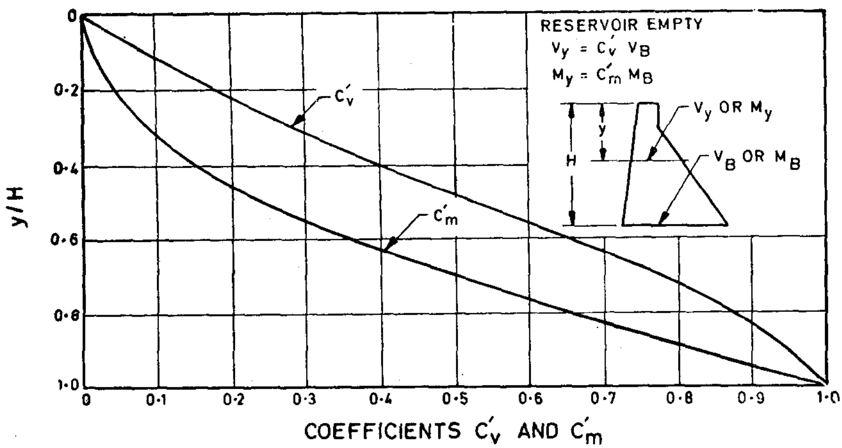


FIG. 11 VALUES OF C'_v AND C'_m ALONG THE HEIGHT OF DAM

7.3.1.2 *Effect of vertical earthquake acceleration* — The effect of vertical earthquake acceleration is to change the unit weight of water and concrete or masonry. An acceleration upwards increases the weight and an acceleration downwards decreases the weight. To consider the effect of vertical earthquake acceleration, the vertical seismic coefficient would be as follows:

a) *For seismic coefficient method of design* — At the top of the dams it would be 0·75 times the α_h value given in **3.4.2.3** (a) and reducing linearly to zero at the base.

b) *For response spectrum method of design* — At the top of the dam it would be 0·75 times the value of α_h given in **7.3.1.1** (b) (2) and reducing linearly to zero at the base.

7.3.1.3 *Effect of earthquake acceleration on uplift forces* — Effect of earthquake acceleration on uplift forces at any horizontal section is determined as a function of the hydrostatic pressure of reservoir and tail-water against the faces of the dam. During an earthquake the water pressure is changed by the hydrodynamic effect. However, the change is not considered effective in producing a corresponding increase or reduction in the uplift force. The duration of the earthquake is too short to permit the building up of pore pressure in the concrete and rock foundations.

7.3.1.4 *Effect of earthquake acceleration on dead silt loads* — It is sufficient to determine the increase in the silt pressure due to earthquake by considering hydrodynamic forces on the water up to the base of the dam and ignoring the weight of the silt.

7.3.2 *Earthquake Forces for Overflow Sections* — The provisions for the dam as given in **7.3.1** to **7.3.1.4** will be applicable to over-flow sections as well. In this case, the height of the dam shall be taken from the base of the dam to the top of the spillway bridge for computing the period as well as shears and moments in the body of the dam. However, for the design of the bridge and the piers, the horizontal seismic coefficients in either direction may be taken as the design seismic coefficient for the top of the dam worked out in **7.3.1** and applied uniformly along the height of the pier.

7.4 Earth and Rockfill Dams and Embankments

7.4.1 *General* — It is recognized that an earth dam or embankment vibrates when subjected to ground motion during an earthquake requiring thereby a dynamic analysis of the structure for its design. Nevertheless, currently accepted design procedure is based on the assumption that the portion of the dam above the rupture surface is rigid. Therefore, the method given in **7.4.2** which assumes additional horizontal and vertical loads on the soil mass within the rupture surface shall be adopted. It is, however, desirable to carry out dynamic analysis for final design of important dams in order to estimate deformations in dams in probable future earthquakes.

7.4.2 *Seismic Force on Soil Mass*

7.4.2.1 The procedure for finding out the seismic coefficient which will depend upon the height of the dam and the lowest point of the rupture surface shall be as follows:

a) Determine the fundamental period of the structure from the formula:

$$T = 2{\cdot}9\, H_t \sqrt{\rho / G}$$

where

T = fundamental period of the earth dam in s,
H_t = height of the dam above toe of the slopes,
ρ = mass density of the shell material, and
G = modulus of rigidity of the shell material.

NOTE — The quantity $\sqrt{G/\rho}$ is the shear wave velocity through the material of the dam and may be used if known instead of ρ and G.

b) Determine S_a/g for this period T and 10 percent damping from average acceleration spectrum curves given in Fig. 2.

c) Compute design seismic coefficient α_h using **3.4.2.3** (b).

7.4.2.2 For checking slope failure with the lowest point of the rupture surface at any depth y below top of dam, the value of equivalent uniform seismic coefficient shall be taken as:

$$\alpha_y = \left(2{\cdot}5 - 1{\cdot}5 \frac{y}{H}\right) \alpha_h$$

where

H = total height of the dam.

7.4.3 *Stability of the Upstream Slope*

7.4.3.1 The stability of the upstream slope of an earth or rockfill dam shall be tested with full reservoir level with horizontal forces due to earthquake acting in upstream direction and vertical forces due to earthquake (taken as one half of horizontal) acting upwards.

7.4.3.2 For preliminary design, a factor of safety of unity shall be accepted as being adequate for ensuring stability of upstream slope. The factor of safety need be tested only for failure surface which passes through the lower half of the dam.

7.4.4 *Stability of Downstream Slope* — The provision of **7.4.3** shall also apply in determining stability of the downstream slope except that the horizontal force due to earthquake should be considered acting in the downstream direction.

7.4.5 *Miscellaneous* — Earthquake forces shall not be normally included in stability analysis for the construction stage or for the reservoir empty condition. However, where the construction or operating schedule requires the reservoir empty condition to exist for prolonged periods, earthquake forces may be included and may be calculated based on 50 percent of the value obtained from **7.4.3** or **7.4.4**.

Provisions in **7.4.3** and **7.4.4** modified to suit the conditions of empty reservoir shall apply for testing the stability of the upstream and downstream slopes.

Junctions between spillways and abutments shall be constructed with great care in view of the damage that may be caused by differential vibrations of the dam and the spillway.

8. RETAINING WALLS

8.1 Lateral Earth Pressure — The pressure from earthfill behind retaining walls during an earthquake shall be as given in **8.1.1** to **8.1.4**. In the analysis, cohesion has been neglected. This assumption is on conservative side.

8.1.1 *Active Pressure Due to Earthfill* — The general conditions encountered for the design of retaining walls are illustrated in Fig. 12A. The active pressure exerted against the wall shall be:

$$P_a = \frac{1}{2} w h^2 C_a$$

where

P_a = active earth pressure in kg/m length of wall,

w = unit weight of soil in kg/m^3,

h = height of wall in m, and

$$C_a = \frac{(1 \pm \alpha_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{\dfrac{\sin(\phi + \delta)\sin(\phi - \iota - \lambda)}{\cos(\alpha - \iota)\cos(\delta + \alpha + \lambda)}\right\}^{\frac{1}{2}}}\right]^2$$

the maximum of the two being the value for design,

α_v = vertical seismic coefficient — its direction being taken consistently throughout the stability analysis of wall and equal to $\frac{1}{2}\alpha_h$

ϕ = angle of internal friction of soil,

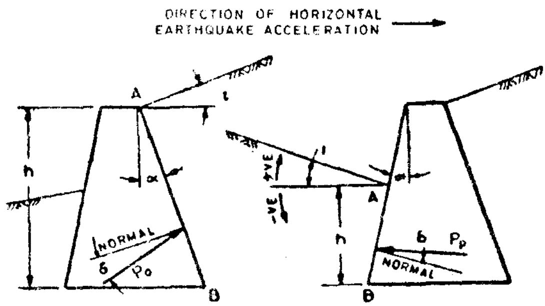
$\lambda = \tan^{-1} \dfrac{\alpha_h}{1 \pm \alpha_v}$

α = angle which earth face of the wall makes with the vertical,

ι = slope of earthfill,

δ = angle of friction between the wall and earthfill, and

α_h = horizontal seismic coefficient [*see* **3.4.2.3** (a)].



12A Active Pressure 12B Passive Pressure

FIG. 12 EARTH PRESSURE DUE TO EARTHQUAKE ON RETAINING WALLS

8.1.1.1 The active pressure may be determined graphically by means of the method described in Appendix H.

8.1.1.2 *Point of application* — From the total pressure computed as above subtract the static active pressure obtained by putting $\alpha_h = \alpha_v = \lambda = 0$ in the expression given in **8.1.1.** The remainder is the dynamic increment. The static component of the total pressure shall be applied at an elevation $h/3$ above the base of the wall. The point of application of the dynamic increment shall be assumed to be at mid-height of the wall.

8.1.2 *Passive Pressure Due to Earthfill* — The general conditions encountered in the design of retaining walls are illustrated in Fig. 12B. The passive pressure against the walls shall be given by the following formula :

$$P_p = \tfrac{1}{2} w h^2 C_p$$

where

P_p = passive earth pressure in kg/m length of wall;

$$C_p = \frac{(1 \pm \alpha_v)\cos^2(\phi + \alpha - \lambda)}{\cos\lambda \cos^2\alpha \cos(\delta - \alpha + \lambda)} \times \left[\frac{1}{1 - \left\{ \frac{\sin(\phi + \delta)\sin(\phi + \iota - \lambda)}{\cos(\alpha - \iota)\cos(\delta - \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2$$

the minimum of the two being the value for design; w, h, α, ϕ and ι are as defined in **8.1.1**; and

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

8.1.2.1 The passive pressure may be determined graphically by means of the method described in Appendix J.

8.1.2.2 *Point of application* — From the static passive pressure obtained by putting $\alpha_h = \alpha_v = \lambda = 0$ in the expression given in **8.1.2**, subtract the total pressure computed as above. The remainder is the dynamic decrement The static component of the total pressure shall be applied at an elevation $h/3$ above the base of the wall. The point of application of the dynamic decrement shall be assumed to be at an elevation 0·66 h above the base of the wall.

8.1.3 *Active Pressure Due to Uniform Surcharge* — The active pressure against the wall due to a uniform surcharge of intensity q per unit area of the inclined earthfill surface shall be:

$$(P_a)_q = \frac{qh \cos\alpha}{\cos(\alpha - \iota)} C_a$$

8.1.3.1 *Point of application* — 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 *Passive Pressure Due to Uniform Surcharge* — The passive pressure against the wall due to a uniform surcharge of intensity q per unit area of the inclined earthfill shall be:

$$(P_p)_q = \frac{q_h \cos\alpha}{\cos(\alpha - \iota)} C_p$$

8.1.4.1 *Point of application* — The dynamic decrement in passive pressures due to uniform surcharge shall be applied at an elevation of 0·66 *h* above the base of the walls while the static component shall be applied at mid-height of the wall.

8.2 Effect of Saturation on Lateral Earth Pressure

8.2.1 For saturated earthfill, the saturated unit weight of the soil shall be adopted as in the formulae described in **8.1**.

8.2.2 For submerged earthfill, the dynamic increment (or decrement) in active and passive earth pressure during earthquakes shall be found from expressions given in **8.1.1** and **8.1.2** with the following modifications:

a) The value of δ shall be taken as $\frac{1}{2}$ the value of δ for dry backfill.

b) The value of λ shall be taken as follows:

$$\lambda = \tan^{-1} \frac{w_s}{w_s - 1} \times \frac{\alpha_h}{1 \pm \alpha_v}$$

where

w_s = saturated unit weight of soil in gm/cc,

α_h = horizontal seismic coefficient [*see* **3.4.2.3** (a)], and

α_v = vertical seismic coefficient which is $\frac{1}{2}$ α_h.

c) Buoyant unit weight shall be adopted.

d) From the value of earth pressure found out as above, subtract the value of earth pressure determined by putting $\alpha_h = \alpha_v = \lambda = 0$ but using buoyant unit weight. The remainder shall be dynamic increment.

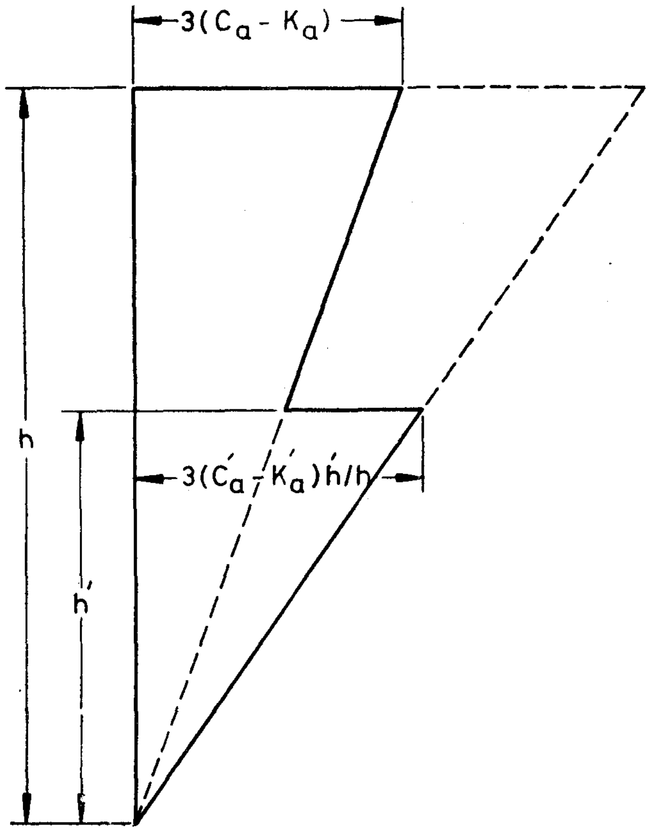
8.2.3 Hydrodynamic pressure on account of water contained in earthfill shall not be considered separately as the effect of acceleration on water has been considered indirectly.

8.3 Partially Submerged Backfill

8.3.1 The ratio of the lateral dynamic increment in active pressures to the vertical pressures at various depths along the height of wall may be taken as shown in Fig. 13.

The pressure distribution of dynamic increment in active pressures may be obtained by multiplying the vertical effective pressures by the coefficients in Fig. 13 at corresponding depths.

NOTE — The procedure may also be used for determining the distribution of dynamic pressure increments in **8.1.1.2** and **8.1.3.1**.



C_a is computed as in **8.1.1** for dry (moist) saturated backfills.
C'_a is computed as in **8.1.1** and **8.2.2** for submerged backfills.
K_a is the value of C_a when $\alpha_h = \alpha_v = \lambda = 0$.
K'_a is the value of C'_a when $\alpha_h = \alpha_v = \lambda = 0$.
h' is the height of submergence above the base of the wall.
h is the height of the retaining wall.

FIG. 13 DISTRIBUTION OF THE RATIO $\frac{\text{LATERAL DYNAMIC INCREMENT}}{\text{VERTICAL EFFECTIVE PRESSURE}}$ WITH HEIGHT OF WALL

8.3.2 A similar procedure as in **8.3.1** may be utilized for determining the distribution of dynamic decrement in passive pressures.

8.4 Concrete or Masonry Inertia Forces — Concrete or masonry inertia forces due to horizontal and vertical earthquake accelerations are the products of the weight of wall and the horizontal and vertical seismic coefficients respectively (*see* **3.4.2** and **3.4.5**).

NOTE — To ensure adequate factor of safety under earthquake condition, the design shall be such that the factor of safety against sliding shall be 1·2 and the resultant of all the forces including earthquake force shall fall within the middle three-fourths of the base width provided. In addition, bearing pressure in soil should not exceed the permissible limit.

9. NOTATIONS AND SYMBOLS

9.1 The various notations and letter symbols used in the formulae and in the body of the standard shall have the meaning as given in Appendix K.

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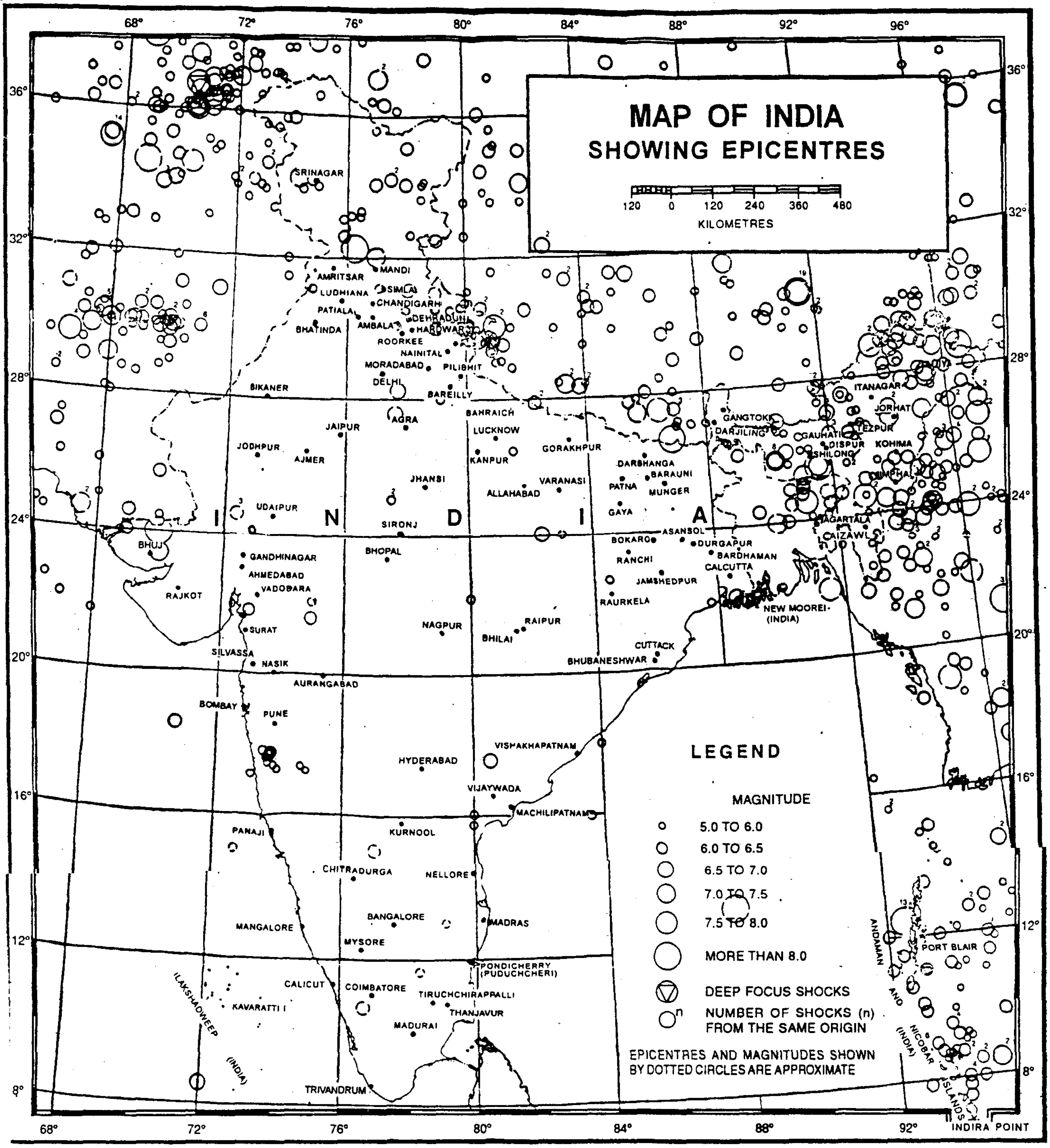
APPENDIX A

(*Clause* 0.7.1)



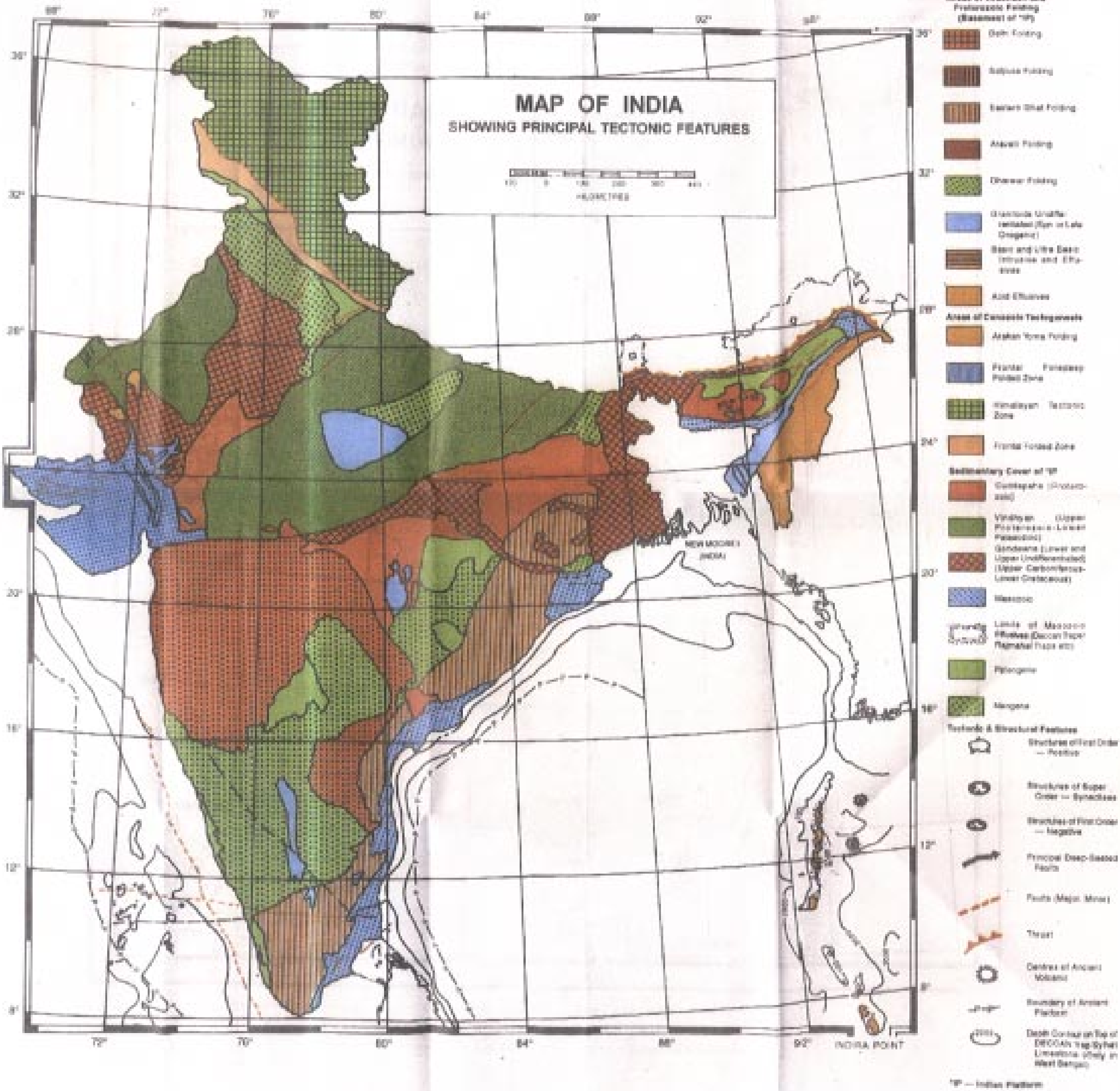
The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

Responsibility for the correctness of internal details shown on the maps rests with the publishers.

Based upon Survey of India map with the permission of the Surveyor General of India.

APPENDIX B

(Clause 0.7.1)



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

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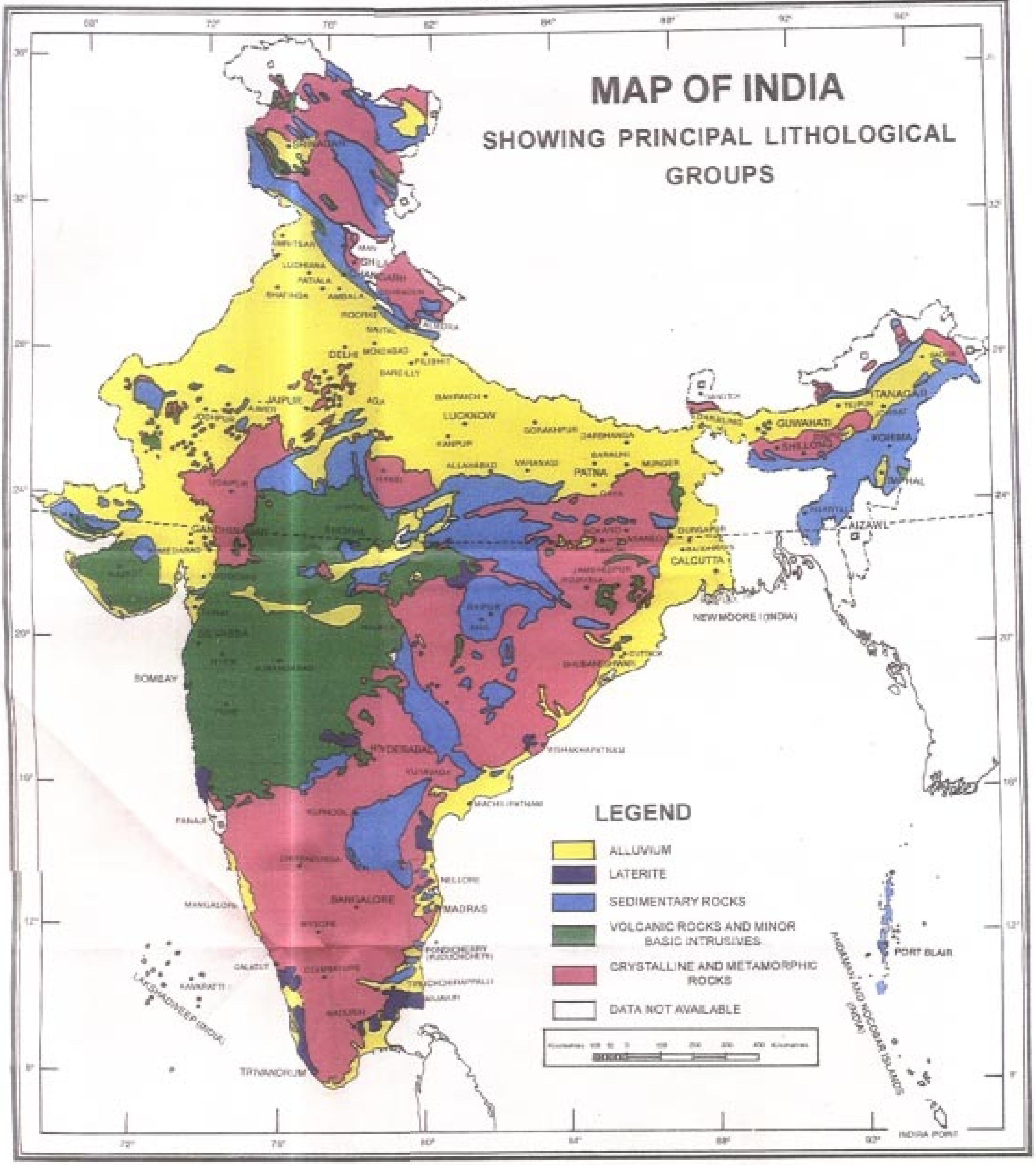
APPENDIX C

(*Clause* 0.7.1)



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

Responsibility for the correctness of internal details shown on the maps rests with the publisher.

Based upon Survey of India map with the permission of the Surveyor General of India.

NOTE — For further details of the Map of India on Lithological Groups, reference may be made to the Geological Map of India, Seventh Edition, 1976, Scale 1 : 200,000, published by Geological Survey of India.

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APPENDIX D

(*Clause* 2.7)

EARTHQUAKE INTENSITY SCALES

D-1. MODIFIED MERCALLI INTENSITY SCALE (ABRIDGED)

Class of Earthquake	*Remarks*
I	Not felt except by a very few under specially favourable circumstances
II	Felt only by a few persons at rest, specially on upper floors of buildings; and delicately suspended objects may swing
III	Felt quite noticeably indoors, specially on upper floors of buildings but many people do not recognize it as an earthquake; standing motor cars may rock slightly; and vibration may be felt like the passing of a truck
IV	During the day felt indoors by many, outdoors by a few, at night some awakened; dishes, windows, doors disturbed; walls make creaking sound, sensation like heavy truck striking the building; and standing motor cars rocked noticeably
V	Felt by nearly everyone; many awakened; some dishes, windows, etc,. broken; a few instances of cracked plaster; unstable objects overturned; disturbance of trees, poles and other tall objects noticed sometimes; and pendulum clocks may stop
VI	Felt by all, many frightened and run outdoors; some heavy furniture moved; a few instances of fallen plaster or damaged chimneys; and damage slight
VII	Everybody runs outdoors, damage negligible in buildings of good design and construction; slight to moderate in well built ordinary structures; considerable in poorly built or badly designed structures; and some chimneys broken, noticed by persons driving motor cars
VIII	Damage slight in specially designed structures; considerable in ordinary but substantial buildings with partial collapse; very heavy in poorly built structures; panel walls thrown out of framed structures; falling of chimney, factory stacks, columns, monuments, and walls; heavy furniture overturned, sand and mud ejected in small amounts; changes in well water; and disturbs persons driving motor cars

Class of Earthquake	*Remarks*
IX	Damage considerable in specially designed structures; well designed framed structures thrown out of plumb; very heavy in substantial buildings with partial collapse; buildings shifted off foundations; ground cracked conspicuously; and underground pipes broken
X	Some well built wooden structures destroyed; most masonry and framed structures with foundations destroyed; ground badly cracked; rails bent; landslides considerable from river banks and steep slopes; shifted sand and mud; and water splashed over banks
XI	Few, if any, masonry structures remain standing; bridges destroyed; broad fissures in ground, underground pipelines completely out of service; earth slumps and landslips in soft ground; and rails bent greatly
XII	Total damage; waves seen on ground surfaces; lines of sight and levels distorted; and objects thrown upward into the air

D-2. COMPREHENSIVE INTENSITY SCALE

D-2.1 The scale was discussed generally at the inter-governmental meeting convened by UNESCO in April 1964. Though not finally approved, the scale is more comprehensive and describes the intensity of earthquake more precisely. The main definitions used are as follows:

a) *Type of Structures (Buildings):*

Structure A Buildings in field-stone, rural structures, unburnt-brick houses, clay houses.

Structure B Ordinary brick buildings, buildings of the large block and prefabricated type, half timbered structures, buildings in natural hewn stone.

Structure C Reinforced buildings, well built wooden structures.

b) *Definition of Quantity:*

Single, few	About 5 percent
Many	About 50 percent
Most	About 75 percent

c) *Classification of Damage to Buildings:*

Grade 1	Slight damage	Fine cracks in plaster; fall of small pieces of plaster
Grade 2	Moderate damage	Small cracks in walls; fall of fairly large pieces of plaster, pantiles slip off; cracks in chimneys; parts of chimney fall down

Grade 3	Heavy damage	Large and deep cracks in walls; fall of chimneys
Grade 4	Destruction	Gaps in walls; parts of buildings may collapse; separate parts of the building lose their cohesion; and inner walls collapse
Grade 5	Total damage	Total collapse of buildings

d) *Intensity Scale*:

I Not noticeable.

The intensity of the vibration is below the limit of sensibility; the tremor is detected and recorded by seismographs only

II Scarcely noticeable (very slight).

Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings

III Weak, partially observed only.

The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects, somewhat more heavily on upper floors

IV Largely observed.

The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors and dishes rattle. Floors and walls crack. Furniture begins to shake. Hanging objects swing slightly. Liquids in open vessels are slightly disturbed. In standing motor cars the shock is noticeable

V Awakening:

a) The earthquake is felt indoors by all, outdoors by many- Many sleeping people awake. A few run outdoors. Animals become uneasy. Buildings tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Unstable objects may be overturned or shifted. Open doors and windows are thrust open and slam back again. Liquids spill in small amounts from well-filled open containers. The sensation of vibration is like that due to heavy object falling inside the buildings

b) Slight damages in buildings of Type A are possible

c) Sometimes change in flow of springs

VI Frightening:

a) Felt by most indoors and outdoors. Many people in buildings are frightened and run outdoors. A few persons lose their balance. Domestic animals run out of their stalls. In few instances dishes and glassware may break, books fall down. Heavy furniture may possibly move and small steeple bells may ring

b) Damage of Grade 1 is sustained in single buildings of Type B and in many of Type A. Damage in few buildings of Type A is of Grade 2.

c) In few cases cracks up to widths of 1 cm possible in wet ground; in mountains occasional landslips; change in flow of springs and in level of well water are observed

VII Damage of buildings:

a) Most people are frightened and run outdoors. Many find it difficult to stand. The vibration is noticed by persons driving motor cars. Large bells ring

b) In many buildings of Type C damage of Grade 1 is caused; in many buildings of Type B damage is of Grade 2. Most buildings of Type A suffer damage of Grade 3, few of Grade 4. In single instances landslips of roadway on steep slopes; cracks in roads; seams of pipelines damaged; cracks in stone walls

VIII Destruction of buildings:

a) Fright and panic; also persons driving motor cars are disturbed. Here and there branches of trees break off. Even heavy furniture moves and partly overturns. Hanging lamps are damaged in part

b) Most buildings of Type C suffer damage of Grade 2, and few of Grade 3. Most buildings of Type B suffer damage of Grade 3, and most buildings of Type A suffer damage of Grade 4. Many buildings of Type C suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist. Tombstones overturn. Stone walls collapse.

c) Small landslips in hollows and on banked roads on steep slopes; cracks in ground up to widths of several centimetres. Water in lakes becomes turbid. New reservoirs come into existence. Dry wells refill and existing wells becomes dry. In many cases change in flow and level of water is observed, IX General damage to buildings:

a) General panic; considerable damage to furniture. Animals run to and fro in confusion and cry

b) Many buildings of Type C suffer damage of Grade 3, and a few of Grade 4. Many buildings of Type B show damage of Grade 4, and a few of Grade 5. Many buildings of Type A suffer damage of Grade 5. Monuments and columns fall. Considerable damage to reservoirs; underground pipes partly broken. In individual cases railway lines are bent and roadway damaged

c) On flat land overflow of water, sand and mud is often observed. Ground cracks to widths of up to 10 cm, on slopes and river banks more than 10 cm; furthermore a large number of slight cracks in ground; falls of rock, many landslides and earth flows; large waves in water. Dry wells renew their flow and existing wells dry up

X General destruction of buildings:

a) Many buildings of Type C suffer damage of Grade 4, and a few of Grade 5. Many buildings of Type B show damage of Grade 5; most of Type A have destruction of Grade 5; critical damage to dams and dykes and severe damage to bridges. Railway lines are bent slightly. Underground pipes are broken or bent. Road paving and asphalt show waves

b) In ground, cracks up to widths of several centimetres, sometimes up to 1 metre. Parallel to water courses occur broad fissures. Loose ground slides from steep slopes. From river banks and steep coasts, considerable landslides are possible. In coastal areas, displacement of sand and mud; change of water level in wells; water from canals, lakes, rivers, etc, thrown on land. New lakes occur

XI Destruction:

a) Severe damage even to well built buildings, bridges, water dams and railway lines; highways become useless; underground pipes destroyed

b) Ground considerably distorted by broad cracks and fissures, as well as by movement in horizontal and vertical directions; numerous landslips and falls of rock. The intensity of the earthquake requires to be investigated specially

XII Landscape changes:

a) Practically all structures above and below ground are greatly damaged or destroyed

b) The surface of the ground is radically changed. Considerable ground cracks with extensive vertical and horizontal movements are observed. Falls of rock and slumping of river banks over wide areas, lakes are dammed; waterfalls appear, and rivers are deflected. The intensity of the earthquake requires to be investigated specially.

APPENDIX E

(*Clause* 3.4.2.1 and *Table* 2)

BASIC HORIZONTAL SEISMIC COEFFICIENTS FOR SOME IMPORTANT TOWNS

Town	*Zone*	*Basic Horizontal Seismic Coefficient* α_0	*Town*	*Zone*	*Basic Horizontol Seismic Coefficient* α_0
Agra	III	0·04	Bikaner	III	0·04
Ahmadabad	III	0·04	Bokaro	III	0·04
Ajmer	I	0·01	Bombay	III	0·04
Allahabad	II	0·02	Burdwan	III	0·04
Almora	IV	0·05	Calcutta	III	0·04
Ambala	IV	0·05	Calicut	III	0·04
Amritsar	IV	0·05	Chandigarh	IV	0·05
Asansol	III	0·04	Chitrgaurad	I	0·01
Aurangabad	I	0·01	Coimbatore	III	0·04
Bahraich	IV	0·05	Cuttack	III	0·04
Bangalore	I	0·01	Darbhanga	V	0·08
Barauni	IV	0·05	Darjeeling	IV	0·05
Bareilly	III	0·04	Dehra Dun	IV	0·05
Bhatinda	III	0·04	Delhi	IV	0·05
Bhilai	I	0·01	Durgapur	III	0·04
Bhopal	II	0·02	Gangtok	IV	0.05
Bhubaneswar	III	0·04	Gauhai	V	0·08
Bhuj	V	0·08	Gaya	III	0·04

Town	*Zone*	*Basic Horizontal Seismic Coefficient* α_0
Gorakhpur	IV	0·05
Hyderabad	I	0·01
Imphal	V	0·08
Jabalpur	III	0·04
Jaipur	II	0·02
Jamshedpur	II	0·02
Jhansi	I	0·01
Jodhpur	I	0·01
Jorhat	V	0·08
Kanpur	III	0·04
Kathmandu	V	0·08
Kohima	V	0·08
Kurnool	I	0·01
Lucknow	III	0·04
Ludhiana	IV	0·05
Madras	II	0·02
Madurai	II	0·02
Mandi	V	0·08
Mangalore	III	0·04
Monghyr	IV	0·05
Moradabad	IV	0·05
Mysore	I	0·01
Nagpur	II	0·02
Nainital	IV	0·05
Nasik	III	0·04
Nellore	II	0·02
Panjim	III	0·04
Patiala	III	0·04
Patna	IV	0·05
Pilibhit	IV	0·05
Pondicherry	II	0·02
Pune	III	0·04
Raipur	I	0 01
Rajkot	III	0·04
Ranchi	II	0·02
Roorkee	IV	0·05
Rourkela	I	0·01
Sadiya	V	0·08
Simla	IV	0·05
Sironj	I	0·01
Srinagar	V	0·08
Surat	III	0·04
Tezpur	V	0·08
Thanjavur	II	0·02
Tiruchirapalli	II	0·02
Trivandrum	III	0·04
Udaipur	II	0·02
Vadodara	III	0·04
Varanasi	III	0·04
Vijayawada	III	0·04
Visakhapatnam	II	0·02

NOTE — The coefficients given are according to **3.4.2.1** and should be suitably modified for important structures in accordance with **3.4.2.3, 4.4** and **7.1** and should be read along with other provisions of the standard.

APPENDIX F

(*Clause* 3.4.2.1 and *Table* 2)

SPECTRA OF EARTHQUAKE

F-1. GENERAL

F-1.1 Spectrum of an earthquake is the representation of the maximum dynamic response of idealized structures during an earthquake. The idealized structure is a single degree of freedom system having a certain period of vibration and damping. The maximum response is plotted against the natural period of vibration and can be expressed in terms of maximum absolute acceleration, maximum relative velocity or maximum relative displacement. For the purpose of design, acceleration spectra are very useful, as they give the seismic force on a structure directly by multiplying it with the generalized or modal mass of the structure.

F-2. AVERAGE SPECTRA

F-2.1 Prof. G. W. Housner has proposed average spectra on the basis of studies on response spectra of four strongest earthquakes that have occurred in USA (*see* Fig. 2 which shows the average acceleration spectra).

F-2.2 To take into account the seismicity of the various zones, the ordinate of the average spectra are to be multiplied by a factor F_0. This factor F_0 depends on the magnitude, duration and form of the expected earthquake, distance of the site from expected epicentre, soil conditions and resistance deformation characteristics of the structure, etc. For elastic design with permissible increase in stresses or load factors as given in **3.3**, approximate values of this factor are given in Table 2.

NOTE — It may be pointed out that during the expected maximum intensity of earthquake in the various seismic zones, structures will be subjected to a bigger force. But the capacity of the structure in plastic range will be available for absorbing the kinetic energy imparted by the earthquake. Therefore, the structural details are to be worked out in such a manner that it can undergo sufficient plastic deformations before failure [*see* **1.2** and **3.3.2** (b) (Note 3)].

F-3. DAMPING IN STRUCTURES

F-3.1 The variety of damping displayed in different types of structures has made the choice of a suitable damping coefficient for a given structure largely a matter of judgement. However, some values are given below to indicate the order of damping coefficient in various types of structures:

a) Steel structures	2 to 5 percent of critical
b) Concrete structures	5 „ 10 „ „ „

c)	Brick structures in cement mortar	5 to 10 percent of critical
d)	Timber structures	2 ,, 5 ,, ,, ,,
e)	Earthen structures	10 ,, 30 ,, ,, ,,

NOTE — It may be mentioned here that in the elastic range, damping displayed by structures is much lower than that given above. It may lie between 1 and 4 percent for the above type of structures at low stresses. The values given thus presume some inelastic deformations or fine cracking to take place when this order of damping will occur. However, for obtaining design seismic coefficient, the values of damping mentioned in relevant clauses shall apply.

F-4. METHOD OF USING THE SPECTRA

F-4.1 Let the period of a structure be 0·8 second and the damping 5 percent critical. Further let the soil-foundation system give factor, $\beta = 1{\cdot}2$ and let the structure have an importance factor, $I = 1{\cdot}5$. Referring to Fig. 2, the spectral acceleration, S_a is 0·12 g. If the structure has mass $M = 12{\cdot}0$ kg sec^2/cm and is to be located in Zone V, the design horizontal seismic coefficient α_h would be [*see* **3.4.2.3** (b)]:

$$\begin{aligned} \alpha_h &= \beta\, I F_o\, (S_a/g) \\ &= 1{\cdot}2 \times 1{\cdot}5 \times 0{\cdot}4 \times 0{\cdot}12 \\ &= 0{\cdot}086\,4 \end{aligned}$$

Therefore, horizontal seismic force

$$\begin{aligned} P &= \alpha_h M g \\ &= 0{\cdot}086\,4 \times 12{\cdot}0 \times 981 \\ &= 1\,017{\cdot}1 \text{ kg} \end{aligned}$$

APPENDIX G

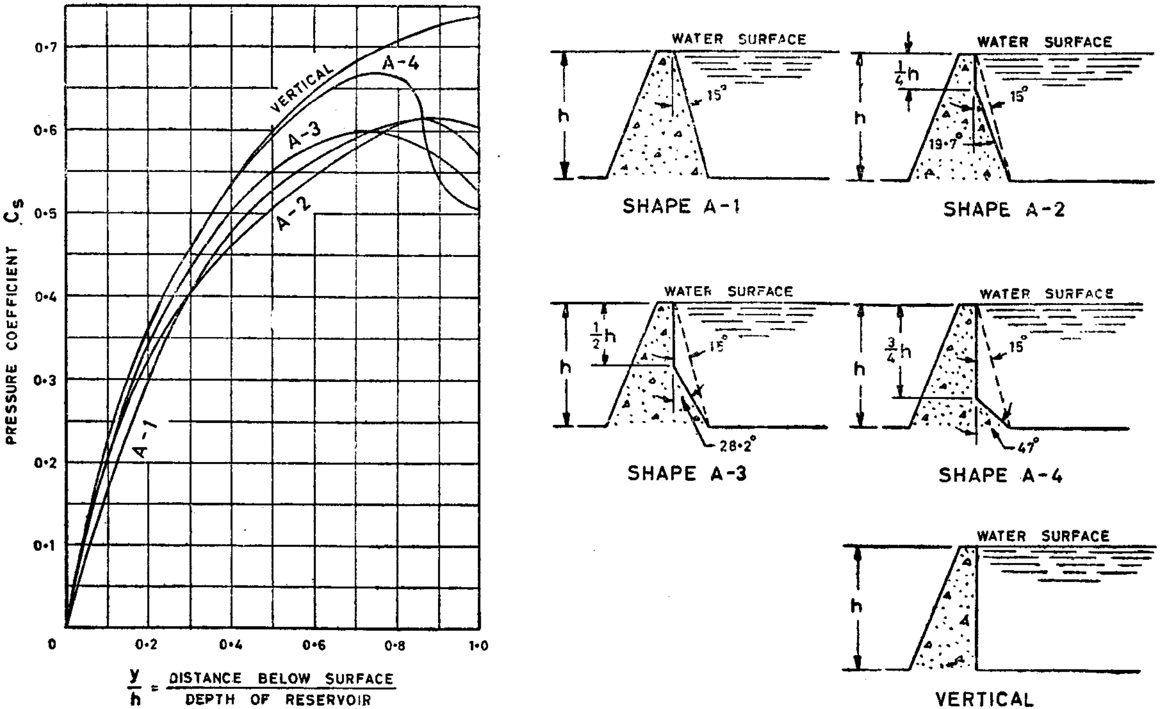
(*Clause* 7.2.1.1)

VARIATION OF THE COEFFICIENT C_s WITH SHAPES AND DEPTHS

G-1. The increase in water pressure on the surface of dam due to horizontal earthquake forces depends upon the shape of the dam and varies with depth. In the equation specified in **7.2.1.1**, the coefficient C_s defines the magnitude and distribution of the increased pressure.

G-2. C_s is a function of the shape of dam and is independent of the magnitude and intensity of the earthquake.

G-3. The magnitude of C_s for various shapes of dams, illustrated in Fig. 14 to 18, assuming water as incompressible, has been established by laboratory experiments. For more detailed analysis, these values may be adopted.

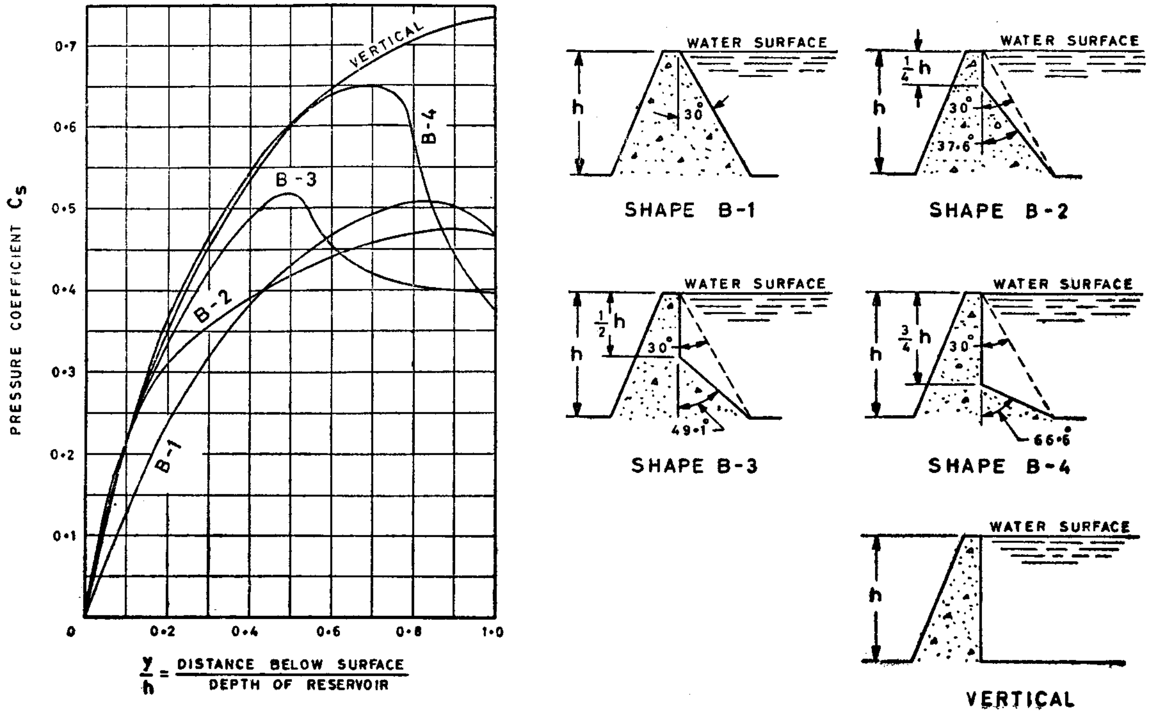


$$p = C_s \, \alpha_h \, wh$$

where

p = hydrodynamic pressure at depth y,
C_s = coefficient which varies with shape and depth,
α_h = horizontal seismic coefficient (*see* **7.1**),
w = unit weight of water, and
h = maximum depth of reservoir.

FIG. 14 VALUES OF C_s FOR COMBINATION SLOPES IN WHICH THE INCLUSIVE ANGLE IS 15° AND VERTICAL PORTION OF UPSTREAM FACE IS VARIABLE

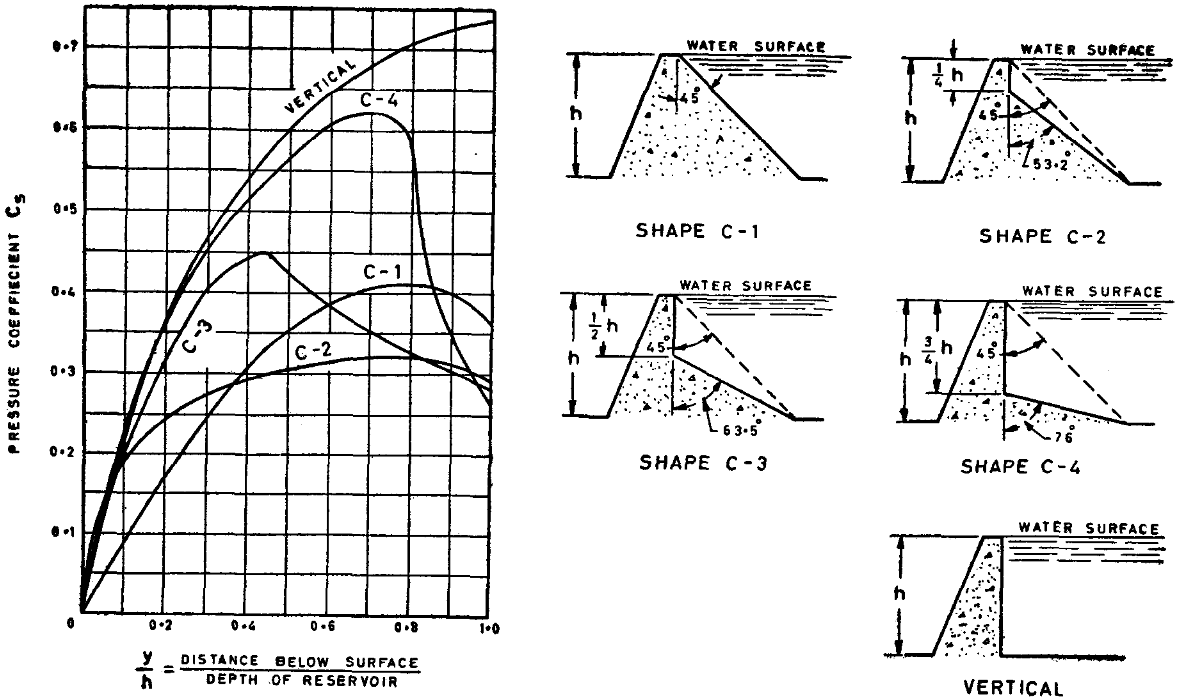


$$p = C_s\, \alpha_h\, wh$$

where

p = hydrodynamic pressure at depth y,
C_s = coefficient which varies with shape and depth,
α_h = horizontal seismic coefficient (*see* **7.1**),
w = unit weight of water, and
h = maximum depth of reservoir.

FIG. 15 VALUES OF C_s FOR COMBINATION SLOPES IN WHICH THE INCLUSIVE ANGLE IS 30° AND VERTICAL PORTION OF UPSTREAM FACE IS VARIABLE

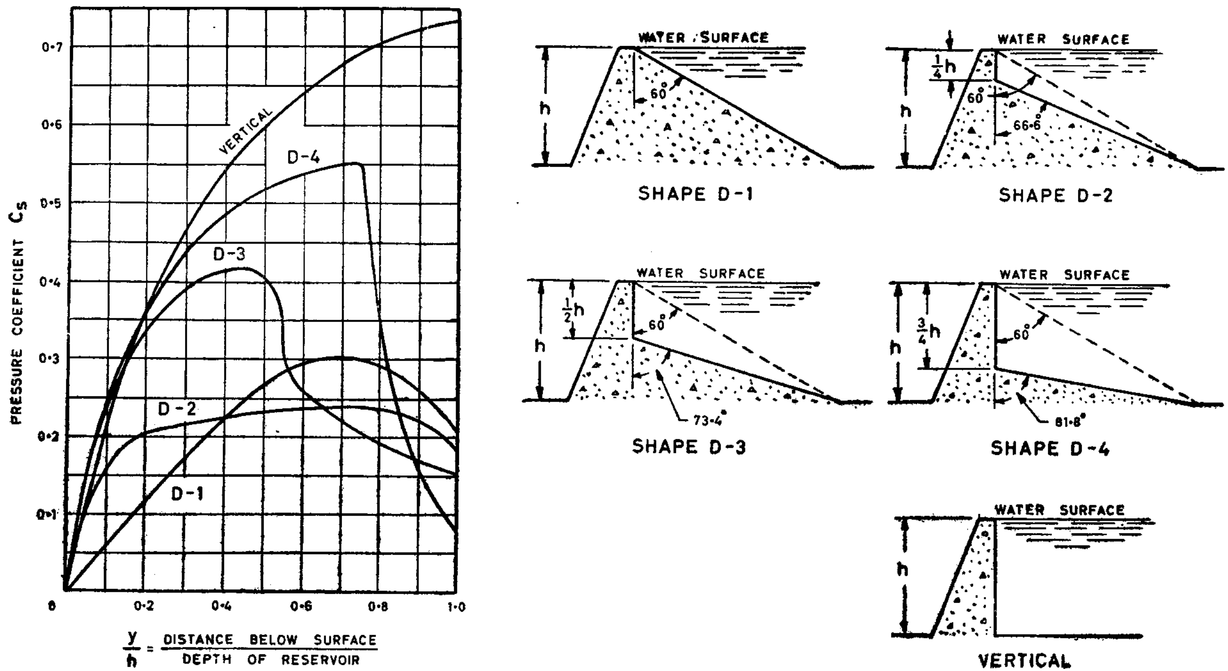


$$p = C_s \alpha_h wh$$

where

p = hydrodynamic pressure at depth y,
C_s = coefficient which varies with shape and depth,
α_h = horizontal seismic coefficient (*see* **7.1**),
w = unit weight of water, and
h = maximum depth of reservoir.

FIG. 16 VALUES OF C_s FOR COMBINATION SLOPES IN WHICH THE INCLUSIVE ANGLE IS 45° AND VERTICAL PORTION OF UPSTREAM FACE IS VARIABLE

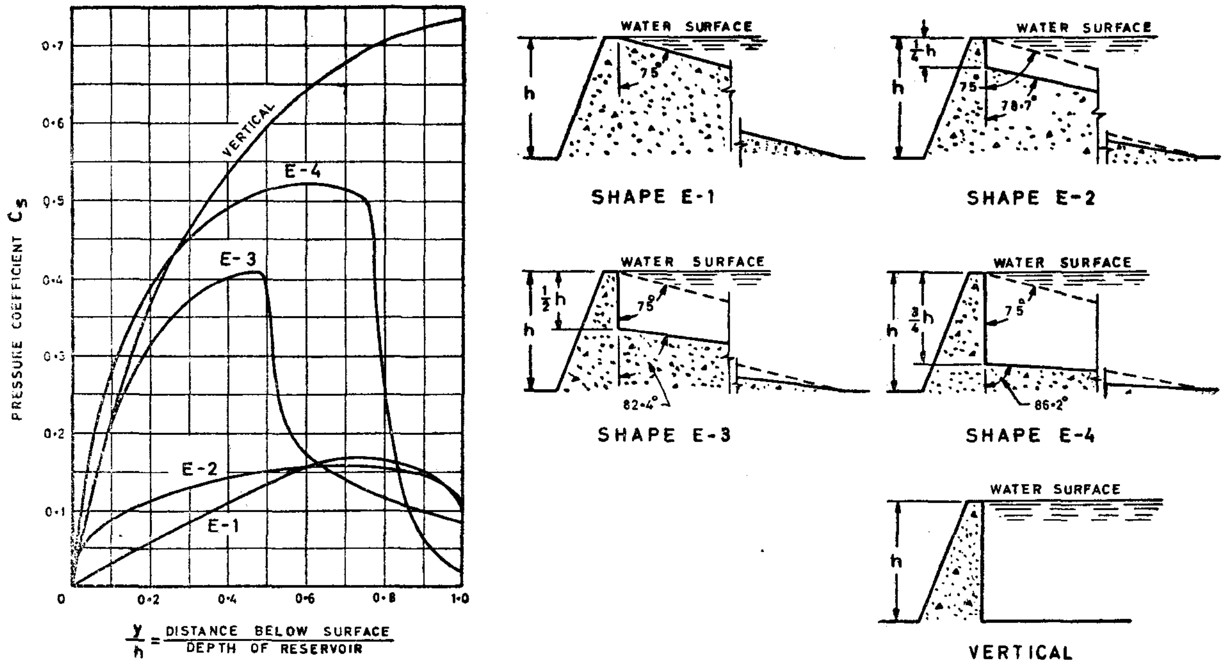


$$p = C_S \, \alpha_h \, wh$$

where

p = hydrodynamic pressure at depth y,
C_S = coefficient which varies with shape and depth,
α_h = horizontal seismic coefficient (*see* **7.1**),
w = unit weight of water, and
h = maximum depth of reservoir.

FIG. 17 VALUES OF C_S FOR COMBINATION SLOPES IN WHICH THE INCLUSIVE ANGLE IS 60° AND VERTICAL PORTION OF UPSTREAM FACE IS VARIABLE



$$p = C_s \, \alpha_h \, wh$$

where

p = hydrodynamic pressure at depth y,
C_s = coefficient which varies with shape and depth,
α_h = basic horizontal seismic coefficient (*see* **7.1**),
w = unit weight of water, and
h = maximum depth of reservoir.

Fig. 18 Values of C_s for Combination Slopes in Which the Inclusive Angle is 75° and Vertical Portion of Upstream Face is Variable

APPENDIX H

(*Clause* 8.1.1.1)

GRAPHICAL DETERMINATION OF ACTIVE EARTH PRESSURE

H-1. METHOD

H-1.1 Make the following construction (*see* Fig. 19):

Draw BB' to make an angle ($\phi - \lambda$) with horizontal. Assume planes of rupture Ba, Bb, etc, such that $Aa = ab = bc$, etc. Make $Ba' = a'b' = b'c'$ etc, on BB' equal to Aa, ab, bc, etc, in length. Draw active pressure vectors from a', b', etc, at an angle ($90° - \delta - \alpha - \lambda$) with BB' to intersect corresponding assumed planes of rupture. Draw the locus of the intersection of assumed planes of rupture and corresponding active pressure vector (modified Culmann's line) and determine the maximum active pressure vector X parallel to BE.

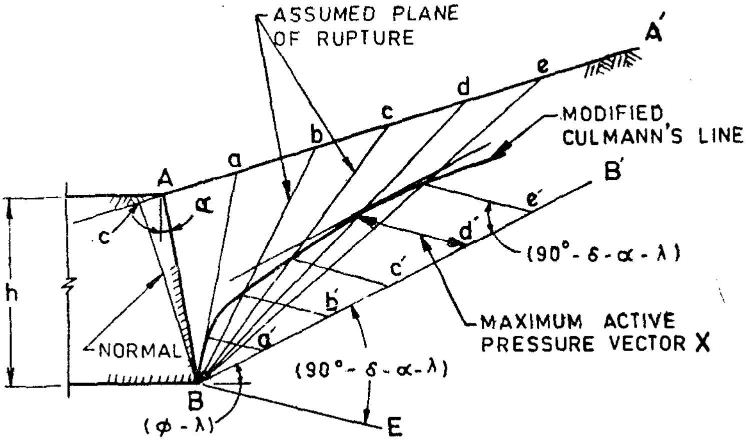


FIG. 19 DETERMINATION OF ACTIVE EARTH PRESSURE BY GRAPHICAL METHOD

H-1.2 The active earth pressure shall be calculated as follows:

$$P_a = \frac{1}{2}\left(\frac{1 \pm \alpha_v}{\cos \gamma}\right) w\, XBC$$

where

X = active pressure vector,

BC = prependicular distance from B to AA' as shown in Fig. 19, and

P_a, w, α_v and λ are as defined in **8.1.1**.

APPENDIX J

(*Clause* 8.1.2.1)

GRAPHICAL DETERMINATION OF PASSIVE EARTH PRESSURE

J-1. METHOD

J-1.1 Make the following construction (*see* Fig. 20):

Draw BB' to make an angle ($\phi - \lambda$) with the horizontal. Assume planes of rupture Ba, Bb, etc, such that $Aa = ab = bc$, etc. Make $Ba' = a'b' = b'c'$, etc, on BB' equal to Aa, ab, bc, etc, in length. Draw passive pressure vectors from a', b', etc, at an angle ($90° - \alpha + \delta + \lambda$) with BB' to intersect corresponding assumed planes of rupture. Draw the locus of the intersection of assumed planes of rupture and corresponding passive pressure vector (modified Culmann's line) and determine the minimum passive pressure vector X parallel to BE.

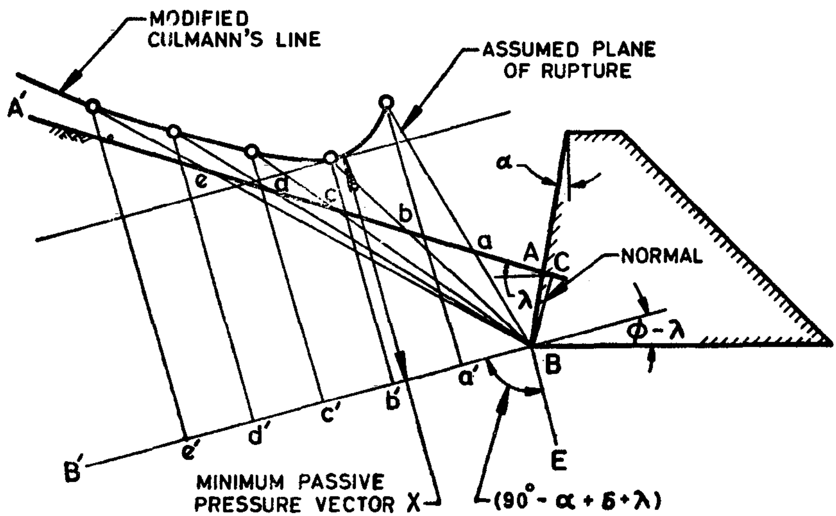


FIG. 20 DETERMINATION OF PASSIVE EARTH PRESSURE BY GRAPHICAL METHOD

J-1.2 The passive pressure shall be calculated as follows:

$$P_p = \tfrac{1}{2}\left(\frac{1 \pm \alpha_v}{\cos \lambda}\right) w\, XBC$$

where

X = passive pressure vector,

BC = perpendicular distance from B to AA' as shown in Fig. 20, and

P_p, w, α_v and λ are as defined in **8.1.2**.

APPENDIX K

(*Clause* 9.1)

NOTATIONS AND SYMBOLS

K-1. The following notations and letter symbols shall have the meaning indicated against each, unless otherwise specified in the body of the standard:

A = Area of cross-section at the base of the structure shell in stacklike structures

B = Base width of the dam

C = Coefficient defining flexibility of structure

C_a = Coefficient for determining active earth pressure (for dry-moist-saturated backfills)

C'_a = Coefficient for determining active earth pressure (for submerged backfills)

C_e = Coefficient depending on submerged portion of pier and enveloping cylinder

C_m = Maximum value of C_s

C'_m = Coefficient to determine bending moment at any section from base moment in dams

C_p = Coefficient for determining passive earth pressure

C_r = Mode participation factor

C_s = Coefficient which varies with shape and depth of dam

C_T = Coefficient depending on slenderness ratio of structure, used for determining T

C_v = Coefficient depending on slenderness ratio, used for determining V

C'_v = Coefficient to determine shear at any section from base shear in dams

d = Dimension of building in a direction parallel to the applied seismic force

DL = Dead load on the structure

EL = Value of earthquake load adopted for design

E_s = Modulus of elasticity of the material of the structure

F = Total horizontal force for submerged portion of pier

F_0 = Seismic zone factor

g = Acceleration due to gravity

G = Modulus of rigidity of the shell material of earth and rock-fill dam

h = Height of water stored in tank, *or*

= Depth of reservoir, *or*

= Height of retaining wall

h' = Height of stacklike structure above the base, *or*

= Height of submergence above base of retaining walls

$\bar{h}$ = Height of centre of gravity of stacklike structure or dam above base

h_1 = Height measured from the base of the building to the roof or any floor, i

H = Total height of the main structure of the building, *or*

= Height of submerged portion of pier, *or*

= Height of water surface from the level of deepest scour, *or*

= Height of dam

H_t = Height of dam above toe of the slopes

I = Importance factor

k = Slenderness ratio of stacklike structure

K = Performance factor for buildings

K_a = Value of C_a for static active earth pressure conditions

K'_a = Value of C'_a for static active earth pressure conditions

l = Half the (longer) length of the rectangular tank

l' = Half the width of strip in circular tank

LL = Superimposed (live) load on the structure

M = Design bending moment at a distance x' from top, in a stacklike structure

M_B = Base moment

M_h = Hydrodynamic moment in submersible bridges

M_y = Bending moment at depth y below top of dam

n = Number of storeys including basement storeys

p = Hydrodynamic pressure in submersible bridges or dams, *or*

= Hydrodynamic pressure at any location, x, from the centre of rectangular tank

p_b = Pressure on the bottom of the tank or bottom of submerged portion of the pier

P_w = Pressure on the wall of the tank

P_a = Active earth pressure due to earthfill

P_p = Passive earth pressure due to earthfill

$(P_a)_q$ = Active earth pressure due to uniform surcharge

$(P_p)_q$ = Passive earth pressure due to uniform surcharge

q = Intensity of uniform surcharge

Q_i = Lateral forces at any roof or floor, i

$Q_i^{(r)}$ = Load acting at any floor level, i, due to mode of vibration

r = Mean radius of structural shell of circular stacklike structures

r_e = Radius of gyration of structural shell at the base section of stacklike structures

R = Radius of circular tank

S_a = Spectral acceleration

t = Thickness of structural shell of circular stacklike structure

T = Fundamental time period of vibration of structure

UL = Ultimate load for which the structure or its element should be designed

V = Design shear force in stacklike structure at distance x' from the top

V_1 = Total shear due to horizontal component of hydrodynamic force at the elevation at which the slope of the dam face commences

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

V_B = Base shear

V_h = Hydrodynamic shear in submersible bridges

V_i = Shear force acting at floor, level, i

$V_i^{(r)}$ = Absolute value of maximum shear at the ith storey, in the rth mode

V_y = Shear force at depth y below top of the dam

w = Unit weight of water, *or*
Unit weight of soil

w_m = Unit weight of material of dam

w_s = Saturated unit weight of soil

W = Total dead load + appropriate amount of live load in buildings, *or*
Total weight of masonry or concrete in the dam

W_e = Weight of the water of the enveloping cylinder

W_h = Increase (or decrease) in vertical component of load due to hydrodynamic force

W_i = Dead load + appropriate amount of live load of the roof or any floor, i

W_m = Weight of bridge mass under consideration ignoring reduction due to buoyancy or uplift

W_t = Total weight of stacklike structure including weight of lining and contents above base

x = Location in a rectangular tank from the centre of the tank

x' = Distance from the top of stacklike structure

y = Depth of location or section below the water surface or top of the dam

α = Angle which earth face of the wall makes with the vertical

α_0 = Basic seismic coefficient

α_h = Design horizontal seismic coefficient

α_v = Vertical seismic coefficient

α_y = Equivalent uniform seismic coefficient at depth y below top of dam

β = Soil-foundation system factor

γ = Constant used to determine shear force at any floor

δ = Angle of friction between the wall and earthfill

Δ = Static horizontal deflection at the top of the tank under a static horizontal force

θ = Angle between the face of the dam and the vertical

ι = Slope of the earthfill

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

ρ = Mass density of the shell material of earth and rockfill dam

ϕ = Angle of internal friction of soil

ϕ' = Angle subtended by centre line of circular tank in plan, with chord width of 2 l'

$\phi_i^{(r)}$ = Mode shape coefficient obtained from free vibration analysis at floor, i

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TO

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(Fourth Revision)

(Page 7, Fig. 1, footnotes) - Add the following new sentence in the end:

'Lakshadweep falls under seismic zone III.'

(BDC 39)

Reprography Unit, BIS, New Delhi, India