A state-of-the-art review on seismic design of bridges - Part II : CALTRANS, TNZ and Indian codes

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In a companion paper[1], the historical development of the seismic design provisions for bridges in USA are studied and it is shown how the experience of the performance of bridges during past earthquakes has been translated into the AASHTO code. In this paper, code provisions on seismic design of bridges in California, USA (CALTRANS code) and New Zealand (TNZ code) are reviewed in detail, together with those in the Indian codes (IS : 1893-1984 and IRC : 6-1966). In the American and New Zealand codes, more realistic ground accelerations are explicitly considered in design, suitable response reduction factors are used to account for ductility and overstrength, and the principle of capacity design is liberally used. This results in a bridge structure which is likely to display ductile behaviour in the event of very strong shaking. Moreover, these codes provide specifications for vertical hold-down devices and horizontal linkage elements; these devices provide positive anchorage and stability against lateral and longitudinal displacements and against vertical uplift. Finally, to prevent loss-of-span type of collapses, these codes require minimum seating widths. It is seen that, generally, the seismic design force for bridges in the current Indian codes is extremely low, particularly for the connections (such is not the case for the buildings). The Indian codes require major changes regarding (i) more realistic earthquake ground accelerations (ii) consideration of flexibility of structure in design force calculation, and (iii) provision of proper response reduction factors for different elements of the bridges to account for ductility and overstrength. Also, there is an urgent need to incorporate provisions on vertical hold-down devices and horizontal linkage elements, and minimum seat widths.

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The historical development of the seismic design codes for bridges in USA has been presented in a companion paper[1] along with a review of the AASHTO code of USA[2]. This paper presents a review of the CALTRANS code[3] of USA, the Transit NZ code[4] of New Zealand, and the Indian codes[5,6]. This paper shows that the New Zealand code has the least gap between state-of-the-art and state-of-the-practice. For instance, it uses an inelastic design spectrum to specify the seismic design force. Such an approach is currently not being followed in the design codes of USA. Also, the use of a soil amplification factor as a function of the natural period of the bridge system is a special feature in the CALTRANS code; in the AASHTO code, the site coefficient which reflects the type of soil is independent of the natural period of the bridge system. In light of the performance of bridges in India during the past earthquakes[8] and the above mentioned review of the design codes of USA and New Zealand, the paper raises the relevant issues for consideration in the next revision of the Indian code provisions for seismic design of bridges.

CALTRANS code (USA)

This standard specification is published by the California Department of Transportation (CALTRANS) for the purpose of seismic design of freeway/highway bridges in the State of California, USA. This code has been a model code for many others in the profession of code development.

Design philosophy

In this code, the design force is obtained in two steps. Firstly, the elastic forces generated in the members and connections under the maximum credible earthquake are obtained. Then, depending on the capability of a component to provide ductile behaviour, the above maximum elastic forces are divided by the reduction factor of that component to arrive at its design forces. Thus, different components of a bridge are recognised to have different ductility and overstrength in them. Since inelasticities are expected in substructures, the concept

of capacity design is used in the design of substructures and foundations.

The CALTRANS code is primarily based on the strength criteria. There are no provisions to control the lateral displacements (drift) in bridge structures. However, provisions accounting for relative displacements between adjacent components of the superstructure and between the superstructure and supports are available. These specifications focus on the design of horizontal linkage elements between adjacent spans or on the design of vertical hold-down devices at the supports, be it piers or abutments.

Seismic forces are required to be determined for two independent loading conditions in perpendicular directions, usually along longitudinal and transverse axes, of the bridge.

Design force and reduction factors

The elastic response spectra (5 percent damping) at the site for the maximum credible event(s) are obtained by the product of peak rock accelerations A, acceleration spectra in rock R, and soil amplification factor S, described in the following sub-sections.

Peak rock accelerations, A

Seismic risk at different locations in California is specified in CALTRANS code through contour maps where the contours join all locations of equal peak rock acceleration, that is, maximum expected acceleration A at bedrock or rock-like material. Since local soil conditions will influence the acceleration at a site, these contours are prepared assuming rock strata. The effect of local soil conditions is incorporated in design through soil amplification factor. The peak rock acceleration contours are drawn up to 0.7g at 0.1g intervals.

Acceleration spectra in rock, R

The code provides normalised acceleration spectra R in rock, for the different ranges of peak rock accelerations, Fig 1, as a function of the natural period of the bridge T. Note that the maximum spectral amplification used in these curves is 2.6. Also, the shape of spectrum appears different from usual because (i) the natural period is plotted on a log scale, and (ii) the natural period axis starts at 0.1 s. The elastic spectrum for 5 percent damping on rock can thus be obtained for any location in California by multiplying the peak rock ac-
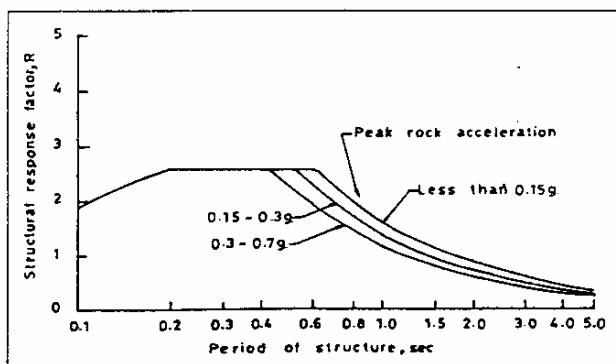
celeration, A with the normalised rock spectra, R.

Soil amplification factor, S

Depending on the type and depth of soil over bedrock, motions at the ground are modified from what is produced at the rock level. Thus, the soil amplification factor S, which is the ratio of the peak acceleration at the ground to that at the bedrock, has been developed. The CALTRANS code categorizes soil sites into four categories : thickness of alluvium of 0-10 feet, 10-80 feet, 80-150 feet, and > 150 feet. For alluvium thickness of 0-10 feet, $S = 1.0$. For the three categories, the plots of S versus T for different values of peak rock acceleration (ranging from 0.1g to 0.7g) are given in Fig 2.

Fundamental natural period of bridge, T

While using the equivalent static method, the fundamental natural period, in seconds, of the bridge as a whole in any direction may be estimated by the relation

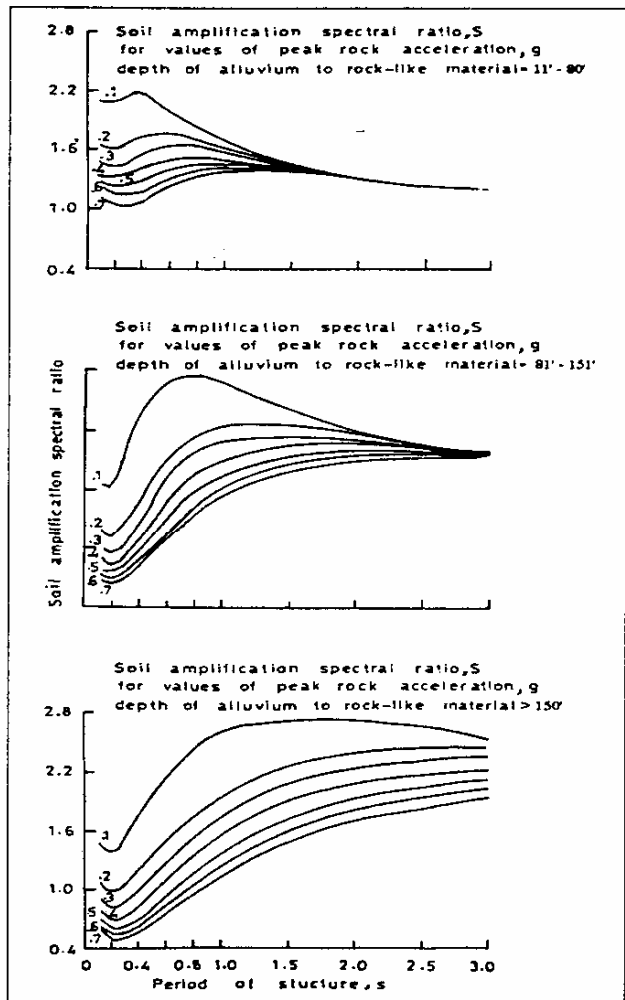$$T = 0.32 \sqrt{\frac{W}{P}} \qquad (1)$$

Fig 2 Normalised rock spectra R[3]

Fig 1 Normalised rock spectra R[3]

where W is the dead load of the bridge and P is the total uniform force applied to the superstructure which will cause one-inch maximum horizontal deflection in the considered direction of loading. Thus, P represents the total stiffness of the superstructure, supporting members and surrounding soil.

Ductility and risk reduction factor, Z

Seismic design forces and moments for individual members as per the CALTRANS code shall be determined by dividing the individual elastic seismic member forces with the appropriate ductility and risk reduction factor, Z. This factor, which is similar to the response reduction factor discussed earlier in this report, is shown in Fig 3 for the different components of the bridge. Factor Z accounts for ductility and risk to damage as seen in the past earthquakes. It was seen that low-level bridges with periods less than 0.6 s were much less vulnerable to collapse than the more flexible bridges.

The Z factor is taken to be a higher value for low-height bridge superstructures owing to favourable experiences in past earthquakes. Hence, the Z factor is gradually reduced with increase in T beyond 0.6 s. This implies higher design force for bridge superstructures that are higher. For a single-column bent,

$$Z = \begin{cases} \left(6 \right) & \text{if } T \leq 0.6 \text{ s} \\ 6\left[1 - \dfrac{T-0.6}{2.4} \right] & \text{if } T > 0.6 \text{ s} \end{cases} \qquad (2)$$

and for a multi-column bent

$$Z = \begin{cases} \left(8 \right) & \text{if } T \leq 0.6 \text{ s} \\ 8\left[1 - \dfrac{T-0.6}{2.4} \right] & \text{if } T > 0.6 \text{ s} \end{cases} \qquad (3)$$

The Z factor for substructures, that is, piers, abutment walls and wingwalls, is taken as 2.0. These elements have
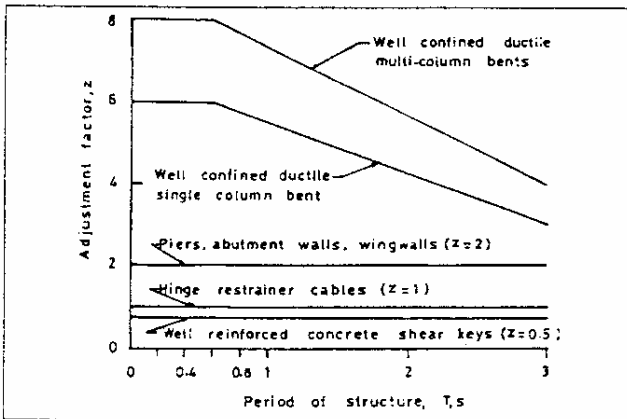

Fig 3 Ductility and risk reduction factor Z'

much less ductility and no redundancy. Hence, the Z values are lower than those used in case of superstructures. The Z factor for restraining devices is also independent of the natural period of the structure. The value of Z for hinge restrained cable is taken as 1.0 and that for well-restrained concrete shear keys is taken as 0.8. These values indicate that restrainers and shear keys are being designed for the maximum expected elastic forces or more. These low values ensure that the components are ńot stressed beyond yield. Failure of these components may lead to collapse conditions and thus lower Z values are used.

Method of analysis

The CALTRANS code permits two methods of analysis, namely, equivalent static analysis and dynamic analysis, depending on the nature of the bridge. The design forces may be estimated by using the ARS spectra and reducing the moments and forces by the ductility and risk reduction factor, Z. However, the deflections determined by using the ARS spectra can be assumed to be realistic and are not to be reduced.

Equivalent static analysis

In case of relatively simple bridges, which have well-balanced stiffness, supporting bents (or substructures) of approximately equal stiffness, little or no skew, tangent or very large radius of alignment, relatively light substructures and no intermediate expansion joints, the equivalent static analysis may be used even though the dynamic analysis method is preferred. When using this method of analysis, a minimum value of ARS equal to 0.4 is imposed. However, there is no such minimum force requirement when using dynamic analysis. On the other hand, the equivalent static analysis is the preferred method to obtain forces in hinge restrainers.

The seismic load may be assumed as an equivalent uniform static load equal to ARS times W, applied at the vertical centre of gravity of the total bridge structure. The distribution of this seismic force to individual members shall be in accordance to the stiffness of the superstructure, substructure and the restraint at the abutments. The elastic forces computed in the different bridge components are then divided by the adjustment factor Z; the forces so obtained are to be used as design forces.

Dynamic analysis

Bridge structures with irregular configuration or support stiffnesses are required to be analysed by this method. The method employs modal analysis of the lumped mass space frame of the bridge subjected to ground motion. The ground motion may be given by the given response spectrum (ARS spectrum) or it may be an equivalent site-specific elastic response spectrum (5 percent damping). The model of the bridge structure shall also include the restraint offered by the soil. This dynamic analysis technique is particularly preferred to ascertain the forces and moments in column members and transverse keys.

The commentary on the CALTRANS code clearly states that the assumption that the usually recommended dynamic analysis procedure (which is an elastic procedure) will allow

the prediction of earthquake forces very accurately is not correct. At best, elastic dynamic analysis can provide a good distribution of the forces in the bridge structure and a general estimate of deformations that can be expected. However, issues like cumulative damage need to be included to refine the dynamic analysis to give "reasonable" results.

Combination of orthogonal seismic forces

To account for the directional uncertainty of the earthquake ground motion, two load cases are recommended. The seismic loads are calculated individually along two orthogonal directions. Usually, one may consider the longitudinal axis and the transverse axis of the bridge as these orthogonal directions. These forces are then combined by "100 percent + 30 percent rule" to obtain design forces. This rule is explained in the following and illustrated in Fig 4.

The forces and moments resulting from the two analyses of the bridge system subjected to the seismic load along the two orthogonal directions, say direction 1 and direction 2, shall be combined as below :

(i) **Seismic load case 1** : Combine the forces and moments resulting from the analysis with seismic load along direction 1 with 30 percent of the corresponding forces and moments from the analysis with seismic load along direction 2.
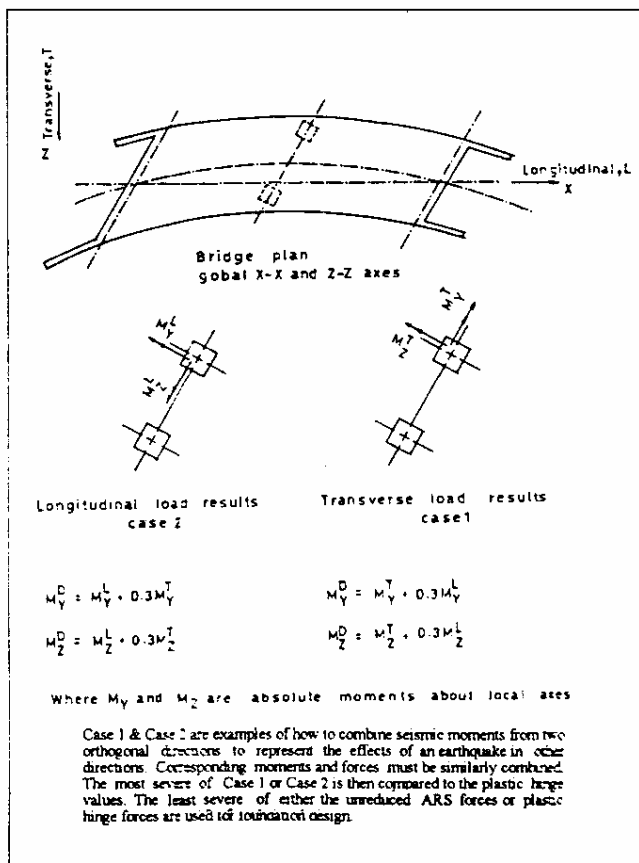

Longitudinal load results
case 2

Transverse load results
case 1

$$M_y^D = M_y^L + 0.3 M_y^T$$

$$M_z^D = M_z^L + 0.3 M_z^T$$

$$M_y^D = M_y^T + 0.3 M_y^L$$

$$M_z^D = M_z^T + 0.3 M_z^L$$

Where M_y and M_z are absolute moments about local axes

Case 1 & Case 2 are examples of how to combine seismic moments from two orthogonal directions to represent the effects of an earthquake in other directions. Corresponding moments and forces must be similarly combined. The most severe of Case 1 or Case 2 is then compared to the plastic hinge values. The least severe of either the unreduced ARS forces or plastic hinge forces are used for foundation design.

Fig 4 Combination of orthogonal seismic forces[3]

(ii) **Seismic load case 2** : Combine the forces and moments resulting from the analysis with seismic load along direction 2 with 30 percent of the corresponding forces and moments from the analysis with seismic load along direction 1.

Special cases

The code requires that all bridge structures at sites adjacent to faults, at sites with unusual geologic conditions, unusual bridge structure types, and bridges whose fundamental natural periods are greater than 3 s, be treated as special cases. The design forces for such bridges shall be based on approved site-specific soil response and dynamic analysis.

Restraining features to limit relative displacements

Monolithic superstructures are preferable to reduce the joint pull-apart and subsequent collapse. However, when girder-bearing systems are used, protection against relative displacements between the superstructure and substructure is provided for through special vertical hold-down devices, horizontal hinge restrainers and fixed restraining devices by "design force" clauses. Interestingly, despite the numerous loss-of-span type of failures in California, there is no requirement of minimum seating width in the CALTRANS code. However, *Memo to Designers* of CALTRANS on abutments mentions AASHTO requirements on minimum seat widths. Further, CALTRANS uses a 24 inch minimum seat on all bridges, and additional seat width at abutments on high skew[9].

Restrainers along longitudinal and transverse directions

Positive longitudinal restraint is required to be provided between adjacent sections of superstructure at all intermediate expansion joints. These restraints, for example, hinge restrainers or flexible single-direction restraining devices, are expected to limit the superstructure displacement. The forces in these restrainers are to be determined using the equivalent static method. When estimating the total stiffness of the frame moving away from the joint, the longitudinal stiffness of one adjacent superstructure frame, restraint at the abutment, gaps at the joints and gaps at the restrainers are to be considered. Further, in case of simple multiple spans, only one span is required to be considered at a time. However, the code requires that forces in shear keys and other types of fixed restraining devices be determined by using the dynamic analysis.

In single span bridges, detailed analysis is not required to estimate the forces in restrainers. The forces in the connections between superstructure and substructure are required to be evaluated as per equivalent static method. When superstructures are fixed to abutments in the transverse direction, their natural period of vibration may be taken as zero.

Restrainers in vertical direction

Where the vertical seismic force opposes and exceeds 50 percent of the dead-load reaction at any support or intermediate

hinge, the CALTRANS code requires that vertical hold-down devices be provided at that support. The minimum design force for such devices shall be the greater of (i) 10 percent of the dead load reaction, and (ii) 1.2 times the net uplift force.

Substructure design

The design of the substructure components is based on the capacity design concept. In calculating the probable plastic moment capacities at the base of the column, the possible overstrength in materials beyond their specified nominal characteristic strengths are to be used. For reinforced concrete columns, a strength reduction factor of 1.3 is recommended for use in the calculation of overstrength plastic hinge capacity from the nominal moment capacities. This strength re-

Fig 5 Potential platic hinge locations and the method for calculating the design shear[3]

duction factor of 1.3 for reinforced concrete columns increases the ultimate strength, as against the values less than 1.0 normally used in design. In the CALTRANS code, this method of determining the plastic hinge capacities is uniformly applicable to all bridges in the State of California.

The code specifies that columns in substructures be designed for the following loads.

Design moment

The design moment for column is obtained by dividing the seismic member forces by the appropriate factor, Z.

Shear force

The design shear force is determined from the probable plastic moment of the column section and the distance between the plastic hinges. Some examples of potential plastic hinge locations are also discussed in the commentary of the code, Fig 5. The length of the plastic hinge region may be assumed to be the largest of (i) largest lateral dimensions of the prismatic portion of the column, (ii) one-sixth the length of the column, and (iii) 24 inches. In case of flared columns, the length of plastic hinge regions may be assumed to be the above quantity enhanced by the flare length.

Axial force

The design axial force is the unfactored dead load axial force plus or minus the axial force developed resulting from the formation of the plastic hinges in the substructure.

Foundation design

Damages in foundations are not easily detectable. Hence, unfavourable brittle failures in them, if any, are avoided by designing them for forces which are envisaged corresponding to the structure undergoing the maximum possible ductile response (that is, the concept of capacity design). The CALTRANS code requires that the design forces for foundations of bents and piers shall be the smaller of (i) the maximum elastic force, and (ii) the force at the foundation due to formation of plastic hinge at the base of the column in the substructure.

The code specifies that the ultimate soil or pile capacity be used for resisting the seismic foundation loads.

TNZ Bridge Manual (New Zealand)

This draft specification[4] is published by Transit New Zealand (the organisation in-charge of national roads), for the purpose of seismic design of highway bridges in New Zealand. The Bridge Manual of Transit New Zealand, here-in-after called the TNZ code, today stands as the design code with the least gap between the states-of-the-art and of-the-practice in so far as the seismic design of bridges is concerned.

Design philosophy

The TNZ code is a strong advocate of the concept of capacity design. The design approach involves the choice of an intended mode of structural behaviour during strong shaking, followed by design and detailing of members to ensure that
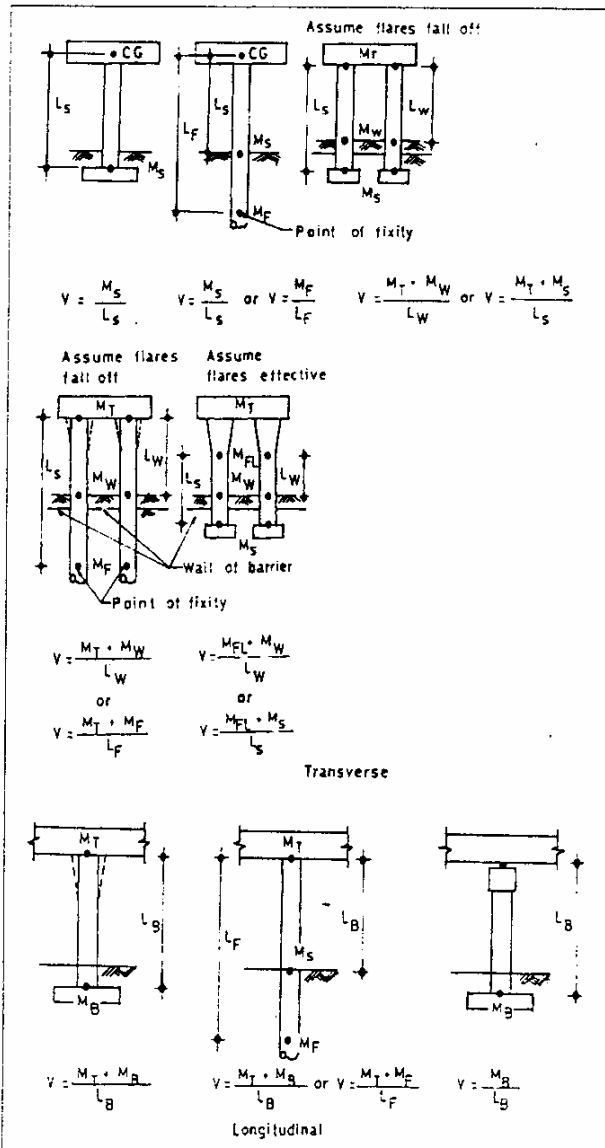
the structure behaves as intended. Sufficient strength capacity is provided elsewhere in the structure to ensure that the chosen energy dissipation mechanism does indeed develop in the event of a major earthquake. The TNZ code recognizes the need to calculate the design force due to ground motion in the two orthogonal directions to the bridge to account for the directional uncertainty of the ground motion. However, no combination of these forces is specified.

Importance categories and risk factor, R

The TNZ code defines three importance categories for bridges depending on the average number of vehicles per day expected at the time of design, on location vis-a-vis motorways and railways, and on whether on a national or provincial highway. Based on this importance category, a factor to be used in determining the design force is introduced keeping in mind a different seismic return period for each of the categories. This factor, named the risk factor R, takes a value of 0.9, 1.1 and 1.3, the last one being the value for the most important bridges, implying higher design seismic force for them.

Zone factor, Z

A zone factor Z, which reflects the peak ground acceleration at the site, is given in the form of a contour map for the New Zealand islands. The TNZ code takes a minimum value of Z as 0.4 and a maximum value of 0.8. This implies that peak ground accelerations in the range of 0.4 g to 0.8 g are envisaged in the country.

Ductility, inelastic design spectra and soil type

The TNZ code discusses in detail the different types of structures from the point of view of ductility. It requires designers to use the actual characteristic of the structure to evaluate the design displacement ductility factor μ. Here, indirect reference is made to the use of nonlinear analysis in obtaining μ. The maximum allowable design displacement ductility factor μ is shown in Table 1 as a function of the different rein-

Table 1 : Maximum allowable values of design displacement ductility factor [TNZ code]

Energy dissipation system	μ
Ductile or partially ductile structure (Type I), in which plastic hinges form at design load intensity, above ground or normal (or mean tide) water level	6
Ductile or partially ductile structure (Type I), in which plastic hinges form in reasonably accessible positions, example, less than 2 m below ground, but not below normal (or mean tide) water level	4
Ductile or partially ductile structure (Type I), in which plastic hinges are inaccessible, forming more than 2 m below ground or below normal (or mean tide) water level, or at a level reasonably predictable	3
Partially ductile structure (Type II)	
Spread footings designed to rock (unless a larger value can be specifically justified)	
Hinging in racked piles in which earthquake load induces large axial forces	2
'Locked-in' structure (T = 0)	1
Elastic structure	

Note : The design ductility factor for structures of limited capacity or demand is to be determined from actual structure characteristics
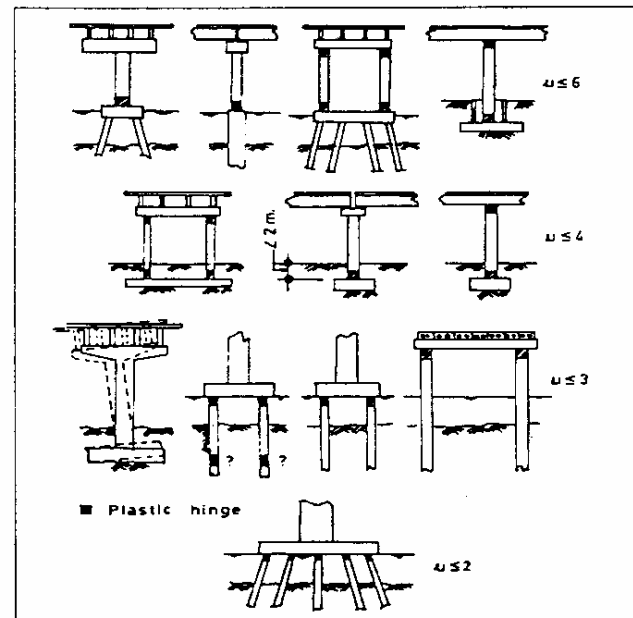
forcement detailing schemes recommended in the reinforced concrete code[10]. Fig 6 gives examples of maximum value of ductility allowed by TNZ.

The TNZ code specifies the basic acceleration coefficient C_μ, which reflects the response amplification (due to the structure flexibility) and the response reduction factor (due to ductility and overstrength). This coefficient C_μ is given for design displacement ductility factor μ values of 1, 2, 3, 4 and 6, for two soil types and for a given fundamental natural period T of the bridge structure in the considered direction of design earthquake. This coefficient C_μ in the form of smooth spectra is shown in Fig 7 for two soil types : namely, normal soils and flexible soils. Foundations resting on soils that can liquefy are to be treated as special cases, and detailed studies are required to ascertain the likely ground response. The spectrum curves in Fig 7 show in the low period range (i) dotted lines, and (ii) solid line plateau. The code requires that the ordinates for first mode response should not be less than the plateau values. For higher mode responses, the dotted portion of the curves may be used instead of the plateau. It is interesting to note that TNZ is amongst the few codes to use inelastic spectra in design as given in Fig 7.

Design force level

The seismic analysis is performed by constructing the basic design spectrum given by the product of basic acceleration coefficient C_μ, zone factor Z, the risk factor R. Notice that no response reduction factor is being used in the process. The main component of response reduction factor, namely ductility reduction factor and the overstrength, are accounted for through C_μ. The effect of ductility reduction factor comes directly by means of different curves for C_μ for different values of ductility. Even though TNZ code does not mention it as such, it is apparent that the overstrength comes into de-

Fig 6 Examples of maximum value of μ allowed by TNZ[4]

sign through the fact that the ordinate of C_μ curves at zero period is 0.4 and not 1.0; this implies an overstrength of 2.5 being assumed. The code allows three types of analysis, namely (i) equivalent static force analysis, (ii) modal analysis (response spectrum method of analysis) and (iii) inelastic time history analysis. The code provides detailed specifications on material properties to be used in analysis (as against most other codes which leave out this and thereby provide ample scope for large variations in material properties and hence in natural period calculations).

The TNZ code specifically states that the design forces and moments in individual members calculated by the response spectrum analysis shall be at least 80 percent of the values obtained by the equivalent static analysis method.

Equivalent static force analysis

The design lateral force is given by

$$.H = C_\mu ZRW \geq 0.05\ W, \tag{4}$$

where, W is the total seismic weight (dead weight plus part of superimposed weight). The other parameters are as described in the previous sections.

Modal analysis

The modal analysis is to be performed with the design inelastic response spectrum C_μ for the specific soil condition
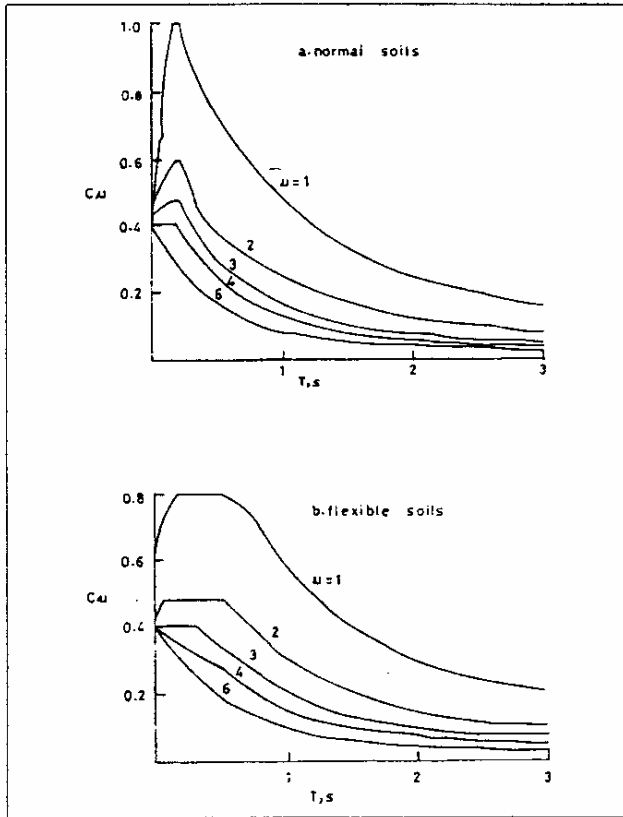

Fig 7 Variation of basic acceleration coefficient C_μ with natural period for different soil types and different ductility[4]

and the chosen level of ductility, factored by the zone factor Z and the risk factor R described in the previous sections. The code specifies that sufficient number of modes shall be taken in the analysis to ensure that the effective mass so included is at least 90 percent of the total bridge mass.

Inelastic time history analysis

Inelastic time history analysis is permitted with synthetic acceleration ground motions appropriate to the site conditions. These synthetic acceleration time histories shall be such that the 5 percent damping inelastic response spectra factored by Z and R factors shall be comparable with the design spectra given by $C_\mu ZR$. In fact, the ordinates of the synthetic ground motion spectra are required to be within 90 percent of the corresponding design spectra ordinates over the range of the first three natural periods of the bridge in the considered direction of motion. Further, the duration of strong shaking in the ground motions records is required to be the larger of 15 s or 5 times the fundamental natural period of the structure.

Vertical seismic response

The TNZ code requires the bridge superstructure to remain elastic under both positive and negative vertical acceleration. The peak vertical acceleration is specified as

$$a_v = 0.67C_E ZRg \tag{5}$$

where, C_E is the basic elastic force coefficient for elastic structures (that is, C_μ for ductility of 1 taken from the curves of Fig 7, for T given by the natural period of vertical vibration), Z and R are as defined earlier, and g is acceleration due to gravity.

Design of substructures and foundations

The TNZ code uses the capacity design principle in the design of substructures and foundations. For calculating the overstrength capacities of members, reference is made to the design code for buildings[11]. No reduction factors or increase in permissible stresses is stated in the TNZ code.

Design for relative displacements

Relative deformations between superstructure and substructures, and between adjacent sections of superstructure are to be provided for through special vertical hold-down devices at the substructure (that is, piers or abutments) and horizontal linkage elements between adjacent spans. In addition to these, the code also specifies minimum seating widths of the superstructure spans over the substructure. Some of the important provisions of the TNZ code regarding structural integrity and relative displacements are given below.

- The code provides clear specifications regarding structural clearances at locations where relative movement between structural elements is designed to occur.

- Horizontal linkage elements, either tight or loose, are required between all simply supported span ends and their piers and between the two parts of the superstructure at a hinge in the longitudinal beam system. Acceptable means of linkages include linkage bolts, shear keys and bearings specifically designed for the
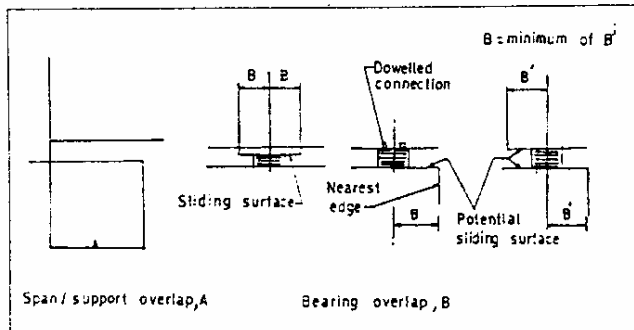
Fig 8 Overlap definition[4]

purpose. Elastomeric bearings with shear dowels are not acceptable means.

The horizontal linkage elements are required to have a dependable strength not less than the forces generated under design seismic conditions, nor less than 0.2 times the dead load of the smaller of the two superstructure elements. In case of a suspended span between two longer lengths, the strength is based on the longer of the two superstructure elements.

The vertical hold-down devices are required to be provided at all supports and structural hinges where the vertical upward seismic reaction under design earthquake conditions is more than 50 percent of the dead load reaction. These devices shall have sufficient strength to prevent uplift of span from its support or separation of the two hinged members under design earthquake conditions and shall have a minimum dependable strength (strength considering the factors of safety) of 20 percent of dead load reaction.

The TNZ code has requirements of minimum seating widths of the spans over the top of substructure (that is, piers and abutments). These are specified in *Table* 2 as minimum overlap requirements. The code distinguishes between the span-support overlap lengths and bearing overlap lengths, *Fig* 8. The former addresses the dislodging of the span from the support while the latter only covers the dislodging of the

Table 2 : Minimum overlap requirements [TNZ Code]

	Span support overlap	*Bearing overlap*
No linkage system	2.0 E + 100 mm (400 mm minimum)	1.25 E
Loose linkage system	1.5 El + 100 mm (300 mm minimum)	1.0 El
Tight linkage system	200mm	-

Note :

E = Relative movement between span and support, from median temperature position at construction time, under design earthquake conditions,

$$EQ + SG + \frac{TP}{3}$$

El = Equivalent relative movement at which the loose linkage operates, that is, El ≥ E

EQ, SG and TP are displacements resulting from load conditions described in the code

bearings (which are fixed to the span) from the support. The relative horizontal movements between superstructures and substructures are required to be occurring simultaneously along both longitudinal and transverse directions.

Review of IS : 1893-1984

Design philosophy

This standard is published by the Bureau of Indian Standards, New Delhi, for the purpose of seismic design of several types of structures including bridges. In this section, the reference to the IS code refers to only the part corresponding to bridges (there are some internal inconsistencies in the code between different sections).

The code discourages the use of masonry and plain concrete arch bridges with spans more than 10 m in the severe seismic zones IV and V. Further, it states that seismic forces need not be designed for in case of (i) slab culverts, box culverts and pipe culverts; and (ii) bridges of length less than 60 m with spans less than 15 m in seismic zones I, II and III.

The code mentions that the modal analysis is necessary for the following bridges in seismic zones IV and V (however, the code provides no information on the specifications for the modal analysis) : (i) suspension bridge, bascule bridge, cable stayed bridge, horizontally curved girder bridge, arch (RC and steel) bridge (ii) bridge with substructure height from foundation base to pier top more than 30 m and (iii) bridge with span more than 120 m.

In the design of the substructure, earthquake forces are to be calculated based on the depth of scour caused by the discharge corresponding to the annual flood. It is assumed that the design earthquake forces and forces due to maximum flood do not occur simultaneously.

Design force level

The IS code, while referring to the seismic coefficient method, suggests that the seismic design force F shall be computed by

$$F = \begin{cases} \beta\,I\,\alpha_o W_m & \text{for horizontal force} \\ 0.5\,\beta\,I\,\alpha_o W_m & \text{for vertical force,} \end{cases} \quad (6)$$

where, β is the soil-foundation system factor, I is the importance factor, and α_o is the basic horizontal seismic coefficient which reflects the seismic zone. The soil-foundation system factor β takes value of 1.0 to 1.2 and 1.5 depending on type of foundation and type of soil. The importance factor I assumes a value of 1.5 for important bridges and 1.0 for regular bridges. The basic horizontal seismic coefficient α_o takes the values of 0.01, 0.02, 0.04, 0.05 and 0.08 for seismic zones I, II, III, IV and V, respectively. W_m is the seismic weight (dead weight plus part of superimposed weight) considered, excluding buoyancy or uplift. The code provides details on what fraction of design live load is to be considered. Note that there are two major omissions in this equation, which appear to be inadvertent : the above equation does not incorporate a term reflection the variation in design spectrum with the natural period of the structure, and the performance fac-

tor. Both these factors are provided in the same code for the case of buildings.

As far as the strength design of the bridge is concerned, the above is the most significant provision of the code. It is obvious that the design force is same for all elements of the bridge irrespective of the ductility of the element. Moreover, the magnitude of the design force is extremely low for most elements, particularly for the connections; just about 8 percent of gravity in the most severe seismic zone.

Design considerations

Superstructure

The code requires the consideration of vertical acceleration in design. Under the simultaneous action of horizontal and vertical acceleration, the code requires superstructures to have a factor of safety of at least 1.5 against overturning in the transverse direction.

The IS code draws attention to an important aspect of relative displacements between superstructure and substructure. It requires that superstructures of bridges be "properly secured to the piers" by suitable methods, particularly in the severe seismic zones IV and V, to prevent them from being dislodged off the bearings during earthquake shaking. However, the code does not provide guidelines on how to do it "properly".

Hydrodynamic forces due to earthquake motion

The IS code provides some specifications on the consideration of hydrodynamic forces on the submerged portion of the piers. Also, provisions are given for hydrodynamic pressure on the submerged superstructure of submersible bridges.

Review of IRC: 6-1966

This standard is published by the Indian Roads Congress, New Delhi, for the purpose of seismic design of highway bridges in India. The IRC: 6-1966 specifications (1985 print), hereinafter called the IRC code, are identical (except for some editorial changes) to the IS: 1893-1984 provisions discussed above with regard to the design seismic force calculations. However, IRC code does not have provisions on hydrodynamic forces under earthquake excitation.

The IRC code provides that (i) all bridges in zone V shall be designed for seismic forces (ii) major bridges, that is, with total length of more than 60 m, in zones III and IV are to be designed for seismic forces and (iii) bridges in zone I and II need not be designed for seismic forces. However, there is no mention in the IRC code of minimum seating width, vertical hold down devices and horizontal linkages.

Conclusions

The IS code in its current form requires major modifications to bring it at par with the bridge code of countries with advanced seismic provisions. The following is a critical discussion of the most important aspects; possible areas where changes are to be made immediately are also indicated. Comments made in this section on IS code are equally applicable to the IRC code.

Design philosophy

The IS code specifications for bridges closely mimic the specifications for buildings not withstanding the well-known differences in the seismic behaviour of buildings and bridges. Further, on some issues, the bridge specifications are even poorer than the clauses for buildings, perhaps because more attention has been paid to the past revision of building provisions. The design force specifications for bridges in the IS code do not follow a clear philosophy. All the components in a bridge structure are to be designed for the same level of force with respect to the maximum elastic force. While the IS code focuses on the strength design aspect, no regard is shown to the deformations in the structure. This is in stark contrast to the numerous experiences in India and abroad during past earthquakes, where failures are attributed to deformational aspects of the structure than the strength. This factor was recognized by advanced seismic codes more than two decades ago. This is a major lacuna in the IS code.

Design force level and design spectrum

It is now well understood internationally that the level of seismic design coefficient is generally higher for bridges than for buildings. This is because, unlike in buildings, bridges do not have non-structural components and have little or no redundancy. For example, it is of interest to note that (i) the peak ground acceleration used in the US bridge codes is about twice as high as the value used in the US building codes, and (ii) the response modification factor used in the US bridge code is much lower than that in the case of buildings. The net result is that in the US, bridges are designed for much higher seismic coefficient than the buildings. This experience has somehow not found place in the IS code. We continue to provide for unrealistically low levels of seismic design force for bridges (the level of design seismic force for buildings in the IS code are quite reasonable). The basic horizontal seismic coefficient α_o of 0.08 for the most severe seismic zone V, is too small. In the US and New Zealand, the peak ground acceleration used in the bridge codes is as high as 0.8 g together with a response amplification factor of about 2.5 in low period range! The IS code does not recognize these issues.

The calculation of design seismic force does not incorporate the response amplification due to the structure flexibility (even though the structural flexibility is properly accounted in the IS code provisions for buildings; both in static as well as dynamic methods) In fact, the code does not use any design spectra for the purpose of estimating the seismic design force. The fundamental natural period T of the structure is not used in the design force computations.

In the 1984 revision of the IS code, a new factor named performance factor K, (to account for different seismic behaviour of structures with different ductility, different overstrength, and different redundancy) was added in the building provisions. Unfortunately no consideration for the same was included in the provisions on bridges. As a result, Indian code continues to provide for the same level of design force for different components of the bridges irrespective of the different expected performances of such components.

The design force for the connections between the super-structure and the substructure in the Indian code remain extremely low. This, together with the absence of any holding down and other devices, will be disastrous in the event of strong shaking; this will lead to collapse of superstructure similar to what was seen in the collapse of Gawana bridge in the Uttarkashi earthquake of 1991[12,8].

There is a need that the current soil-foundation factor β in the IS code be replaced by a soil factor depending on the type of soil at the site.

In the IS code, the importance factor I is 1.5 for important bridges and 1.0 for others. However, the distinction between important bridges and ordinary bridges is left to the discretion of the designer. It is preferable if the code can define the important bridges for this purpose. The IS code does not recognize the variation in possible detailing in RC structures. Hence, there is no mention of ductility provision in this code. It may be noted here that a separate IS code is now available for ductile detailing of RC structure[13], and the code must distinguish in seismic design force between bridges detailed as per this code and those not detailed as per this code.

Capacity design of columns, piers, and foundations

Even though the IS code is based on strength design criteria, proper attention is not paid to preventing brittle failures in the structure. In the sequence of load transfer from the superstructure to the foundations, it is possible that under a given lateral force, the brittle modes of failure may take place prior to the hinging in the ductile regions. The American and New Zealand codes cover this possibility through the approach of capacity design. Similar provisions should be added in the IS code.

Relative displacements and seating widths

Indian codes remain very much deficient with regard to provisions on relative displacements and seating widths. There is a need to provide in the Indian code specific clauses on requirements of vertical hold-down devices and horizontal linkage elements. Provisions need to be added on minimum seat widths to prevent collapse of superstructures.

Acknowledgements

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