(Reaffirmed 0)

भारतीय मानक

Indian Standard

IS 1893 (Part 3) : 2014

संरचनाओं के भूकम्परोधी डिजाइन के मानदंड

भाग 3 पुल और रोकथाम वाली दीवारें

Criteria for Earthquake Resistant Design of Structures

Part 3 Bridges and Retaining Walls

ICS 91.120.25

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Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard (Part 3) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

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 occurring at much larger time intervals at any site, 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 intense construction activity all over the country. It is to serve this purpose that IS 1893 : 1962 'Recommendations for earthquake-resistant design of structures' was first published in 1962 and subsequently revised in 1966, 1975 and 1984.

Further, with a view to keep abreast with the rapid development and extensive research that has been carried out in the field of earthquake-resistant design of various structures, the Committee has decided to cover the provisions for different types of structures in separate parts. Hence IS 1893 has been split into the five parts. The other parts in the series are:

Part 1 General provisions and buildings

Part 2 Liquid retaining tanks - elevated and ground supported

Part 4 Industrial structures including stack like structures

Part 5 Dams and embankments

This standard (Part 3) contains provisions for the design of new bridges and for seismic evaluation of existing bridges in the process of their seismic upgradation and retrofitting. Unless otherwise stated, this standard shall be read necessarily in conjunction with IS 1893 (Part 1), which contains provisions that are general in nature and applicable to all types of structures.

For the purpose of determining design seismic forces, the country is classified into four seismic zones as per Fig. 1 of IS 1893 (Part 1).

This standard has been formulated to ensure that bridges possess at least a minimum strength to withstand earthquakes. The intention is not to prevent damage to them due to the most severe shaking that they may be subjected to during their lifetime. Actual forces that appear on different portions of bridge during earthquakes may be greater than the design seismic forces specified in this standard. However, ductility arising from material behaviour, detailing and over strength arising from the additional reserve strength in them over and above the design force are relied upon to account for this difference in actual and design lateral loads.

The reinforced and pre-stressed concrete components shall be designed to be under reinforced so as to cause a tensile failure. Further, they should be suitably designed to ensure that premature failure due to shear or bond does not occur. Ductility demand under seismic shaking is usually not a major concern in bridge superstructures. However, the seismic response of bridges is critically dependent on the ductile characteristics of the sub-structures, foundations and connections. Provisions for appropriate ductile detailing of reinforced concrete members applicable to sub-structures and foundations and connections for bridges are given in Annex B.

Some of the major and important modifications made in this revision as compared to IS 1893 : 1984 are as follows:

a) Seismic zone factors are the same as included in IS 1893 (Part 1) : 2002. Four methods, namely seismic coefficient method, response spectrum method, time history method and non-linear push over analysis are given for estimating design forces which recognizes the flexibility of bridges.

(*Continued on third cover*)

Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 3 BRIDGES AND RETAINING WALLS

1 SCOPE

1.1 The standard (Part 3) deals with the assessment of earthquake forces and design of new bridges on highways, railways, flyover bridges, pedestrian bridges, submersible bridges, utility bridges and aqueducts. The earthquake effect on retaining walls and bridge abutments are also covered. The hydrodynamic effect of water on submerged sub-structure and method of assessment of liquefaction potential of soil is also included. The methodology of estimation of seismic forces given in this standard may be employed for seismic evaluation of the existing bridges and retrofitting of such structures.

1.2 This standard deals with the earthquake resistant design of regular bridges in which the seismic actions are mainly resisted at abutments or through flexure of piers, that is, bridges composed of vertical pier-foundation system supporting the deck structure with/without bearings. However, for all special and major bridges, detailed dynamic studies shall be undertaken.

1.3 This standard does not deal with the construction features relating to earthquake resistant design of bridges.

2 REFERENCES

The standards listed at Annex A contain provisions which through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated at Annex A.

3 TERMINOLOGY

The definitions given in **3** of IS 1893 (Part 1) and the following shall apply.

3.1 Active Tectonic Fault — A seismotectonic fault is considered to be active when there is an average slip rate of at least 1 mm/year and topographic evidence of seismic activity with in Holocene times (Past 11 000 years).

3.2 Asynchronous Motion — The spatial variability of the seismic action means that the motion at different supports of the bridge is assumed to be different and as a result, the definition of the seismic action shall not be based on the characterization of motion at a single point, as is usually the case.

3.3 Base — It shall be the base of pier or top of well in case of well foundation, base of pier or top of pile cap in case of pile foundation and base of pier in case of open foundation.

3.4 Capacity Design — The design procedure used in structures of ductile behaviour to secure the hierarchy of strengths of the various structural components necessary for leading to the intended configuration of plastic hinges and for avoiding brittle failure modes.

3.5 Dynamic Analysis Method — A seismic analysis method in which the dynamic behaviour of a structure during an earthquake is obtained considering dynamic characteristics of the structure and characteristics of the ground motion by solving the equations of motion of the structure.

3.6 Design Seismic Displacement — The displacement induced by design seismic actions.

3.7 Effects of Earthquake — The effects of earthquake motion that shall be considered in seismic design of bridge include inertial force, displacements, earth pressure, hydrodynamic pressure and liquefaction of soil.

3.8 Isolation Bearing — A bearing support used for a bridge with seismic isolation device having a function to appropriately increase the natural period of the bridge with the controlled damping results in decrease of forces in the structure and displacements in the bearing for better overall performance.

3.9 Special Regular Bridge — The bridges specified under regular bridges but single span more than 120 m or pier height measured from founding level to the top of pier cap to be more than 30 m. In case of pile foundation pier height shall be considered from the point of fixity.

3.10 Modal Analysis — A dynamic analysis method in which response is calculated by combination of response in various modes of vibration.

3.11 Retrofitting — It is upgrading the strength of existing structure in order to increase its capacity to withstand effect of future earthquakes by addition of structural elements, dampers or similar devices. The retrofitting may be required for, (a) seismically deficient structure; (b) earthquake damaged structure; and (c) due to modifications made to increase live load capacity of structure.

3.12 Regular Bridge — A regular bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). A bridge shall be considered regular for the purpose of this standard, if

a) it is straight or it describes a sector of an arch which subtends an angle less than 90º at the center of the arch;
b) the adjacent piers do not differ in stiffness by more than 25 percent (Percentage difference shall be calculated based on the lesser of the two stiffness); and
c) girder bridges, T-beam bridges, truss bridges, hammer head bridges, bridges having single or multiple simply supported spans with each span less than 120 m and pier height above foundation level less than 30 m.

3.13 Seismic Coefficient Method — A seismic analysis method in which seismic force equal to the weight of the structure/component multiplied by design acceleration coefficient is applied statically at the centre of mass of the structure/component.

3.14 Seating Width — The distance between the end of the girder to the top edge of a sub-structure to prevent the girder from being dislocated in the event of an unexpectedly large relative displacement between super and sub-structure.

3.15 Seismic Links — Restrainers through which part or all of the seismic action may be transmitted. Used in combination with bearings and they are usually provided with appropriate slack so as to be activated only in case when the design seismic displacements is exceeded.

3.16 Special and Irregular Types of Bridges — The bridges with innovative designs and bridges such as suspension bridge, cable stayed bridge, arch bridge, bascule bridge and irregular bridges such as skew bridge of angle 30º and above with span more than 60 m shall be categorized under these types.

3.17 Unseating Prevention System — A structure installed to prevent a superstructure from unseating due to an earthquake. It may comprise of an adequate seat length, devices to prevent excessive displacement, jumping and preventing structure from dislodging from supports. It may be in various forms such as; stopper, cable restrainer, bolts, clamps, etc.

4 GENERAL PRINCIPLES AND DESIGN CRITERIA

4.1 General Principles

4.1.1 All components of the bridge, that is, superstructure, sub-structure, bearing, foundation and soil are susceptible to damage in the event of strong ground shaking. The earthquake resistant design shall consider the effect of earthquake motions on each component of the bridge following the provisions of this standard.

4.1.2 The design shall ensure that seismic resistance of the bridge and its components are adequate to meet the specified design requirement so that emergency communication after an earthquake shall be maintained for the design basis earthquake.

4.1.3 Masonry and plain concrete arch bridges with spans more than 10 m shall not be built in the seismic Zones IV and V.

4.1.4 Box, pipe and slab culverts need not be designed for earthquake forces. Bridges of total length not more than 60 m and individual span not more than 15 m need not be designed for earthquake forces other than in Zones IV and V.

4.1.5 Seismic forces on aqueduct structures and flyover bridges shall be calculated as for any other bridge. The effect of inertia force of flowing water mass in aqueduct shall be calculated on the basis of assumptions in **6.5**.

4.1.6 Hydrodynamic pressure on walls of water trough in case of aqueduct shall be considered on the basis of provision of IS 1893 (Part 2).

4.1.7 The liquefaction potential of foundation soil shall be investigated where necessary shall be according to **21**.

4.1.8 When relative movement between two adjacent units of a bridge are designed to occur at a separation/expansion joint, sufficient clearance shall be provided between them, to permit the relative movement under design earthquake conditions to freely occur without inducing damage. Where the two units may be out of phase, the clearance to be provided may be estimated as the square root of the sum of squares of the calculated displacements of the two units under maximum elastic seismic forces.

4.1.9 Special design studies shall be called for the following cases:

a) Consideration of asynchronous ground

motion when, (1) geological discontinuities or marked topographical features are present; and (2) single span is greater than 600 m, even if there are no geological discontinuities.

b) In case of bridges over potentially active tectonic faults, the probable discontinuity of the ground displacement shall be estimated and accommodated either by adequate flexibility of the structure or by provision of suitable movement of joints.

c) Bridge located in near field, that is, within 10 km near fault area of known active tectonic fault.

4.2 Design Criteria

4.2.1 *Site Specific Spectrum*

For special bridges as defined in **3.9** and **3.16** in seismic Zones IV and V where soil conditions are poor consisting of marine clay or loose fine sand and silt (for example where the soil up to 30 m depth has SPT (N values - uncorrected) equal to or less than 20 and for bridges located near a known fault (near - field) or the area is known for complex seismotectonic geological setting, detailed investigations shall be carried out to obtain the site specific spectrum. Site specific spectrum is also required for bridges with spans greater than 150 m. Such a spectrum shall be used for design in place of code spectrum subject to minimum requirements specified in this standard.

4.2.2 *Seismic Safety of Bridge in Longitudinal and Transverse Directions*

The design of the bridge shall be made for the effect of earthquake motions occurring in the traffic direction (longitudinal direction), across traffic direction (transverse direction) and vertical direction. The simultaneous action of the motions shall be considered, where necessary according to provisions of this standard.

4.2.3 *Elastomeric Bearing*

Elastomeric bearings shall generally be used to transmit vertical loads, rotations and horizontal forces other than those due to seismic. In case, in-plane horizontal seismic forces are to be transmitted using these bearings, they shall be checked using minimum dynamic frictional value and minimum vertical load, including combined effects of horizontal and vertical components of earthquake. The bearings shall be suitably anchored in the sub-structure and superstructure. Suitable devices for preventing dislodgement of superstructure need to be incorporated. In such cases, for design of foundation, value of R is to be taken as 1. Bearings should be tested for cyclic loadings for which specialist literature should be consulted.

4.2.4 *Effect of Soil-Structure-Interaction*

This standard specifies design of bridges founded on rock and medium soil, which do not liquefy or slide during the ground shaking. For bridges founded on soft soils and in cases where deep foundations are used, detailed studies of soil structure interaction are required. The soil structure interaction may not be considered for open foundations on rocky strata.

Soil flexibilities included in modelling sub-structure and foundation of the bridge for soil structure interaction, generally lead to longer natural period and hence lower seismic forces. However on the other hand, consideration of soil flexibilities shall result in larger lateral deflections. Soil parameters, like, elastic properties and spring constants shall be properly estimated. In many cases one obtains a range of values of soil properties. In such cases, the highest values of soil stiffness shall be used for calculating natural period and lowest value shall be used for calculating deflection.

4.2.5 *Design for Strength and Ductility*

The earthquake resistant design of bridge shall be based on both strength and ductility. Reinforced and pre-stressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur, subject to the provisions of IS 456 and IS 1343 and as per relevant codes of Indian Roads Congress or Indian Railways Code.

4.2.6 *Inter Linking of Spans*

The interlinking of spans to prevent it from being dislodged off its bearings is desirable alternatively continuous construction should be encouraged. The greater redundancy and energy dissipation capacity in the structure are desirable features for better performance in earthquake.

4.2.7 *Capacity Design*

The design seismic force in this standard for bridges is lower than the maximum expected seismic force on them. However, to ensure good performance at low cost, the difference in the design seismic force and the maximum expected seismic force shall be accounted for through additional safety provisions. The capacity design provisions shall be applicable to important bridges in seismic Zone III and to all bridges in seismic Zones IV and V. These provisions are meant for bridges having reinforced concrete sub-structures; however, if steel sub-structures are used in high seismic zones, reference should be made to specialist literature. Annex B describes the detailing procedure for Reinforced Concrete Structures.

4.2.8 *Earthquake Damaged Bridges*

For seismic retrofitting of earthquake damaged bridges, seismic evaluation should be carried out following the methods of seismic analysis recommended in this standard taking into consideration the reduced stiffness and capacity due to cracking and damage.

4.2.9 *Retrofitting of Existing Bridges*

Seismically deficient bridges should be evaluated using the provisions of this standard to determine the need of retrofitting. The prioritization of bridges for retrofitting should be based upon seismicity, age, deterioration and importance of the bridge. The retrofitting of bridges shall consist of upgrading the strength to meet the requirement of this standard.

5 DESIGN PHILOSOPHY

5.1 Serviceability Limit State

The design of bridge should meet the serviceability limit state under design basis earthquake (DBE). The parts of the bridge intended to contribute to energy dissipation shall undergo minor damage without giving rise to need for reduction of traffic or immediate repair. Specialist literature to be consulted for limit state analysis using MCE (involving non-linear analysis and time history method).

5.2 Ultimate Limit State

The design of bridge should meet non-collapse requirement that is, ultimate limit state under maximum considered earthquake (MCE). While designing as per IS 456, DBE may be considered. The bridge shall retain its structural integrity and adequate residual resistance, although considerable damage may occur in some portions of the bridge. The structure should be able to sustain emergency traffic, inspections and repair could be performed easily after the earthquake.

The bridge superstructure however shall in general be protected from the formation of plastic hinges and from unseating due to extreme seismic displacements under MCE.

5.3 Ductile Behaviour

The reinforced and pre-stressed concrete components shall be designed as under-reinforced so as to cause a tensile failure. Further, they should be suitably designed to ensure that premature failure due to shear or bond does not occur. Stresses induced in the superstructure due to earthquake ground motion are usually quite nominal. Therefore, ductility demand under seismic shaking has not been a major concern in the bridge superstructures during past earthquakes. However, the seismic response of bridges is critically dependent on the ductile characteristics of the sub-structures. Provisions for appropriate ductile detailing of reinforced concrete members given in Annex B shall be applicable to sub-structures. Bridges shall be designed such that under severe seismic shaking plastic hinges form in the sub-structure, rather than in the deck or foundation.

NOTE – Specialist literature to be consulted for asynchronous ground motion.

6 ASSUMPTIONS

The following assumptions shall be made in the earthquake analysis of bridges:

a) The seismic forces due to design basis earthquake (DBE) should not be combined with design wind forces.

b) The scour to be considered for design shall be based on mean design flood. In the absence of detailed data, the scour to be considered for design shall be 0.9 times the maximum design scour depth (*see* Note).

   NOTE — The designer is cautioned that the maximum seismic scour case may not always govern in design condition.

c) The earthquake accelerations should be applied to full mass in case of submerged structures and not on buoyant mass.

d) The seismic force on live load in bridges should not be considered in longitudinal direction. The seismic force on live load should be considered in transverse direction as explained in the **7.1**.

e) The seismic force on flowing mass of water in the longitudinal direction in case of aqueducts should not be considered, however seismic force on this water mass be considered in transverse direction. The hydrodynamic action of water on the walls of water carrying trough shall be considered according to the provisions of code on liquid retaining structures.

f) The earthquake accelerations on embedded portion of bridge foundations should be reduced as explained in **9.3**.

g) The value of static elastic modulus of material, where required, may be taken for dynamic analysis unless a more definite value is available for use in seismic condition.

7 LOAD COMBINATIONS

When earthquake forces are combined with other forces such as dead load and live load, the load factor for plastic design of steel structures and partial safety factors for limit state design of reinforced concrete structures and pre-stressed concrete structures shall be considered. Load factors may be used as in IRC/IRS codes with the provision that when earthquake load

(EL) and dead load (DL) are combined, load factor shall be minimum 1.5; and when seismic load is combined with all other loads, load factor shall be minimum 1.2.

7.1 Seismic Force on Live Load

The seismic force due to live load shall not be considered when acting in the direction of traffic, but shall be considered in the direction perpendicular to traffic.

The live load on the bridges for highways shall be 20 percent of design live load, 0 percent for rural roads and 30 percent of design live load for railway bridges without impact.

NOTE — The bridge owner authorities can modify these percentages on the basis of location of bridge and intensity of traffic.

7.2 Seismic Load Combinations

7.2.1 The seismic forces shall be assumed to come from any horizontal direction. For this purpose, two separate analysis shall be performed for design seismic forces acting along two orthogonal horizontal directions. The design seismic force resultant (that is axial force, bending moments, shear forces, and torsion) at any cross-section of a bridge component resulting from the analyses in the two orthogonal horizontal directions shall be combined according to the expressions below:

a) $\pm EL_x \pm 0.3 EL_y$

b) $\pm 0.3 EL_x \pm EL_y$

where

EL_x = force resultant due to full design seismic force along x direction, and

EL_y = force resultant due to full design seismic force along y direction.

When vertical seismic forces are also considered, the design seismic force resultants at any cross-section of a bridge component shall be combined as below:

1) $\pm EL_x \pm 0.3 EL_y \pm 0.3 EL_z$

2) $\pm 0.3 EL_x \pm EL_y \pm 0.3 EL_z$

3) $\pm 0.3 EL_x \pm 0.3 EL_y \pm EL_z$

where EL_x and EL_y are as defined above and EL_z is the force resultant due to full design seismic force along the vertical direction.

As an alternative to the procedure given above, the forces due to the combined effect of two or three components can be obtained on the basis of square root of sum of square (SRSS), that is $\sqrt{EL_x^{\,2} + EL_y^{\,2}}$ or $\sqrt{EL_x^{\,2} + EL_y^{\,2} + EL_z^{\,2}}$.

7.3 Vertical Component of Seismic Action

7.3.1 The effect of the vertical seismic component on sub-structure and foundation may, as a rule, be omitted in Zones II and III. The vertical accelerations should be specially considered in bridges with large spans, those in which stability is the criteria of design and in situations where bridges are located in near field. However the effect of vertical seismic component is particularly important in the following components/ situations and needs to be investigated:

a) Pre-stressed concrete decks;

b) Bearings and linkages; and

c) Horizontal cantilever structural elements.

7.3.2 The seismic zone factor for vertical ground motions, when required may be taken as two-thirds of that for horizontal motions given in Table 2 of IS 1893 (Part 1). However, the time period for the superstructure has to be worked out separately using the characteristic of the superstructure for vertical motion, in order to estimate $\frac{S_a}{g}$ for vertical acceleration. The natural time period of superstructure can be estimated using appropriate modelling and free vibration analysis using computer. However, for simply supported superstructure with uniform flexural rigidity, the fundamental time period T_v, for vertical motion can be estimated using the expression:

$$T_v = \frac{2}{\pi} l^2 \sqrt{\frac{m}{EI}}$$

where

l = span;

m = mass per unit length; and

EI = flexural rigidity of the superstructure.

8 CALCULATION OF NATURAL PERIOD OF BRIDGE

8.1 Simply Supported Bridges

Where the vibration unit of sub-structure can be idealized as a single cantilever pier carrying the superstructure mass, resting on well, pile or open foundation, the fundamental period shall be calculated from the following equation:

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

where

δ = horizontal displacement at the top of pier due to horizontal force (= mg) where

m = lumped mass at the top of pier.

In general pier shall be considered fixed at the foundation level. However, in case of soft soil or deep foundations, soil flexibility may be considered in the calculation of natural period as per **5.2.4**.

8.2 Other Types of Bridges

Where idealization by a single cantilever pier model is not possible, the natural periods of vibration may be calculated by free vibration analysis of an appropriate mathematical model of bridge superstructure, bearing, sub-structure, foundation and soil.

9 METHOD OF CALCULATING SEISMIC FORCES

9.1 The following methods of seismic analysis may be employed for calculation of seismic forces in bridges:

a) Seismic coefficient method (SCM);
b) Response spectrum method (RSM);
c) Time history method (THM); and
d) Nonlinear pushover analysis (NPA).

The recommended method of analysis for different category of bridges and earthquake level is given in Table 1. The linear analysis considering elastic behaviour is required for DBE.

9.2 Seismic Coefficient Method

The seismic force to be resisted by bridge components shall be computed as follows:

$$F = A_h W$$

where

F = horizontal seismic force to be resisted;

W = weight of mass under consideration ignoring reduction due to buoyancy or uplift; and

A_h = design horizontal seismic coefficient as determined from **9.4.1**.

9.3 For embedded portion of foundation at depths exceeding 30 m below scour level, the seismic force due to foundation mass may be computed using design seismic coefficient equal to $0.5A_h$.

For portion of foundation between the scour level and up to 30 m depth, the seismic force due to that portion of foundation mass may be computed using seismic coefficient obtained by linearly interpolating between A_h at scour level and 0.5 A_h at a depth 30 m below scour level.

Table 1 Method of Seismic Analysis of Bridges
(*Clause* 9.1)

Earthquake Level	Category of Bridge Type		
	Regular	Special Regular	Special Irregular
(1)	(2)	(3)	(4)
DBE	SCM	RSM THM	RSM THM NPA

NOTE – In case of MCE, non-linear analysis and Time History Method shall be adopted for regular, special regular and special irregular bridges.

9.4 Response Spectrum Method (RSM)

The following steps are required in RSM:

a) Formulation of an appropriate mathematical model consisting of lumped mass system using 2D/3D beam elements. The mathematical model should suitably represent dynamic characteristics of superstructure, bearings, sub-structure, foundation and soil/ rock springs. In rock and very stiff soil fixed base may be assumed.
b) Determination of natural frequency and mode shapes following a standard transfer matrix, stiffness matrix, finite element method or any other standard approach.
c) Determine total response by combining responses in various modes by: (1) by mode combination procedure such as SRSS, CQC, etc, or (2) time-wise superposition of responses using ground motion time history(s). In **9.1** (a) and **9.1** (b), A_h shall be computed as explained below.

9.4.1 *Horizontal Seismic Coefficient, A_h*

The design horizontal seismic coefficient, A_h shall be determined from following expression of **6.4.2** of IS1893 (Part 1).

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

Provided that for any structure with $T < 0.1$ *s*, the value of A_h shall not be taken less than $Z/2$, whatever be the value of I/R.

where

Z = zone factor;

I = importance factor (*see* Table 2);

R = response reduction factor (*see* Table 3); and

$\frac{S_a}{g}$ = average acceleration coefficient for rock or soil sites as given in Fig. 1.

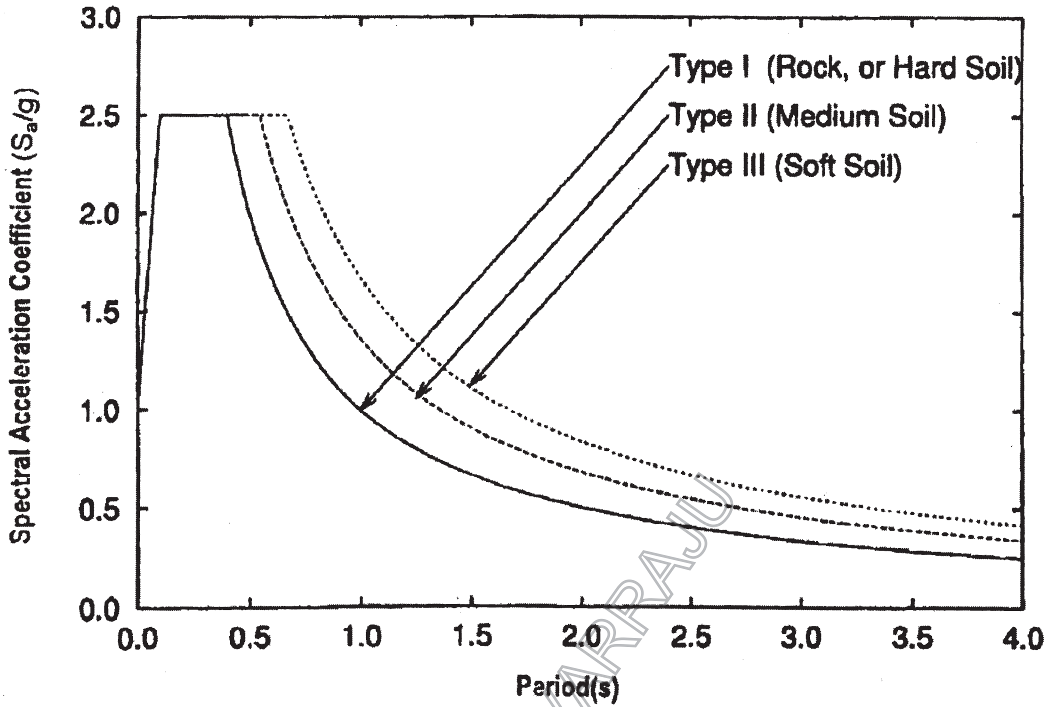


FIG.1 RESPONSE SPECTRA FOR ROCK AND SOIL SITES FOR 5 PERCENT DAMPING

9.4.2 *Design Vertical Seismic Coefficient, A_v*

The design vertical seismic coefficient may be adopted as in **7.3.2**.

9.4.3 *Design Seismic Coefficient for Different Soils and Damping*

Mathematical expressions of **5.4.2** of IS 1893 (Part 1) can be used to compute S_a/g for different soil/rock types. Table 3 of IS 1893 (Part1) can be used for damping other than 5 percent.

9.4.4 *Importance Factor, I*

Bridges are designed to resist design basis earthquake (DBE) level, or other higher or lower magnitude of forces, depending on the consequences of their partial or complete non-availability, due to damage or failure from seismic event. The level of design force is obtained by multiplying (*Z*/2) by factor '*I*', which represents seismic importance of the structure. Combination of factors considered in assessing the consequences of failure, and hence choice of factor '*I*', include *inter alia*.

Importance factor depends upon the following:

a) Extent of disturbance to traffic and possibility of providing temporary diversion;
b) Availability of alternative routes;
c) Cost of repairs and time involved, which depend on the extent of damages, minor or major;
d) Cost of replacement, and time involved in reconstruction in case of failure; and
e) Indirect economic loss due to its partial or full non-availability.

Importance factors are given in Table 2 for different types of bridges.

9.4.5 *Response Reduction Factor*, *R*

The response reduction factor for different components is given in Table 3.

9.5 TIME HISTORY METHOD (THM)

The dynamic analysis of a bridge by time history method may be carried out using direct step-by-step method of integration of equations of motion. At least three spectrum compatible time histories shall be used, when site-specific time histories are not available. The spectrum used to generate these time histories shall be the same as used for the modal analysis. Their duration shall be consistent with their magnitude and source characteristics of design basis earthquake. The total duration of time history shall be about 30s of which the strong motion part shall be not less than 6s. This analysis can be carried out using a standard software package.

9.6 Non-linear Pushover Analysis (NPA)

It is a static non-linear analysis carried out to determine lateral load *versus* displacement at control point in the structure for the purpose of determining capacity of

Table 2 Importance Factor
(*Clause* 9.4.4)

Sl No. (1)	Seismic Class (2)	Illustrative Examples of Bridges (3)	Importance Factor 'I' (4)
i)	Normal bridges	All bridges except those mentioned in other classes	1
ii)	Important bridges	a) River bridges and flyovers inside cities	1.2
		b) Bridges on national and state highways	1.2
		c) Bridges serving traffic near ports and other centres of economic activities	1.2
		d) Bridges crossing railway lines	1.2
iii)	Large critical bridges in all Seismic Zones	a) Long bridges more than 1 km length across perennial rivers and creeks	1.5
		b) Bridges for which alternative routes are not available	1.5
iv)	Railway bridges	a) All important bridges irrespective of route.	1.5
		b) Major bridges on group A, B and C routes (Route classification as per IRP way manual)	1.5
		c) Major bridges on all other routes.	1.25
		d) All other bridges on group A, B and C routes	1.25
		e) All other bridges	1.0

NOTE — While checking for seismic effects during construction, the importance factor of 1 shall be considered for all bridges in all zones.

Table 3 Response Reduction Factor R for Bridge Components
(*Clause* 9.4.5)

Sl No. (1)	Structure, Component or Connection (2)	R (3)
i)	Superstructure	1.0
ii)	Sub-structure:	
	a) Reinforced concrete piers with ductile detailing cantilever type, wall type	3.0
	b) Reinforced concrete piers without ductile detailing*, cantilever type, wall type	2.5
	c) Masonry piers (un reinforced) cantilever type, wall type	1.5
	d) Reinforced concrete, framed construction in piers, with ductile detailing, columns of RCC bents, RCC single column piers	4.0
	e) Steel framed construction	2.5
	f) Steel cantilever piers	1.0
	g) Steel trussed arch	1.5
	h) Reinforced concrete arch	3.5
	k) Abutments of mass concrete and masonry	1.0
	m) R.C.C. abutment	2.5
	n) Integral frame with ductile detailing, and	4.0
	p) Integral frame without ductile detailing	3.3
iii)	Bearings (Elastomeric, pot, knuckle, roller-rocker)	0.8
iv)	Expansion joints and connections within a span of structure, hinge	1.0
v)	Stoppers in bearings	1.0
vi)	Foundations (well, piles or open).	1.0

NOTE — Response reduction factor, *R* should be taken as 1.0 for calculating displacements.

the structure. The analysis can be performed using a standard software package. The method can be employed for design of special bridges and to determine capacity of existing structures for the purpose of retrofitting.

10 HYDRODYNAMIC FORCE ON SUB-STRUCTURE

10.1 The hydrodynamic force on submerged portion of pier and foundation up to mean scour level shall be assumed to act in a horizontal direction corresponding to that of earthquake motion. The total horizontal force is given by the following formula:

$$F = C_e A_h W_e$$

where

C_e = coefficient (*see* Table 4);
A_h = design horizontal seismic coefficient;
W_e = weight of the water in the enveloping cylinder,
= $\rho_w \pi a^2 H$, see **10.3**;
ρ_w = unit weight of water;
H = height of submerged portion of pier; and
a = radius of enveloping cylinder.

10.2 Hydrodynamic Pressure Distribution

The hydrodynamic pressure distribution on submerged portion of bridge pier is given in Fig. 2. The coefficients C_1, C_2, C_3 and C_4 are given in Table 5. The pressure distribution, Fig. 2, along the height of pier is drawn by assuming the value of C_1 from 0.1 to 1.0 in Table 5;

Table 4 Values of C_e
(*Clause* 10.1)

Sl No. (1)	*H/a* (2)	C_e (3)
i)	1.0	0.390
ii)	2.0	0.575
iii)	3.0	0.675
iv)	4.0	0.730

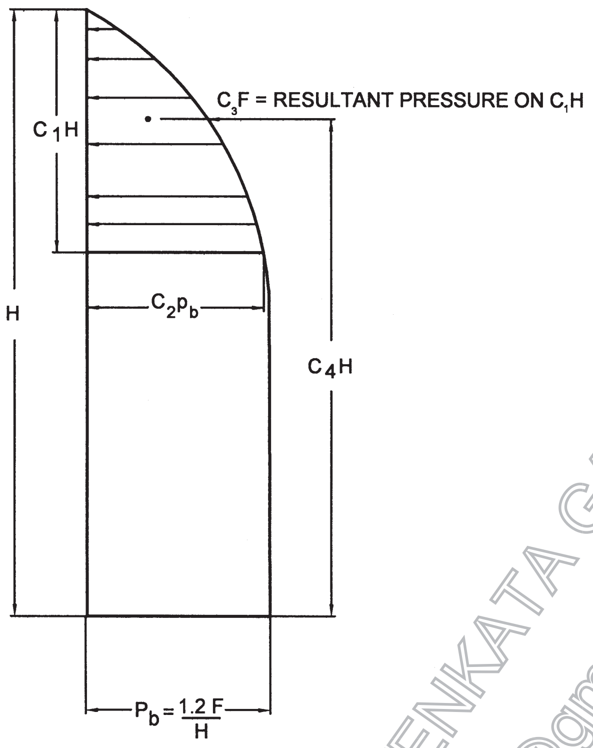


FIG. 2 DIAGRAM SHOWING HYDRODYNAMIC PRESSURE DISTRIBUTION

this implies selecting a point on the vertical axis with origin at top, then other coefficients are read horizontally from the table to generate the pressure curve and determine other coefficients mentioned on the curve.

Table 5 Coefficients C_1, C_2, C_3 and C_4
(*Clause* 10.1)

Sl No. (1)	C_1 (2)	C_2 (3)	C_3 (4)	C_4 (5)
i)	0.1	0.410	0.026	0.934
ii)	0.2	0.673	0.093	0.871
iii)	0.3	0.832	0.184	0.810
iv)	0.4	0.922	0.289	0.751
v)	0.5	0.970	0.403	0.694
vi)	0.6	0.990	0.521	0.639
vii)	0.8	0.999	0.760	0.532
viii)	1.0	1.000	1.000	0.428

10.3 Typical cases of submerged portions of piers and the enveloping cylinders are shown in Fig. 3.

10.4 The earth pressure on the back of abutments, wing walls and return walls of bridge shall be calculated as given in **22** (*see also* 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.

11 SUPERSTRUCTURE

11.1 The superstructure shall be designed for the design seismic forces as specified in **9** plus other loads required in design load combinations.

11.2 Under simultaneous action of horizontal and vertical accelerations, the superstructure shall have a factor of safety of at least 1.5 against overturning under DBE condition.

11.3 The superstructure shall be secured, when necessary to the sub-structure in all zones through bearings possessing adequate vertical holding down devices and/or unseating prevention system for superstructure. These devices should be used for suspended spans also with the restrained portion of the superstructure. However, frictional forces in the devices should not be relied upon for preventing dislodging and jumping of superstructure.

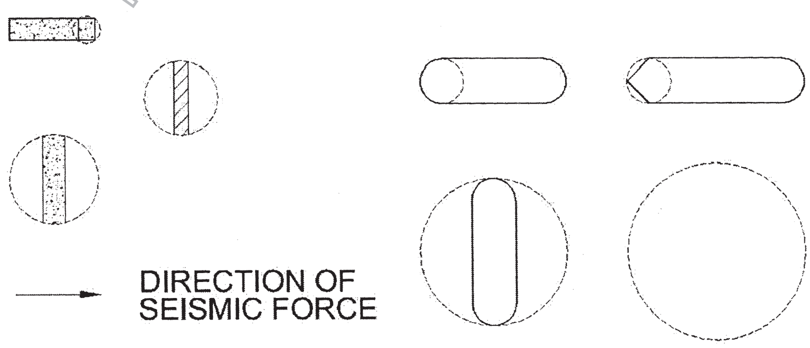


FIG. 3 CASES OF ENVELOPING CYLINDER

12 BEARINGS

12.1 The fixed bearings should be designed to withstand the horizontal and vertical seismic forces, which are expected to transmit these forces in the event of ground motion.

12.2 In the case of movable bearings, the bearings shall be able to accommodate designed displacements. The displacements beyond design values shall be restrained by stoppers.

12.3 Any out of phase motion of piers, if envisaged, shall be considered in working out design seismic displacement in bearings.

12.4 The bearings that are permitted to move in longitudinal direction but restrained in transverse direction shall be designed for estimated design seismic force in transverse direction.

13 VERTICAL HOLD-DOWN DEVICES

13.1 Vertical hold-down devices shall be provided at all supports (or hinges in continuous structures), where resulting vertical force *U* due to the maximum elastic horizontal and vertical seismic forces (combined as per **7**) opposes and exceeds 50 percent of the dead load reaction *D*.

13.2 Where vertical force *U*, due to the combined effect of maximum elastic horizontal and vertical seismic forces, opposes and exceeds 50 percent, but is less than 100 percent, of the dead load reaction *D*, the vertical hold-down device shall be designed for a minimum net upward force of 10 percent of the downward dead load reaction that would be exerted if the span were simply supported.

13.3 If the vertical force *U*, due to the combined effect of maximum horizontal and vertical seismic forces, opposes and exceeds 100 percent of the dead load reaction *D*, then the device shall be designed for a net upward force of 1.2 (*U*-*D*); however, it shall not be less than 10 percent of the downward dead load reaction that would be exerted if the span were simply supported.

14 SEATING WIDTH

The bearing seat width S_E, in mm, between the end of girder and edge of sub-structure, Fig. 4 and minimum S_E between the ends of girder at suspended joint should be not less that the following values:

S_E = 203+1.67 L+6.66 H for seismic Zones II and III

S_E = 305+2.50 L+10.0 H for seismic Zones IV and V

where

L = length of the superstructure to the adjacent expansion joints or to the end of superstructure. In case of bearings under suspended spans, it is the sum of the lengths of two adjacent portions of the superstructure. In case of single span bridges, it is equal to the length of the superstructure, in m.

H = average height of all columns or piers supporting the superstructure to the next expansion joint, for bearings at abutments, in m. It is equal to zero for single span bridges. For bearings at column or piers, it is the height of column or pier. For bearings under suspended spans, it is the average height of two adjacent columns or piers.

15 ANTI-DISLODGING ELEMENTS IN THE HORIZONTAL DIRECTION

Anti-dislodgement elements shall be provided between adjacent sections of the superstructure at supports and at expansion joints. Anti-dislodgement elements like

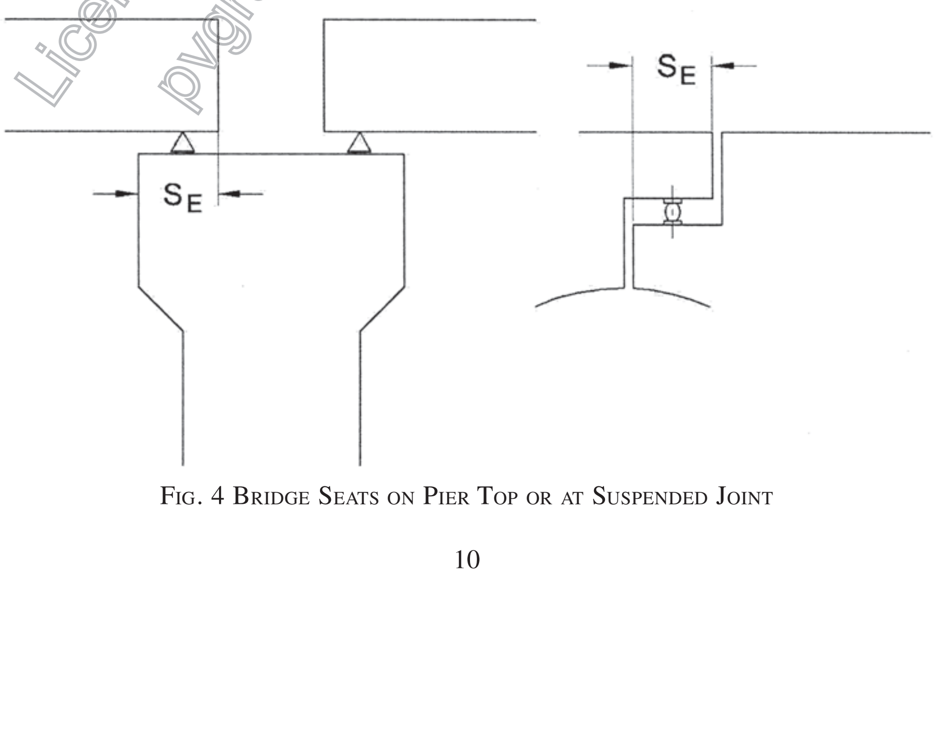


FIG. 4 BRIDGE SEATS ON PIER TOP OR AT SUSPENDED JOINT

reaction blocks and seismic arrestors shall be designed for, at least twice the seismic force.

The linkages, if provided, shall be designed for at least, elastic seismic acceleration coefficient, A_h times the weight of the lighter of the two adjoining spans or parts of the structure as in the case of suspended spans.

If the linkage is at locations where relative deformations are designed to occur, then sufficient slack must be allowed in the linkage so that linkages start functioning only when the design relative displacement at the linkage is exceeded and linkage becomes effective, after overcoming the designed slack in the linkage.

When linkages are provided at columns or piers, the linkage of each span may be connected to the column or pier instead of to the adjacent span.

16 SUBMERSIBLE BRIDGES

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

$$p = 8.75 A_h \sqrt{Hy}$$

where

p = hydrodynamic pressure, in kPa;

A_h = design horizontal seismic coefficient as given in **9.4.1**;

H = height of water surface from the level of deepest scour (*see* **4.5.2**) in m; and

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

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

$$V_y = 2/3\ py$$

$$M_y = 4/15\ py^2$$

where

V_y = hydrodynamic shear, in kPa; and

M_y = hydrodynamic moment, in kPa-m.

17 SPECIAL DUCTILITY REQUIREMENTS FOR BRIDGES

17.1 General Requirement

The bridge shall be designed so that its behaviour under design seismic action is ductile. The capacity design provisions shall be applicable to regular, special and irregular types of bridges in Zones III, IV and V.

The intended plastic hinges shall be provided with adequate ductility measures to ensure the required overall structure ductility.

17.2 Detailing for Ductility

The compliance with provisions of Annex B should be made in general to ensure the availability of adequate local and overall structure ductility.

18 DETAILING FOR CONTROL OF DISPLACEMENTS

In addition to ensure overall ductility, structural and non-structural detailing must ensure satisfactory behaviour of the bridge under design seismic displacement.

The design value of the displacement d_{ED} under seismic condition for providing clearances in critical components shall be determined as follows:

$$d_{ED} = d_E + d_G \pm d_{TS}$$

where

d_E = design seismic displacement determined from linear analysis considering R = 1;

d_G = displacement due to permanent and quasi-permanent action measured in long term such as shrinkage, creep and post-tensioning;

d_{TS} = displacement due to thermal movements = 0.4 d_T; and

d_T = design displacement due to thermal movement.

19 SEISMIC RETROFITTING OF BRIDGES

19.1 General Provision

The decision to retrofit shall be based on the overall consideration of seismicity, vulnerability and importance of the bridge.

The need to retrofit shall be determined on the basis of one of the standard procedures such as capacity-demand ratio method, non-linear pushover analysis and time history method.

The objective of retrofitting should be to meet at least the requirement of present seismic code considering residual life of the structure.

19.2 Retrofit Techniques

19.2.1 On the basis of deficiency observed after seismic evaluation, suitable retrofit techniques should be selected. Some retrofit techniques for various components are given below.

19.2.2 *Superstructure*

Horizontal or vertical motion restrainers, inter linking of spans, pre-stressing, using dampers.

19.2.3 *Sub-structure*

Concrete jacketing, steel jacketing, carbon fiber winding, composite jacket of fibre glass and other composites.

19.2.4 *Bearings*

Replacement of bearings by new bearings that could accommodate displacements, provision of stoppers, clamps/vertical holding down devices, replacement of bearings by isolation devices.

19.2.5 *Foundation*

Strengthening of existing foundation by enlargement of size, increasing number of piles, jacketing.

19.3 Effectiveness of Retrofit Techniques

The retrofit structure should be analyzed and re-designed to check its effectiveness following standard procedures. The experimental methods of testing effectiveness of techniques may be carried out on components/models by quasi-static testing or on shaking table.

20 SPECIAL EARTHQUAKE PROTECTION AND RESISTANT DEVICES

20.1 Seismic Base Isolation

Seismic base isolation devices can be designed and introduced on the top of piers to increase fundamental period of bridge and thus reduce the seismic forces in sub-structure. Seismic base isolation devices can be used in place of traditional bearings. The devices may comprise of elastomeric bearings, high damping rubber bearing, lead rubber bearing and friction-pendulum system. In addition to isolation bearing a damping device in the form of a damper is also provided. The isolation device shall be useful when natural period of bridge is less than 2 s.

20.1.1 The isolation system should be designed following the standard procedure. The choice of characteristics of future ground motion is important in this design approach.

20.2 Shock Transmission Units (STU)

In the case of bridge spans with continuous superstructure, the seismic force developed at deck level can be distributed uniformly to various piers by providing special devices such as STUs. The design is made such that these units permit slow movements, while under severe shaking these devices restrain the movement and engage various piers in sharing the shear forces. The STUs may consist of viscous dampers or such similar devices installed between sub-structure and superstructure. The STUs should be designed following a standard literature.

20.3 EARTHQUAKE RESISTANT DESIGN WITH SPECIAL FEATURES

The bridges with typical site conditions can be designed with special earthquake resistant features. The earthquake force generated in the superstructure can be resisted by one or more specially made abutments, the earthquake force in superstructure can be resisted by one or more piers with fixed bearings, while other supports may be provided with movable bearings. A balance should be maintained between the strength and the flexibility requirements of the horizontal supports. High flexibility reduces the level of design force but increases movement at the joints and moveable bearings and may lead to high second order effects.

21 FOUNDATIONS

21.1 Liquefaction may occur in the case of saturated cohesion-less soil during earthquake vibrations. The liquefaction potential of sites liable to liquefy should be estimated by specialist literature.

21.2 The remedial measures for liquefaction should be undertaken, if necessary. The structural design of bridge may be modified if required to account for such effects.

21.3 Safety against overturning/sliding shall be checked as per relevant IRC/IRS Codes.

22 RETAINING WALLS

22.1 This clause is applicable to ϕ soil only. The reference may be made to **25** for soil with cohesion.

22.1.1 *Dynamic Active Earth Pressure Due to Backfill*

Figure 5 shows a wall of height h, inclined with an angle α with vertical, retaining dry/moist cohesionless earth fill. The dynamic active earth pressure exerted against the wall shall be:

$$\left(P_{A\gamma}\right)_{dyn} = \frac{1}{2}\gamma h^2 C_a$$

where

$(P_{A\gamma})_{dyn}$ = dynamic total active earth pressure, in kN/m length of wall;

γ = unit weight of soil, in kN/m^3; and

h = height of wall, in m.

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

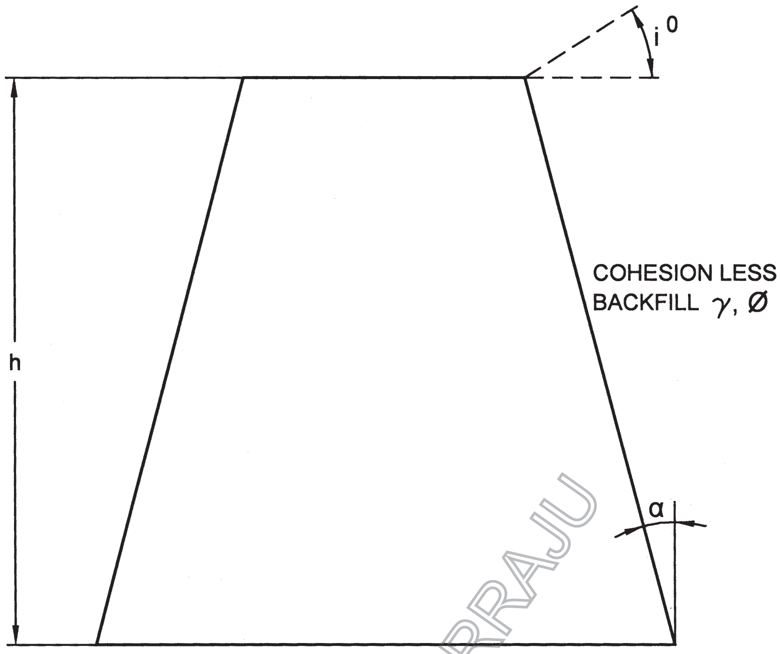


FIG. 5 CROSS-SECTION OF RETAINING WALL

where

A_v = vertical seismic coefficient – its direction being taken consistently throughout the stability analysis of wall and equal to $\frac{2}{3} A_h$;

ϕ = angle of internal friction of soil;

$\lambda = \tan^{-1}\left[\frac{A_h}{1 \pm A_v}\right]$;

α = angle with earth face of the wall makes with the vertical;

i = slope of earthfill;

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

A_h = horizontal seismic coefficient.

The expression of $(C_a)_{dyn}$ gives two values depending on the sign of A_v. For design purpose higher of the two values shall be taken.

22.1.1.1 The active pressure may be determined graphically by means of the method described in Annex C.

22.1.1.2 *Point of application*

From the total earth pressure computed as above subtract the static active pressure obtained by putting $A_h = A_v = \lambda = 0$ in the expression given in **22.1.1**. 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 an elevation 0.5 h above the base.

22.1.1 *Dynamic Passive Earth Pressure Due to Backfill*

The dynamic passive earth pressure against the walls shall be given by the following formula:

$$\left(P_{p\gamma}\right)_{dyn} = \frac{1}{2}\gamma \; h^2 \; C_p$$

where

$(P_{py})_{dyn}$ = dynamic passive earth pressure length of wall, in kg/m; and

$$C_p = \frac{(1 \pm A_v)\cos^2(\phi + \alpha - \lambda)}{\cos\lambda \quad \cos^2\alpha \quad \cos \quad (\delta + \alpha + \lambda)} \times$$

$$\left[\frac{1}{1 - \left\{\frac{\sin \;(\phi + \delta)\; \sin \;(\phi + i - \lambda)}{\cos \;(\alpha - i)\; \cos \;(\delta - \alpha + \lambda)}\right\}^{\frac{1}{2}}}\right]^2$$

For design purposes, the lesser value of C_p shall be taken out of its two values corresponding to $\pm A_v$.

22.1.2.1 The passive pressure may be determined graphically by means of the method described in Annex D.

22.1.2.2 *Point of application*

From the total passive earth pressure computed as above subtract the static earth pressure obtained by putting $A_h = A_v = \lambda = 0$ in the expression given in **22.1.2**. 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 0.5 h above the base of the wall.

22.1.3 *Active Pressure Due to Uniform Surcharge*

The active pressure against the wall due to a uniform surcharge of intensity q, kN per unit area of the inclined earthfill surface shall be:

$$\left(P_{\mathrm{Aq}}\right)_{\mathrm{dyn}} = \frac{qh\cos\alpha}{\cos(\alpha - i)} C_{\mathrm{a}}$$

22.1.3.1 *Point of application*

The dynamic increment in active pressure 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.

22.1.4 *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:

$$\left(P_{\mathrm{Pq}}\right)_{\mathrm{dyn}} = \frac{qh\cos\alpha}{\cos(\alpha - i)} C_{\mathrm{p}}$$

22.1.4.1 *Point of application*

The dynamic decrement in passive pressure 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 an elevation 0.5 h above the base.

23 EFFECT OF SATURATION ON LATERAL EARTH PRESSURE

23.1 For saturated earthfill, the saturated unit weight of the soil shall be adopted as in the formulae described in **22.1**.

23.2 For submerged earthfill, the dynamic increment (or decrement) in active and passive earth pressure during earthquakes shall be found from expressions given in **22.1.1** and **22.1.2** with the following modifications:

a) The value of δ shall be taken as ½ the value of δ for dry/moist backfill.

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

$$\lambda = \tan^{-1}\left[\frac{\gamma_{\mathrm{s}}}{\gamma_{\mathrm{s}} - 10} \times \frac{A_{\mathrm{h}}}{1 \pm A_{\mathrm{v}}}\right]$$

where

γ_{s} = saturated unit weight of soil, in kN/m^3;

A_{h} = horizontal seismic coefficient (*see* **10.4.1**); and

A_{v} = vertical seismic coefficient which is $\frac{2}{3} A_{\mathrm{h}}$.

c) Buoyant unit weight shall be adopted.

d) From the value of earth pressure found out as above, subtract the value of static earth pressure determined by putting $A_{\mathrm{h}} = A_{\mathrm{v}} = \lambda = 0$. The remainder shall be dynamic increment.

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

24 PARTIALLY SUBMERGED BACKFILL

24.1 The ratio of the lateral dynamic increment in active pressure due to backfill to the vertical pressures at various depths along the height of wall may be taken as shown in Fig. 6a.

The pressure distribution of dynamic increment in active pressures due to backfill may be obtained by multiplying the vertical effective pressures by the coefficients in Fig. 6a at corresponding depths.

NOTE — C_{a} is computed as in **22.1.1** for dry, moist and saturated backfills and C'_a is computed as in **22.1.1** and **23.2** for sub-merged backfills.

K_{a} = value of C_{a} when $A_{\mathrm{h}} = A_{\mathrm{v}} = \lambda = 0$

K'_a = value of C'_a when $A_{\mathrm{h}} = A_{\mathrm{v}} = \lambda = 0$

h' = height of submergence above the base of the wall

Lateral dynamic increment due to surcharge multiplying with q is shown in Fig. 6b.

24.2 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.

25 ACTIVE EARTH PRESSURE DUE TO c-ϕ SOIL

25.1 Active earth pressure due to c-ϕ soil as backfill. Fig. 7 shows a section of retaining wall retaining c-φ soil as backfill which also carries a uniform surcharge of intensity q.

25.2 AFCD is cracked zone in c-ϕ soil, CD being 'H_{c}' given by following expression:

$$H_{\mathrm{c}} = \frac{2c}{\gamma}\sqrt{N_{\varphi}} = nH$$

where

$N\varphi = \tan^2(45° + \phi/2)$;

n = Non-dimensional factor describing the depth of tension crack; and

γ = Dry or moist unit weight of soil.

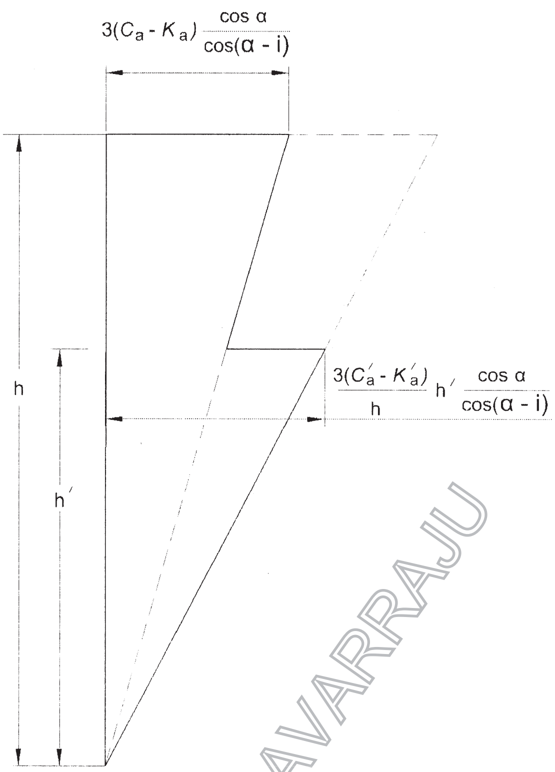


FIG. 6A DISTRIBUTION OF THE RATIO = $\dfrac{\text{LATERAL DYNAMIC INCREMENT DUE TO BACKFILL WITH HEIGHT OF WALL}}{\text{VERTICAL EFFECTIVE PRESSURE}}$

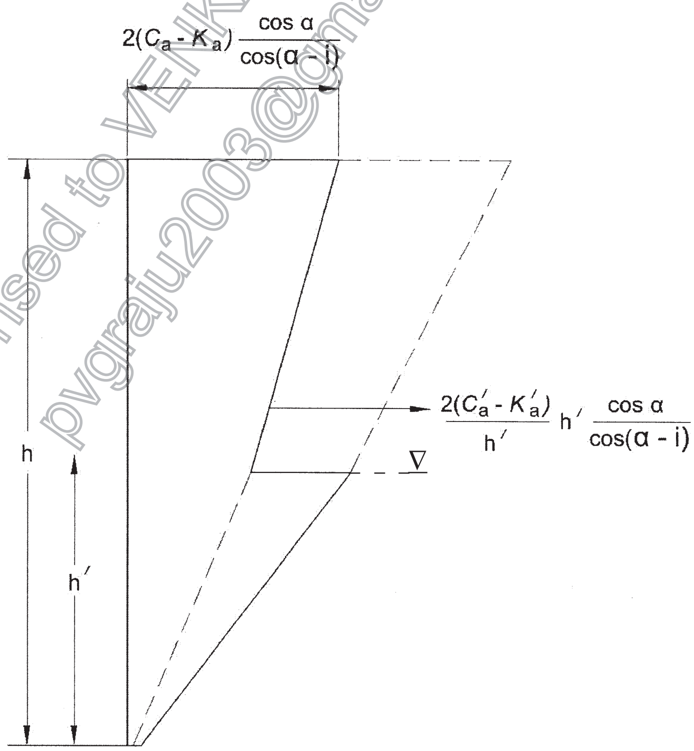


FIG. 6B DISTRIBUTION OF THE RATIO = $\dfrac{\text{LATERAL DYNAMIC INCREMENT DUE TO SURCHARGE WITH HEIGHT OF WALL}}{\text{SURCHARGE INTENSITY}}$

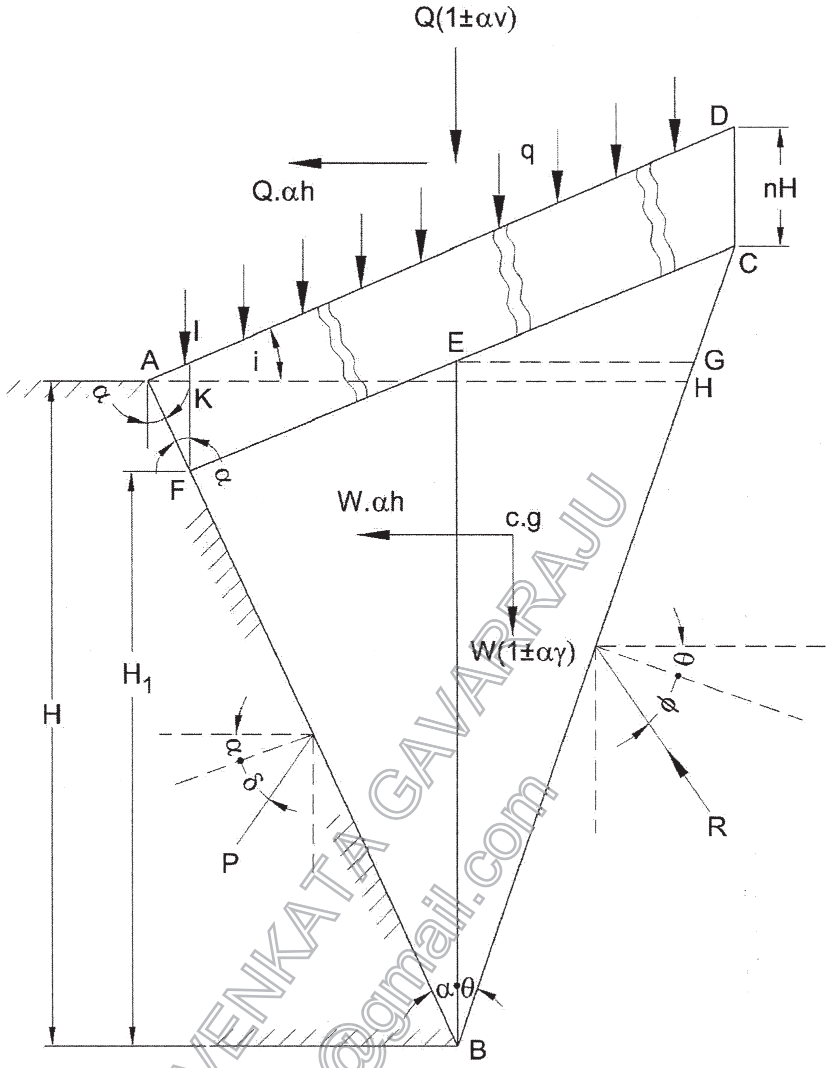


FIG. 7 FORCES ACTING ON FAILURE WEDGE IN ACTIVE STATE FOR SEISMIC CONDITION IN c-ϕ SOIL

25.3 The general expression of computing dynamic active earth pressure is given as:

$$(\mathrm{P_A})_{\mathrm{dyn}} = \frac{1}{2}\gamma H^2 (N_{\mathrm{a\gamma m}})_{\mathrm{dyn}} + qH\left(N_{\mathrm{aqm}}\right)_{\mathrm{dyn}} - cH(N_{\mathrm{acm}})_{\mathrm{dyn}}$$

where $(N_{\mathrm{a\gamma m}})_{\mathrm{dyr}}$, $(N_{\mathrm{aqm}})_{\mathrm{dyr}}$, and $(N_{\mathrm{acm}})_{\mathrm{dyr}}$, are dynamic earth coefficients which depend on ϕ, α, i, δ, A_{h}, A_{v} and n.

25.4 For static case ($A_{\mathrm{h}} = A_{\mathrm{v}} = 0$), earth pressure coefficients are designated as $(N_{\mathrm{a\gamma m}})_{\mathrm{st}}$, $(N_{aqm})_{\mathrm{st}}$ and $(N_{\mathrm{acm}})_{\mathrm{st}}$. For various values of parameters ϕ, α, i and n, these earth pressure coefficients are shown in Figs. 8 to 12. It may be noted that $(N_{\mathrm{acm}})_{\mathrm{st}}$ is given in Fig. 8 for $i = 0$ and $n = 0$ case. For other values of i and n, these curves can be used by multiplying the obtained values of $(N_{\mathrm{acm}})_{\mathrm{st}}$ by the factor:

$$\left[1 - \frac{n \ \cos \ i \ \cos \ \alpha}{\cos \ (\alpha - i)}\right]$$

For $i = 0$ and $n = 0$ values of $(N_{\mathrm{aqm}})_{\mathrm{st}}$ and $(N_{\mathrm{ayy}})_{\mathrm{st}}$ are same and shown in Fig. 9 a.

For intermediate value of ϕ, α, i and n linear interpolation may be done.

It was further found that $(N_{\mathrm{acm}})_{\mathrm{dyr}}$ is independent to A_{h} and A_{v}. Therefore the value given in Fig. 8 may be adopted as $(N_{\mathrm{acm}})_{\mathrm{dyn}}$.

26 Values of $(N_{\mathrm{aqm}})_{\mathrm{dyn}}$ and $(N_{\mathrm{ayy}})_{\mathrm{dyn}}$ may be obtained by multiplying the values of $(N_{\mathrm{aqm}})_{\mathrm{st}}$ and $(N_{\mathrm{ayy}})_{\mathrm{st}}$ by non-dimensional factors λ_1 and λ_2 respectively. Their values are given in Table 5 for some selected values of ϕ, α, n, i and A_{v}. For intermediate values of these parameters, linear interpolation may be done.

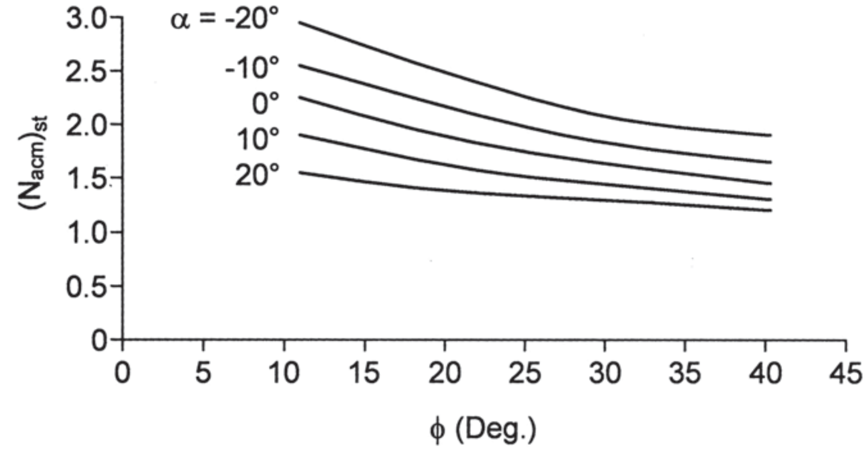
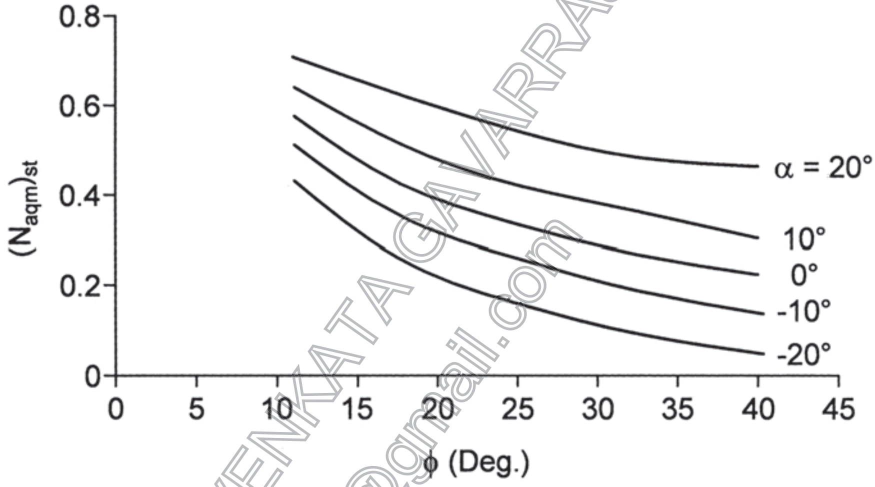
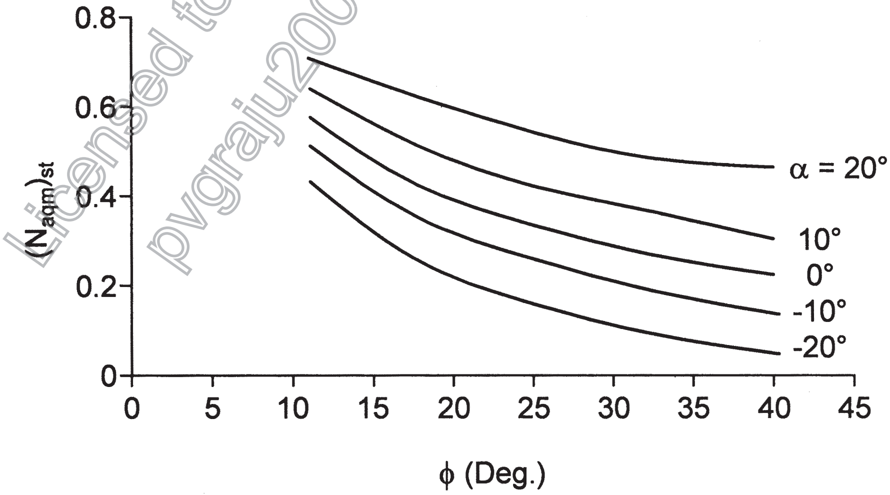


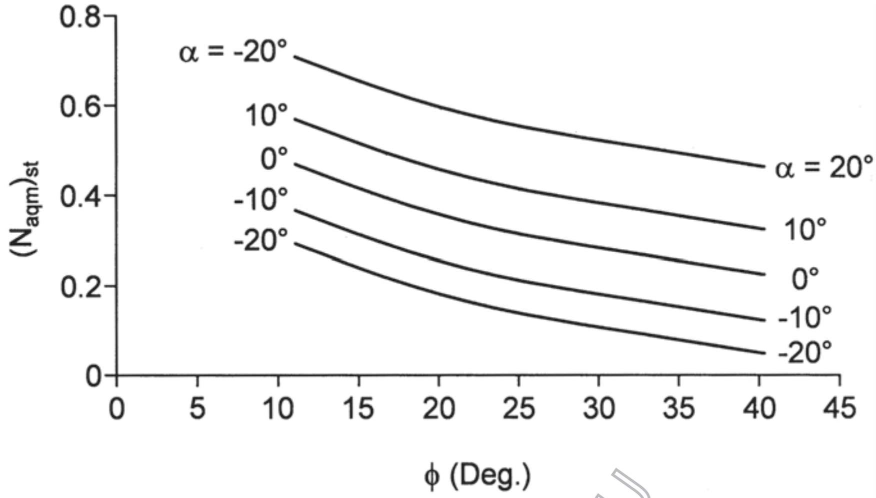
FIG. 8 $(N_{acm})_{st}$ *VERSUS* ϕ *FOR* $n = 0$, $i = 0°$



9A $(N_{aqm})_{st}$ *versus* f *for* $n = 0$, $i = 0°$

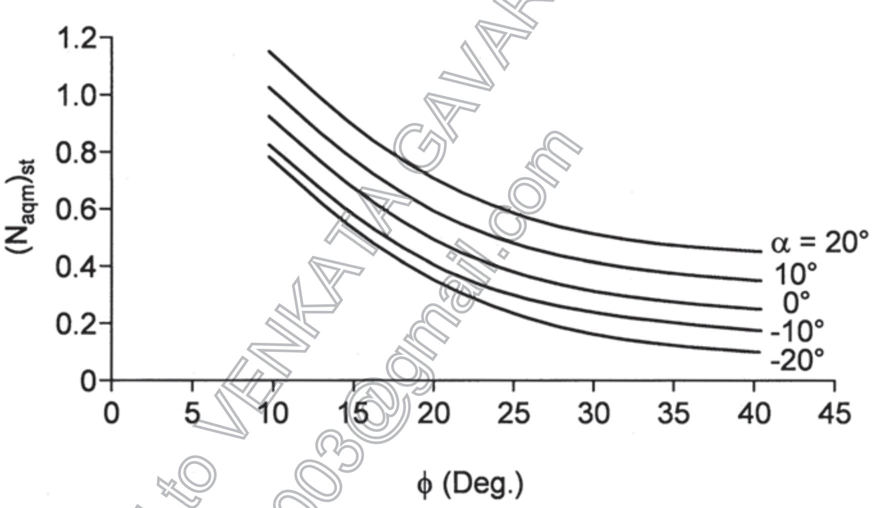


9B $(N_{aqm})_{st}$ *versus* f *for* $n = 0.2$, $i = 0°$

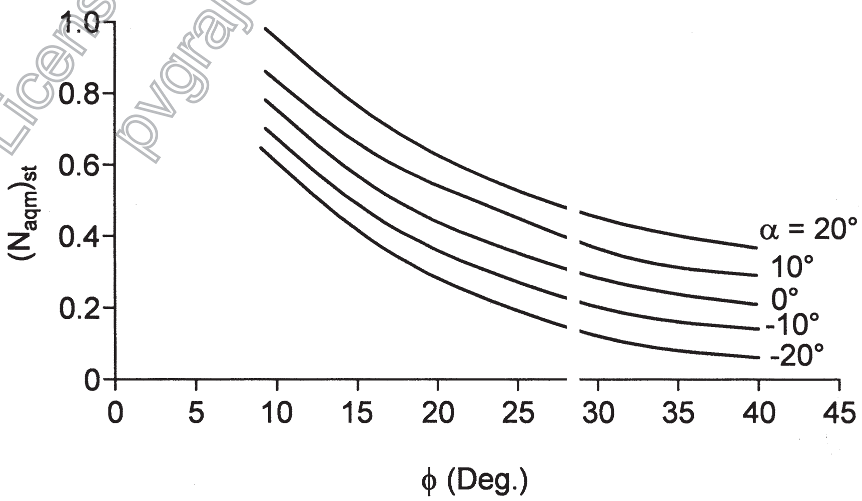


9C $(N_{aqm})_{st}$ *versus* f *for* n = 0.4, i = 0°

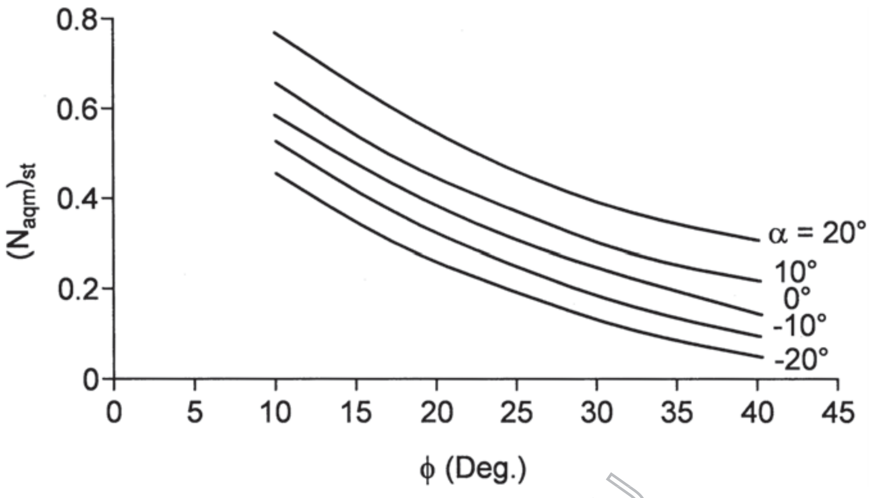
FIG. 9 EARTH PRESSURE COEFFICIENT (naqm)st for 0° SLOPE



10A $(N_{aqm})_{st}$ VERSUS ϕ FOR n = 0, i = 10°

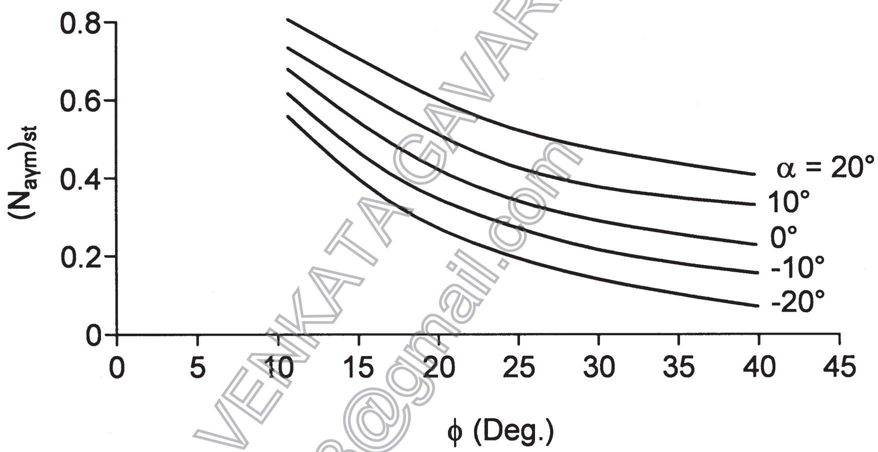


10B $(N_{aqm})_{st}$ VERSUS ϕ FOR n = 0.2, i = 10°

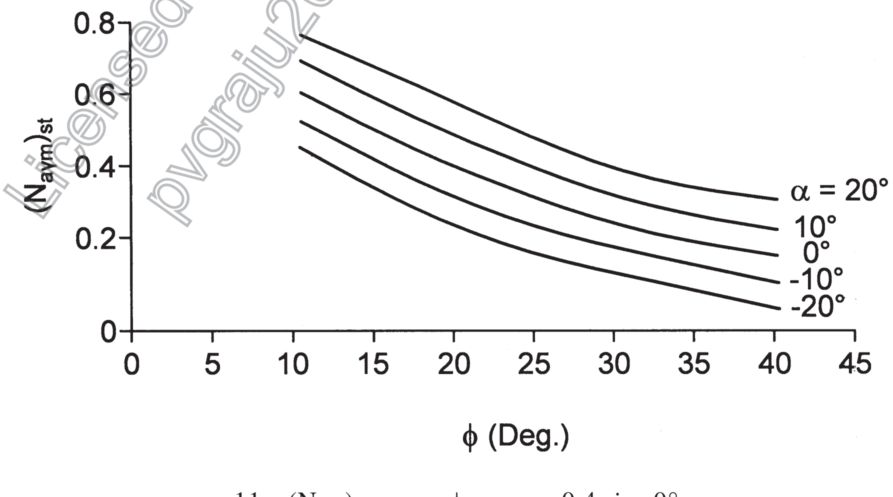


10C $(N_{aqm})_{st}$ *VERSUS* ϕ *FOR* $n = 0.4$, $i = 10°$

FIG. 10 EARTH PRESSURE COEFFICIENT (naqm)st FOR 10° SLOPE

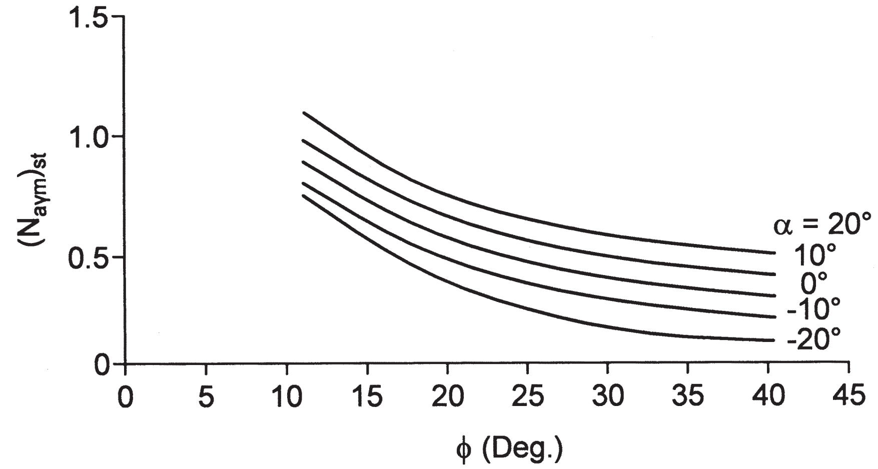


11A $(N_{aym})_{st}$ *VERSUS* ϕ *FOR* $n = 0.2$, $i = 0°$

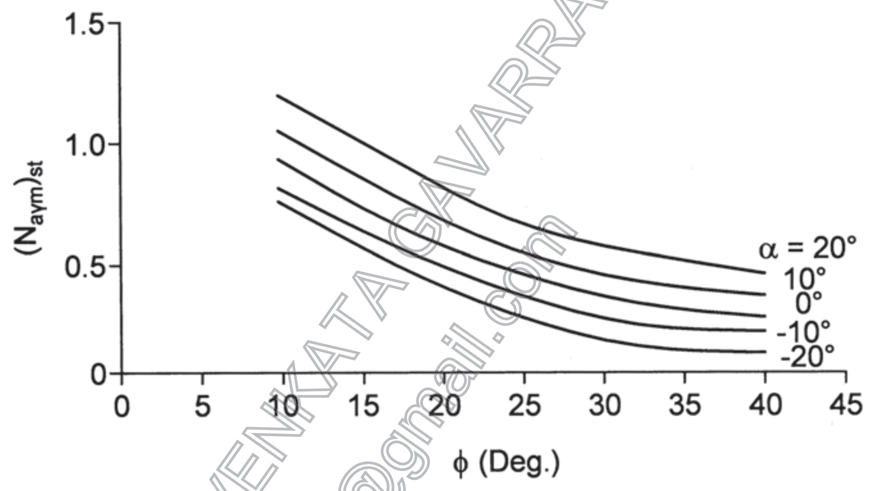


11B $(N_{aym})_{st}$ *VERSUS* ϕ *FOR* $n = 0.4$, $i = 0°$

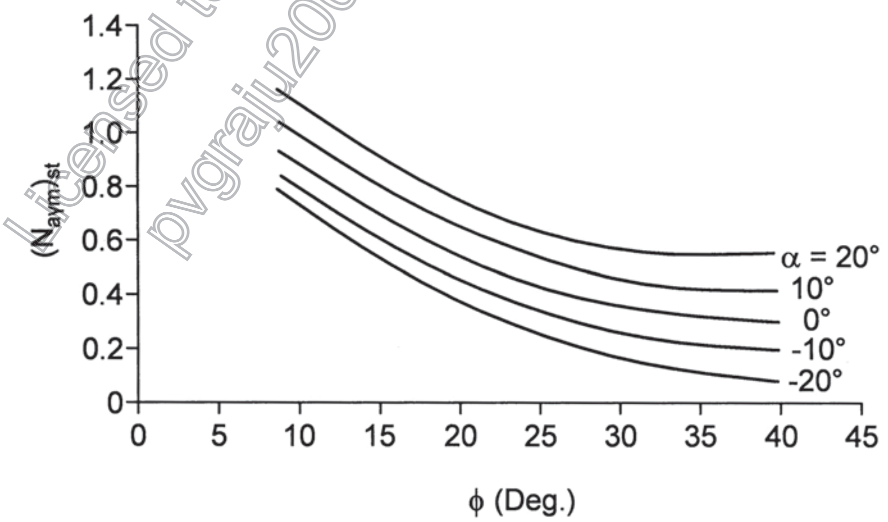
FIG. 11 EARTH PRESSURE COEFFICIENT (naym)st for 0° Slope



12A $(N_{aym})_{st}$ *VERSUS* φ *FOR* $n = 0$, $i = 10°$



12B $(N_{aym})_{st}$ *VERSUS* φ *FOR* $n = 0.2$, $i = 10°$



12C $(N_{aym})_{st}$ *VERSUS* φ *FOR* $n = 0.4$, $i = 10°$

FIG. 12 EARTH PRESSURE COEFFICIENT (naym)ST FOR 10° SLOPE

Table 5 Values of λ_1 and λ_2

(*Clause* 26)

$n = 0$, $i = 0°$, $\lambda_1 = \lambda_2$

ϕ	$A_h = 0.05$			$A_h = 0.10$			$A_h = 0.15$		
	α = -20°	α = 0°	α = 20°	α = -20°	α= 0°	α = 20°	α = -20°	α = 0°	α = 20°
20°	1.153	1.117	1.098	1.327	1.247	1.208	1.52	1.393	1.333
30°	1.187	1.133	1.108	1.392	1.278	1.228	1.62	1.438	1.36
40°	1.241	1.156	1.123	1.51	1.327	1.258	1.807	1.513	1.407

$n = 0$, $i = 10°$, $\lambda_1 = \lambda_2$

ϕ	$A_h = 0.05$			$A_h = 0.10$			$A_h = 0.15$		
	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°
20°	1.193	1.153	1.136	1.435	1.345	1.313	1.773	1.618	1.574
30°	1.206	1.151	1.128	1.441	1.323	1.276	1.712	1.52	1.449
40°	1.254	1.168	1.137	1.542	1.356	1.292	1.869	1.566	1.466

$n = 0.4$, $i = 0°$, λ_1

ϕ	$A_h = 0.05$			$A_h = 0.10$			$A_h = 0.15$		
	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°
20°	1.206	1.117	1.073	1.44	1.247	1.154	1.709	1.393	1.246
30°	1.259	1.133	1.086	1.554	1.279	1.182	1.887	1.438	1.286
40°	1.382	1.155	1.101	1.827	1.327	1.21	2.336	1.513	1.33

$n = 0.4$, $i = 10°$, λ_1

ϕ	$A_h = 0.05$			$A_h = 0.10$			$A_h = 0.15$		
	α = -20°	α = 0°	α =20°	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α =20°
20°	1.254	1.153	1.104	1.582	1.345	1.239	2.055	1.618	1.436
30°	1.281	1.151	1.105	1.616	1.323	1.225	2.009	1.52	1.364
40°	1.394	1.168	1.114	1.868	1.357	1.242	2.424	1.566	1.384

$n = 0.4$, $i = 0°$, λ_2

ϕ	$A_h = 0.05$			$A_h = 0.10$			$A_h = 0.15$		
	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°
20°	1.169	1.117	1.09	1.356	1.247	1.192	1.57	1.393	1.307
30°	1.205	1.133	1.102	1.433	1.279	1.214	1.684	1.438	1.337
40°	1.27	1.156	1.116	1.575	1.327	1.243	1.917	1.513	1.382

$n = 0.4$, $i = 10°$, λ_2

ϕ	$A_h = 0.05$			$A_h = 0.10$			$A_h = 0.15$		
	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°	α = -20°	α = 0°	α = 20°
20°	1.211	1.153	1.127	1.478	1.345	1.292	1.854	1.618	1.536
30°	1.226	1.151	1.122	1.488	1.323	1.262	1.79	1.52	1.425
40°	1.287	1.168	1.13	1.619	1.357	1.277	1.997	1.566	1.442

ANNEX A

(*Clause* 2)

LIST OF REFERRED INDIAN STANDARDS

IS No.	*Title*
456 : 2000	Code of practice for plain and reinforced concrete(*fourth revision*)
1343 : 2012	Code of practice for pre-stressed concrete (*first revision*)
1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement — Specification (*fourth revision*)
1893	Criteria for earthquake resistant design of structures:
(Part 1) : 2002	General provisions and buildings
(Part 2) : 2014	Liquid retaining tanks — Elevated and ground supported

ANNEX B

(*Foreword*, *Clauses* 4.2.7, 5.3 *and* 17.2)

DUCTILE DETAILING

B-0 GENERAL

The detailing rules given have been chosen with the intention that reliable plastic hinges should form at the top and bottom of each pier column, or at the bottom only of a single stem pier under horizontal loading and that the bridge should remain elastic between the hinges (*see* Fig. 13). The aim is to achieve a reliable ductile structure. Repair of plastic hinges is relatively easy.

Design strategy to be used is based on assumption that the plastic response shall occur in the sub-structure.

B-1 SPECIFICATION

B-1.1 Minimum grade of concrete should be M25 (f_{ck} = 25 MPa).

B-1.2 Steel reinforcement having elongation more than 14.5 percent and conforming to other requirements of IS 1786 shall be used.

B-2 LAYOUT

a) The use of circular column is preferred for better plastic hinge performance and ease of construction.

b) The bridge must be proportioned and detailed by the designer so that plastic hinges occur only at the controlled locations (for example pier column ends) and not in other uncontrolled places.

B-3 LONGITUDINAL REINFORCEMENT

The area of the longitudinal reinforcement shall not be less than 0.8 percent and not more than 6 percent of the gross cross-section area A_g. Splicing of flexural region is not permitted in the plastic hinge region. Lap shall not be located within a distance of 2 times the maximum column cross-sectional dimension from the end at which hinge may occur. The splices should be proportioned as a tension splice.

B-3.1 Curtailment of longitudinal reinforcement in piers due to reduction in seismic bending moment towards top.

B-3.1.1 The reduction of longitudinal reinforcement at mid-height in piers should not be carried out except in tall pier.

B-3.1.2 In case of high bridge piers such as of height equal to 30 m or more, the reduction of reinforcement at mid height may be done. In such cases the following method should be adopted:

a) The curtailment of longitudinal reinforcement shall not be carried out in the section six times the least lateral column dimension from the location where plastic hinge in likely to occur.

b) The interval between hoop ties is specified to be less than 150 mm in a reinforcement position. The interval between hoop ties shall not change abruptly, the change must be gradual.

B- 4 TRANSVERSE REINFORCEMENT

The transverse reinforcement for circular columns shall consist of spiral or circular hoops. Continuity of these reinforcements should be provided by either [*see* Figs. 14 (a) *and* 14(b)]:

a) *Welding* — The minimum length of weld should be 12 times the bar diameter, and the minimum weld throat thickness should be 0.4 times the bar diameter.

b) *Lapping* — The minimum length of lap

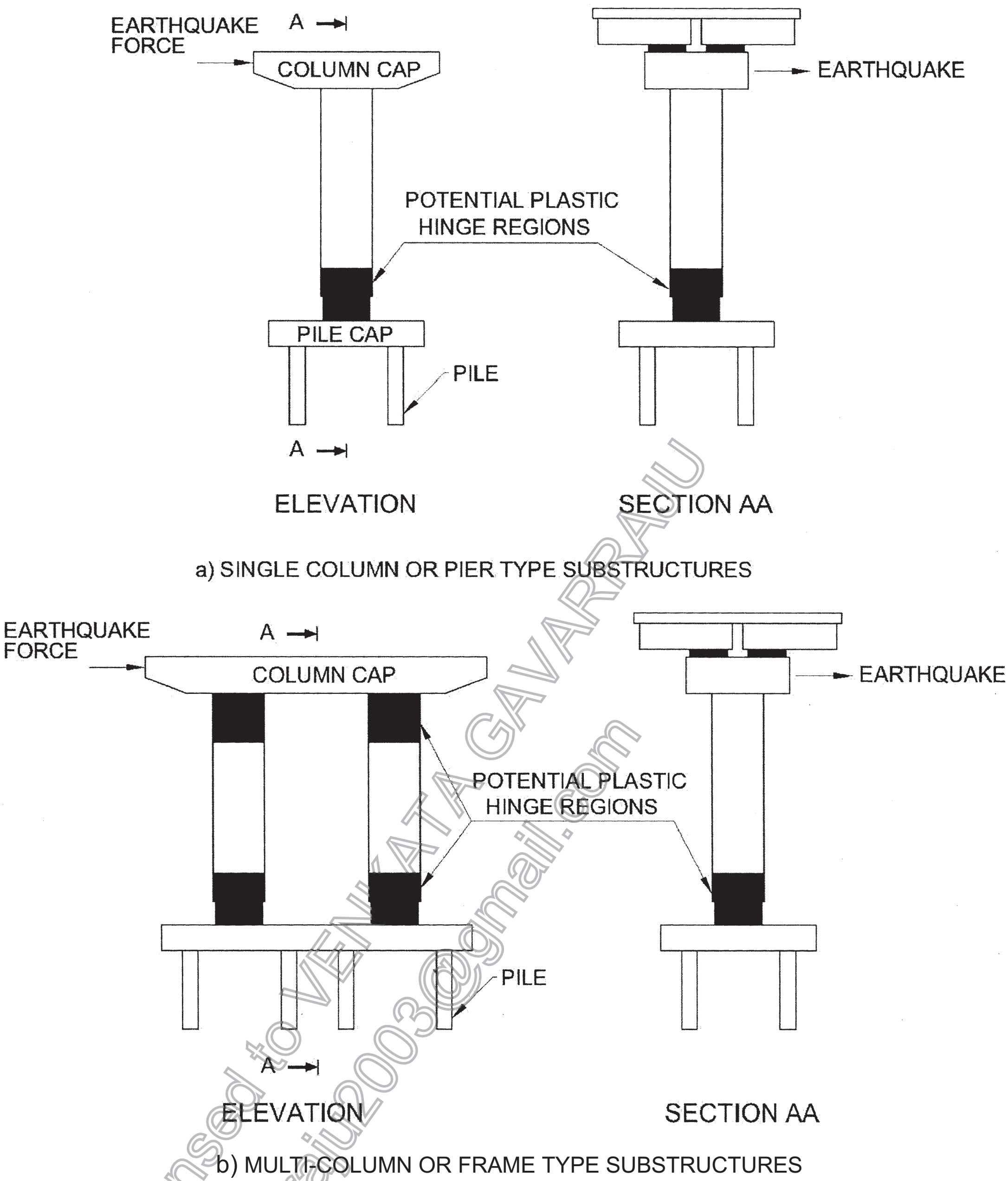


FIG. 13 POTENTIAL LOCATION OF PLASTIC HINGES IN SUBSTRUCTURES

should be 30 times the bar diameter and each end of the bar anchored with 135° hooks with a 10 diameter extension into the confined core. Splicing of the spiral reinforcement in the plastic hinge region should be avoided.

In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135° hook with a 10 diameter extension at each end that is embedded in the confined core Fig. 14 (c). When hoop ties are joined in any place other than a corner the hoop ties shall overlap each other by a length 40 times the bar diameter of the reinforcing bar which makes the hoop ties with hooks as specified above.

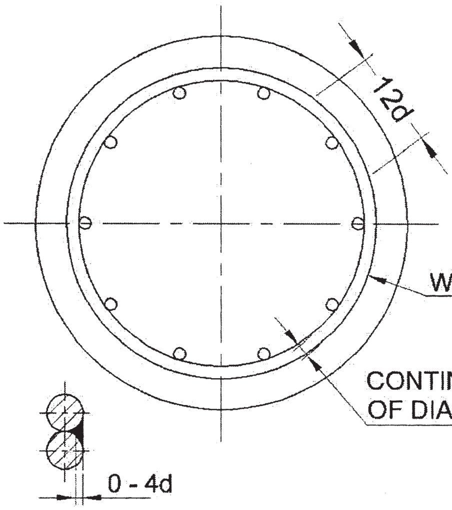
Joint portion of hoop ties for both circular and rectangular hoops should be staggered.

B-5 DESIGN OF PLASTIC HINGE REGIONS

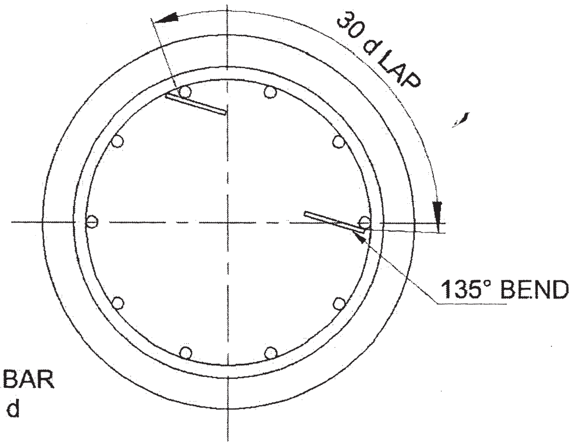
B-5.1 Seismic Design Force for Sub-structure

Provisions given for the ductile detailing of RC members subjected to seismic forces shall be adopted for supporting components of the bridge. Further, the design shear force at the critical section (s) of sub-structures shall be the lower of the following:

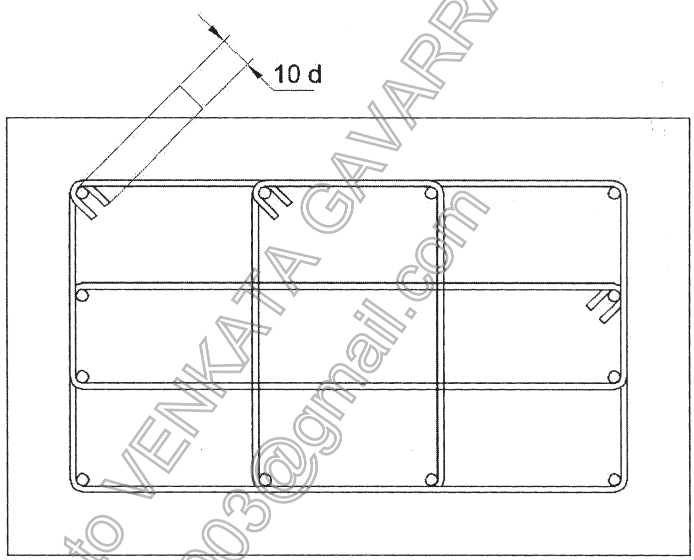
a) Maximum elastic shear force at the critical section of the bridge component divided by



a) WELDING IN CIRCULAR HOOPS



b) LAPPING IN CIRCULAR HOOPS



c) RECTANGULAR HOOPS

FIG. 14 TRANSVERSE REINFORCEMENT IN COLUMN

the response reduction factor for the components as per Table.

b) Maximum shear force that develops when the sub-structure has maximum moment that it can sustain (that is the over strength plastic moment capacity as per **B-5.2**) in single column or single-pier type sub-structure, or maximum shear force that is developed when plastic moment hinges are formed in the sub-structure so as to form a collapse mechanism in multiple column frame type or multiple-pier type sub-structures, in which the plastic moment capacity shall be the over strength plastic moment capacity as per **B-5.2**.

c) In a single-column type or pier type sub-structure, the critical section is at the bottom of the column or pier as shown in Figs. 13(a). And, in multi-column frame-type sub-structures or multi-pier sub-structures, the critical sections are at the bottom and/or top of the columns/piers as shown in Figs. 13 (b).

B-5.2 Over Strength Plastic Moment Capacity

The over strength plastic moment capacity at the reinforced concrete section shall be taken as 1.3 times the ultimate moment capacity based on the usual partial safety factors recommended by relevant design codes for materials and loads, and on the actual dimensions of members and the actual reinforcement detailing adopted.

B-5.3 Special Confining Reinforcement

Special confining reinforcement shall be provided at the ends of pier columns where plastic hinge can occur. This transverse reinforcement should extend for a distance from the point of maximum moment over the plastic hinge region over a length l_o. The length l_o shall not be less than, 1.5 times the column diameter or 1.5 times the large cross sectional dimension where yielding occurs, 1/6 of clear height of the column for frame pier (that is when hinging can occur at both ends of the column), 1/4 of clear height of the column for cantilever pier (that is when hinging can occur at only one end of the column) or 600 mm.

B-5.4 Spacing of Transverse Reinforcement

The spacing of hoops used as special confining reinforcement shall not exceed 1/5 times the least lateral dimension of the cross-section of column or 6 times the diameter of the longitudinal bar or 150 mm.

The parallel legs of rectangular stirrups shall be spaced not more than 1/3 of the smallest dimension of the concrete core or more than 300 mm centre to centre. If the length of any side of the stirrups exceeds 300 mm, a cross tie shall be provided. Alternatively, overlapping stirrups may be provided within the column.

B-5.5 Amount of Transverse Steel to be Provided

B-5.5.1 The area of cross-section, A_{sh}, of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sw} = 0.09SD_k\left[\frac{A_g}{A_c} - 1.0\right]\frac{f_{ck}}{f_y}$$

or, $$A_{sw} = 0.024SD_k\frac{f_{ck}}{f_y}$$

whichever is the greater

where

A_{sh} = area of cross-section of circular hoop;

S = pitch of spiral or spacing of hoops, in mm;

D_k = Diameter of core measured to the outside of the spiral or hoops, in mm;

f_{ck} = characteristic compressive strength of concrete;

f_y = yield stress of steel (of circular hoops or spiral);

A_g = gross area of the column cross-section; and

A_c = Area of the concrete core $= \frac{\pi}{4}D_k^2$.

B-5.5.2 The total area of cross-section of the bar forming rectangular hoop and cross ties, A_{sh} to be used as special confining reinforcement shall not be less than

$$A_{sw} = 0.24Sh\left[\frac{A_g}{A_k} - 1.0\right]\frac{f_{ck}}{f_y}$$

or, $$A_{sw} = 0.096Sh\frac{f_{ck}}{f_y}$$

where

h = longer dimension of the rectangular confining hoop measured to its outer face; and

A_k = area of confined core concrete in the rectangular hoop measured to its outer side dimensions.

NOTE — Crossties where used should be of the same diameter as a peripheral hoop bar and A_k shall be measured as the overall core area, regardless the hoop area. The hooks of crossties shall engage peripheral longitudinal bars.

B-5.5.2.1 Unsupported length of rectangular hoops shall not exceed 300 mm.

B-5.5.3 For ductile detailing of hollow cross-section of pier, special literature may be referred.

B-6 DESIGN OF COMPONENTS BETWEEN THE HINGES

Once the position of the plastic hinges has been determined and these regions detailed to ensure a ductile performance, the structure between the plastic hinges is designed considering the capacity of the plastic hinges. The intention here is,

a) to reliably protect the bridge against collapse so that it shall be available for service after a major shaking.
b) to localize structural damage to the plastic hinge regions where it can be controlled and repaired.

The process of designing the structure between the plastic hinges is known as 'capacity design'.

B-6.1 Column Shear and Transverse Reinforcement

To avoid a brittle shear failure, design shear force for pier shall be based on over strength moment capacities of the plastic hinges and given by:

$$V_u = \frac{\Sigma M^\circ}{h}$$

where

ΣM° = sum of the over strength moment capacities of the hinges resisting lateral loads, as detailed. In case of twin pier this would be the sum of the over strength moment capacities at the top and bottom of the column. For single stem piers the over strength moment capacity at the bottom only should be used.

h = clear height of the column in the case of a column in double curvature; height to be calculated from point of contra-flexure in the case of a column in single curvature.

Outside the hinge regions, the spacing of hoops shall not exceed half the least lateral dimension of the column or 300 mm.

B-7 DESIGN OF JOINTS

Beam-column joints should be designed properly to resist the forces caused by axial load, bending and shear forces in the joining members. Forces in the joint should be determined by considering a free body of the joint with the forces on the joint member boundaries properly represented.

The joint shear strength should be entirely provided by transverse reinforcement. Where the joint is not confined adequately (that is where minimum pier and pile cap width is less than three column diameters) the special confinement requirement should be satisfied.

ANNEX C

(*Clause* 22.1.1.1)

GRAPHICAL DETERMINATION OF DYNAMIC ACTIVE EARTH PRESSURE

C-1 MODIFIED CULMANN'S GRAPHICAL CONSTRUCTION (*see* Fig. 15)

Different steps in modified construction for determining dynamic active earth pressure are as follows:

a) Draw the wall section along with backfill surface on a suitable scale.
b) Draw *BS* at an angle ($\phi - \psi$) with the horizontal.
c) Draw *BL* at an angle of ($90 - \alpha - \delta - \psi$) below *BS*.
d) Intercept BD_1 equal to the resultant of the weight W_1 of first wedge ABC_1 and inertial forces ($\pm W_1\alpha_v$ and $W_1\alpha_h$). The magnitude of this resultant is $\overline{W}_1$

$$\overline{W}_1 = W_1\sqrt{(1 \mp \alpha_v)^2 + \alpha_h^2}$$

e) Through D_1 draw $D_1 E_1$ parallel to *BL* intersecting BC_1 at E_1.
f) Measure $D_1 E_1$ to the same force scale as BD_1. The $D_1 E_1$ is the dynamic earth pressure for trial wedge.
g) Repeat steps (d) to (f) with BC_2, BC_3, etc, as trial wedges.
h) Draw a smooth curve through E_1, E_2 and E_3. This is the modified Culmann's line.
j) Draw a line parallel to *BS* and tangential to this curve. The maximum coordinate in the direction of *BL* is obtained from the point of tangent and is the dynamic active earth pressure $(P_A)_{dyr}$.

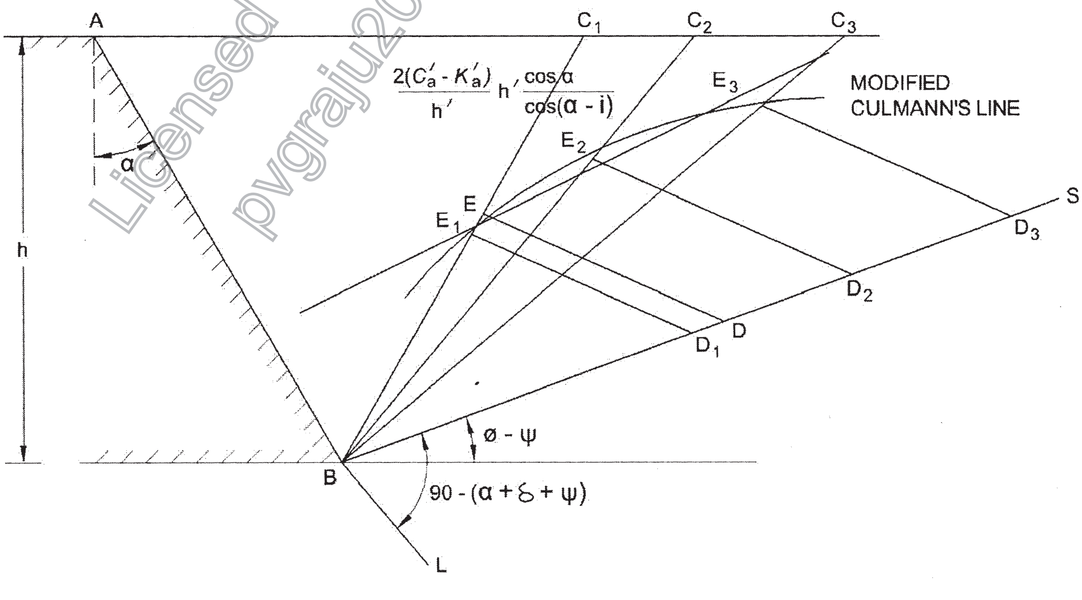


FIG. 15 MODIFIED CULMANN'S CONSTRUCTION FOR DYNAMIC ACTIVE EARTH PRESSURE

ANNEX D

(*Clause* 22.1.2.1)

GRAPHICAL DETERMINATION OF DYNAMIC PASSIVE EARTH PRESSURE

D-1 MODIFIED CULMANN'S GRAPHICAL CONSTRUCTION

For determining the passive earth pressure draw *BS* at ($\phi - \psi$) below horizontal. Next draw *BL* at ($90 - \alpha - \delta - \psi$) below *BS*. The other steps for construction remain unaltered (*see* Fig. 16).

Effect of uniformly distributed load and line load on the back fill surface may be handled in the similar way as for the static case.

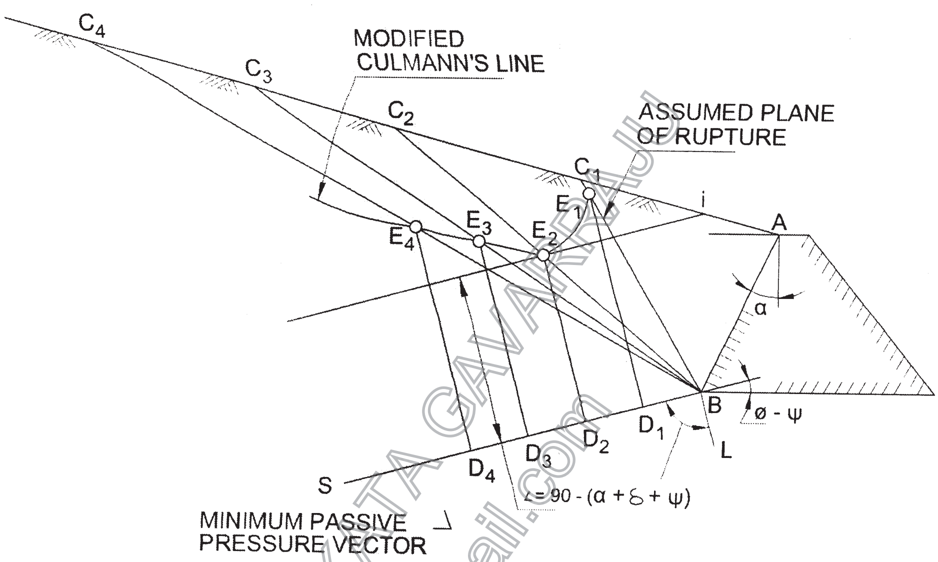


FIG. 16 MODIFIED CULMANN'S CONSTRUCTION FOR DYNAMIC PASSIVE EARTH PRESSURE

ANNEX E

(*Foreword*)

COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

Chairman

Dr A. S. Arya (up to 6 September 2010)
Building Materials and Technology Promotion Council, New Delhi

Dr D. K. Paul (after 6 September 2010)
Indian Institute of Technology Roorkee, Roorkee

Organization	*Representative(s)*
Association of Consulting Engineers, Bangalore	Shri Umesh B. Rao
	Shri B.V. Ravindra Nath (*Alternate*)
Atomic Energy Regulatory Board, Mumbai	Dr P.C. Basu
	Shri Roshan A. D. (*Alternate*)
Bharat Heavy Electrical Limited, New Delhi	Shri Ravi Kumar
	Dr C. Kameshwara Rao (*Alternate*)
Building Materials & Technology Promotion Council, New Delhi	Shri J. K. Prasad
	Shri Pankaj Gupta (*Alternate*)
Central Building Research Institute, Roorkee	Shri Navjeen Saxena
	Shri Ajay Chaurasia (*Alternate*)
Central Public Works Department, New Delhi	Shri Bhagwan Singh
	Shri S. P. Lokhande (*Alternate*)
Central Soils and Materials Research Station, New Delhi	Shri N. P. Honkandavar
	Shri S. L. Gupta (*Alternate*)
Central Water & Power Research Station, Pune	Shri I. D. Gupta
	Shri S. G. Chaphalkar (*Alternate*)
Central Water Commission, New Delhi	Director, CMDD (E & NE)
	Director, Embankment (*Alternate*)
DDF Consultants Pvt. Ltd, New Delhi	Dr (Shrimati) Pratima R. Bose
Delhi College of Engineering, Delhi	Shri Alok Verma
Department of Atomic Energy, Kalpakkam	Shri S. Ramanujam
	Shri R. C. Jain (*Alternate*)
Directorate General of Border Roads, New Delhi	Shri A. K. Dixit
Engineer-in-Chief's Branch, New Delhi	Brig. B. D. Pandey
	Shri Ravi Sinha (*Alternate*)
Engineers India Limited, New Delhi	Shri Vinay Kumar
	Ms Ila Dass (*Alternate*)
Gammon India Limited, Mumbai	Shri V. N. Haggade
	Shri J. N. Desai (*Alternate*)
Geological Survey of India, Lucknow	Shri Harsh Gupta
	Dr Kiran Mazumdar (*Alternate*)
Housing & Urban Development Corporation Ltd, New Delhi	Shrimati Bindu Jeswani
	Shri Surinder Gera (*Alternate*)
Indian Concrete Institute, Chennai	Dr A. R. Santhakumar
Indian Institute of Technology Bombay, Mumbai	Dr Ravi Sinha
	Dr Alok Goyal (*Alternate*)
Indian Institute of Technology Hyderabad, Hyderabad	Dr C. V. R. Murty
Indian Institute of Technology Kanpur, Kanpur	Dr Durgesh C. Rai
Indian Institute of Technology Madras, Chennai	Dr A. Meher Prasad
Indian Institute of Technology Roorkee, Roorkee	Prof Ashok Jain
Indian Institute of Technology, Gandhinagar,	Dr S. K. Jain

Organization	*Representative(s)*
Indian Meterological Department, New Delhi	Shri Surya Bali Jaiswar Shri Rajesh Prakash (*Alternate*)
Indian Road Congress, New Delhi	Secretary General Director (*Alternate*)
Indian Society of Earthquake Technology, Roorkee	Prof D. K. Paul Prof H. R. Wason (*Alternate*)
Maharashtra Engineering Research Institute, Nasik	Superintending Engineer (Earth Dam) Executive Director (Earth Dam) (*Alternate*)
Ministry of Road Transport and Highways, New Delhi	Shri R. K. Pandey Shri Virendra Kumar (*Alternate*)
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National Geophysical Research Institute (CSIR), Hyderabad	Dr M. Ravi Kumar Dr N. Puranchadra Rao (*Alternate*)
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School of Planning and Architecture, New Delhi	Dr V. Thiruvendgadam
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Tandon Consultants Pvt Limited, New Delhi	Dr Mahesh Tandon Shri Vinay K. Gupta (*Alternate*)
Tata Consulting Engineers, Mumbai	Shri K. V. Subramanian Shri C. K. Ravindranathan (*Alternate*)
Vakil-Mehta-Sheth Consulting Engineers, Mumbai	Ms Alpa R. Sheth Shri R. D. Chaudhari (*Alternate*)
Visvesvaraya National Institute of Technology, Nagpur	Dr O. R. Jaiswal Dr R. K. Ingle (*Alternate*)
Wadia Institute of Himalayan Geology, Dehradun	Dr Sushil Kumar
In personal capacity (*174/2 F, Solanipram, Roorkee*)	Dr S. K. Thakkar
In personal capacity (*36 Old Sneh Nagar, Wardha Raod, Nagpur*)	Shri L. K. Jain
In personal capacity (*C-2/155, West Enclave, Pitam Pura New Delhi*)	Dr K. G. Bhatia
In personal capacity (*K-L / 2 Kavi Nagar, Ghaziabad*)	Dr A. K. Mittal
BIS Directorate General	Shri A. K. Saini, Scientist 'F' & Head (Civil Engg) [Representing Director General (*Ex-officio*)]

Member Secretary
Shri S. Chaturvedi
Scientist 'E' (Civil Engg), BIS

(*Continued from second cover*)

b) The concept of ductility and over-strength is brought in the draft explicitly, by introducing the response reduction factors.
c) Different response reduction factors have been proposed for the different components of the bridge, depending on the redundancy, expected ductility and over-strength in them.
d) The design force level for bridge has been raised from the existing level and brought in line with IS 1893 (Part 1) : 2002.
e) The concept of capacity design is introduced in the design of connections, sub-structures and foundations.
f) The soil-foundation factor is dropped. The effect of soil on response is represented in the response spectrum.
g) Provision for dislodging of girders in the bearings is introduced.
h) Use of vertical hold-down devices, stoppers, restrainers and horizontal linkage elements to account for the large displacements generated during seismic shaking is recommended for preventing falling of spans.
j) A minimum width of seating of superstructure over sub-structures to avoid dislodging of spans from atop the sub-structure is required for all bridges.
k) The method of computing earth pressures for $c-\phi$ soil is included in the section on retaining walls.

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 units used with the items covered by the symbols shall be consistent throughout this standard, unless specifically noted otherwise.

The composition of the Committee responsible for the formulation of this standard is given in Annex E.

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 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Bureau of Indian Standards

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Copyright

Review of Indian Standards

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