



सत्यमेव जयते

GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS

RDSO GUIDELINES ON SEISMIC DESIGN OF RAILWAY BRIDGES

(Version 1.0)

REPORT NO. BS – 118

NOVEMBER - 2015

**BRIDGE & STRUCTURES DIRECTORATE
RESEARCH DESIGNS AND STANDARDS ORGANISATION
LUCKNOW-226011**



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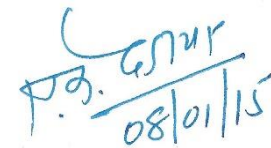
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PREFACE

The Guidelines on Seismic Design of Railway Bridges were initially prepared by IIT-Kanpur in joint consultation with RDSO in year 2010 and circulated to all the Railways for their comments on the same. Advice were sought from the different PSU's and Metro Railways as well. The provisions of the IITK-RDSO guidelines were put up before the Bridge Standard Committee for its acceptance and subsequent adoption in Bridge rules and other design codes. However, in 80th and 81st BSC only a very small portion of this Guideline was accepted. After obtaining advice from all concerned and also based upon the provisions of IRC-6, it was felt that the response reduction factor used in the IIT-K RDSO Guidelines were too conservative. There was a major disagreement on Live load factor as well. Accordingly, some design parameters of IIT-K RDSO guidelines were modified and presented before the 82nd BSC in January-2014. The proposed modifications in IITK-RDSO Guidelines were approved by the BSC along with recommendation to incorporate the new design provisions in all the relevant Codes and Manuals through correction slips.

The current Seismic design provisions of Bridge rules are based upon IS 1893-1984. IS code was completely revised in 2002 incorporating the latest design philosophy. The new methods of IS 1893-2002 were based on the international practices and it took into account the Flexibility and Ductility of the structure while calculating design forces. Indian Road Congress also adopted the new IS provisions replacing the old provisions in IRC-6. Indian Railways have not yet upgraded its Seismic design codes to the current Indian and International standards. The current provisions of the Bridge rule, where Seismic accelerations are not related to the flexibility of Bridge is not very rational and it has now become obsolete. Provision of ductility in the structure allows us to take lesser forces in the design, as the structure can survive severe seismic shaking through large deflections in plastic range, consequently dissipating more energy. But our current Bridge rule does not take any advantage of ductility of the structure in design, giving no incentive for providing ductility features in the Bridge Substructure.

The provisions of this Guidelines will be soon incorporated in the Bridge Rules, Codes and Manuals through correction slips. The Guidelines have been simplified and made more concise so that it can be easily grasped and put to use in the design offices. With the issue of this Guideline, the earlier issued IIT-K RDSO Guidelines stands withdrawn.



A.K Dadarya
(ED/ B&S/ RDSO)

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1. Terminology

For the purpose of these guidelines, the following terms are defined

Base

The level at which inertia forces generated in the substructure and superstructure are transferred to the foundation.

Centre of Mass

The point through which the resultant of the masses of a system acts. This point corresponds to the center of gravity of the system.

Closely-Spaced Mode

Closely-Spaced modes of a structure are those of its natural modes of vibration whose natural frequencies differ from each other by 10 percent or less of the lower frequency.

Critical Damping

The minimum damping above which free vibration motion is not oscillatory.

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.

Design Acceleration Spectrum

Design acceleration spectrum refers to an average smoothened plot of maximum acceleration as a function of natural frequency or time period of vibration for a specified damping ratio for Earthquake excitations at the base of a single degree of freedom system.

Design Basis Earthquake (DBE)

It is the Earthquake which can reasonably be expected to occur at least once during the design Life of the structure

Design Horizontal Acceleration Coefficient

It is a horizontal acceleration coefficient that shall be used to obtain design horizontal seismic force on structures.

Design Seismic Force

The seismic force prescribed by this standard for each bridge component that shall be used in its design. It is obtained as the maximum elastic seismic force divided by the appropriate response reduction factor specified in this standard for each component.

Design Seismic Force Resultant (V)

The force resultant (namely axial force, shear force, bending moment or torsional moment) at a cross-section of the bridge due to *design seismic force* for shaking along a considered direction applied on the structure.

Ductility

Ductility of a structure, or its members, is the capacity to undergo large inelastic deformations without significant loss of strength or stiffness.

Ductile Detailing

The preferred choice of location and amount of reinforcement in reinforced concrete structures to provide for adequate ductility in them. In steel structures, it is the design of members and their connections to make them adequately ductile.

Epicentre

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

Focus

The point inside earth on the fault where the slip starts that causes the earthquake.

Importance Factor (I)

A factor used to obtain the design seismic force depending on the importance of the structure.

Linear Elastic Analysis

Analysis of the structure considering linear properties of the material and load-versus deformation characteristics of the different component of the structure.

Liquefaction

Liquefaction is the state in saturated cohesion less soil wherein the effective shear strength is reduced to negligible value during an earthquake due to pore pressures caused by vibrations approaching the total confining pressure. In this situation, the soil tends to behave like a fluid mass.

Magnitude

The magnitude of earthquake is a number which is a measure of energy released in an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer would register due to the earthquake at an epicentral distance of 100 km.

Maximum Considered Earthquake (MCE)

Maximum considered earthquake is the largest reasonably conceivable earthquake that appears possible in the Earthquake Zone.

Maximum Elastic Force Resultant (F_{net}^e)

The force resultant (namely axial force, shear force, bending moment or torsional moment) at a cross-section of the bridge due to *maximum elastic seismic force* for shaking along a considered direction applied on the structure.

Maximum Elastic Seismic Force (F^e)

The maximum force in the bridge component due to the expected seismic shaking in the considered seismic zone.

Modal Mass (M_k)

Modal mass of structure subjected to horizontal or vertical ground motion is a part of total seismic mass of the structure that is effective in mode k of vibration. The modal mass for a given mode has a unique value irrespective of scaling of the mode shape.

Mode Shapes Coefficient (Φ_{jk})

The spatial pattern of vibration when the structure is vibrating in its normal mode k is called as mode shape of vibration of mode k . Φ_{jk} is coefficient for j^{th} node in k^{th} mode.

Natural Period

Natural period of a structure is its time period of undamped vibration.

- (a) Fundamental Natural Period: It is the highest modal time period of vibration along the considered direction of earthquake motion.
- (b) Modal Natural Period: The modal natural period of mode k is the time period of vibration in mode k .

Normal Mode

Mode of vibration at which all its masses attain maximum values of displacements and rotations simultaneously, and they also pass through equilibrium positions simultaneously.

Over strength

Strength considering all factors that may cause an increase, e.g., steel strength being higher than the specified characteristic strength, effect of strain hardening in steel with large strains, and concrete strength being higher than specified characteristic value.

P- Δ Effect

It is the secondary effect on shears and moments of frame members due to action of the vertical loads, interacting with the lateral displacement of structure resulting from seismic forces.

Response Spectrum Acceleration Coefficient (S_a/g)

It is factor denoting the design acceleration spectrum of the structure subjected to earthquake ground motion, and depends on natural period of vibration and damping of structures.

Response Reduction Factor (R)

The factor by which the actual lateral force, that would be generated if the structure were to remain elastic during the most severe shaking that is likely at that site, shall be reduced to obtain the design lateral force.

Response Spectrum

It is a representation of the maximum response of idealized single degree of freedom systems of different periods for a fixed value of 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.

Restrainer

A steel rod, steel cable, rubber-impregnated chain, or similar device that prevents a superstructure from becoming unseated during an earthquake.

Seismic Mass

Seismic weight divided by acceleration due to gravity.

Seismic Weight (W)

Total dead load plus part of live load.

Skew

The angle between the centerline of the superstructure and a horizontal line perpendicular to the abutments or bents.

Soil Profile Factor

A factor used to obtain the elastic acceleration spectrum depending on the soil profile underneath the structure at the site.

Strength

The usable capacity of a structure or its members to resist the applied loads.

Stiffness of Piers

The force required to produce unit deformation in the pier under a lateral load applied at its top.

Substructure

Elements such as piers, abutments, and foundations that support the superstructure.

Superstructure

The bridge elements supported by the substructure.

Zone Factor (Z)

It is a reasonable estimate of effective peak ground acceleration for the Maximum Considered Earthquake (MCE) in the Earthquake Zone. " $Z/2$ " is the effective peak ground acceleration of Design Basis Earthquake (DBE). " $Z/2$ " is multiplied by the Response acceleration coefficient S_a/g to obtain design response spectrum.

2. Symbols

a	Structural width in the direction of hydrodynamic pressure
A	Elastic seismic acceleration coefficient
A_o	Sectional area of the substructure
A_c	Area of the concrete core $= \frac{\pi}{4} D_k^2$
A_g	Gross area of the column cross section
A_k	Elastic seismic acceleration coefficient of mode
A_r	As per APPENDIX C, Area of confined core concrete in the rectangular hoop measure to its outer side dimensions
A_{sh}	Area of cross-section of circular hoop
B	Structural width perpendicular to hydrodynamic pressure
B	Bonded plan dimension or bonded diameter in loaded direction of rectangular bearing or diameter of circular bearing,
B_l	Damping coefficient (Table -10)
C_e	Hydrodynamic force coefficient
C_j	Fraction of missing mass for j^{th} mode.
C_1, C_2, C_3, C_4	Pressure coefficients to estimate flow load due to stream on the substructure
D_k	Diameter of core measured to the outside of the spiral or hoops
d_i	Thickness of any layer
E_c	Modulus of elasticity of concrete
EDC	Energy dissipated per cycle (Figure – 10)
E_x, E_y	Earthquake force in x-and y-direction respectively
E_s	Modulus of elasticity of steel
F	Hydrodynamic force on substructure; (also, Horizontal force in kN applied at center of mass of superstructure for one mm horizontal deflection of bridge along considered direction of horizontal force)
F_e	Inertia force due to mass of a bridge component under earthquake shaking along a direction
F_{missing}	Lateral force associated with missing mass
f_{ck}	Characteristic strength of concrete at 28 days in MPa.
f_y	Yield stress of steel
$\{F_k^e\}$	Inertia force vector due to mass of bridge under earthquake shaking along a direction in mode k
F_p	Maximum Positive force
F_n	Maximum Negative force
F_{net}^e	Maximum elastic force resultants at a cross-section due to all modes considered

F_{max}	Maximum force
F_y	Yield Force
g	Acceleration due to gravity
h	Longer dimension of the rectangular confining hoop measured to its outer face
H_p	Height of Pier
I	Importance Factor
K	Bulk modulus of elastomer
K_d, K_u	Post – elastic stiffness, Elastic (unloading) stiffness , Effective stiffness resp.
K_{eff}	(Clause 17.1 and Figure – 10)
K_i^e	Smaller effective stiffness
K_j^e	larger effective stiffness
L	Length (in <i>meters</i>) of the superstructure to the adjacent expansion joint or to the end of superstructure. In case of bearings under suspended spans, it is sum of the lengths of the two adjacent portions of the superstructure. In case of single span bridges, it is equal to the length of the superstructure
m	Number of modes of vibration considered
m_j	Total mass of the j^{th} mode
$[m]$	Seismic mass matrix of the bridge structure
M_y	Moment Capacity of the column/pier section at the first yield of the reinforcing steel
$\sum M^0$	Sum of the over strength moment capacities of the hinges resisting lateral loads
$\bar{N}$	Average SPT value of the soil profile
N_i	Standard penetration resistance of layer i
P_k	Modal participation factor of mode k of vibration
p_b	Pressure due to fluid on submerged superstructures
Q_d	Characteristic strength
R	Response Reduction Factor
r_1, r_2, r_3	Force resultants due to full design seismic force along two principal horizontal directions and along the vertical direction, respectively
S	Pitch of spiral or spacing of hoops
$\frac{S_a}{g}$	Bridge flexibility factor along the considered direction
$\left(\frac{S_a}{g} \right)_k$	Bridge flexibility factor of mode k of vibration
t_i	Thickness of i^{th} layer

T_1	Fundamental natural period of vibration of bridge in considered direction
T_k	Natural Period of Vibration of mode k
T_r	Total elastomer thickness
$u(s)$	Displacement at position s caused in the acting direction of inertial force when the force corresponding to the weight of the superstructure and substructure above the ground surface for seismic design is assumed to act in the acting direction of inertial force
V	Lateral Shear Force
V^e	Maximum elastic force resultant at a cross-section of a bridge component
V_{net}	Design seismic force resultant in any component of the bridge due to all modes considered
W	Seismic weight, which includes full dead load and part live load
W_b, W_1, W_2	Widths of seating at bearing supports at expansion ends of girders.
W_e	Weight of water in a hypothetical enveloping cylinder around a substructure
Z	Seismic zone factor
$\{1\}$	Vector consisting of unity (one) associated with translational degrees of freedom in the considered direction of shaking, and zero associated with all other degrees of freedom
δ	Displacement at the acting position of inertial force of the superstructures when the force corresponding to 80% of the weight of the substructure above the ground surface for seismic design and all weight of the superstructure portion supported by it is assumed to act in the acting direction of inertial force (m)
Δ_p	Maximum positive displacement
Δ_n	Maximum negative displacement
Δ_{max}	Maximum bearing displacement (Figure 10)
Δ_y	yield displacement
ΔF_{ed}	Additional vertical load due to seismic overturning effects, base on peak response under the design seismic action
β	Ratio of natural frequencies of modes i and j , Also equivalent damping ratio (Sec.17.5.8)
$\{\phi_k\}$	Mode shape vector of the bridge in mode k of vibration
ϕ_{jk}	Mode shape coefficient for j^{th} , degree of freedom in k^{th} mode of vibration
ϕ_y	Yield Curvature
λ	Net response due to all modes considered
λ_k	Response in mode k of vibration
$\lambda_{missing}$	Maximum response of missing mass

3. Introduction

The present guidelines deal with the seismic design of new Railway bridges and these may not be used for seismic evaluation of existing bridges. Bridges and portions thereof shall be designed and constructed, to resist the effects of design seismic force specified in these guidelines as a minimum. The design approach adopted is to ensure that structure possess at least a minimum strength to withstand a minor earthquake (<DBE), which occur frequently, without damage; resist moderate earthquake (DBE) without significant structural damage though some damage may occur; and aims that structure withstand a major earthquake without collapse. Actual forces that appear on the bridges during an earthquake are much greater than the design forces specified in here.

However, ductility, arising from inelastic material behaviour and detailing, and overstrength, arising from the additional reserve strength in structures over and above the design strength are relied upon to account for this difference in actual and design lateral loads.

4. General Principles

4.1 Ground Motion

The characteristics (intensity, duration, etc..) of seismic ground vibrations expected at any location depends upon the magnitude of earthquake, the depth of focus, distance from the epicentre, characteristics of the path through which the seismic waves travel, and the soil strata on which the structure stands. The random ground motions, which cause the structures to vibrate, can be resolved in any three mutually perpendicular directions. Generally, two horizontal and one vertical component of ground motion is considered. The predominant direction of ground vibration is usually horizontal.

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

4.3 Masonry and plain concrete arch bridges with spans more than 12 m shall not be built in the seismic zones IV and V.

4.4 Box and pipe culverts need not be analysed for seismic forces.

4.5 Following bridges need not be analysed for seismic forces:

- (a) In Zones II & III, bridges with overall length less than 60m or spans less than 15m
- (b) Single span bridges upto 30m span

However, these bridges shall be provided with:

- i. The minimum seating width as per Clause 14.3
- ii. The connections in the restrained direction between superstructure and substructure shall be designed for elastic seismic force from superstructure.

- 4.6** For specific cases of bridges, some additional studies/analysis should be required, which are described in Table 1.

Table 1 - Cases Requiring Special Studies/Analysis

Sr. No.	Cases in which additional special studies/analysis is required	Special studies/analysis
1.	In zone IV and V, bridges with individual span length more than 120 m and/or pier height is more than 30 m.	Modelling of the bridge including geometrical nonlinearity, P-delta effect and soil-structure interaction is needed. Pushover analysis may be done to ascertain the energy dissipation characteristics of ductile members. (Details given in APPENDIX E)
2.	Continuous deck bridge of length larger than 600 m	Spatial variation of ground motion shall be considered.
3.	Geological discontinuity exists at the site	Spatial variation of ground motion shall be considered.
4.	Bridge site close to a fault (< 10 km) which may be active.	Site specific spectrum shall be obtained. Else, near source modifications as per Clause 6.1 and 6.8.4 shall be done. Specialist literature shall be required to obtain site specific spectrum. If bridge is crossing the fault, detailed geological studies shall be performed to estimate past movements across the fault. Bridge to be designed so as to withstand the expected fault displacements. Help from geological / seismological persons with enough experience will be required to calculate fault movement.
5.	In zone IV and V, if the soil condition is poor, consisting of marine clay or loose sand (e.g., where the soil up to 30m depth has average SPT N value equal to or less than 20)	Site specific spectrum shall be obtained.
6.	Site with loose sand or poorly graded sands with little or no fines. Liquefiable soil.	Liquefaction analysis is required (Details given in APPENDIX G). Liquefaction is the act or process of transforming any substance into a liquid state. In non -cohesive soils it is the transformation of the soil in the solid state to the liquefied state due to the increase in the

		pore pressure and the consequent reduction in the effective stress.
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4.7 Assumptions

The following assumptions are made in the earthquake-resistant design of bridges:

- Earthquake causes impulsive ground motions, which are complex and random in character, changing in period and amplitude, and each lasting for a 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.

Note: However, there are exceptions where resonance-like conditions have been seen to occur between long distance waves and tall structures founded on deep soft soils.

- Earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves. Similarly, earthquake motion need not be considered to occur simultaneously with other extreme environmental conditions such as thermal, which have low probability of occurrences.
- 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 dynamic conditions.

5. Conceptual Considerations

Conceptual design suggestions in terms of preferred configuration, superstructure, substructure and ground conditions are given in Table below, along with the non-preferred types, for which special design and detailing are required. These considerations shall be followed as much as practically possible and a balance shall be maintained between functional requirements, cost and seismic resistance features.

Seismically preferred and not Preferred Aspects of Bridges

	Seismically preferred	Seismically not preferred
1.0	Straight bridge alignment	Curved bridge alignment
1.1	Normal piers	Skewed piers
1.2	Uniform pier stiffness	Varying pier stiffness
1.3	Uniform span stiffness	Varying span stiffness
1.4	Uniform span mass	Varying span mass
2.0	Superstructure	
2.1	a) Simply supported spans b) Integral bridges	Continuous spans
2.2	Short spans	Long spans
2.3	Light spans	Heavy spans
2.4	No intermediate hinges within span	Intermediate hinges
3.0	Substructure	
3.1	Wide seats	Narrow seats
3.2	Multiple column	Single column
4.0	Ground conditions	
4.1	Stiff, Stable soil	Unstable soil

6. Design Criteria

6.1 Seismic Zone Map

For the purpose of determining design seismic forces, the country is classified into four seismic zones. A seismic zone map of India is shown in Fig. 1. The peak ground acceleration (PGA) (or zero period acceleration, ZPA), associated with each zone, is called zone factor, Z. The zone factor is given in Table 3. Zone factors for some important towns are given in Appendix D

Table 3 - Zone Factor Z for Horizontal Motion

Seismic Zone	II	III	IV	V
Z	0.10	0.16	0.24	0.36

Note: Near Source Effect

For bridges which are within a distance of 10 km from a known active fault, seismic hazard shall be specified after detailed geological study of the fault and the site condition. In absence of such detailed investigation, the near-source modification in the form of 20% increase in zone factor may be used.

6.2 - Importance Factor

The values of importance factor I, for different bridges are given in Table 4. The importance factor reflects strategic importance of the route and functionality of the bridge in the post-earthquake period.

Table 4

Category	Importance Factor	Bridges included
Category I Bridge	1.5	i) All important bridges irrespective of route. ii) Major bridges on group A, B and C routes. (For route classification see IRPW Manual)
Category II Bridge	1.25	i) Major bridges on all other routes. ii) All other bridges on group A, B and C routes.
Other Bridge	1.0	All other bridges

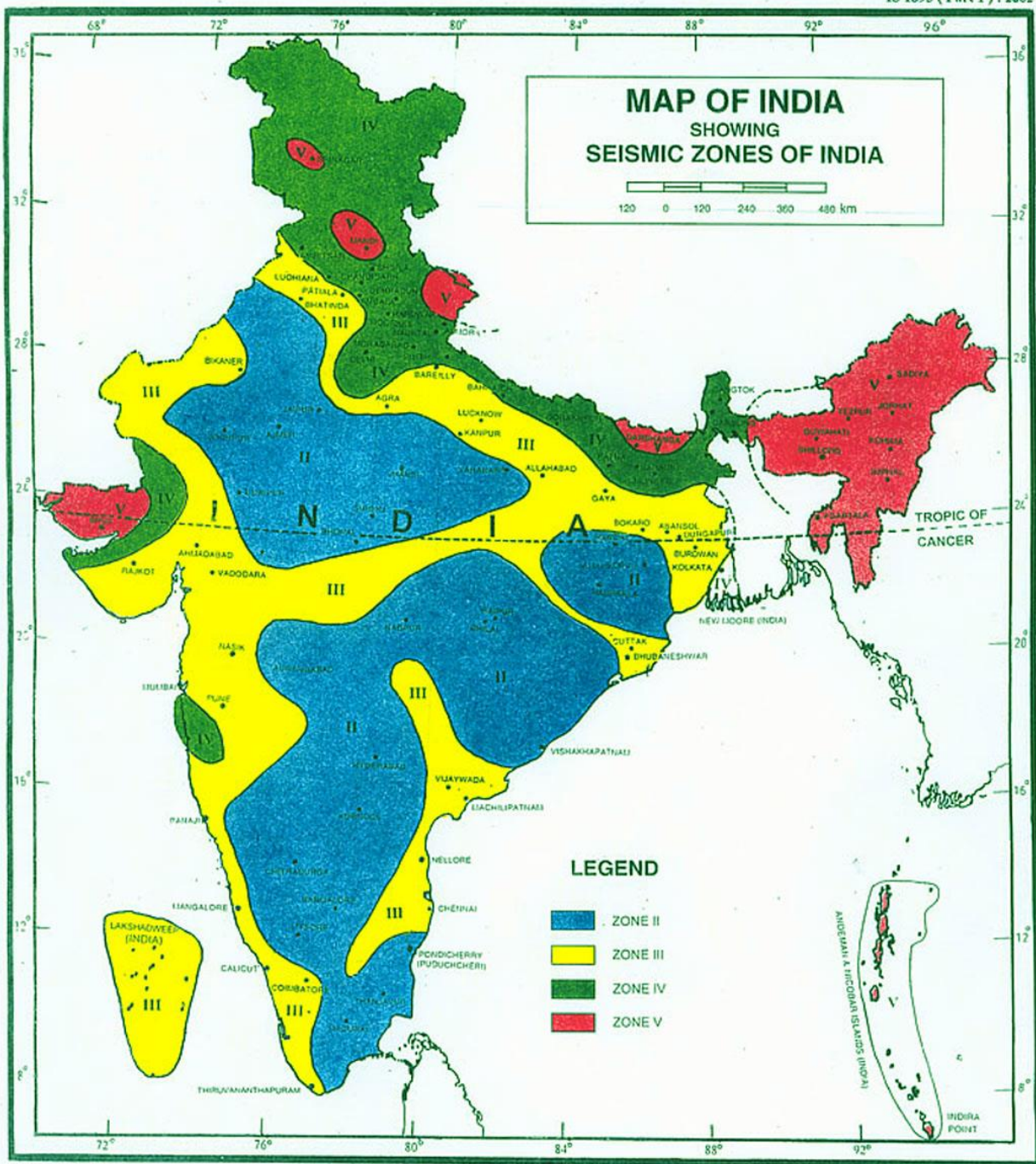


Fig. 1 Seismic zone map of India

6.3 Methods of Calculating Design Seismic Force

6.3.1 The seismic forces for bridges shall be generally estimated by Seismic Coefficient Method (Single Mode Method using response spectrum) described in Section 7.0. Response Spectrum Method (Multi Mode Method) described in Section 8 shall be used in zones IV and V in following cases:

- (a) Irregular bridge as defined in section 6.3.5.2
- (b) Individual span more than 80m
- (c) Continuous bridge
- (d) Height of top of pier / abutment from the base of foundation is more than 30m.

6.3.2 The Time History method described in Section 9.0 shall be used in following cases:

- (i) To verify the result of Response Spectrum Method for irregular bridges in zone IV, and V.
- (ii) Bridges with special devices like Shock Transmission Units (STU), and seismic isolation devices, time history method is mandatory.

6.3.3 The Pushover analysis described in Section 10.0 may be used to ascertain the nonlinear load carrying capacity and ductility of pier with more than 50 m height from the base of foundation and individual span more than 120 m.

6.3.4 For applying seismic forces obtained using Seismic Coefficient Method or Response Spectrum Method and for applying earthquake ground motion in Time History Method (THM), the mathematical model of bridges shall be used. This model shall appropriately model the stiffness of superstructure, bearings, piers and bridge ends. Analysis of bridge model under dead load, live load and seismic loads gives bending moment, shear and axial forces in various bridge components.

6.3.5 Regular and Irregular Bridges

6.3.5.1 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 purposes of this guidelines, if

- (a) It is straight or it describes a sector of an arc which subtends an angle less than 90° at the center of the arc, and
- (b) The adjacent piers do not differ in stiffness by more than 25%. (Percentage difference shall be calculated based on the lesser of the two stiffness's as reference).
- (c) If multi-column piers are used then the stiffness of the stiffest columns within piers shall not be 25% more than the stiffness of the most flexible column in that pier.

6.3.5.2 Irregular Bridge

All bridges not conforming to Clause 6.3.5.1 shall be considered irregular. Further, arch bridges of span exceeding 30m, cable stayed bridges, suspension bridges, and other innovative bridge forms shall also be treated as irregular bridges.

6.4 Seismic Weight and Live Load

6.4.1 Seismic Weight

The seismic weight of the superstructure shall be taken as its full dead load plus appropriate amount of live load. The seismic weight of the substructure and of the foundation shall be their respective full dead load. Buoyancy and uplift shall be ignored in the calculation of seismic weight.

Note— In the Seismic Coefficient Method (Clause 7.0), for simply supported regular bridges, single degree of freedom (SDOF) model is used to obtain time period and in this model only 80% of pier weight is considered in the seismic weight of Pier (for calculating forces within piers and foundations). The forces within the superstructure shall be calculated only for the seismic mass of the superstructure components and shall not include any mass from Piers/ Substructure.

6.4.2 Live load in seismic weight

- (i) No live load (train load) shall be considered while calculating horizontal seismic forces along the direction of traffic (Longitudinal direction).
- (ii) The horizontal seismic forces in the direction perpendicular to traffic (transverse direction) shall be calculated using 50 percent live load (excluding impact effect).
- (iii) The vertical seismic force shall be calculated using 50 percent live load (excluding impact effect).

6.4.3 Seismic Mass

The seismic mass of a bridge component is its seismic weight divided by the acceleration due to gravity. Generally, analysis for horizontal seismic forces is adequate. When vertical motion is to be considered, the design seismic forces shall be combined as per clause 6.5.3.

6.5 Combination of Seismic Components

The seismic forces shall be assumed to act in any direction. For design purpose, the analysis is done for earthquake motion in two orthogonal horizontal directions and one vertical direction.

6.5.1 For regular bridges, the two orthogonal horizontal directions are usually the longitudinal and transverse direction of the bridges (Fig 2a). For such bridges analysis shall be done for seismic forces in longitudinal and transverse directions. The seismic force resultants (Bending Moment, Shear Force and Axial Force) at any component obtained from the analysis in longitudinal and transverse directions shall be considered separately.

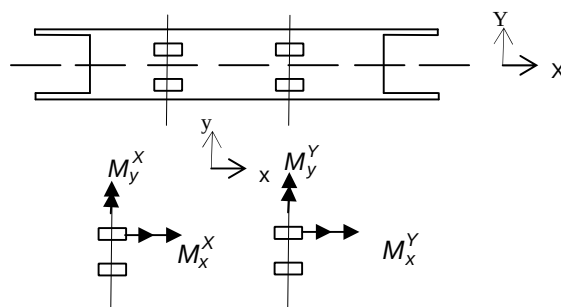


Fig 2 A: Seismic forces for straight bridge (Clause 6.5.1)

M_y^x = Bending Moment in y-direction when force is applied in X Direction

M_x^X = Bending Moment in x-direction when force is applied in X Direction

M_y^Y = Bending Moment in y-direction when force is applied in Y Direction

M_y^X = Bending Moment in x-direction when force is applied in Y Direction

For Straight Bridge, M_y^X and M_x^Y are zero.

6.5.2 For irregular bridges, particularly, skew bridge (Fig. 2b), design seismic force resultants shall be obtained along x-and y-direction. The design seismic force resultant (Bending Moment, Shear Force and Axial Force) at any component shall be obtained as follows:

(a) $\pm r_1 \pm 0.3r_2$

(b) $\pm 0.3r_1 \pm r_2$

where

r_1 = Force resultant due to full design seismic force along x direction,

r_2 = Force resultant due to full design seismic force along y direction.

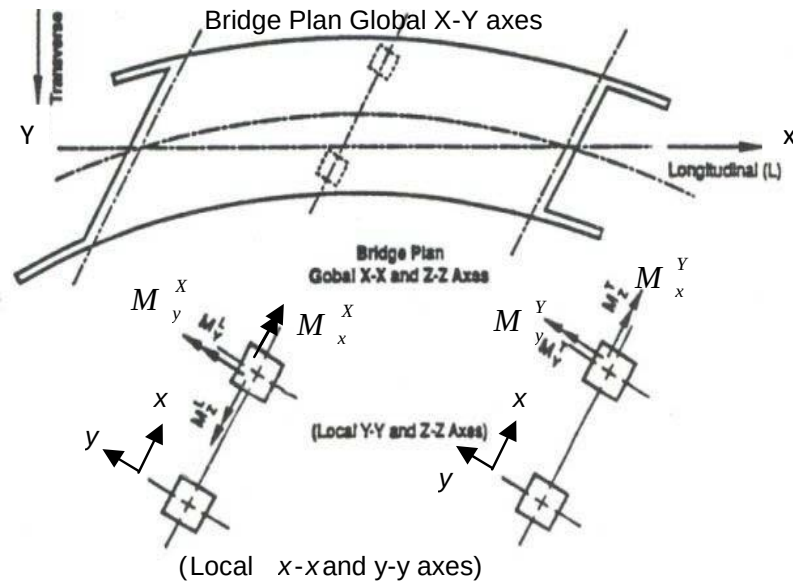


Fig. 2 b: Combination of orthogonal seismic forces for Skew Bridge (Clause 6.5.2).

Design Seismic Force Resultant for Bending Moment

	Moments for ground motion along X-axis	Moments for ground motion along Y-axis
Design Moments	$M_x = M_x^X + 0.3M_y^X$	$M_y = M_y^Y + 0.3M_x^Y$
	$M_x = 0.3M_x^X + M_y^Y$	$M_y = 0.3M_y^Y + M_x^X$
	Where, M_x and M_y are absolute moments about local axes.	

6.5.3 When vertical seismic forces are also considered, (Clause 6.8.1), then for regular bridges, the design seismic force resultants shall be obtained for the X-, Y- and Z-direction separately. For irregular bridges, the design seismic force resultant at any component shall be computed as follows:

$$(a) \pm r_1 \pm 0.3r_2 \pm 0.3r_3$$

$$(b) \pm 0.3r_1 \pm r_2 \pm 0.3r_3$$

$$(c) \pm 0.3r_1 \pm 0.3r_2 \pm r_3$$

Where r_1 and r_2 are as defined in Clause 6.5.2, and r_3 is the force resultant due to full design seismic force along the vertical or z-direction.

6.5.4 As an alternative to the procedure in 6.5.2 and 6.5.3, 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

$$\sqrt{r_1^2 + r_2^2} \text{ or } \sqrt{r_1^2 + r_2^2 + r_3^2}$$

Where r_1, r_2 and r_3 are as defined in Clause 6.5.2 or 6.5.3.

6.6 Damping and soil Properties

6.6.1 Damping

In general, 5% damping shall be considered.

6.6.1.1 If well foundation is used, then 10% damping shall be used.

6.6.1.2 In case the guard rails are effectively provided, on single span of bridge up to 30 m overall length, 10 % damping in longitudinal direction can be considered. However, in the transverse direction damping will not change.

6.6.2 Increase in Allowable Pressure in Soils

When earthquake force is included then allowable pressure in soil and rock shall be increased as stipulated in Table 5. Bearing pressure for foundation and pile capacity shall be determined by working stress method only.

6.6.3 The values for allowable bearing pressure in soil given in Table 5 applies to the upper 30m of the soil profile. Profiles containing distinctly different soil layers shall be subdivided into layers, each designated by a number that ranges from 1 (at the top) to n (at the bottom), where there are a total of n layers in the upper 30 meters, and a weighted average will be obtained as follows:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}}$$

where $\sum_{i=1}^n d_i$ is equal to 30 m, N_i is the standard penetration resistance of layer i , not to

exceed 100 blows per 300 mm as directly measured in the field without correcting, and d_i is the thickness of any layer i between 0 and 30m.

6.6.4 Soil Structure Interaction

Soil flexibility should be considered in the seismic analysis of bridges, whenever deemed necessary. This is particularly important for foundations in soft soil conditions and in cases where deep foundations are used. Soil flexibility leads to longer natural period and hence lowers seismic forces, however, on the other hand, it results in larger lateral deflections. Soil parameters, like, elastic properties and spring constants shall be properly estimated. In many cases, one gets a range of values for soil properties. In such cases, the highest values of soil stiffness shall be used for calculating the natural period and seismic forces, and lowest value shall be used for calculating the deflection.

Table 5 – Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

S. No.	Foundation	Type of soil Mainly Constituting the Foundation		
		Type I Rock or Hard Soil	Type II Stiff Soil	Type III Soft Soils
(1)	(2)	(3)	(4)	(5)
i)	Piles passing through any soil but resting on soil type I	50	50	--
ii)	Piles not covered under item i	--	25	--
iii)	Raft foundations	50	50	--
iv)	Combined isolated RCC footing with tie beams	50	25	--
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations.	50	25	--
vi)	Well foundation	50	25	--

NOTES

1. The allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.
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.
3. Desirable minimum field values of N- If soils of smaller N-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.

Seismic Zone Level	Depth below Ground (in meters)	N-Values	Remarks
III, IV and V	≤ 5	15	For values of depths between 5m and 10m, linear interpolation is recommended.
	≥ 10	20	
II (for important bridges only)	≤ 5	15	
	≥ 10	20	

4. The values of N (uncorrected values) are at the founding level and the allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.
5. The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy.
6. IS 1498 and IS 2131 may also be referred.

Type of Soil	
Soil Type	Definition
Type I: Rock or Hard Soils	<ul style="list-style-type: none"> Well graded gravel (GW) or well graded sand (SW) both with less than 5% passing 75 μm sieve (Fines); Well graded Gravel – Sand mixtures with or without fines (GW-SW); Poorly graded Sand (SP) or clayey sand (SC), all having N above 30; Stiff to hard clays having N above 16, where N is the Standard Penetration Test value.
Type II: Stiff Soils	<ul style="list-style-type: none"> Poorly graded sands or Poorly graded sands with gravel (SP) with little or no fines having N between 10 and 30; Stiff to medium stiff fine-grained soils, like Silts of Low compressibility (ML) or Clays of Low Compressibility (CL) having N between 10 and 16.
Type III: Soft Soils	<p>All soft soils other than SP with $N < 10$. The various possible soils are</p> <ul style="list-style-type: none"> Silts of Intermediate compressibility (MI); Silts of High compressibility (MH); Clays of Intermediate compressibility (CI); Clays of High compressibility (CH); Silts and Clays of Intermediate to High compressibility (MI-MH or CI-CH); Silt with Clay of Intermediate compressibility (MI-CI); Silt with Clay of High compressibility (MH-CH).

6.7 Combination of Seismic Design Forces with Other Forces

The design seismic force resultant at a cross-section of a bridge component shall be appropriately combined with those due to other forces as per Table 12 of IRS Concrete Bridge Code (reprint 2014). However, in lieu of combination 2 of Clause 11.0 of IRS Concrete Bridge Code, following load combinations shall be used:

(A) Ultimate limit state design

- 1) $1.25DL + 1.5 DL(S) + 1.5EQ + 1.4 PS + 1.7 EP$
- 2) $1.25DL + 1.5DL(S) + 0.5(LL + LL (F)) + 1.2EQ + 1.7 EP + 1.4PS + 1.4HY + 1.4BO$
- 3) $0.9DL + 0.8DL(S) + 1.5EQ + 1.4 PS + 1.7 EP$

(B) Serviceability Limit State

- 1) $1.0 DL + 1.2 DL(S) + 1.0 EQ + 1.0 EP + 1.0PS + 1.0HY + 1.0BO$
- 2) $1.0 DL + 1.2 DL(S) + 0.5(LL + LL(F)) + 1.0EQ + 1.0 EP + 1.0PS + 1.0HY + 1.0 BY$

(C) During the construction stage, following load combination shall be used:

$$1.0DL + 1.2DL(S) + 0.8EQ + 1.0ER + 1.3EP + 1.0PS + 1.0HY + 1.0BO$$

Where,

DL = dead load,

$DL(S)$ = superimposed dead load,

LL = live load, The live load (LL) includes impact effect, longitudinal forces (tractive and braking), and centrifugal force.

$LL (F)$ = live load on footpath,

EQ = earthquake load,
EP = earth pressure,
ER = erection load such as cranes, machines etc.
PS = prestressing load,
HY = hydrodynamic load, BO = buoyancy load,
SH = shrinkage load,
CR = creep load,
TE = temperature load.

6.8 Vertical Motions

The seismic zone factor for vertical ground motions, when required (as explained below), may be taken as two-thirds of that for horizontal motions given in Table 3.

6.8.1- In zone IV and V the effects of vertical component shall be considered for all elements of the bridge. The effect of the vertical seismic component may be omitted for all elements in zones II and III, except for the following cases,

- Prestressed concrete girders / Composite girders.
- Bearings, hold down devices, and linkages.
- Horizontal cantilever structural elements such as cantilevers of deck slabs and cantilever bridges.
- Situations where stability (overturning /sliding) becomes critical.
- Bridge sites located near fault.

6.8.2- For superstructure with span up to 80 m, the effect of vertical motion can be considered by analysing the superstructure for 25% additional dead weight in upward and downward direction.

6.8.3- For superstructure with span more than 80m, analysis for vertical ground motion shall be done.

Such analysis requires time period of superstructure in vertical direction. Time period for the superstructure has to be worked out separately using the property of the superstructure, in order to estimate the seismic acceleration coefficient (S_a/g) for vertical acceleration. It can be done by free vibration analysis of superstructure using standard structural analysis software. 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 is the span, m is the mass per unit length, and EI is the flexural rigidity of the superstructure.

When ultimate limit state is used, effective flexure rigidity equal to 50% of gross flexural rigidity shall be taken for concrete superstructure (RCC and Prestressed girders and slabs).

6.8.4 For locations, within 10 km of active fault, seismic zone factor for vertical ground motion may be taken as equal to that for horizontal motion. (Which shall include the 20% increase in horizontal PGA as per Clause 6.1).

7. Single mode Response Spectrum Method (or Seismic Coefficient Method)

The method can be employed by using the following step-wise procedure:

- Obtain the horizontal elastic acceleration coefficient due to design earthquake, which is same for all components. (Clause 7.1)
- Obtain the seismic weight of each component. (Clause 6.4)
- Obtain the seismic inertia forces generated in each component by multiplying quantities in (a) and (b) above. (Clause 7.2.1)
- Apply these inertia forces generated in each of the components at the center of mass of the corresponding component, and conduct a linear elastic analysis of the entire bridge structure to obtain the stress resultants at each cross-section of interest.
- Obtain the design stress resultants in any component by dividing the maximum elastic stress resultants obtained in (d) above by the response reduction factor prescribed for that component. (Clause 7.3)

7.1 Elastic Seismic Acceleration Coefficient

The Elastic Seismic Acceleration Coefficient A_h due to design earthquake along a considered direction shall be obtained as

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

Where,

Z = Zone Factor, given in Table 3, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The Factor 2 in the denominator of Z is used so as to reduce Maximum considered earthquake (MCE) zone factor to the factor of Design Basis Earthquake (DBE)

I = Importance Factor, given in Table 4,

$\frac{S_a}{g}$ = Response Spectrum Acceleration Coefficient along the considered direction given as follows:

For rocky, or hard soil sites (Type I)

$$\frac{S_a}{g} = \begin{cases} 2.50 & T \leq 0.40 \\ 1.00/T & 0.40 \leq T \leq 3.00 \\ 0.33 & T \geq 3.00 \end{cases}$$

For medium soil sites (Type II)

$$\frac{S_a}{g} = \begin{cases} 2.50 & T \leq 0.55 \\ 1.36/T & 0.55 \leq T \leq 3.00 \\ 0.45 & T \geq 3.00 \end{cases}$$

For soft soil sites (Type III)

$$\frac{S_a}{g} = \begin{cases} 2.50 & T \leq 0.67 \\ 1.67/T & 0.67 \leq T \leq 3.00 \\ 0.56 & T \geq 3.00 \end{cases}$$

T = Fundamental natural period of the bridge along the considered direction.

The soil types are described in Table 5.

A plot of $\frac{S_a}{g}$ is given in Fig.3 for 5% damping. For other damping values, the multiplying factors are given in Table 6.

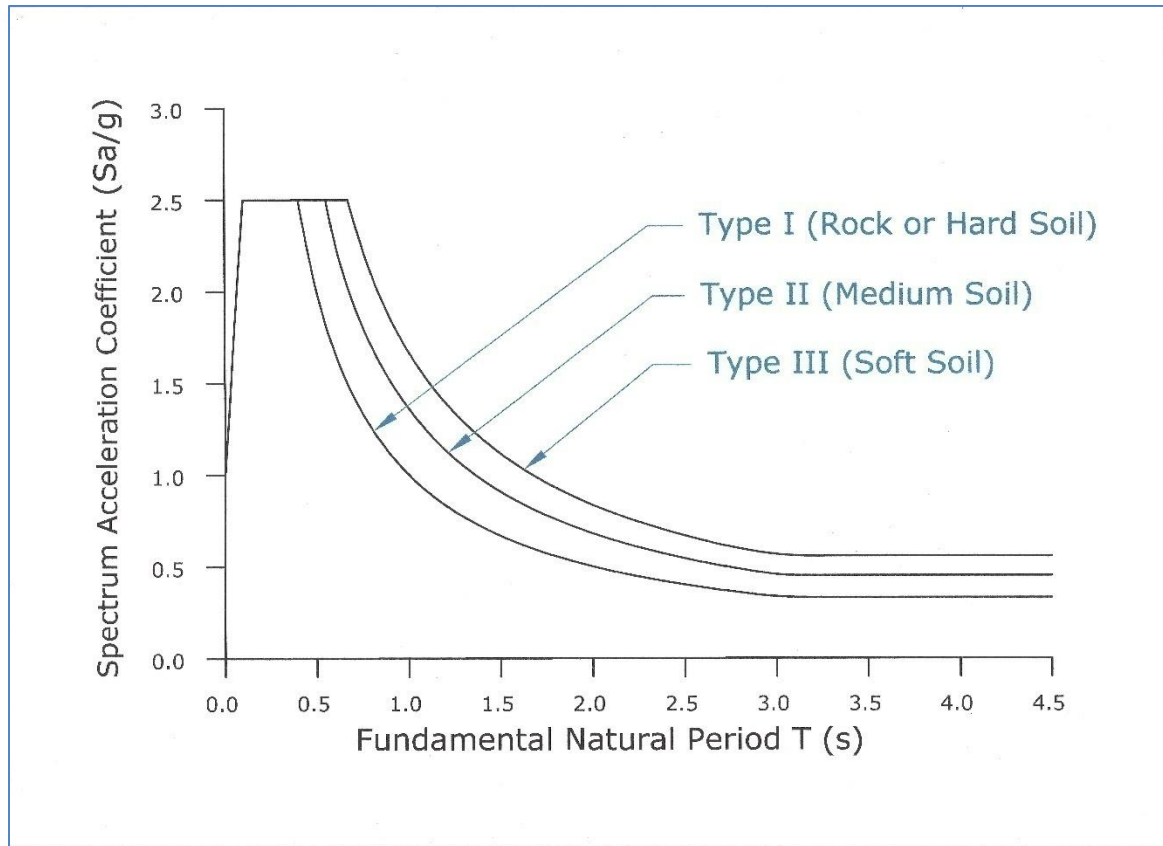


Fig. 3 Response Spectrum for 5% damping for Seismic Coefficient Method (Clause 7.0)

Table 6 Multiplying Factors for Other Damping percentages

Damping %	2	5	10
Factor	1.4	1.0	0.8
Application	Prestressed Concrete, Steel and Composite steel elements	Reinforced Concrete elements	Retrofitting of old bridges with RC piers

7.1.1 Fundamental Natural Period

Fundamental time period of the bridge member is to be calculated by any rational method of analysis.

The fundamental period can also be calculated by the method given below:

- 1) For simply supported bridges, the design vibration unit consists of one pier and a superstructure portion supported by it (because each pier is supporting two half superstructure units). The fundamental natural period T shall be calculated from the following equation:

$$t = \sqrt[2]{\frac{W}{1000F}}$$

W = Full Weight of the superstructure, 80% weight of substructure, and appropriate amount of live load in kN.

F = Horizontal force in kN required to be applied at the centre of mass of superstructure for one mm horizontal deflection at the top of pier/abutment for the earthquake in the transverse direction, and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

- 2) For multi-span integral bridges (continuous bridges), the design vibration unit consists of a number of substructures and superstructure portions supported by it (Fig. C-3c). The fundamental natural period (T) shall be calculated by any suitable method. For example, Rayleigh's method may be used as follows:

$$T = \sqrt[2]{\delta}$$

$$\delta = \frac{\int W(s)u(s)^2 ds}{\int W(s)u(s)ds}$$

$W(s)$ =Weight of the superstructure and substructure at position s (kN)

$u(s)$ = Displacement at position s caused in the acting direction of inertial force when the force corresponding to the weight of the superstructure and substructure above the ground surface for seismic design is assumed to act in the acting direction of inertial force.

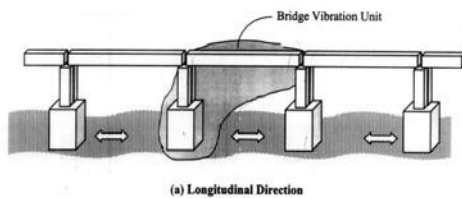


Fig. C3a – Design vibration unit in longitudinal direction

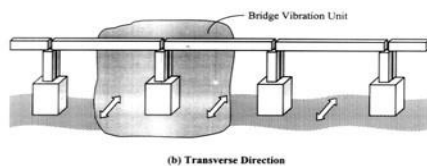
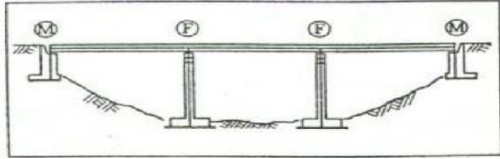


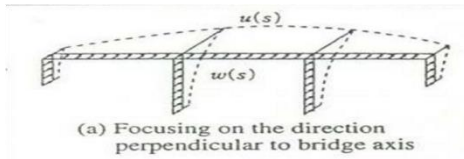
Fig. C3b – Design vibration unit in transverse direction

In case of idealisation as SDOF, 80% of pier mass shall be lumped at the top of pier.

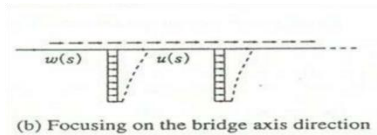
In response spectrum analysis where free vibration analysis is carried out to obtain natural time period, total weight of substructure is considered.



Continuous bridge; F = fixed and M = movable bearings



For transverse direction



For longitudinal direction

Fig. C-3c Design vibration unit for Continuous Bridge

Note - 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 fundamental natural period as per the Clause 6.6.4.

7.1.1.1 For ultimate limit state, the cracked flexural stiffness of reinforced concrete pier shall be used. The cracked flexural stiffness is the initial slope of the moment curvature ($M\Phi$) curve and is given by

$$E_c I_{eff} = \frac{M_y}{\phi_y}$$

where,

M_y is the moment capacity of the column/pier section at the first yield of the reinforcing steel, and ϕ_y is the yield curvature.

In the absence of more rigorous estimate, effective moment of inertia, I_{eff} , can be taken as 0.75 times gross moment of inertia, I_g .

7.2 Maximum Elastic Forces and Deformations

The inertia forces due to mass of each component or portion of the bridge as obtained from Clause 7.2.1 shall be applied at the center of mass of the corresponding component or portion of

the bridge. A linear static analysis of the bridge shall be performed for these applied inertia forces to obtain the force resultants (e.g., bending moment, shear force and axial force) and deformations (e.g., displacements and rotations) at different locations in the bridge. The stress resultants V^e and deformations so obtained are the maximum elastic force resultants (at the chosen cross-section of the bridge component) and the maximum elastic deformations (at the chosen nodes in the bridge structure), respectively.

Inertia Force Due to Mass of Each Bridge Component

The inertia force due to the mass of each bridge component (e.g., superstructure, substructure and foundation) under earthquake ground shaking along any direction shall be obtained from

$$F^e = A_h W$$

where

A_h = Elastic Seismic Acceleration Coefficient along the considered direction of shaking obtained as per Clause 7.1, and

W = Seismic weight as discussed in Clause 6.4.

The part of foundation embedded below the scour level shall not be considered to produce any seismic forces.

7.3 - Design Seismic Force Resultants for Bridge Components

The design seismic force resultant V at a cross section of a bridge component due to earthquake shaking along a considered direction shall be given by

$$V = \frac{V^e}{R}$$

Where,

V^e = Maximum elastic force resultant at the chosen cross-section of that bridge component from Clause 7.2, and

R = Response Reduction Factor for the component as given in Table 7.

Response Reduction Factor shall not be applied for calculation of displacements of Bridge Elements and for Bridge as a whole.

Table 7: Response Reduction Factor **R** for Bridge Components and Connections

Bridge Component		R with ductile detailing	R without ductile detailing
Superstructure		N.A	2.0
Substructure			
(i)	Masonry/ PCC piers, Abutments	NA	1.5
(ii)a	RCC wall pier and abutments in transverse direction (where plastic hinge cannot develop)	NA	1
(ii)b	RCC wall piers and abutments in longitudinal direction (where plastic hinge can develop)	3.0	2.5
(iii)	RCC long piers where hinges can develop	4.0	2.5
(iv)	Column (RCC or Steel)	4.0	2.5
Bearings		2.0	2.0
Connectors and Stoppers (Reaction blocks) Those restraining dislodgement or drifting away of bridge elements.		<p>When connectors and stoppers are designed to withstand seismic forces primarily, R value shall be taken as 1.0.</p> <p>When connectors and stoppers are designed as additional safety measures in the event of failure of bearings, R value specified for substructure above shall be used.</p>	

Note: 1) These parts of structural element of foundation which are not in contact with soil and transferring load to it, are to be treated as part of substructure element.

2) In case of elastomeric bearings, the value of R for substructure shall be taken as half of values given in Table 7.

3) For stability analysis of well foundation by conventional method seismic force can be further reduced by a factor of 2.0.

8. Response Spectrum Method (Multi mode Method)

The Response Spectrum Method requires the evaluation of natural periods and mode shapes of several modes of vibration of the structure. This method requires dynamic analysis, by a competent structural engineer.

8.1 Elastic Seismic Acceleration Coefficient in Mode k

The elastic seismic acceleration coefficient A_k for mode k shall be determined by:

$$A_k = \frac{Z}{2} \times I \times (S_a / g)_k$$

Where Z and I are as defined in Clause 7.1, and period (T_k) of k^{th} mode.

$\left(\frac{S_a}{g}\right)_k$ is the seismic acceleration coefficient for mode k given by expression.

For rocky, or hard soil sites (Type I)

$$\left(\frac{S_a}{g}\right)_k = \begin{cases} 2.50 & T_k \leq 0.40 \\ 1.00/T_k & 0.40 \leq T_k < 3.00 \\ 0.33 & T_k \geq 3.00 \end{cases}$$

For medium soil sites (Type II)

$$\left(\frac{S_a}{g}\right)_k = \begin{cases} 2.50 & T_k \leq 0.55 \\ 1.36/T_k & 0.55 \leq T_k < 3.00 \\ 0.45 & T_k \geq 3.0 \end{cases}$$

For soft soil sites (Type III)

$$\left(\frac{S_a}{g}\right)_k = \begin{cases} 2.50 & T_k \leq 0.67 \\ 1.67/T_k & 0.67 \leq T_k < 3.00 \\ 0.56 & T_k \geq 3.0 \end{cases}$$

where T_k is the natural period of vibration of mode k of the bridge. For modes other than the

fundamental mode, the bridge flexibility factor $\left(\frac{S_a}{g}\right)_k$ for $T_k \leq 0.0.1$ sec may be taken as $\left(\frac{S_a}{g}\right)_k =$

$$1 + 15T_k$$

A plot of $\left(\frac{S_a}{g}\right)_k$ versus T_k is given in Fig. 4 for 5% damping. Table 6 gives the multiplying factors

for obtaining spectral values for various other damping percentages.

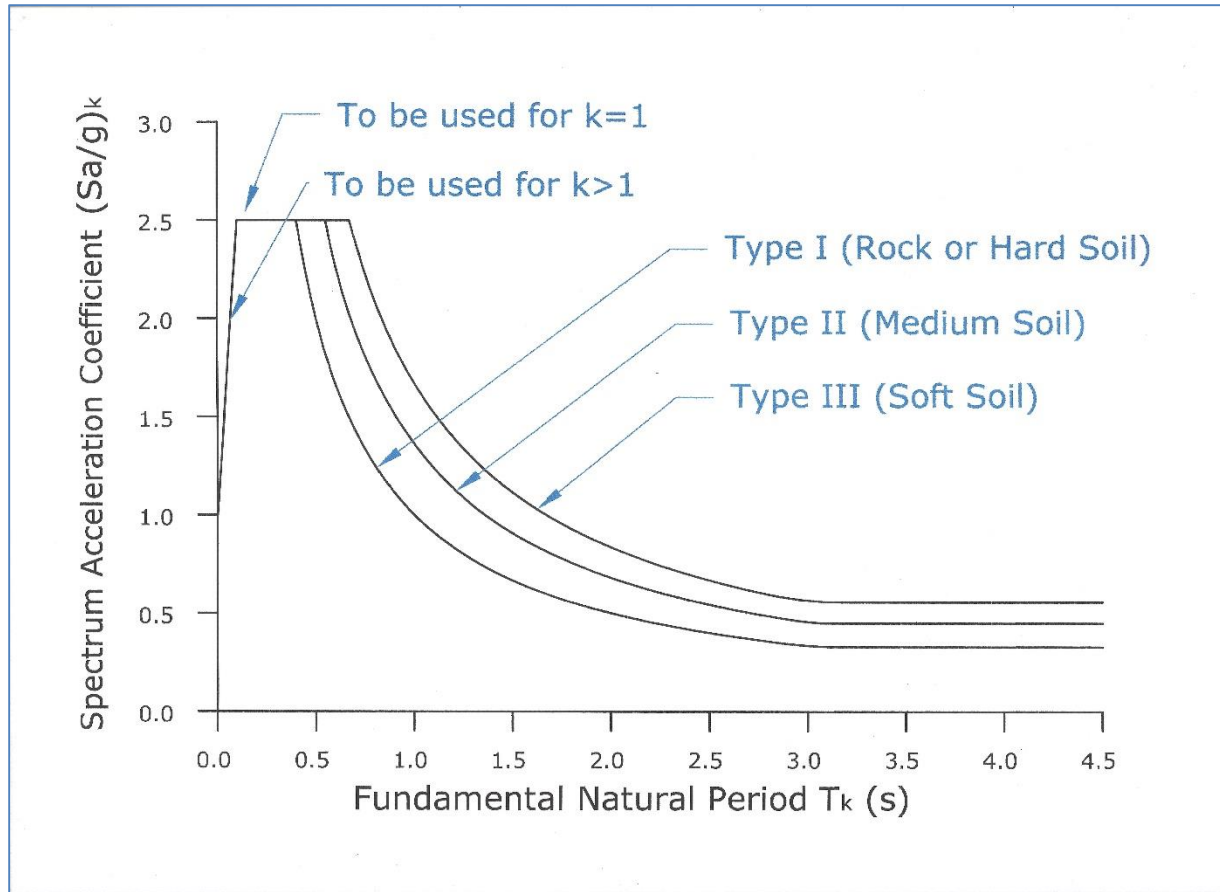


Fig. 4 Acceleration response spectrum for 5% damping to be used for response spectrum method

8.2 Inertia Force due to Mass of Bridge at Node j in Mode k

The effect of seismic shaking can be quantified as concentrated seismic inertia forces and moment corresponding to the translational and rotational degrees of freedom, respectively, at each node of the discretised model of the bridge structure (a typical discretised model is shown in Fig. C3). Each mode of vibration contributes to these seismic inertia forces and moments. The maximum elastic force at j^{th} node in k^{th} mode is given by

$$F_k^j = m_j \phi_k P_k A_k g$$

The force vector $\{F_k^e\}$ of maximum elastic inertia forces at different nodes in mode k of vibration due to earthquake shaking along a considered direction shall be obtained as:

$$\{F_k^e\} = [m] \{\phi_k\} P_k A_k g$$

[m] where = Seismic mass matrix of the bridge structure, as defined in Clause 8.2.1,

$\{\phi_k\}$ = Mode shape vector of vibration mode k of the bridge structure obtained from free vibration analysis,

P_k = Modal participation factor of vibration mode k of the bridge structure for a given direction of earthquake shaking

$$= \frac{\{\varphi_k\}^T [m] \{1\}}{\{\varphi_k\}^T [m] \{\varphi_k\}}$$

A_k = Elastic seismic acceleration coefficient for mode k as defined in Clause 8.1,

g = Acceleration due to gravity, and

$\{1\}$ = Vector consisting of unity (one) associated with translational degrees of freedom in the considered direction of shaking, and zero associated with all other degrees of freedom.

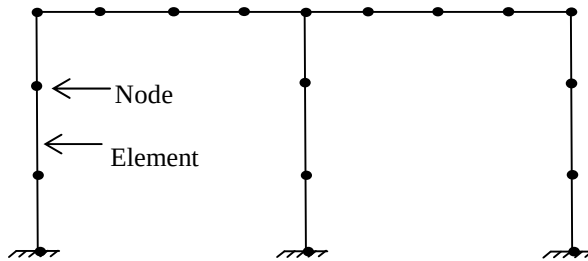


Fig C3:- Mathematical Model of Bridge

8.2.1 Seismic Mass Matrix

The seismic mass matrix of the bridge structure shall be constructed by considering its seismic mass lumped at the nodes of discretisation. The seismic mass of each bridge component shall be estimated as per Clause 6.4, and shall be proportionally distributed to the nodes of discretisation of that bridge component.

8.3 Maximum Elastic Forces and Deformations

The maximum elastic seismic forces in mode k obtained from Clause 8.2 shall be applied on the bridge and a linear static analysis of the bridge shall be performed to evaluate the maximum elastic force resultants F_k^e (e.g., bending moment, shear force and axial force) and the maximum elastic deformations (e.g., displacements and rotations) in mode k at different locations (or nodes) in the bridge for a considered direction of earthquake shaking.

The maximum elastic force resultants F_{net}^e and the maximum elastic deformations, due to all modes considered, for the considered direction of earthquake shaking, shall be obtained by combining those due to the individual modes as follows:

(a) If the structure does not have closely-spaced modes, then the maximum response λ due to all modes considered may be estimated by the square root of sum of squares (SRSS) method as:

$$\lambda = \sqrt{\sum_{k=1}^m (\lambda_k)^2}$$

Where,

λ_k = Absolute value of response in mode k , and

m = Number of modes being considered

(b) If the structure has a few closely-spaced modes, then the maximum response (λ) due to these modes shall be obtained by the absolute sum method as:

$$\lambda^* = \sum_{c=1}^r |\lambda_c|$$

where the summation is for the closely-spaced modes only. This maximum response due to closely-spaced modes (λ^*) is then combined with those of the remaining well-separated modes by the square root of sum of square (SRSS) method in a) above.

8.3.1 The number of modes to be considered in the analysis shall be such that at least 90% of the total seismic mass of the structure is included in the calculations of response for earthquake shaking along each principal direction. If modes with natural frequency beyond 33 Hz are to be considered, modal combination (Clause 8.3 (a) and 8.3 (b)) shall be carried out only for modes with natural frequency less than 33 Hz. Modes with natural frequency exceeding 33 Hz shall be treated as rigid modes and accounted for through missing mass correction discussed below:

At degree of freedom j , the missing mass is given by

$$C_j m_j = \left(1 - \sum_{k=1}^n P_k \phi_{kj} \right) m_j$$

Where

P_k = Modal participation factor for mode k ,

ϕ_{kj} = Mode shape coefficient for j^{th} , degree of freedom in k^{th} mode of vibration

m_j = Total mass of the j^{th} mode,

c_j = Fraction of missing mass for j^{th} mode.

Lateral force associated with missing mass is

$$F_j^{\text{missing}} = c_j m_j \left(\frac{Z}{2} I \right)$$

The structure will be statically analyzed for this set of lateral inertial forces and response λ^{missing} will be obtained. The response λ^{missing} will be combined with response λ for flexible modes by the square root of sum of square (SRSS) method in a) above.

8.4 Design Seismic Force Resultants in Bridge Components

The design seismic force resultant V_{net} at any cross-section in a bridge component for a considered direction of earthquake shaking shall be determined as

$$V_{\text{net}} = \frac{F_{\text{net}}^e}{R}$$

Where the maximum elastic force resultant F_{net}^e due to all modes considered is as obtained in Clause 8.3, and Response Reduction Factor R of that component of bridge is as per Table 7. However, Response Reduction Factor shall not be applied for calculation of design displacements.

8.5 Multi-directional Shaking

When earthquake ground shaking is considered along more than one direction, the design seismic force resultants obtained from Clause 7.3 or 8.4 at a cross-section of a bridge component due to earthquake shaking in each considered direction, shall be combined as per Clause 6.5.

9. Time History Method

In Time History Method, dynamic analysis of bridge is carried out for specified earthquake ground motion. In this method, dynamic response (i.e. response varying with time) is obtained.

9.1 Modelling of Bridge

In order to carry out time history analysis, a suitable mathematical model of the bridge shall be developed. The model shall adequately represent the mass distribution and stiffness of superstructure, bearings, pier, abutment and foundation. The damping characteristics shall also be adequately included in the model. For analysis, mass of live load as per clause no 6.4.2 shall be included in the model. The pier can be considered to be fixed at the foundation level.

9.2 Analysis

Analysis may be carried out using modal superposition method or direct numerical direction under consideration. Time step to be used in the analysis shall be suitably chosen and sensitivity of the solution to time step shall be ascertained.

9.3 Ground Motion

Ground acceleration time histories shall have characteristics that are representative of seismic environment of the site and local site conditions. Time histories from actual recorded events with similar magnitude, fault distance and local site condition shall be selected.

The ground motions selected shall have peak ground acceleration value of $Z \times I$, where, Z is zone factor and I is importance factor. At least three ground motions shall be used, and maximum response of the three cases, shall be taken as design value. If more than seven time histories are used, then, average response can be used as design value.

9.3.1 Scaling of Time Histories

Time histories to be used in the analysis, shall be suitably scaled so as to match the design response spectra. The response spectra of time history shall be matched with the design spectra given by

$$S(T) = \gamma \times ZI(S_a / g)$$

The matching shall be such that the average response spectra of the selected time histories shall not be less than the above mentioned design spectra in the periods ranging from $0.2T$ and $1.5T$, where T is the fundamental time period of the bridge in the direction under consideration.

9.3.2 Ground Motions for Two- and Three-Dimensional Analysis

For 2-dimensional analysis, ground motion consists of horizontal acceleration time history in the direction under consideration. If vertical ground motion is to be considered, then, vertical acceleration time history is also used.

For 3-dimensional analysis, ground motions consist of pairs of time histories of appropriate components of horizontal accelerations. For each pair of horizontal acceleration time histories, SRSS response spectrum shall be obtained. This SRSS response spectrum shall be scaled suitably to match with the design response spectrum as described in Clause 9.3.1. If required the vertical acceleration time history shall also be considered.

9.4 Interpretation of Time History Analysis Results

9.4.1 Linear Analysis

From the time history of the response quantity of a particular member, the maximum value will be the design value. This maximum value shall be divided by $2R$, where R is the response reduction factor for that member.

9.4.2 Nonlinear Analysis

Nonlinear analysis is used for verifying if the provided strength is sufficient to accommodate the expected inelastic deformation. For the nonlinear analysis, the bridge model shall include nonlinear properties.

In the analysis, ground motions in two directions shall be applied simultaneously along with the dead loads and other loads.

The results of nonlinear analysis shall not be divided by factor $2R$.

10. Pushover Analysis

The design force is obtained by dividing the elastic force by R value. In some instances, mentioned in Table.1 energy dissipating capacity may be ascertained by a push over analysis to ensure that the required displacement demand is being met with. The details regarding push over analysis are given in Appendix – E.

11. Superstructure

11.1 The superstructure shall be designed for the design seismic forces specified in Clauses 7.0 or 8.0 along with the other appropriate loads.

The superstructure shall be designed for lesser of following forces:

- a) Elastic seismic forces i.e. seismic forces with $R = 1.0$
- b) Forces developed when over strength plastic moment hinges are formed in the substructure, as described in Appendix C.

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. In this calculation, the forces to be considered on the superstructure shall be the maximum elastic forces generated in the superstructure, as calculated using Clauses 7.2 and 8.3.

11.3 The superstructure shall be secured to the substructure in seismic zones IV and V, through vertical hold-down device and anti-dislodging elements in horizontal direction as specified in Clauses 11.3.1 and 11.3.2, respectively. These vertical hold-down devices and anti-dislodging elements may also be used to secure the suspended spans, if any, with the restrained portions of the superstructure. However, the frictional forces shall not be relied upon in the design of these hold-down devices or anti-dislodging elements.

11.3.1 - Vertical Hold-Down Devices

“In Zone IV & V, vertical hold-down devices shall be provided at all supports where resulting vertical seismic force opposes and exceeds 50% of the dead load reaction”.

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

11.3.1.2 - If the vertical force U, due to the combined effect of maximum horizontal and vertical seismic forces, opposes and exceeds 100% 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% of the downward dead load reaction that would be exerted if the span were simply supported.

11.3.2 Horizontal Linkage Elements

Horizontal linkage elements are anti-dislodging devices. Positive horizontal linkage elements (high tensile wire strand ties, cables, and dampers) shall be provided between adjacent section of the superstructure at supports and at expansion joints within a span.

11.3.2.1 – The linkage shall be designed for at least the elastic seismic horizontal coefficient times the weight of the lighter of the two connected spans or parts of the structure.

11.3.2.2 - If the linkage is at locations where relative deformation are permitted in the design then, sufficient slack must be allowed in the linkage so that linkages start functioning only when the relative design displacement at the linkage is exceeded.

11.3.2.3 - “When linkages are provided at columns or piers, the linkage of each span may be connected to the column or pier instead of the adjacent span. Alternatively, reactions blocks may be provided as per sub-para 11.3.2.4

11.3.2.4- Reaction blocks (or seismic arrestors) when used as anti-dislodging elements shall be designed for seismic force equal to 1.5 times the elastic seismic coefficient multiplied by tributary weight of spans corresponding to that pier/abutment.

12. Substructure

12.1 Scour Depth

The scour to be considered for design shall be based on mean annual flood. In the absence of detailed data the scour to be considered for design shall be 0.9 times the maximum design scour depth. The maximum scour case may not be seismically governing design condition. Therefore, the design under seismic loading must be checked for minimum scour depth condition as well, where increased stiffness of foundation system can result in lower time periods and consequently higher forces.

12.2 Hydrodynamic Force

12.2.1For the submerged portion of the pier, the total horizontal hydrodynamic force along the direction of ground motion is given by

$$F=C_e A_h W_e$$

where C_e is a coefficient given by Table 8, depending on the height of submergence of the pier relative to that of the radius of a hypothetical enveloping cylinder (Fig. 5); and A_h is the elastic seismic acceleration coefficient as per Clause 7.1 or 8.1; and W_e is the weight of the water in the hypothetical enveloping cylinder. The pressure distribution due to hydrodynamic effect on pier is given in Fig. 6; the coefficients C_1 , C_2 , C_3 and C_4 in Fig. 6 are given in Table 9.

12.2.2 In response spectrum analysis, to account for hydrodynamic pressure, additional weight of water shall be added over the submerged depth of pier. The weight of water to be added at a height of 3/7H from the ground level, is given by:

$$W_{wp} = \frac{3}{4} W_0 A_o \frac{b H_p}{a} \left(1 - \frac{b}{4H} \right)$$

for $b/H < 2.0$

$$W_{wp} = \frac{3}{4} W_0 A_o \frac{b H_p}{a} \left(0.7 - \frac{b}{10H} \right)$$

for $2.0 < b/H < 4.0$

$$W_{wp} = \frac{9}{40} W_0 A_o \frac{b H_p}{a}$$

for $4.0 < b/H$

where,

b = structural width perpendicular to hydrodynamic pressure,

a = structural width in the direction of hydrodynamic pressure,

A_o = sectional area of the substructure,

W_0 = density of water,

H_p = pier height and

H = height of submerged portion of pier

Table - 8. Values of C_e

$\left(\frac{\text{Height of Submerged Portion}(H)}{\text{Radius of Enveloping Cylinder}} \right)$	1.0	2.0	3.0	4.0
C_e	.390	.575	.675	.730

Table - 9. Coefficients C_2 , C_3 and C_4 as a function of C_1

C_1	C_2	C_3	C_4
0.1	0.410	0.026	0.9345
0.2	0.673	0.093	0.8712
0.3	0.832	0.184	0.8103
0.4	0.922	0.289	0.7515
0.5	0.970	0.403	0.6945
0.6	0.990	0.521	0.6390
0.8	0.999	0.760	0.5320
1.0	1.000	1.000	0.4286

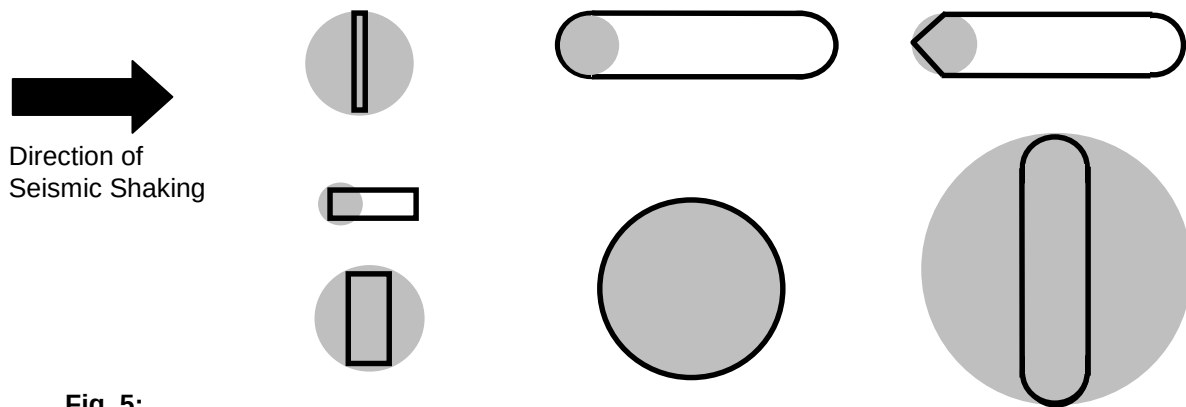


Fig. 5:
Hypothetical Enveloping Cylinders to Estimate Hydrodynamic Forces on Substructures due to Seismic Shaking (Clause 12.2)

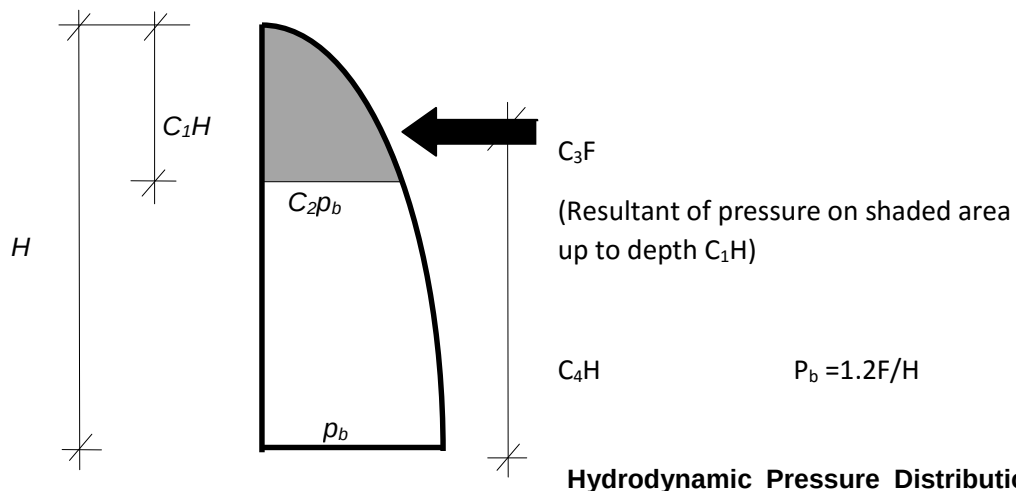


Fig. 6:
Substructure due to Steam Flow (Clause 12.2.2)

12.2.3 Analysis for Vertical Acceleration

While carrying out the analysis for vertical acceleration, the added mass of water for hydrodynamic effect shall not be considered.

12.3 Design Seismic Force

The design seismic forces for the substructure shall be obtained as the maximum elastic force on it (as defined in Clause 12.3.1) divided by the appropriate response reduction factor given in Table 7.

12.3.1 Maximum Elastic Seismic Forces

The maximum elastic seismic force resultants at any cross-section of the substructure shall be calculated considering the following forces:

- (a) Maximum elastic seismic forces transferred from the superstructure to the top of the substructure
- (b) Maximum elastic seismic forces applied at its center of mass due to the substructure's own inertia forces. Reduction due to buoyancy shall be ignored in the calculation of seismic weight.
- (c) Hydrodynamic forces acting on piers as per Clause 12.2, and
- (d) Modification in earth-pressure due to earthquake acting on abutments as per Appendix F.

12.4 Substructure of Continuous Girder Superstructure

12.4.1 When the superstructure of a multi-span bridge consists of a single continuous girder resting on a restrained bearing (in longitudinal direction) over one of the piers and on sliding bearings over the other piers, the design seismic force at the top of the substructures along the longitudinal direction of the bridge shall be taken as follows:

- (a) For the pier supporting the restrained bearing, it shall be the full elastic seismic force transmitted from the superstructure to the top of the pier in the longitudinal direction divided by the appropriate response reduction factor, assuming no friction between the other sliding bearings and the corresponding piers.
- (b) For the other piers supporting the sliding bearings, it shall be the horizontal friction force generated on the pier due to the superstructure resting on the pier considering the maximum possible friction between the sliding bearings and the top of the pier.

12.4.2 In transverse direction, the seismic force from superstructure is to be transmitted to the substructures in proportion to their lateral stiffness.

12.4.3 While considering the stability of the substructure, such as, wingwalls, abutments etc., against overturning, the minimum factor of safety shall be 1.5 under simultaneous action of maximum elastic seismic forces in both horizontal and vertical directions during the earthquake.

13. Foundations

13.1 For design of foundation, the seismic loads should be taken as 1.25 times the forces transmitted to it by substructure, so as to provide sufficient margin to cover the possible higher forces transmitted by substructure arising out of its overstrength.

13.2 - Following factor of safety shall be adopted for seismic design of foundation under ultimate condition:

Factor of safety against overturning - 1.5

Factor of safety against sliding - 1.25

Notes:

Note 1: No live load to be considered when the net effect has a stabilizing effect.

Note 2: Area under tension need not be checked provided above criteria for overturning and sliding is satisfied.

13.3 In loose sands or poorly graded sands with little or no fines, vibrations due to earthquake may cause liquefaction or excessive total and differential settlements. In Zones IV and V, the founding of bridges on such sands should be avoided unless appropriate methods of compaction or stabilization are adopted. Liquefaction analysis procedure is given in APPENDIX G. Foundation should be taken to sufficient depth below the layers of soil which are susceptible to liquefaction.

14. Connections

The connection between the superstructure and substructure is achieved through bearings. The primary functions of the bearings are to resist the vertical loads due to dead load and live load and to allow for superstructure movements (translation and rotation) due to live load and temperature changes. The design of bearings is governed by the force to be resisted and the extent of movement (translation and rotation) it can accommodate. During seismic event, the lateral seismic forces from superstructure are transferred to substructure through bearings. The bearing shall possess sufficient strength to resist these seismic forces.

14.1 Design Force for Connections

14.1.1 Seismic Zone II and III

The connections between adjacent sections of the superstructure or between the superstructure and the substructure shall be designed to resist at least horizontal seismic force in the restrained directions equal to 0.2 times the vertical dead load reaction at the bearing, irrespective of the number of spans.

14.1.2 Seismic Zone IV and V

The connection between the superstructure and substructure, and the substructure and foundation shall be designed to resist the smaller of the following forces:

- a) Maximum elastic horizontal seismic force obtained from analysis and transferred through the connection in the restrained directions, divided by the appropriate Response reduction factor R as applicable to connections, which are given in Table 7.
- b) Maximum horizontal force, when over strength plastic moment hinges are formed in the substructure.

14.2 Displacement at Connections

14.2.1 - Separation between Adjacent Units

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 calculated 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 given by Clauses 7.2 or 8.3.

14.3 Minimum Seating Width Requirements

The widths of seating W (in mm) at supports measured normal to the face of the abutment/pier/pedestal of bearings/restrained portion of superstructure from the closest end of the girder shall be following:

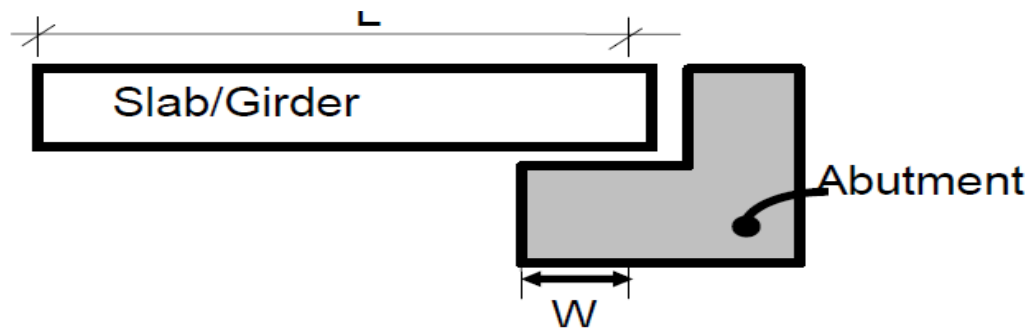
The value specified below:

$$W = \begin{cases} 300 + 1.5L + 6H_p & \text{for seismic zones II and III} \\ 500 + 2.5L + 10H_p & \text{for seismic zones IV and V} \end{cases}$$

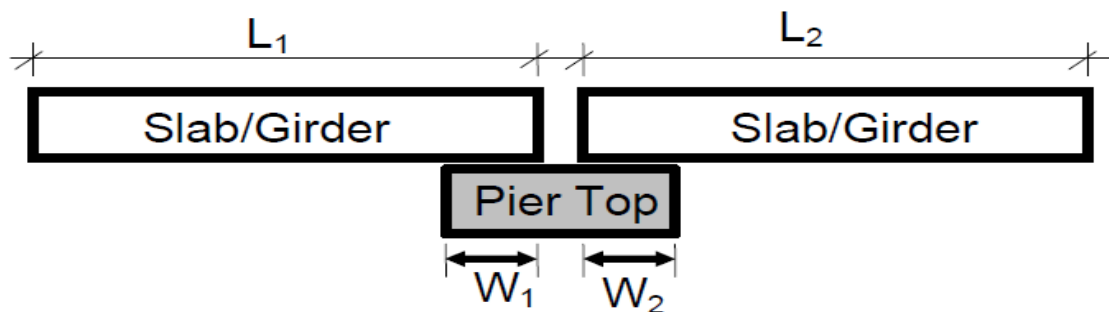
Where

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

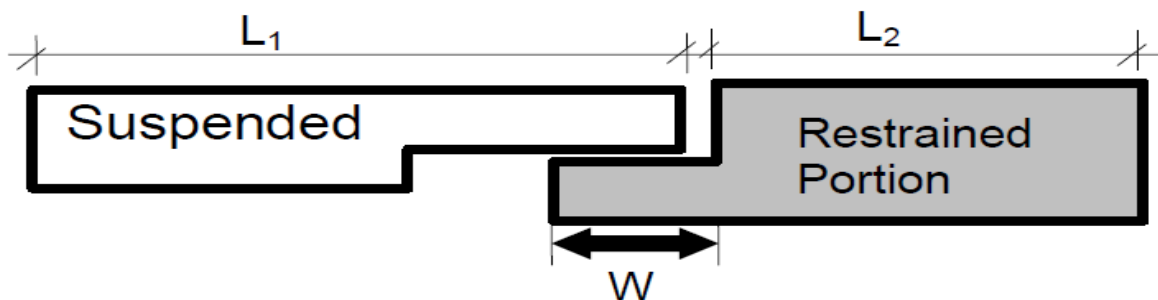
For bearings at abutments, H_p is the average height (in meters) of all columns supporting the superstructure to the next expansion joint. It is equal to zero for single span bridges. For bearings at columns or piers, H_p is the height (in meters) of column or pier. For bearings under suspended spans, H_p is the average height (in meters) of the two adjacent columns or piers. Graphical representation of seating widths is shown in Fig. below



(a) Abutment



(b) Column or Pier



(c) Suspended Span on Restrained Portion of Superstructure

15. Special Ductile Detailing Requirements for Bridges Substructures

The design seismic force 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 in design/detailing. (These provisions are meant for bridges having reinforced concrete substructures; however, if steel substructures are used in high seismic zones, reference should be made to specialist literature.) APPENDIX C describes the detailing procedure.

16. Special Devices

Special devices like seismic isolation devices, shock transmission units (STU) and dampers may be employed to improve the seismic performance of bridges. However, appropriate analysis and testing shall be carried out before installation.

16.1 Seismic Isolation Devices

Section 17 provides details regarding bridges with seismic isolation.

16.2 Shock Transmission Units

Multi-span bridges with continuous superstructure may be provided with restrained bearings over only one pier/abutment. In order to distribute the seismic forces generated by the superstructure to other pier(s)/abutment(s), STUs may be introduced after adequate testing, between superstructure and other pier(s)/abutment(s) where free/guided bearings are used. However, specialist literature shall be consulted for the details of such STUs and for their design in bridges subjected to seismic effects. STUs should facilitate the breathing of the bridge due to thermal and shrinkage effects.

STUs shall be accessible for inspection and maintenance/replacement.

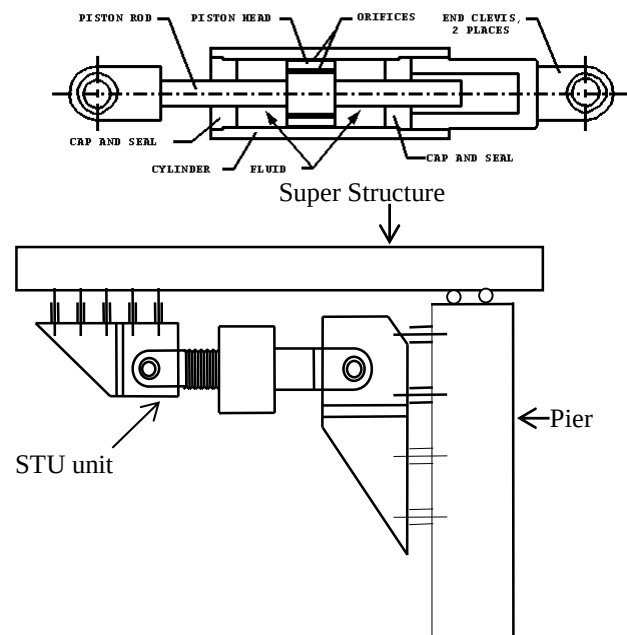


Fig 7 Typical Shock Transmission Unit

17. Bridges with Seismic Isolation

17.1 General

Seismic isolation devices (bearings) are deployed below the deck and on the top of the pier (Fig. 8). These shall be used for stiff bridges with time period less than 1 sec. The reduction in forces is achieved either by lengthening of time period or increase of damping or both of them. The effect of lengthening of period and increase of damping on the design force is explained in Fig. 9. The increase in damping is achieved by hysteretic energy loss. The isolation bearing is idealized as bilinear spring with hysteresis as shown in Fig. 10; where, K_u is elastic stiffness, K_d is post elastic stiffness, Q_d is characteristic strength and K_{eff} is effective stiffness.

With the use of isolation devices, the lateral displacement of superstructure increases. This increase in displacement shall not cause any adverse effect. Isolation bearings shall not be used for bridges which (a) are on soft soil, (b) which have long natural time period, and (c) which may experience uplift at bearing support. Isolation bearings shall be firmly fixed to the superstructure and substructure by anchor bolts and shall be easily accessible for replacements.

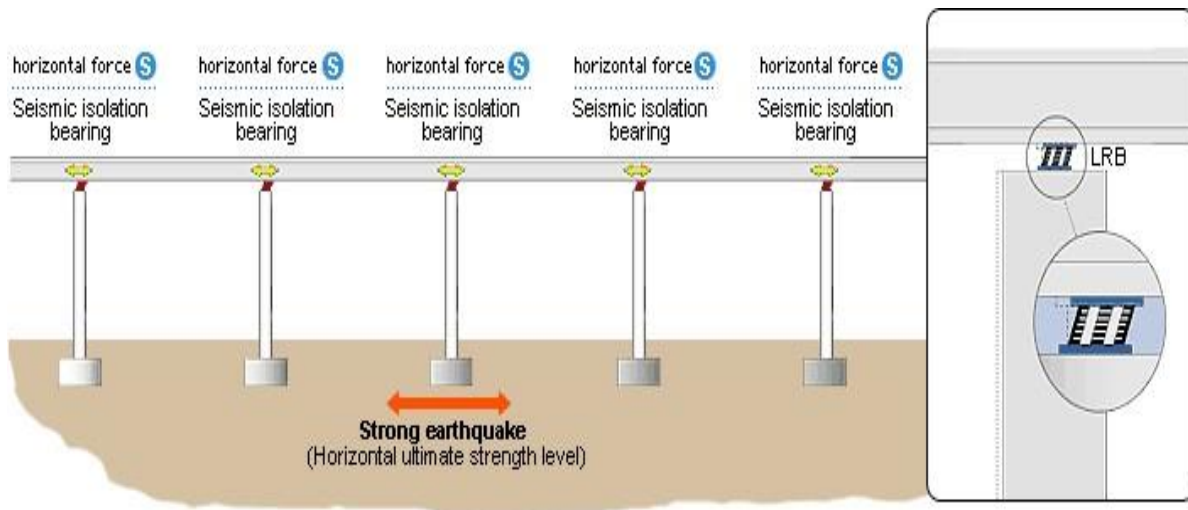


Fig.8 Bridge with seismic isolation

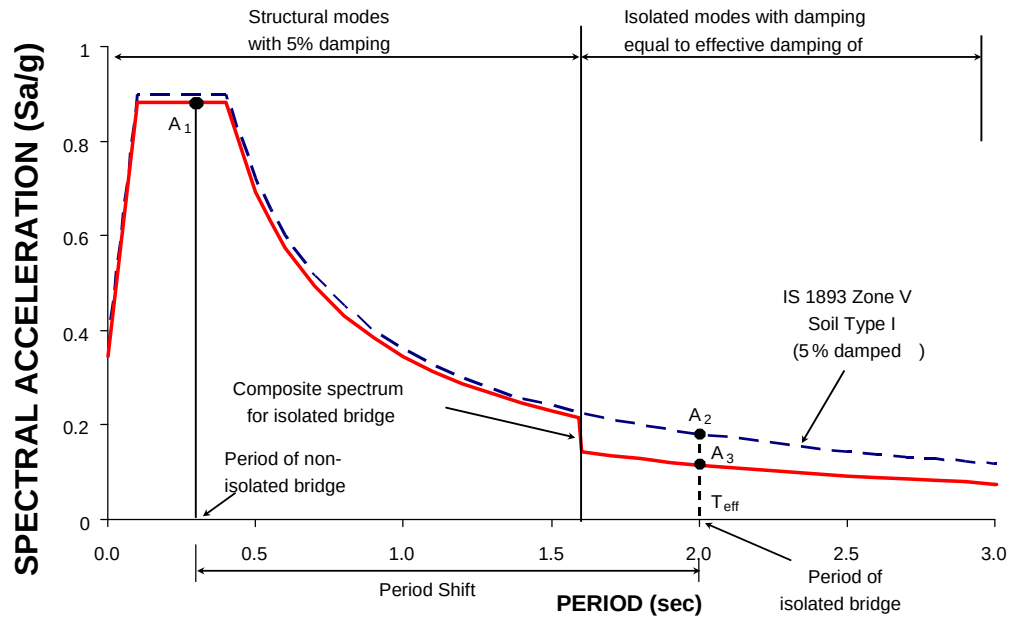


Fig.9 Effect of isolator on spectral acceleration

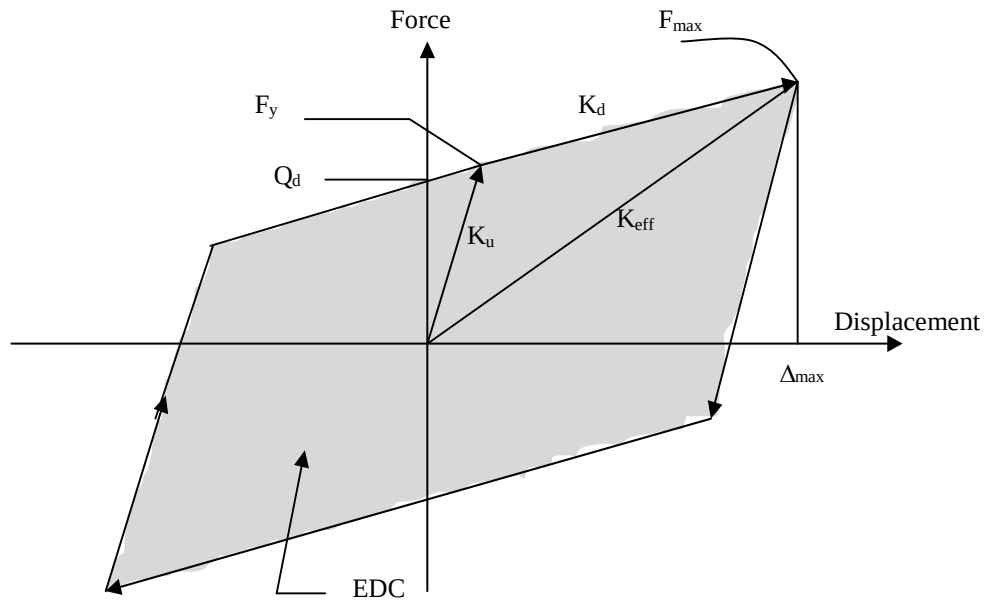


Fig.10 Bilinear force-deflection model for isolator

Q_d =Characteristics Strength

F_y = Yield Force

F_{max} =Maximum Force

K_d =Post-elastic stiffness

K_u = Elastic (unloading) stiffness

K_{eff} = Maximum bearing displacement

EDC = Energy dissipated per cycle = Area of hysteresis loop (shaded)

17.2 Design Criteria

A site-specific seismic hazard analysis shall be carried out to develop design ground motion for base-isolated bridges. This study shall be carried out by professionals with acknowledged expertise to do so and will usually involve geological, seismological, geotechnical and structural inputs. However, if the design ground motion thus arrived at gives a design less conservative than

that from design response spectrum given by $\left(\frac{Z}{2} \cdot I \cdot \frac{S_a}{g}\right)$ then the latter shall govern the design. The

response reduction factor for the substructure shall be taken as half of the values given in Table 7. However, the value of response reduction factor shall not be less than 1.0.

17.3 Analysis Procedure

The seismic coefficient method (single mode method) or response spectrum method (multimode method) can be used. The isolation system shall be idealized as bilinear system (Fig. 10) with linear stiffness as K_{eff} . The analysis shall be done using upper bound properties and lower bound properties. The upper bound properties, which would result in higher value of K_{eff} , would give higher force, and the lower bound properties would give higher deflection. The maximum and minimum values are obtained by multiplying K_d and Q_d with the property modification factors, which depend on velocity, temperature, aging, scragging, travel and contamination. The values of property modification factors are described in Appendix – H.

From the analysis, the isolator deflection, d_i , shall be obtained. Then, the design force for isolator is $F = K_{eff} \cdot d_i$. If uniform load method is used, then, isolator displacement is given by

$$d_1 = \frac{250A_h T_{eff}^2}{B_1} mm$$

Where $T_{eff} = 2\pi \sqrt{\frac{W}{K_{eff} g}}$

Since, the isolator unit has low stiffness, the displacement increases. The clearance in the two orthogonal directions shall be the maximum displacement determined in each of the directions from the analysis. The clearance shall not be less than

$$\frac{200A_h T_{eff}^2}{B_1} mm$$

where, B_i is the damping coefficient corresponding to the effective damping ratio of the isolator unit.

The value of B_i shall be taken from Table 10.

Table 10. Damping Coefficient for Isolated Bridges, B_i

	Damping (Percentage of critical)						
	≤ 2	5	10	20	30	40	50

B_I	0.8	1	1.2	1.5	1.7	1.9	2.0
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In the uniform load method, earthquake force, $F = (A_h) \cdot W$ is applied on the structure. Here, W is weight of the bridge, and A_h is the design seismic

17.4 Requirement of Isolator Unit

17.4.1 Non-seismic Lateral Forces

The isolation system must resist all Non-seismic lateral load combinations applied above the isolation unit. The rigidity against these lateral forces shall be established with the help of tests. If the temperature is likely to be very low in winter, then, the effect of low temperature on either coefficient of friction, shear modulus etc. shall be properly considered. The isolator shall not lose its effectiveness due to low temperature.

17.4.2 Lateral Restoring Force

The isolator unit has more flexibility and high energy dissipating capacity. Hence, in order to avoid cumulative displacement, it must have sufficient restoring force at any given displaced position. In order to ensure that the restoring force is not too less, it is recommended that at any displacement less than the design displacement, the tangent stiffness shall be such that the natural period shall not be more than 6 sec. The restoring force at any displacement shall be more than the restoring force at lower displacement. If the restoring force is constant for all displacements, then, this force shall be at least equal to 1.05 times the characteristics strength, K_d . It is important to note that the forces which do not depend on the displacements, such as damping force may not be used to meet the minimum restoring force requirement.

17.4.3 Vertical Load and Rotational Stability

In laterally un-deformed state, the isolation system shall provide a factor of safety of at least three against the vertical loads. It shall also be shown to be stable under 1.2 times the dead load and vertical load due to seismic force. Further, its stability against the lateral displacement equal to the offset displacement and 1.1 times the total design displacement shall be checked.

The isolator shall have the rotation capacity to accommodate rotation due to dead load, live load and construction misalignment, which shall not be less than 0.005 radians.

17.5 Tests on Isolation System

17.5.1 System Characterization Test

This is to establish characteristics of isolation unit and its various components.

17.5.2 Prototype Test

This is to establish deformation and damping characteristics of the isolator unit.

17.5.3 These tests are done at manufacturing units and the specimens involved in the test are not used. The prototype test is to be conducted on at least two specimen of full size. The system characterization tests are conducted on various components as per the requirements of the corresponding IS codes.

17.5.4 A shake table test on model not less than $1/4^{\text{th}}$ of full model shall be done. Scale factors for this test shall be well established. Wear or travel and fatigue tests are conducted to check if the movements due to thermal displacements and live load rotation can be accommodated. The thermal displacements and live load rotations shall correspond to at least 30 years of expected

movement. The tests shall be applied at the design contact pressure and at $20^{\circ}\text{C} \pm 8^{\circ}\text{C}$. The rate of application shall be not less than 63.5 mm/minute.

17.5.5 The tests shall be done for following minimum:

Bearings – 1.6 km

Dampers attached to the web of the neutral axis – 1.6 km

Dampers attached to the girder bottom – 3.2 km.

17.5.6 – The prototype specimen shall be tested in the following sequence for prescribed number of cycles:

Table – 11: Sequence for Testing of Bearing

Test	Description
(A) Component	
Thermal	Three fully reversed cycle of loads at a lateral displacement corresponding to the maximum thermal displacement. The test velocity shall not be less than 0.075 mm per minute.
Wind and braking	Twenty fully reversed cycles between limits of plus and minus maximum load for a total duration not less than 40 seconds. After the cyclic testing, the maximum load shall be held for 60 seconds.
Seismic -1	Three fully reversed cycles of loading at each of the following multiples of the total design displacement: 1.0, 0.25, 0.50, 0.75, 1.0, and 1.25 in the sequence mentioned. The results of test corresponding to design displacement are used for finding stiffness and damping properties.
Seismic -2	Fully reversed cycles of loading at design displacement for 25 cycles. The test shall be started from a displacement equal to the offset displacement.
The prototype specimen shall be tested in the following sequence for prescribed number of cycles: Wind and braking	Three fully reversed cycles between limits of plus and minus the maximum load for a total duration not less than 40 seconds. After the cyclic testing, the maximum load shall be held for 60 seconds. This test is done to ascertain the survivability of the isolator after the major earthquake.
(B) Prototype	
Seismic performance verification	Three fully reversed cycles of loading at the design displacement. The test verifies service load performance after the major earthquake.

Vertical load	The vertical load carrying capacity shall be demonstrated under $1.2DL + LL$ (seismic) + additional vertical load due to overturning moment.
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17.5.7 The force deflection characteristics of the isolator shall be based on cyclic load test results (seismic test described above) for each fully reversed cycle of loading (Fig. 10). The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$K_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n}$$

where, Δ_p and Δ_n are maximum positive and negative displacements and F_p and F_n are maximum positive and negative forces at Δ_p and Δ_n respectively (Fig. 10).

17.5.8 The equivalent viscous damping ratio (β) is given by

$$\beta = \frac{1}{2\pi} \frac{\text{Total EDC area}}{\sum (K_{eff} d_1^2)}$$

The total *EDC* area shall be taken as the sum of the areas of all isolator units. The hysteresis loop area of each isolator unit shall be taken as the minimum area of the three loops established at the design displacement, d_i is the design displacement at the centre of rigidity of the isolation system in the direction under consideration.

17.6 - System Adequacy

In the above mentioned tests, the performance of isolator unit is considered to be satisfactory, if the following conditions are satisfied:

- (i) The force deflection plots, of all tests on prototype specimen (excluding viscous damper component) shall show positive incremental force-carrying capacity so as to meet the restoring force requirements.
- (ii) In the thermal test on prototype, the maximum measured force shall be less than the design value.
- (iii) In the other tests on prototype, the maximum displacement shall be less than the design displacement.
- (iv) In the three cycles of seismic tests, the average effective stiffness shall be within 10% of the value used in the design.
- (v) In the seismic test, in each of the three cycles, the measured minimum effective stiffness shall not be less than the 80% of the maximum effective stiffness.
- (vi) In the second seismic test (Seismic -2), the minimum effective stiffness shall not be less than 80% of the maximum effective stiffness. Similarly, the minimum area under *EDC* shall not be less than 70% of the maximum *EDC* area.

17.7 Requirements for Elastomeric Bearings

In addition to the normal tests and designs, which are done for non-seismic conditions, the elastomeric bearings shall comply with the design described in this section. The elastomeric bearings shall use steel reinforcement; the use of fabric reinforcement is not permitted.

17.7.1 Shear Strain Components for Isolation Design

The various components of shear strain in the bearing shall be computed as:

Table .12 Shear Strain Components

Component	Shear strain
Shear strain due to vertical load	$\gamma_c \begin{cases} = \frac{3S_1 P}{2ArG(1+2kS^2)} \text{ for } S_1 \leq 15 \\ = \frac{3P \left(1 + \frac{8GkS_1^2}{K}\right)}{4GkSA_r} \text{ for } S_1 > 15 \end{cases}$
Shear strain due to non-seismic lateral displacement	$\gamma_{s,s} = \frac{\Delta_s}{T_r}$
Shear strain due to seismic lateral displacement	$\gamma_{s,eq} = \frac{d_1}{T_r}$
Shear strain due to rotation	$\gamma_r = \frac{B^2 \theta}{2t_1 T_r}$

Where,

K is the bulk modulus of the elastomer, in the absence of measured data, the value of K may be taken as 2000 MPa. The shape factor, S_1 shall be taken as the plan area of the elastomer layer divided by the area of perimeter free to bulge.

Δ_s is non seismic lateral displacement resulting from creep, post-tensioning, shrinkage and thermal effects,

d_i is seismic lateral displacement,

θ is design rotation and shall not be less than 0.005 rad.

T_r is total elastomer thickness,

$\bar{k}$ is the material constant, and

t_i is the thickness of i^{th} layer.

B is bonded plan dimension or bonded diameter in loaded direction of rectangular bearing or diameter of circular bearing

17.7.2 Load Combinations

The elastomeric bearing shall satisfy the following load combinations of shear strains:

$$\gamma_c \leq 2.5$$

$$\gamma_c + \gamma_{s,s} + \gamma_r \leq 5.0$$

$$\gamma_c + \gamma_{s,eq} + 0.5\gamma_r \leq 5.5$$

where, shear strains are as explained in Table 12 above.

17.7.3 Construction Requirements

In addition to non-seismic construction requirements following shall be met with:

- (i) The layers of elastomeric bearings shall integrally bond during vulcanization and cold bonding is not allowed.
- (ii) A 5-minute proof load test with 1.5 times the dead load and live load shall be conducted on each bearing. There shall be no bulging due to poor lamination.
- (iii) All bearings shall be tested in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be average dead load of all bearings and they shall be subjected to five fully reversed cycles of loading at the total design displacement or 50% of elastomer thickness. For each group of similar types of bearings, the effective stiffness and EDC shall be averaged. For individual bearings, the effective stiffness shall be within 20% of design values and EDC shall not be less than 25% of the design value. The average value of effective stiffness of a group shall be within 10% of design value and the EDC value shall not be less than 15% of the design value.

After all the tests, all the bearing shall be visually inspected for defects. If there is lack of bond between rubber and steel, or laminate placement fault, or permanent deformation or surface cracks on rubber that are wider or deeper than $2/3^{rd}$ rubber thickness, then, the bearing shall be rejected.

18. Post-Earthquake Operation and Inspection

The response of railway tracks and bridges to an earthquake would depend on distance from epicenter and nature of attenuation. The post-earthquake train operations in the region shall be cautiously started. Detailed procedure for post-earthquake operations and inspection is explained in Appendix – J.

Appendix – (A) References

- 1) IIT-K RDSO Guidelines on Seismic Design of Railway Bridges –November 2010
- 2) “Manual for Railway Engineering”, American Railway Engineering and Maintenance-of-Way Association (AREMA), USA, 2007.
- 3) “AASHTO LRFD Bridge Design Specifications”, American Association of State Highway and Transportation Officials (AASHTO), USA, 2007.
- 4) “Seismic Design Criteria”, California Department of Transportation (CALTRANS), USA, 2006.
- 5) “Design of Structures for Earthquake Resistance”, Eurocode 8: Part 2: Bridges, European Committee for Standardization, 2005.
- 6) “Bridge Manual”, TRANSIT, Wellington, New Zealand, 2005.
- 7) “Specifications for Highway Bridges”, Part V Seismic Design Japan Road Association, 2003.
- 8) “Seismic Design for Railway Structures”, Railway Technical Research Institute (RTRI), Japan, 2000.
- 9) “Seismic Design Criteria for High Speed Rail Project“, National Center for Research on Earthquake Engineering, Taiwan, 1992.
- 10) Murty, C.V.R. and Jain, S.K. “A Proposed Draft for Indian Code Provisions on seismic design for bridges-Part I: Code”, Journal of Structural Engineering, Vol.26, No. 3, 223-234, 2000.
- 11) Murty, C.V.R. and Jain, S.K. “A Proposed Draft for Indian Code Provisions on seismic design for bridges-Part II: Code”, Journal of Structural Engineering, Vol.27, No. 2, 79-89, 2000
- 12) Skinner ,R.I. , Kelly , T.E. and Robinson , B. “ Seismic Isolation for Designers and Structural Engineers”, Robinson Seismic Ltd.
- 13) “AASHTO Guide Specifications for Seismic Isolation Design “American Association of State Highway and Transportation Officials (AASHTO), USA, 2000.

Appendix – (B) Relevant Codes and Standards

The following Codes/Standards are necessary adjuncts to these guidelines:

- 1) IRC:6-2014 Standard Specification and Code of Practice for Road Bridges,
- 2) IRC:83 (Part III) Standard Specification and Code of (Part III) Practice for Road Bridges Section IX: - Bearings, 2002
- 3) IRS Code of Practice For Plain, Reinforced & Prestressed Concrete For General Bridge Construction, Third Revision, 2004
- 4) IRS Code of Practice For the Design of Sub-Structures and Foundation of Bridge, Second Revision, 2004
- 5) IRS Code of Practice For the Design of Steel or Wrought Iron Bridges Carrying Rail, Road or Pedestrian Traffic, Second Revision, 2004
- 6) IRS Bridge Rules specifying the Loads for Bridge Design of Super Structure and Sub-Rules Structure of bridges, Second Revision, 2004
- 7) IS 1893 (Part I) Criteria for Earthquake Resistant Design of Structures, Part I: General Provisions and Buildings, 2002
- 8) IS 13920 Ductile Detailing of Reinforced Concrete Structure Subjected to Seismic Forces-Code of Practice, 1993

Appendix – (C) Ductile Detailing Specifications

(Clause 15.0)

C-1. 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 (Fig. C-1). 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 will occur in the substructure. However, in case of a wall type substructure, the nonlinear behaviour may occur in the foundation-ground system.

C-2. Specification

C-2.1 Minimum grade of concrete should be M25 ($f_{ck} = 25$ MPa).

C-2.2 Steel reinforcement of grade Fe 415 (see IS 1786: 2008) or less shall be preferred. However, high strength deformed steel bars of grades Fe500 & Fe550 having elongation more than 14.5 percent and conforming to other requirements of IS 1786 : 2008 may also be used for the reinforcement.

C-3. 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 (e.g., pier column ends) and not in other uncontrolled places.

C-4. Longitudinal Reinforcement

The area of the longitudinal reinforcement shall not be less than 0.8 percent nor more than 6 percent, of the gross cross section area A_g . Splicing of flexural region reinforcement 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 hinging can occur. The splices should be proportioned as a tension splice.

C-4.1 The reduction of longitudinal reinforcement at mid-height in piers should not be carried out except in tall pier.

C-4.2 In case of high bridge piers such as of height equal to 30m or more, the reduction of reinforcement at mid height may be done. In such cases the following method should be adopted:

- (i) 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 is likely to occur.
- (ii) The interval between hoop ties is specified to be less than 150mm in a reinforcement position. The interval between hoop ties shall not change abruptly, the change must be gradual.

C-5. Transverse Reinforcement

The transverse reinforcement for circular columns shall consist of spiral or circular hoops. Continuity of these reinforcements should be provided by either (Fig. C -2(a) or C-2.(b)):

- (a) Welding, where the minimum length of weld should be 12 bar diameter, and the minimum weld throat thickness should be 0.4 times the bar diameter.
- (b) Lapping, where the minimum length of lap should be 30 bar diameters 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 (Figure C-2.c). When hoop ties are joined in any place other than a corner the hoop ties shall overlap each other by a length 40 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.

C-6. Design of Plastic Hinge Regions

C-6.1 Seismic Design Force for Substructure

Provisions given in this Appendix for the ductile detailing of RC members subjected to seismic forces shall be adopted for supporting components of the bridge. The design shear force at the critical section(s) of substructures shall be the lower of the following:

- (a) Maximum elastic shear force at the critical section of the bridge component divided by the response reduction factor for that component as per Table 7, and

- (b) Maximum shear force that develops when
 - (i) The substructure has maximum moment that it can sustain (*i.e.*, the overstrength plastic moment capacity as per Clause 6.2) in single-column or single-pier type substructure.
 - (ii) Plastic moment hinges are formed in the substructure so as to form a collapse mechanism in multiple-column frame type or multiple-pier type substructures, in which the plastic moment capacity shall be the overstrength plastic moment capacity as per Clause C 6.2.

In a single-column type or pier type substructure, the critical section is at the bottom of the column or pier as shown in Figure C-1(a) and, in multi-column frame-type substructures or multi-pier substructures, the critical sections are at the bottom and/or top of the columns/piers as shown in Figure C-1(b).

C-6.2 Over strength Plastic Moment Capacity

The over strength plastic moment capacity at a 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.

C-6.3 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,

- (a) 1.5 times the column diameter or 1.5 times the larger cross sectional dimension where yielding occurs
- (b) 1/6 of clear height of the column for frame pier (*i.e* when hinging can occur at both ends of the column)
- (c) 1/4 of clear height of the column for cantilever pier (*i.e* when hinging can occur at only one end of the column)
- (d) 600 mm

C-6.4 Spacing of Transverse Reinforcement

The spacing of hoops used as special confining reinforcement shall not exceed

- (i) 1/5 times the least lateral dimension of the cross section of column,
- (ii) 6 times the diameter of the longitudinal bar,
- (iii) 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 350 mm centre to centre. If the length of any side of the stirrups exceeds 350 mm, a cross tie shall be provided. Alternatively, overlapping stirrups may be provided within the column.

C-6.5 Amount of Transverse Steel to be Provided

C-6.5.1 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

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

or,

$$A_{sh} = 0.024SD_k \frac{f_{ck}}{f_y}$$

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

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

C-6.5.2 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

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

Or,

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

Whichever is the greater

where

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

A_r = Area of confined core concrete in the rectangular hoop measure to its outer side dimensions.

Note: Cross ties where used should be of the same diameter as the peripheral hoop bar and A_r shall be measured as the overall core area, regardless the hoop area. The hooks of cross ties shall engage peripheral longitudinal bars.

C-6.5.3 Unsupported length of rectangular hoops shall not exceed 300mm.

C-6.5.4. For ductile detailing of hollow cross-section of pier special literature may be referred. Some of the provisions for hollow RC piers are:

- i) For hollow cylindrical piers, in the plastic hinge region, the ratio of internal diameter to thickness should not exceed 8.0.
- ii) For wall type hollow piers, in the plastic region, the ratio of clear width of the wall to thickness should not exceed 8.0.

C-7. 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:

- (i) To reliably protect the bridge against collapse so that it will be available for service after a major shaking.
- (ii) 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”.

C-7.1 Column Shear and Transverse Reinforcement

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

$$V_u = \frac{\sum M^O}{h}$$

where

$\sum M^O$ = the sum of the overstrength moment capacities of the hinges resisting lateral loads, as detailed. In case of twin pier this would be the sum of the overstrength moment capacities at the top and bottom of the column. For single stem piers the overstrength 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 calculated 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, nor 300 mm.

C-8. Design of Joints:

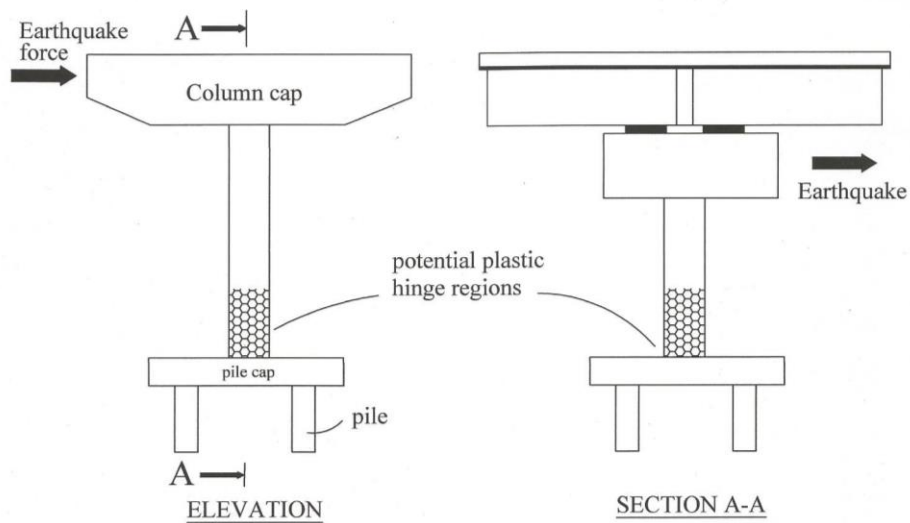
Beam-column joints should be designed properly to resist the forces caused by axial loads, bending and shear forces in the joining members. Forces in the joint should be determined by considering a free body of the joint with the forces on the joint member boundaries properly represented.

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

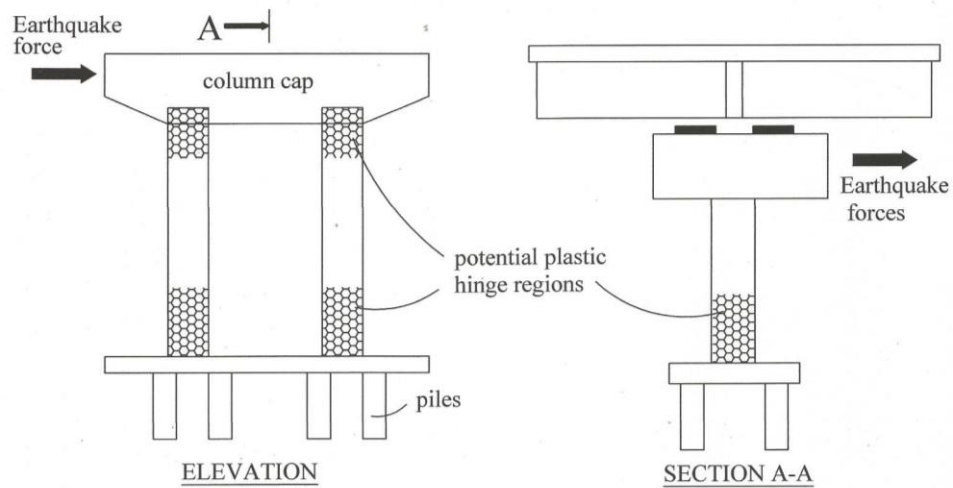
C-8.1 Ductility of all the joints in the structure may be ensured by offsetting the splices / couplers where the area of reinforcement provided is at least twice the area required by analysis staggered 600 mm minimum.

C-8.2 The pier – foundation joint or the slab – pier joint (in case of integral slab – bridges) must be checked for principal tensile stress in the concrete around the junction, following an appropriate prevailing method. The un-cracked joint may be designed by keeping the principal stresses in the joint region below direct tensile strength of concrete. If the joint cannot be prevented from cracking, additional vertical stirrups may be added to the external concrete region around the column.

The joint stresses may be assumed to disperse 45° around the column as per prevailing practices.

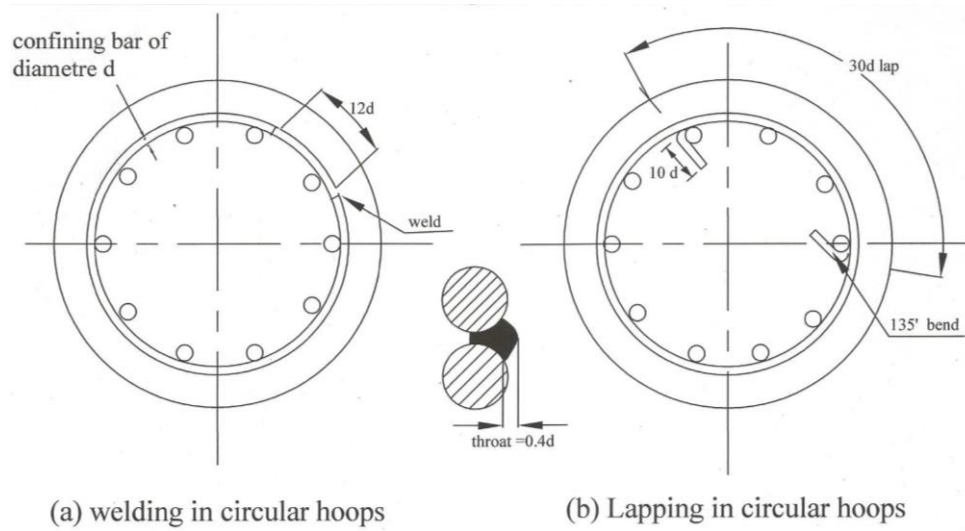
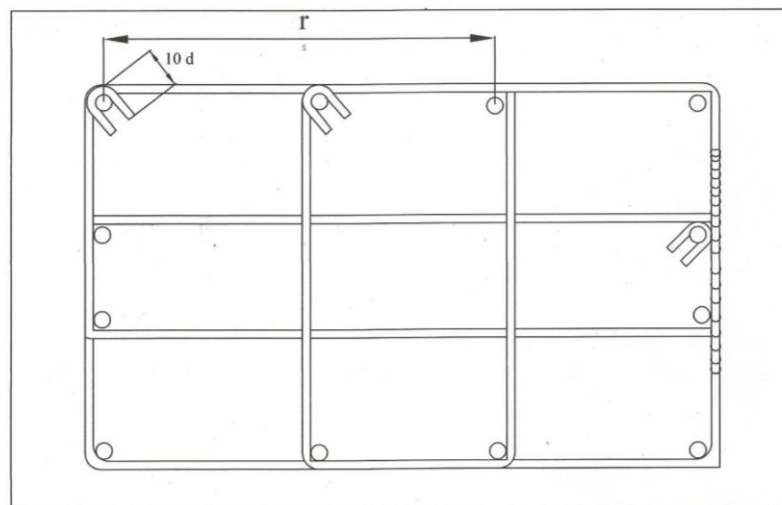


(a) SINGLE COLUMN OR PIER TYPE SUBSTRUCTERS



(b) MULTI COLUMN OR FRAME TYPE SUBSTRUCTERS

(Fig. C-1 Potential location of plastic hinges in substructures: (Clause C-1))



(Fig. C-2 Transverse reinforcement in column (Clause C-5))

Appendix – (D) Zone Factors for Some Important Towns (Clause 6.1)

<i>Town</i>	<i>Zone</i>	<i>Zone Factor, Z</i>	<i>Town</i>	<i>Zone</i>	<i>Zone Factor, Z</i>
Agra	III	0.16	Kanchipuram	III	0.16
Ahmedabad	III	0.16	Kanpur	III	0.16
Ajmer	II	0.10	Karwar	III	0.16
Allahabad	II	0.10	Kohima	V	0.36
Almora	IV	0.24	Kolkata	III	0.16
Ambala	IV	0.24	Kota	II	0.10
Amritsar	IV	0.24	Kurnool	II	0.10
Asansol	III	0.24	Lucknow	III	0.16
Aurangabad	II	0.10	Ludhiyana	IV	0.24
Bahraich	IV	0.24	Madurai	II	0.10
Bangalore	II	0.10	Mandi	V	0.36
Barauni	IV	0.24	Mangalore	III	0.16
Bareilly	III	0.16	Monghyr	IV	0.24
Belgaum	III	0.16	Moradabad	IV	0.24
Bhatinda	III	0.16	Mumbai	III	0.16
Bhilai	II	0.10	Mysore	II	0.10
Bhopal	II	0.10	Nagpur	II	0.10
Bhubaneswar	III	0.16	Nagarjunasagar	II	0.10
Bhuj	V	0.36	Nainital	IV	0.24
Bijapur	III	0.16	Nasik	III	0.16
Bikaner	III	0.16	Nellore	III	0.16
Bokaro	III	0.16	Osmanabad	III	0.16
Bulandshahr	IV	0.24	Panjim	III	0.16
Burdwan	III	0.16	Patiala	III	0.16
Calicut	III	0.16	Patna	IV	0.24
Chandigarh	IV	0.24	Pilibhit	IV	0.24
Chennai	III	0.16	Pondicherry	II	0.10

Chitradurga	II	0.10	Pune	III	0.16
Coimatore	III	0.16	Raipur	II	0.10
Cuddalore	III II	0.16	Rajkot	III	0.16
Cuttack	III	0.16	Ranchi	II	0.10
Darbhanga	V	0.36	Roorkee	IV	0.24
Darjeeling	IV	0.24	Rourkela	II	0.10
Dharwad	III	0.16	Sadiya	V	0.36
Dehra Dun	IV	0.24	Salem	III	0.16
Dharampuri	III	0.16	Simla	IV	0.24
Delhi	IV	0.24	Sironj	II	0.10
Durgapur	III	0.16	Solapur	III	0.16
Gangtok	IV	0.24	Srinagar	V	0.36
Guwahati	V	0.36	Surat	III	0.16
Goa	III	0.16	Tarapur	III	0.16
Gulbarga	II	0.10	Tezpur	V	0.36
Gaya	III	0.16	Thane	III	0.16
Gorakhpur	IV	0.24	Thanjavur	II	0.10
Hyderabad	II	0.10	Thiruvananthapuram	III	0.16
Imphal	V	0.36	Tiruchirappali	II	0.10
Jabalpur	III	0.16	Thiruvannamalai	III	0.16
Jaipur	II	0.10	Udaipur	II	0.10
Jamshedpur	II	0.10	Vadodara	III	0.16
Jhansi	II	0.10	Varanasi	III	0.16
Jodhpur	II	0.10	Vellore	III	0.16
Jorhat	V	0.36	Vijayawada	III	0.16
Kakrapara	III	0.16	Vishakhapatnam	II	0.10
Kalapakkam	III	0.16			

Appendix – (E) Pushover Analysis

(Clause 10.0)

E-1 Pushover analysis is performed to explicitly ascertain the displacement capacity of the bridge structure. This analysis is explained for the reinforced concrete structures. This is done with the help of static nonlinear analysis, in which nonlinear properties of concrete and reinforcing steel are used. The displacement capacity shall be greater than the displacement demand. The procedure explained herein, is based on Caltrans (2006).

E-2 Displacement demand

The displacement demand is twice the elastic displacement obtained using a linear analysis. This displacement demand is doubled due to use of factor $Z/2$ in the seismic force calculation for linear analysis. The single mode method (Clause 7.0) or multi-mode method (Clause 8.0) may be used as per the requirements of Clause 6.3.1. From the displacement demand, Δ_D , the displacement ductility demand is obtained as

$$\mu D = \Delta_D / \Delta_Y$$

where, Δ_Y is yield displacement of the system from its initial position to the formation of plastic hinge.

E-3 Displacement capacity

The local displacement capacity of a member is obtained from its curvature capacity, which is determined from the moment curvature ($M-\phi$) analysis. The expected stress strain curve or material properties of concrete and steel are used. For confined concrete, the Mander's model shown in Fig. E-1 is used, and the stress-strain model shown in Fig. E-2 is used for steel. The moment curvature analysis obtains the curvatures associated with a range of moments for a cross-section, based on the strain compatibility force equilibrium conditions. The $M-\phi$ curve (Fig. E-3) can be idealized with an elastic perfectly plastic curve to estimate the plastic moment capacity of a cross-section. The idealized plastic moment capacity is obtained by balancing the areas between the actual curve and the idealized curve beyond the first reinforcing bar yield point (Fig. E-3).

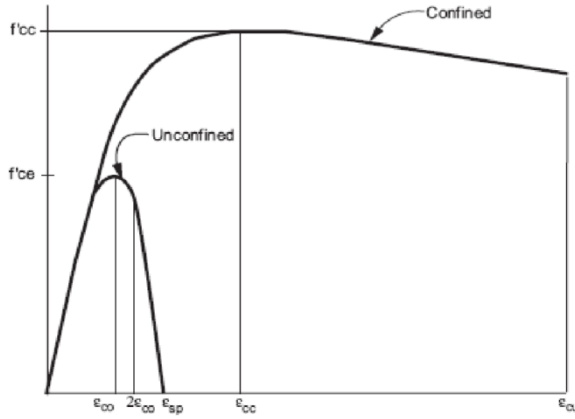
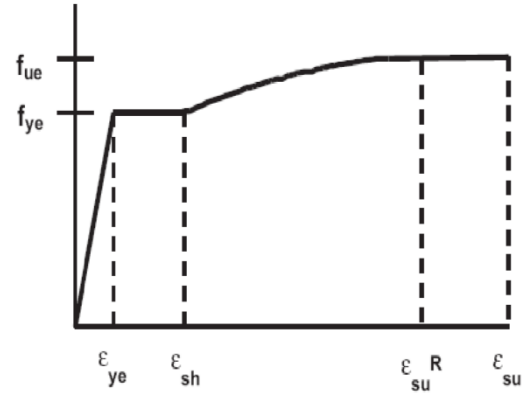


Fig E-1 Stress strain model for concrete Fig E-2 Stress strain model for steel

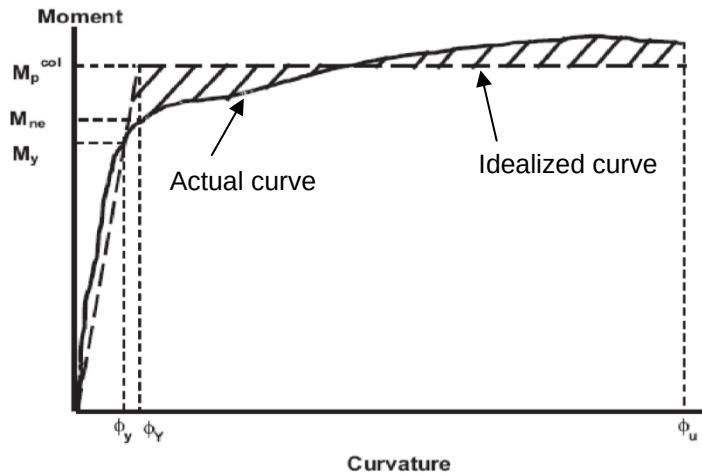


Fig. E-3 Moment curvature (M-Φ) curve

Here, M_p is the plastic moment capacity, M_y is the first reinforcing bar yield point & M_{ne} is the expected nominal moment capacity, ϕ_u is the curvature capacity at the failure limit state defined as the concrete strain reaching ϵ_{cu} or the confinement reinforcing steel reaching the reduced ultimate strain ϵ_{su}^R . Similarly, ϕ_y is the idealized yield curvature defined by an elastic-perfectly plastic representation of $M-\phi$ curve (Fig. E-3). The idealized plastic curvature capacity, ϕ_p , which is assumed constant over plastic hinge length, L_p is given by $\phi_p = \phi_u - \phi_y$. The hinge length, L_p in mm is given by

$$L_p = 0.08L + 0.022f_{ye}d_{bl} \geq 0.044f_{ye}d_{bl} \quad \text{for columns (mm, MPa)}$$

$$L_p = G + 0.044f_{ye}d_{bl} \quad \text{for horizontally isolated flared columns}$$

Here, G is the gap between the isolated flare and the soffit of the bent cap. With reference to Fig. E-4, the plastic rotation capacity, $\theta_p = L_p \times \phi_p$ and

$$\Delta_p = \theta_p \times \left(L - \frac{L_p}{2} \right)$$

Then, the total displacement capacity of the column is given by

$$\Delta_c = \Delta_Y^{col} + \Delta_p$$

where, Δ_Y^{col} is the idealized yield displacement of the column (Fig. E-4).

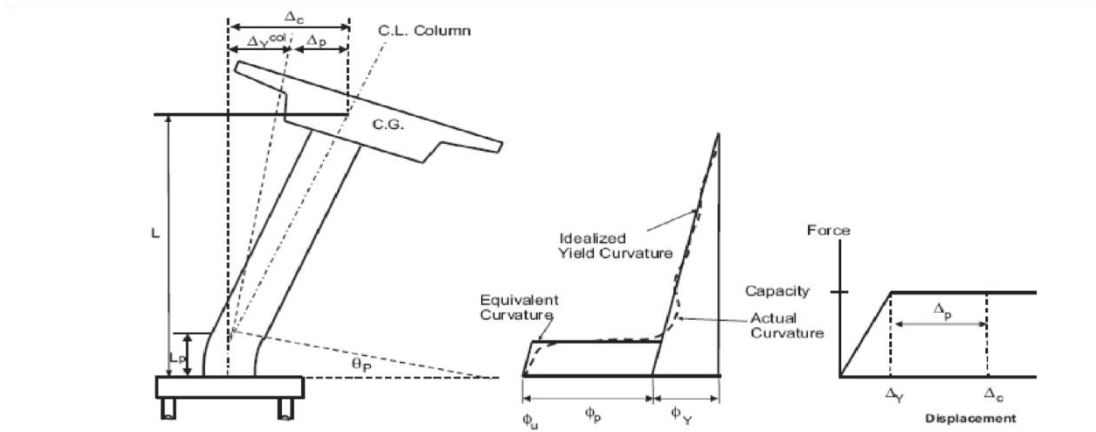


Fig. E-4 Lateral displacement capacity of fixed base column

The displacement capacity Δ_c thus obtained shall be greater than the demand Δ_D obtained from linear static analysis. The above described procedure to obtain the displacement capacity is for a cantilever column, fixed at the base and free at the top. Similarly, analysis can be done for fixed-fixed column. For a frame type substructure, M- ϕ curve is to be given for each member and the analysis becomes more involved, for which help of standard software may be required.

It shall be ensured that the flexural hinge occurs prior to shear failure of column, and hence, the nominal shear capacity shall be greater than the shear force corresponding to plastic hinge. Similarly, capacity protection shall be provided to the other adjacent components such as bent cap, pile cap etc.

Appendix – (F) Dynamic Earth Pressure

(Clause 12.3.1)

F-1. Dynamic earth pressure on abutments

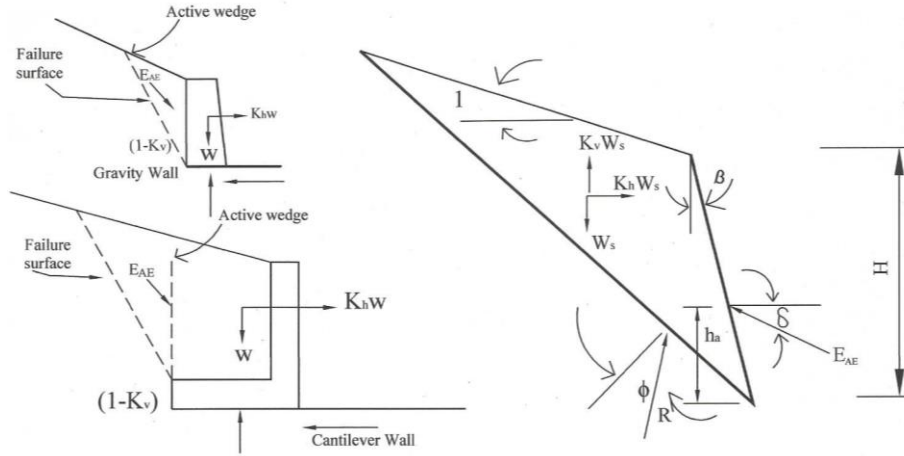


Figure F1: Seismic Active Earth Pressure On Retaining Walls

Figure F 1: Seismic Active Earth Pressure on Retaining Walls

F-1.1 Lateral Earth Pressure - The pressure from earth fill behind retaining walls during an earthquake shall be as given in F.1.1.1 to F.1.4.1. In the analysis, cohesion has been neglected. This assumption is on conservative side.

F-1.1.1 Active Pressure Due to Earth fill -The general conditions encountered for the design of retaining walls are illustrated in Fig. F 1. The total active pressure exerted against the wall shall be the maximum of the two given by the following expression:

$$E_{AE} = \frac{1}{2} \gamma H^2 (1 \pm A_v) K_{AE} \quad (\text{Eq. F.1})$$

Where the seismic active earth pressure coefficient K_{AE} is given by

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos \theta \cos^2 \beta \cos(\delta + \beta + \theta)} \times \left[1 + \frac{\sin(\phi + \delta) \sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta) \cos(\beta - i)} \right]^{-2} \quad (\text{Eq. F.2})$$

and where

γ = unit weight of soil (kN/m³)

H = height of wall in (m) Φ = angle of friction of soil ($^\circ$)

δ = angle of friction between soil and abutment ($^\circ$)

A_h =elastic seismic coefficient [see Clause 9.1]

A_v = vertical seismic coefficient– it's value being taken consistently throughout the stability analysis of wall equal to $2/3 A_h$.

$$\theta = \tan^{-1} \frac{A_h}{1 \pm A_v} \quad (^\circ)$$

i = backfill slope angle ($^\circ$)

β =slope of wall to the vertical ($^\circ$)

F.1.1.2 Point of Application – From the total pressure computed as above subtract the static active pressure obtained by putting $A_v = A_h = \theta = 0$ in the expression given by equation F.1 and F.2. 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.

F.1.2 Passive Pressure Due to Earth fill –The total passive pressure against the walls shall be the minimum of the two given by the following expression:

$$E_{PE} = \frac{1}{2} \gamma H^2 (1 \pm A_v) K_{PE} \quad (\text{Eq.F.3})$$

Where the seismic passive earth pressure coefficient K_{PE} is given by

$$K_{PE} = \frac{\cos^2(\phi - \theta + \beta)}{\cos \theta \cos^2 \beta \cos(\phi - \beta + \theta)} \times \left[1 - \sqrt{\frac{\sin(\phi - \phi) \sin(\phi - \theta + i)}{\cos(\phi - \beta + \theta) \cos(\beta - i)}} \right]^{-2} \quad (\text{Eq. F.4})$$

F.1.2.2 Point of application- From the static passive pressure obtained by putting $A_h = A_v = \theta = 0$ in the expression given by equation F.3 and F.4, subtracts 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.

F.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 earth fill surface shall be:

$$(E_{AE})_q = \frac{qH \cos \beta}{\cos(i - \beta)} \times (1 + A_v) K_{AE} \quad (\text{Eq. F.5})$$

F.1.3.1 Point of application- The dynamic increment in active pressure due to uniform surcharge shall be applied at an elevation of $0.66H$ above the base of the wall, while the static component shall be applied at mid-height of the wall.

F.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 earth fill shall be:

$$(P_{PE})_q = \frac{qH \cos \beta}{\cos(\beta - i)} K_{PE} \quad (\text{Eq. F.6})$$

F.1.4.1 Point of application- The dynamic decrement in passive pressures due to uniform surcharge shall be applied at an elevation of 0.66h above the base of the walls while the static component shall be applied at mid-height of the wall

F.2 Effect of Saturation on Lateral earth Pressure

F.2.1 For saturated earthfill, the saturated unit weight of the soil shall be adopted in the Equation F.1

F.2.2 For submerged earthfill, the dynamic increment (or decrement) in active and passive earth pressures during earthquakes shall be found from expressions given in equation F.2 and F.4 with the following modifications:

- a) The value of δ shall be taken as the value 1/2 of δ for dry backfill.
- b) The value of θ shall be taken as follows:

$$\theta = \tan^{-1} \left[\frac{\gamma_t A_h}{\gamma_b (1 \pm A_v)} \right] \quad (\text{Eq. F.7})$$

Where

γ_t = saturated unit weight of soil (kN/m³)

γ_b = submerged unit weight of soil (kN/m³)

A_h = elastic seismic coefficient

A_v = vertical seismic coefficient = 2/3 A_h

- c) Buoyant unit weight shall be used in equation F.1 and F.3 as the case may be

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

F.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.

F.3 Partially Submerged Backfill

The situations with partial submerged backfill may be handled by weighing unit weights based on the volume of soil in the failure wedge above and below the phreatic surface as shown in Figure F2. Equation F.7 shall be used to calculate θ using γ_{se} instead of γ_b . Then total active and passive pressure can be obtained from equation F.1 and F.2 using equivalent unit weight (γ_{se})

F.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.

NOTE - To ensure adequate factor of safety under earthquake condition, the design shall be such that the factor of safety against sliding shall be 1.25 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.

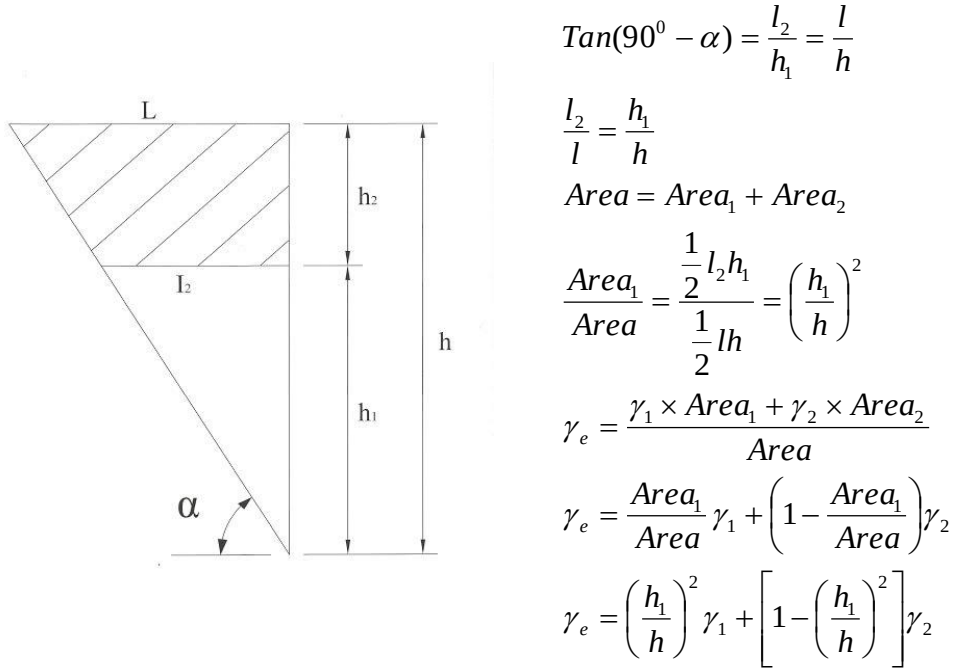


Figure F 2: Effective unit weight for partially submerged backfills

Notes:

- (1) Exact solution when $r_u = 0$
- (2) Approximate Solution when $r_u > 0$.

Appendix – (G) Simplified Procedure for Evaluation of Liquefaction Potential (Clause 13.3)

G-1 Cohesionless Soils

Due to the difficulties in obtaining and laboratory testing of undisturbed representative samples from most potentially liquefiable sites, in-situ testing is often relied upon for assessing the liquefaction potential of cohesion less soils. Liquefaction potential assessment procedures involving both the SPT and CPT are widely used in practice. The most common procedure used in engineering practice for the assessment of liquefaction potential of sands and silts is the *Simplified Procedure*¹. The procedure may be used with either SPT blow count, CPT tip resistance or shear wave velocity measured within the deposit as discussed below:

Step 1: The subsurface data used to assess liquefaction susceptibility should include the location of the water table, either SPT blow count (N), or tip resistance of a standard CPT cone (q_c) or the shear wave velocity, mean grain size (D_{50}), unit weight, and fines content of the soil (percent by weight passing the IS Standard Sieve No. 75 μ).

Step 2: Evaluate the total vertical stress (σ_v) and effective vertical stress (σ'_v) for all potentially liquefiable layers within the deposit.

Step 3: The following equation can be used to evaluate the *stress reduction factor* r_d :

$$r_d = 1 - 0.00765z \quad \text{for } z \leq 9.15 \text{ m and}$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 < z \leq 23 \text{ m}$$

Where z is the depth below the ground surface in meters.

Step 4: Calculate the *critical stress ratio* induced by the design earthquake, CSR , as;

$$CSR = 0.65(a_{\max} / g)r_d(\sigma_v / \sigma'_v)$$

Where σ_v and σ'_v are the total and effective vertical stresses, respectively, at depth z , $a_{\max}$ is the peak horizontal ground acceleration (PHGA), and g is the acceleration due to gravity. In the absence of site specific estimates of $a_{\max}$, the PHGA may be estimated by $a_{\max}/g = ZISa/g$, where Z is the zone factor obtained from Table-3 as described earlier, I is the importance factor as per Table-4 and Sa/g is response spectrum acceleration coefficient obtained from Clause 7.1. For estimating the vertical total and effective stresses, the water table should be assumed at the highest piezometric elevation likely to be encountered during the operational life of the dam or the embankment except where there is a free standing water column. For assessing liquefaction potential of soil layers underneath free standing water column, the height of free standing water should be neglected and water table should be assumed at the soil surface.

For assessing liquefaction susceptibility using the SPT go to Step 5a, for the CPT go to Step 5b, and the shear wave velocity go to Step 5c, to compute cyclic resistance ratio ($CRR_{7.5}$) for M_w 7.5 earthquakes. Cyclic resistance ratio, CRR for sites for earthquakes of other magnitudes or for sites underlain by non-horizontal soil layers or where vertical effective stress exceeds 1 atmospheric pressure is estimated by multiplying $CRR_{7.5}$ by three correction factors, k_m , k_α and k_σ respectively. Here correction factors for magnitude sloped stratigraphy and effective stress has been denoted with symbols k_m , k_α and k_σ , respectively. These correction factors are obtained from figures G-1, G-2 and G-3.

Step 5a:

Evaluate the *standardized SPT blow count* (N_{60}) which is the standard penetration test blow count for a hammer with an efficiency of 60 percent. Specifications of the “standardized” equipment corresponding to an efficiency of 60 percent are given in Table G-1 in the absence of test-specific energy measurement. The standardized SPT blow count is obtained from the equation:

$$N_{60} = N \cdot C_{60}$$

Where C_{60} is the product of various correction factors. Correction factors recommended by various investigators for some common SPT configurations are provided in Table G-2.

Calculate the *normalized standardized SPT blow count*, $(N_1)_{60}$ using $(N_1)_{60} = C_N N_{60}$, where $(N_1)_{60}$ is the standardized blow count

normalized to an effective overburden pressure of 98kPa in order to eliminate the influence of confining pressure. Stress normalization factor C_N is calculated from following expression:

$$C_N = \left(P_a / \sigma'_v \right)^{\frac{1}{2}}$$

Subjected to $C_N \leq 2$, where P_a is the atmospheric pressure. However, the closed-form expression proposed by Liao and Whitman (1986) may also be used:

$$C_N = 9.79 \left(1 / \sigma'_v \right)^{\frac{1}{2}}$$

The Critical Resistance Ratio (*CRR*) or the resistance of a soil layer against liquefaction is estimated from Figure G-5 for representative $(N_1)_{60}$ value of the deposit.

Step 5b:

Calculate normalized cone tip resistance, $(q_{c1N})_{cs}$, using $(q_{c1N})_{cs} = K_c \left(P_a / \sigma'_v \right)^n (q_c / P_a)$

Where q_c is the measured cone tip resistance corrected for thin layers, exponent n has a value of 0.5 for sand and 1 for clay, and K_c is the correction factor for grain characteristics estimated as follows.

$K_c = 1.0$ for $I_c \leq 1.64$ and

$$K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88 \quad \text{for } I_c > 1.64$$

The soil behavior type index, I_c , is given by $I_c = \sqrt{(3.47 - \log Q)^2 + (1.22 + \log F)^2}$

Where $Q = [(q_c - \sigma'_v) / P_q] \left(P_a / \sigma'_v \right)^n$, $F = [f / (q_c - \sigma'_v)] \times 100$, f is the measured sleeve friction and n has the same values as described earlier. Assess susceptibility of a soil to liquefaction using Figure G-6.

The *CRR* for a soil layer is estimated from Figure G-6 using the $(q_{c1N})_{cs}$ value representative of the layer.

Although soils with $I_c > 2.6$ are deemed non-liquefiable, such deposits may soften and deform during earthquakes. General guidance is not available to deal with such possibilities.

Softening and deformability of deposits with $I_c > 2.6$ should thus be treated on a material specific basis.

Step 5c:

Calculate normalized shear wave velocity, V_{s1} , for clean sands using: $V_{s1} = V_s \times (P_a / \sigma'_v)^{0.25}$ subjected to $V_{s1} \leq 1.3V_s$.

The CRR for a soil layer is estimated from Figure G-7 using the V_{s1} value representative of the layer.

Appropriate $CRR_{V_{s1}}$ curve should be used in this assessment depending on the fines content of the layer.

Step 6: Correct $CRR_{7.5}$ for earthquake magnitude (M_w), stress level and for initial static shear using correction factors k_m, k_σ and k_α , respectively, according to:

$$CRR = CRR_{7.5} \cdot k_m \cdot k_\sigma \cdot k_\alpha$$

where, k_m, k_σ, k_α are correction factors, respectively for magnitude correction (Figure G-1), effective overburden correction (Figure G-2) and sloping ground correction (Figure G-3), in combination with Figure G-4. The Critical Stress ratio $CRR_{7.5}$ is estimated from Figure G-5 for SPT, Figure G-6 for CPT and Figure G-7 for shear wave velocity data.

Step 7: Calculate the factor of safety against initial liquefaction, FS , as:

$$FS = CRR / CSR$$

Where CSR is as estimated in Step 4 and CRR is from Step 6a, 6b or 6c. When the design ground motion is conservative, earthquake-related permanent ground deformation is generally small if $FS \geq 1.1$.

G-2 Cohesive Soils

Cohesive soils are often deemed to be non-liquefiable if any one of the following conditions is not satisfied (Figure G-8a):

- Percent (by weight) finer than 5 $\mu m \leq 15$ %
- $w_l \leq 35$ %
- $w_n \leq 0.9 \times w_l$

where w_l is the Liquid Limit and w_n is the Natural Moisture Content, respectively. These conditions are collectively referred to as the Chinese Criteria. Since the Chinese Criteria are not always conservative, Seed et al. recommend the following alternative (Figure G-8b):

- Cohesive soils should be considered liquefiable if $w_l \leq 37$ %, $I_p \leq 12$ % and $w_n \leq 0.85 \leq w_l$, where I_p is the Plasticity Index
- Liquefaction susceptibility of soils should be considered marginal if $w_l \leq 47$ %, $I_p \leq 20$ % and $w_n \leq 0.85 \leq w_l$, where I_p is the Plasticity Index and for such soils liquefaction susceptibility should be obtained from laboratory testing of undisturbed representative samples

Cohesive soils should be considered non-liquefiable if $w_l \geq 47$ % or $I_p \geq 20$ % or $w_n \geq 0.85 \geq w_l$, where I_p is the Plasticity Index

Table G-1: Recommended “Standardized” SPT Equipment

Element	Standard Specification
Sampler	Standard split-spoon sampler with: (a) Outside diameter = 51 mm, and Inside Diameter = 35 mm (constant – i.e., no room for liners in the barrel)
Drill Rods	A or AW-type for depths less than 15.2 m; N- or NW-type for greater depths
Hammer	Standard (safety) hammer: (a) drop hammer (b) weight = 65 kg; (c) drop = 750 mm (d) delivers 60% of the theoretical potential energy
Rope	Two wraps of rope around the pulley
Borehole	100 to 130mm diameter borehole
Drill Bit	Upward deflection of drilling mud (tricone or baffled drag bit)
Blow Count Rate	30 to 40 blows per minute
Penetration Resistant Count	Measured over range of 150 to 450 mm of penetration into the ground

Notes:

- (1) If the equipment meets the above specifications, $N = N_{60}$ and only a correction for overburden are needed.
- (2) This specification is essentially the same to the ASTM D 1586 standard.

Table G-2: Correction Factors for Non-Standard SPT Procedures and Equipment

Correction for	Correction Factor
Nonstandard Hammer Type (<i>DH</i> = doughnut hammer; <i>ER</i> = energy ratio)	$C_{HT} = 0.75$ for <i>DH</i> with rope and pulley $C_{HT} = 1.33$ for <i>DH</i> with trip/auto and <i>ER</i> = 80
Nonstandard Hammer Weight or Height of fall (<i>H</i> = height of fall in mm; <i>W</i> = hammer weight in kg)	$C_{HW} = \frac{H \times W}{63.4 \times 762}$
Nonstandard Sampler Setup (standard samples with room for liners, but used without liners)	$C_{SS} = 1.10$ for loose sand $C_{SS} = 1.20$ for dense sand
Nonstandard Sampler Setup (standard samples with room for liners, but liners are used)	$C_{SS} = 0.90$ for loose sand $C_{SS} = 0.80$ for dense sand
Short Rod Length	$C_{RL} = 0.75$ for rod length 0-3 m
Nonstandard Borehole Diameter	$C_{BD} = 1.05$ for 150 mm borehole diameter $C_{BD} = 1.15$ for 200 mm borehole diameter

Notes:

N = Uncorrected SPT blow count.

$C_{60} = C_{HT} C_{HW} C_{SS} C_{RL} C_{BD}$

$N_{60} = N C_{60}$

C_N = Correction factor for overburden pressure

$(N_1)_{60} = C_N N_{60} = C_N C_{60} N$

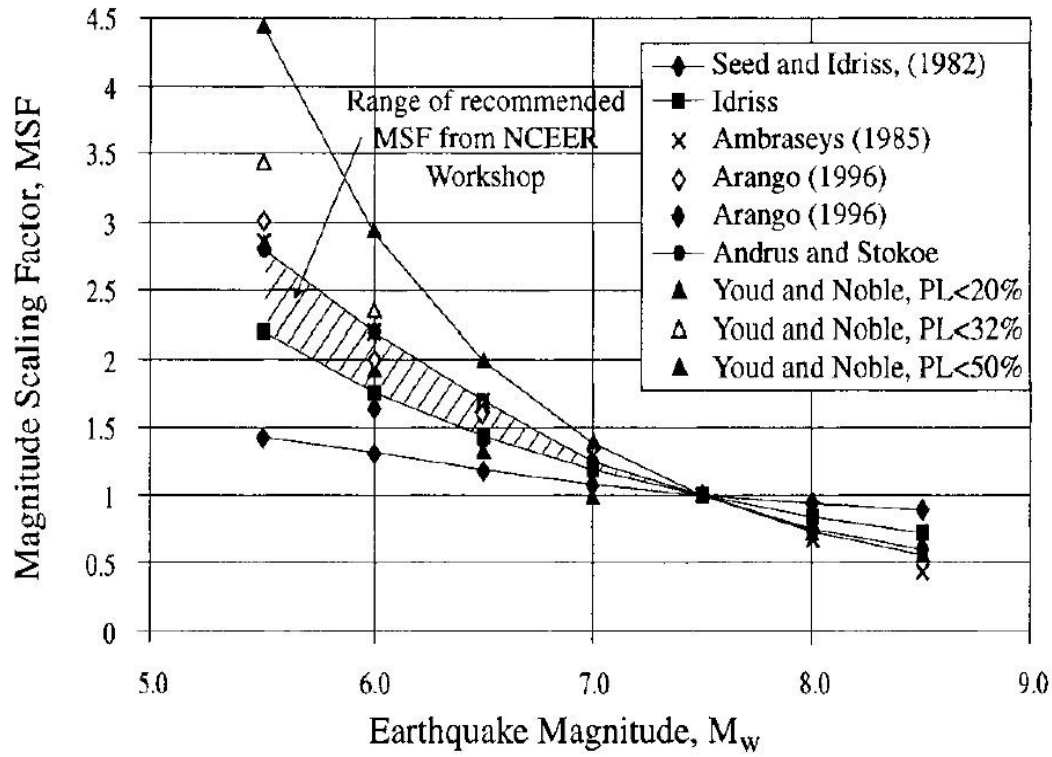


Figure G-1: Magnitude Correction factor

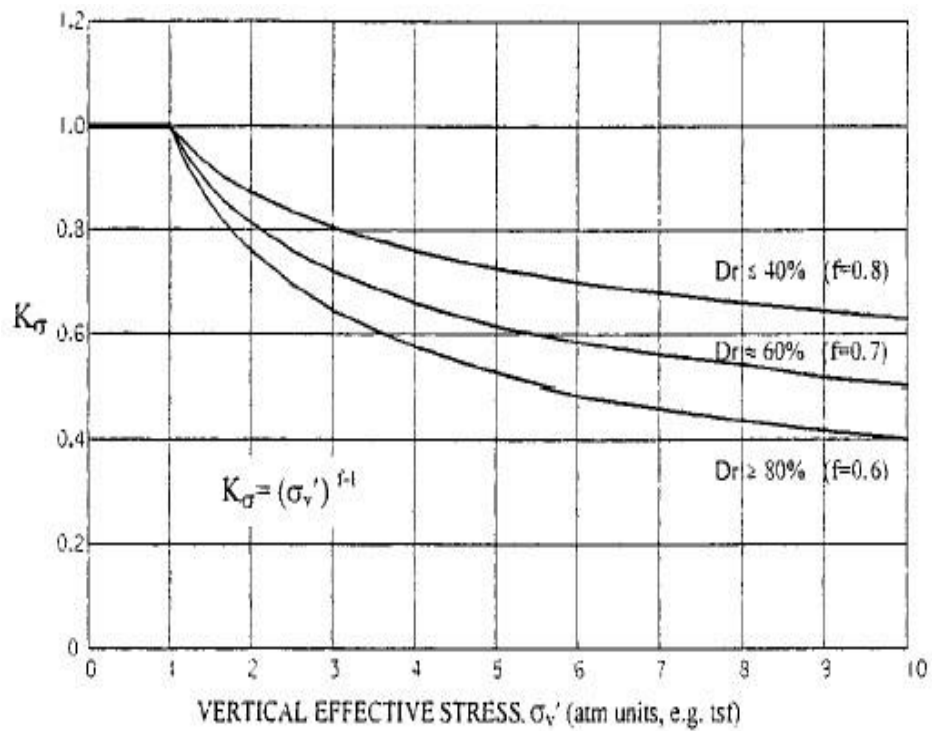


Figure G-2: Stress Correction factor

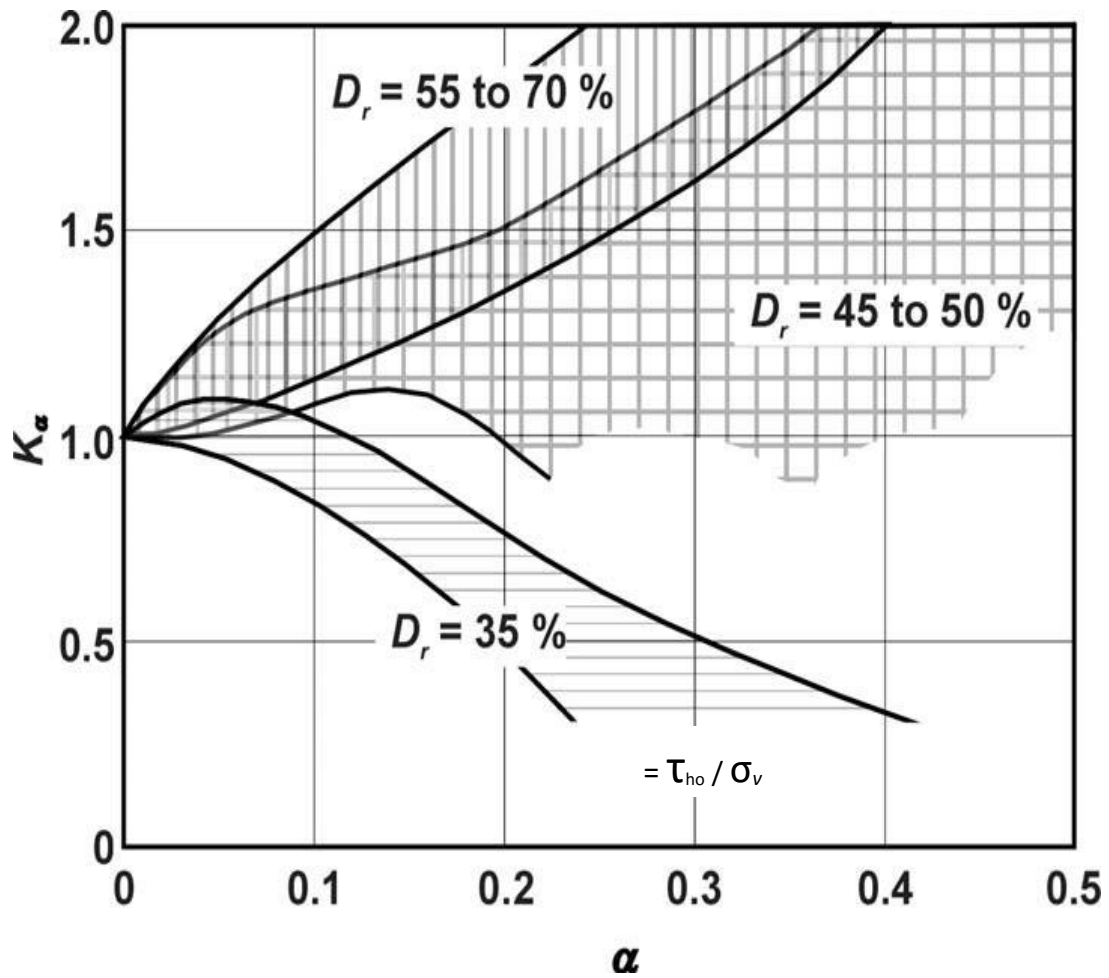


Figure G-3: Correction for initial static shear (Note: Initial static shear for an embankment may be estimated from Figure G-4)



$$\tau_{\max} = \frac{PZ}{\pi\alpha} \left(\log^2 \frac{R_2}{R_1} + \alpha^2 \right)^{1/2}$$

74

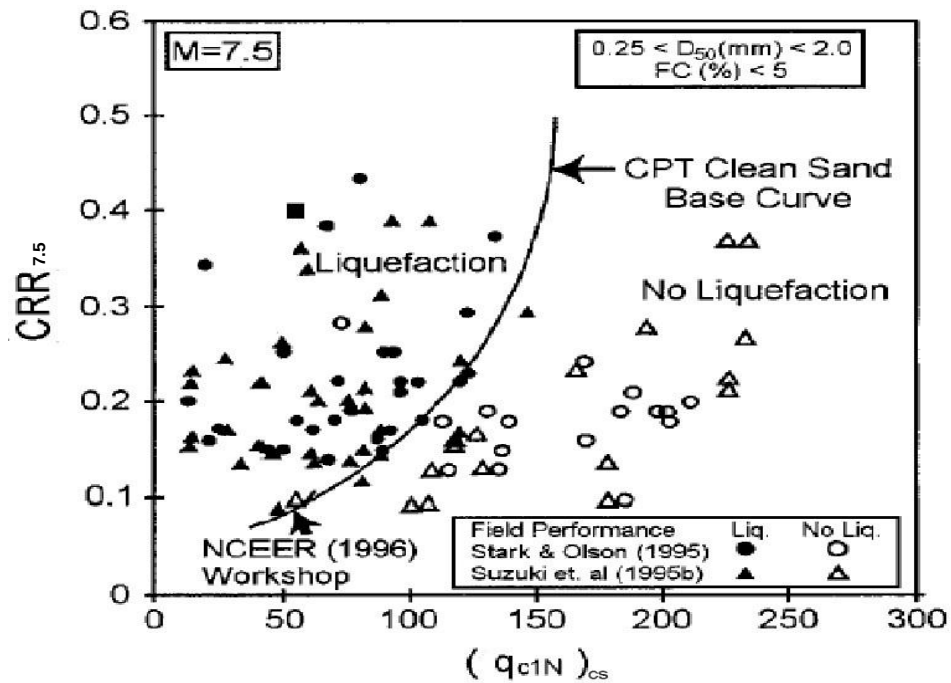


Figure G-6: Relationship between CRR and $(q_{c1N})_{cs}$ for M_w 7.5 earthquakes

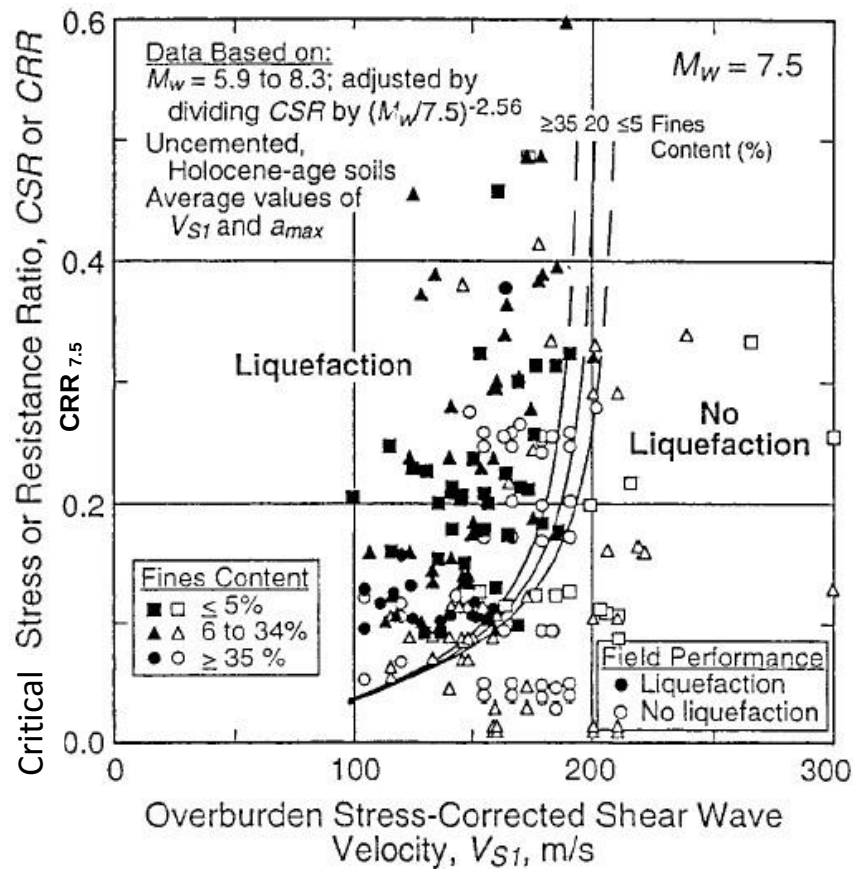


Figure G-7: Relationship between CRR and V_{s1} for M_w 7.5 earthquakes

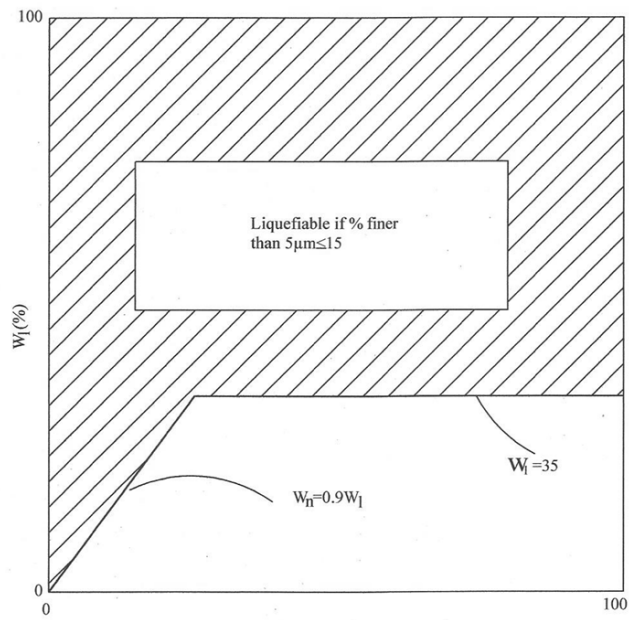


Figure G-8a The Chinese Criteria (Seed et.al.,2003)

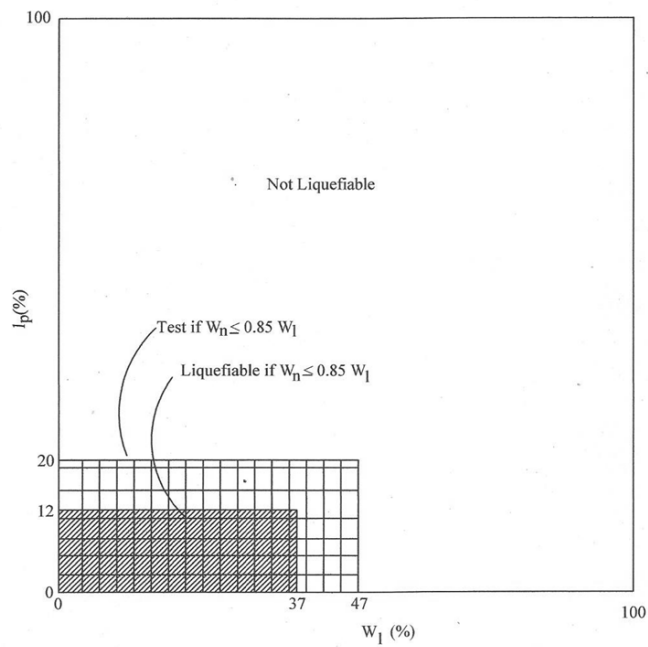


Figure G-8b Proposal of Seed et.al.(2003)

Appendix – (H) System property modification factors

(Clause 17.3)

H-1 General

$$K_{d,max} = K_d \times \lambda_{max,Kd} \quad \text{and} \quad K_{d,min} = K_d \times \lambda_{min,Kd}$$

$$Q_{d,max} = Q_d \times \lambda_{max,Qd} \quad \text{and} \quad Q_{d,min} = Q_d \times \lambda_{min,Qd}$$

These factors are given by

$$\lambda_{min,Kd} = \lambda_{min,t,Kd} \times \lambda_{min,a,Kd} \times \lambda_{min,v,Kd} \times \lambda_{min,tr,Kd} \times \lambda_{min,c,Kd} \times \lambda_{min,scrag,Kd}$$

$$\lambda_{max,Kd} = \lambda_{max,t,Kd} \times \lambda_{max,a,Kd} \times \lambda_{max,v,Kd} \times \lambda_{max,tr,Kd} \times \lambda_{max,c,Kd} \times \lambda_{max,scrag,Kd}$$

$$\lambda_{min,Qd} = \lambda_{min,t,Qd} \times \lambda_{min,a,Qd} \times \lambda_{min,v,Qd} \times \lambda_{min,tr,Qd} \times \lambda_{min,c,Qd} \times \lambda_{min,scrag,Qd}$$

$$\lambda_{max,Qd} = \lambda_{max,t,Qd} \times \lambda_{max,a,Qd} \times \lambda_{max,v,Qd} \times \lambda_{max,tr,Qd} \times \lambda_{max,c,Qd} \times \lambda_{max,scrag,Qd}$$

Where,

t = factors to account for effect of temperature

a = factors to account for effect of aging

v = factors to account for effect of velocity (including freq. for elastomeric bearings)

tr = factors to account for effect of travel (wear)

c = factors to account for effect of contamination (in sliding system)

scrag = factors to account for effect of scragging a bearing (in elastomeric systems)

H-2 Elastomeric bearings

Factors for λ_{max}

Factors for λ_{min}

$$\lambda_{min} = 1.0 \quad \text{for } K_d \text{ and } Q_d$$

$$\lambda_{max,v} = \text{Established by test}$$

$$\lambda_{max,c} = 1.0$$

$$\lambda_{max,tr} = \text{Established by test}$$

$$\lambda_{max,a} = \text{See Table H 2.1}$$

$$\lambda_{max,t} = \text{See Table H 2.2}$$

$$\lambda_{max,scrag} = \text{See Table H 2.3}$$

Table H - 2.1: Value of $\lambda_{\max,a}$

	$\lambda_{\max,a}$	
	K_d	Q_d
Low-Damping natural rubber	1.1	1.1
High-Damping rubber with small difference between scragged and unscragged properties	1.2	1.2
High-Damping rubber with large difference between scragged and unscragged properties	1.3	1.3
Lead	-	1.0
Neoprene	3.0	3.0

Table H - 2.2: Value of $\lambda_{\max,t}$

Minimum Temp for design	$\lambda_{\max,t}$					
	Q_d			K_d		
°C	HDRB ¹	HDRB ²	LDRB ²	HDRB ¹	HDRB ²	LDRB ²
21	1.0	1.0	1.0	1.0	1.0	1.0
0	1.3	1.3	1.3	1.2	1.1	1.1
-10	1.4	1.4	1.4	1.4	1.2	1.1
-30	2.5	2.0	1.5	2.0	1.4	1.3

HDRB = High damping rubber bearing

LDRB = Low damping rubber bearing

¹

Large difference in scragged and unscragged properties (more than 25%)

²

Small differences in scragged and unscragged properties

Table H - 2.3: Value of $\lambda_{max,scrag}$

$\lambda_{max,scrag}$					
Q_d			K_d		
<i>LDRB</i>	<i>HDRB</i> with $\beta_{eff} \leq 0.15$	<i>HDRB</i> with $\beta_{eff} \leq 0.15$	<i>LDRB</i>	<i>HDRB</i> with $\beta_{eff} \leq 0.15$	<i>HDRB</i> with $\beta_{eff} \leq 0.15$
1.0	1.2	1.5	1.0	1.2	1.8

H-3 Sliding Isolation system

Factors for λ_{max}

Factors for $\lambda_{min}\lambda_{max,v}$ does not apply

$\lambda_{min} = 1.0$ for K_d and Q_d

$\lambda_{max,a}$ = See Table H 3.1

$\lambda_{max,c}$ = See Table H 3.2

$\lambda_{max,tr}$ = See Table H 3.3

$\lambda_{max,t}$ = See Table H 3.4

Table H – 3.1: Value of $\lambda_{max,a}$

$\lambda_{max,a}$						
	Unlubricated PTFE		Lubricated PTFE		Bimetallic Interfaces	
	Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed
Condition Environment	Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed
Normal	1.1	1.2	1.3	1.4	2.0	2.2
Severe	1.2	1.5	1.4	1.8	2.2	2.5

Table H – 3.2: Value of $\lambda_{\max,c}$

	$\lambda_{\max,c}$		
	Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
Sealed with stainless steel surface facing down	1.0	1.0	1.0
Sealed with stainless steel surface facing up*	1.1	1.1	1.1
Unsealed with stainless steel surface facing down	1.1	3.0	1.1
Unsealed with stainless steel surface facing up	Not Allowed	Not Allowed	Not Allowed

Table H – 3.3: Value of $\lambda_{\max,tr}$

Cumulative Travel (M)	$\lambda_{\max,c}$		
	Unlubricated PTFE*	Lubricated PTFE	Bimetallic Interfaces
1005			
< 2010	1.1	1.1	To be established by test
> 2010	1.1	3.0	To be established by test

* Test data based on 1/8-inch sheet, recessed by 1/16 inch and bonded.

Table H – 3.4: Value of $\lambda_{\max,t}$

Minimum Temp for design	$\lambda_{\max,t}$		
0_c	Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
21	1.0	1.0	To be established by test
0	1.1	1.3	
-10	1.2	1.5	
-30	1.5	3.0	

APPENDIX J

Post-Earthquake Operations and Inspections

1.0 - Post Earthquake Operations and Inspections

The response of railway tracks and bridges to an earthquake would depend on distance from epicenter and nature of attenuation. The post-earthquake train operations in the region shall be cautiously started.

1.1 - Operations

After an earthquake is reported, the operating department shall notify all the trains and engines within 150 km radius of the reporting area to either stop or run at restricted speed of 10 Kmph (depending upon intensity reported from the area) until magnitude and epicentre (and corresponding response level) have been determined by the Senior Divisional Engineer of the section. After determination of the magnitude and epicenter, response levels given in Table 1 and 2 will govern the operations.

Table – 1 Specified Radius of Different Earthquake

Earthquake Magnitude(Richter)	Response Level	Specified Radius
0- 4.99	I	No action
5.0 – 5.99	II	80 km
6.0 – 6.99	III II	160 km 240 km
7.0 or above	III II	* *

* As directed by CBE, but not less than the radius specified for Earthquakes of magnitude between 6.0 – 6.99 of Richter scale.

Table –2 Details of Response Level

Response level	Details
I	Resume maximum operation speed. The need for inspections will be determined by Sr. DEN responsible for maintenance of P.Way.
II	All trains and engines will run at restricted speed of 30 Kmph over all Major, Important and Girder bridges within the specified radius of the epicenter until inspections have been made by PWI, Asst PWI and ADEN and appropriate speeds established by consulting sectional Sr. DEN.
III	All trains and Engines within the specified radius of the epicenter must stop and may not proceed until proper inspections have been performed by PWI or Asst PWI or BRI or ADEN and appropriate speed restrictions established by consulting Sectional Sr. DEN for damaged bridges and other locations. On all important and Major bridges, before relaxation of the speed to normal, detailed inspection should be carried out by Sectional Sr. DEN and an Engineer deputed by CBE together.

1.2 - Post Earthquake Inspection

The following list provides a general guideline for an inspection procedure:

1.2.1 - Track and Roadbed

During the post-earthquake inspection, following items shall be observed:

- Line, surface and cross level irregularities caused by embankment slides or liquefaction
- Track buckling or pull apart due to soil movement
- Offset across fault rupture
- Disturbed ballast
- Cracks or slope failures in embankments
- Slides and/or potential slides in cuts, including loose rocks that could fall in an aftershock
- Scour due to tsunami in coastal area

Potential for scour or ponding against embankment due to changes in water course

1.2.2 - Bridges

Following an earthquake, inspectors may need to travel by rail between bridges. River bed may get flooded, hence, to quickly reach the bearings; alternate access routes shall be made. In steel bridges following shall be observed carefully:

- Displaced or damaged bearings
- Stretched or broken anchor bolts
- Distress in viaduct tower
- Buckled columns or bracings
- Tension distress in main members or bracings
- Displaced substructure elements

Concrete bridge inspection shall include the following :

- Displacement at bearings
- Displaced substructure elements
- Cracks in superstructure
- Cracks in substructure

Inspection team shall also look for items which may fall on track. At an ROB, attention shall be given to reduced span at bearings, damages to column and Restrainer system. If there area adjacent buildings to railway track, then such buildings shall also be inspected to ensure if they can withstand aftershocks. Inspection team shall also look for damages to the power lines passing over the track.