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A proposed draft for Indian code provisions on seismic design of bridges - Part II+: Commentary

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The provisions for seismic design of bridges contained in IRC:6-1966[1] and IS:1893-1984[2] need to be revised in view of many deficiencies. Part I of this paper discussed a proposed draft on the provisions for seismic design of bridges. This part provides a detailed commentary to explain the proposed codal provisions.

Based on the detailed review of the provisions in the Indian codes[1,4] on the seismic design of bridges and the review[5] of the performance of bridges in India during past earthquakes, a revised draft for provisions on seismic design of bridges has been presented in the companion paper[6]. In order to explain these provisions and to describe the intent behind some of the clauses, this part provides a detailed commentary.

In the following sections, clause numbers are as in Part I of this paper. For instance, clause C3.1.1 of this paper contains discussion about clause 3.1.1 of Part I. Only those clauses of Part I which require discussion are included. The figures and tables of this paper are given numbers, starting with C. Thus, for example, "Table 5" refers to Table 5 of Part I of this paper, while "Table C1" refers to the Table C1 of this paper.

COMMENTARY

Definitions (C0.1)

This section on definitions has been particularly included to define the terms that are added in the code. Two of the important ones are:

(a) The term "Average acceleration spectrum" used in IRC:6-1966[1] has now been dropped. Instead, a term "Elastic horizontal acceleration spectrum" has been introduced. This is because the spectrum used in design may not necessarily be the "average" of the acceleration spectra of the recorded ground motions. In fact, the average acceleration spectrum may undergo modifications before it is prescribed for use in design to account for effects such as ductility and overstrength, and for ensuring safety of both: very short period or long period structures. Further, the said spectrum is used in the estimation of the total elastic force on the structure/component. Thus, the additional word "elastic" appears.

(b) The term "Response reduction factor" has been introduced. Through this factor, the actual lateral force that would be generated, if the structure were to remain elastic during the most severe shaking that is likely to occur at that site, is reduced to obtain the design lateral force. This term has been introduced to clarify the designer that the design lateral force is not the same as the maximum force, that appears on the structure/component, under the maximum expected level of seismic shaking.

Symbols (C0.2)

The existing version of the IRC code[1] considers variation in seismic risk in different parts of the country through "horizontal seismic coefficient α." On the other hand, the code[2] uses the "basic horizontal coefficient α_0" for the same parameter. Hence, in the draft provisions, a new parameter "seismic zone factor" has been defined to distinguish from the earlier parameters and has been assigned the symbol Z.

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Symbol A has been assigned to represent the elastic acceleration spectrum arrived at, after considering the relevant factors such as seismic zone factor Z, importance factor I, bridge flexibility factor C, and soil profile factor S. This spectrum value A is to be finally used for design of a bridge independent of the method of (i.e., static or dynamic analysis used).

GENERAL PRINCIPLES (C1.0)

Scope (C1.1)

The designers may use this draft code both for design of new bridges and for seismic evaluation of the adequacy of design of existing bridges. The designer may choose to design bridges for seismic forces larger than those specified in this code and but not less.

(C1.2): The earthquake codes provide design force which are substantially lower than what a structure is expected to actually experience during strong earthquake shaking. Hence, it is important that the structure be made ductile and statically redundant to allow for alternate load transfer paths. Ductile design and detailing enables a designer to use a lower design force (higher value of response reduction factor R) than for an ordinarily-detailed structure.

(C1.3) Provisions for ductile design and detailing for reinforced concrete structures are provided in IS:13920-1993[7]. However, provisions for ductile detailing of prestressed concrete, steel and prefabricated structures are not yet available in Indian Standards. If such structures are to be designed for high seismic zones, it is expected that the designer will ensure suitable ductility following the advanced seismic codes of other countries (USA, New Zealand and Japan), Since ductility of super structure is usually not a problem in bridges, this clause requires ductile detailing of the substructures, and connections only.

(C.4) Designers are prohibited to consider masonry and plain concrete arch bridges of span more than 10 m as structural systems for bridges in high seismic zones, since these systems do not possess adequate ductility or reserve strength and may not withstand forces due to strong ground shaking.

Ground Motion (C1.5)

The vertical component of seismic forces produce stresses that are usually not accounted for, in the gravity design of prestressed horizontal girders and horizontally cantilevered components. The 1994 Northridge earthquake in USA has clearly shown the vulnerability of horizontal prestressed girders subjected to vertical ground motions. To check the girder for vertical component ground motions, it may be sufficient to consider the girder, except in case of large span bridges, as rigid for vertical vibrations and subjected to zero-period vertical accelerations but no response reduction factor R. (the seismic coefficient as $0.67ZIS$, since $C = 1.0$ at zero-period, and the vertical accelerations to be taken for the purposes of design are 0.67 times that of the horizontal accelerations specified in this code).

In the seismic design of bridges, vertical ground motions are particularly important. Vertical seismic forces may cause dislocation of girders, and additional stress resultant and displacements, particularly in long span bridges. For this reason, this draft recommends that vertical seismic forces shall also be considered in the design. Also, in the overall stability check of bridges, the stability of superstructures or portions thereof that are not monolithic with the substructure, and in the design of vertical hold-down devices at supports, vertical seismic forces shall be considered.

(C1.6): This clause warn designers that the provisions contained in this draft code do not provide safeguard against situations, where soil underlying the structure may undergo instability due to large settlements, sliding or liquefaction.

Assumptions (C1.7)

The elastic modulus of concrete is difficult to specify and varies with the stress level, loading conditions (static or dynamic), material strength, age etc. Hence, there tends to be a very large variation in the value of elastic modulus specified by different design codes even for the same grade of concrete under static conditions. For instance, the value of E given by IS:456-1978[8] is about 1.4 times of the value given by ACI 318-89[9] for the same grade of concrete. Further, the actual strength of concrete shows an increase with time. The value of E given in existing codes[8-9] is the secant modulus, and often leads to conservative estimates of deflections, i.e., the stiffness of the structure is underestimated. On the other hand, for calculation of the design seismic force, it is unconservative to consider a lower stiffness of the structure by taking a low value of modulus of elasticity. This is because a lower stiffness of the structure leads to a high natural period and therefore a lower design seismic coefficient.

Hence, there are no easy answers to the question of, what value of modulus of elasticity to be used for seismic analysis. Considering to enormous variations, this clause allows the designer to use elastic modulus as that for a static condition.

DESIGN CRITERIA (C2.0)

In the current IRC and IS codes, the design seismic forces for bridges are directly specified; this was often misunderstood as the maximum expected seismic force on the bridge. In line with the world wide practice the draft code now distinguishes the actual forces appearing on each bridge component during design earthquake shaking, if the entire bridge structure were to behave linearly elastic for the calculation of design seismic force for that component.

The draft code makes it clear that the design seismic forces on superstructure, substructure and foundations are only a fraction of the maximum elastic forces that would appear on the bridge. Only in connections, the design seismic forces may be equal to (or more than) the maximum elastic forces that would be transmitted through them. However, if capacity design provisions become applicable, the connection design forces may also be less than the maximum elastic forces. This is in contrast with the design forces for any other design loading conditions. For instance, in case of design for wind effects, the maximum forces that appear on the structure are designed for, and no reductions are employed.

The draft code achieves this by the following step-wise procedure:

(a) Obtain the horizontal elastic acceleration coefficient due to design earthquake, which is same for all components;

(b) Obtain the seismic weight of each component;

(c) Obtain the seismic inertia forces generated in each component

(d) Apply these inertia forces 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;

(e) Obtain the *design stress resultant* in any component by dividing the *maximum elastic stress resultant* obtained in (d) above by the *response reduction factor* prescribed for that component.

Seismic Zone Map (C2.1)

The seismic zone map is under revision by the Sectional Committee on Earthquake Engineering of the Bureau of Indian Standards (BIS). However, it is already agreed upon that the new zoning map of India shall have only four seismic zones. As an interim measure till the new zoning map is available, for the purpose of determining seismic forces the current seismic zone map as given in IS:1893-1984[2] is used with seismic zone I merged with seismic zone II. The current IRC:6-1966 uses the same seismic zone map as in IS:1893-1984[2].

Methods of Calculating Design Seismic Force (C2.2)

Both IRC:6-1966[1] and IS:1893-1984[2], follow a very simplistic design force calculation procedure which does not qualify under either the Seismic Coefficient Method or the Response Spectrum Method. The seismic design force computation does not include consideration of flexibility of the bridge. This implies that all bridges in a seismic zone, irrespective of their span and structural system, adopt the same acceleration coefficient in the design.

This draft code includes the effect of bridge flexibility in its design force computation. Further, it permits the use of both the Seismic Coefficient Method (Equivalent static method) and the Response Spectrum Method (Dynamic analysis method). The Seismic Coefficient Method described in part I[6], assumes that (a) the fundamental mode of vibration has the most dominant contribution to seismic force, and (b) masses and stiffnesses are evenly distributed in the bridge resulting in a regular mode shape. However, in long span bridges, higher modes may be important. And, in irregular bridges, the mode shape may not be regular. Hence, this clause suggests the use of multi-mode analysis, namely Response Spectrum Method, for such bridges. The draft code also prescribes that all bridges in the high seismic zones (Zones IV and V) shall be analysed as per the multi-mode (dynamic) method. This is again motivated by the fact that better distribution of forces is achieved by this method.

In both the methods, the accurate modelling of the bridge structure is essential. Because, unlike buildings where the empirical natural period is based on actual measurements of buildings, no such benchmark is available for bridge structures. The large scatter in the bridge geometry, structural system, and the loading conditions makes the determination of an empirical benchmark for natural period of bridges very difficult.

The draft code recognises that bridges (even regular) of spans 100 m or more, and all irregular bridges in seismic zones IV and V, require more detailed analysis and design.

Regular and Irregular Bridge (C2.2.1)

The classification of regular and irregular bridges, included in the draft code is adopted from the AASHTO code[10] While this classification is meant to be used only as a guide, the responsibility of identifying other irregularities in the chosen bridge structure still rests with the designer.

Vertical Motions (C2.3)

The existing codes[1,2] prescribe that the vertical accelerations be taken as one-half of the horizontal accelerations for the purpose of design. However, studies on strong ground motion records in the past earthquakes indicate that the peak ground accelerations (PGA) in the vertical direction is generally about two-thirds of that in the horizontal direction. The draft codal provisions of IS:1893 being discussed in the Earthquake Engineering Sectional Committee of the BIS, include that the seismic zone factor (which reflects the PGA) for vertical motions be taken as two-thirds of that for horizontal motions. The same provision is now included for the seismic design of bridges.

Live Load (C.2.4)

For Calculation of Magnitude of Seismic Forces Only (C2 4.1)

By the live load acting on the span, one usually refer to vehicular traffic. Seismic shaking in the direction of traffic causes the wheels to roll, once the frictional forces are overcome. The inertia force generated by the vehicle mass in this case is smaller than that, if the vehicle mass were completely fastened to the span. Further, the inertia force generated by the vehicle mass due to friction between the deck and wheels, is assumed to be taken care in the usual design for braking forces in the longitudinal direction. Thus, live load is ignored while estimating the seismic forces in the direction of traffic.

On the contrary, under seismic shaking in the direction perpendicular to that of traffic, the rolling of wheels is not possible. Thus, live load is included for shaking in this direction. Here, it is assumed that at the time of the earthquake, 100% of design live load is present on road bridges. Further, since live load is friction supported on the rail or deck, only a portion of the live load could contribute to the seismic forces. Thus, (a) 50% of design live load in case of railway bridges, and (b) 25% of design live load in case of road bridges, is recommended.

When computing the vertical seismic forces, the entire live load is considered to be present on the bridge at the time of the earthquake.

For Calculation of Stresses Due to Live Load, but to be Combined with Stresses due to Seismic Forces (C2.4.2)

As discussed in the clause C2.4.1, it is assumed that, at the time of the earthquake, 100% design live load is present on the span in case of railway bridges and only 50% in case of road bridges.

Seismic Load Combinations (C2.5)

The design ground motion can occur along any direction of a bridge. Moreover, the motion has different directions at different times. The earthquake ground motion can be thought of in terms of its components in the two horizontal directions and one vertical direction. For regular bridges, the two orthogonal horizontal directions (x- and y- directions) are usually the longitudinal and transverse directions of the bridge. For such bridges, it is sufficient to design the bridge for seismic force (ELx and ELy) acting along each of the x- and y-direction separately. During earthquake shaking, when the resultant motion is in a direction other than x and y, the motion can be resolved into x- and y- components, which the elements in the two principal directions are normally designed to withstand.

However, in case of bridges which are irregular, particularly those with skew, design based on considering seismic force in x- and y-directions separately, leads to underdesign of the bridge components. In such case, the bridge should also be designed for earthquake forces acting along the directions in which the structural systems of the substructures are oriented. One way of avoiding to consider too many possible earthquake directions is to design the structure for:

(a) Full design force along x-direction (ELx) acting simultaneously with 30% of the design force in the y-direction (ELy); i.e., ($ELx + 0.3ELy$), and

(b) Full design force along y-direction (ELy) acting simultaneously with 30% of the design force in the x-direction (ELx); i.e., ($0.3ELx + ELy$).

This combination ensures that the components (particularly the substructure) oriented in any direction will have sufficient lateral strength. In case vertical ground motions are also considered, the same principle is then extended to the design force combinations in the three principal directions.

Increase in Permissible Stresses (C2.6).

The increases in permissible stresses in these clauses are the same as in IS: 1893-1984[2].

SEISMIC COEFFICIENT METHOD (C3.0)

Elastic Seismic Acceleration Coefficient A (C3.1)

Several changes have been incorporated in this new elastic seismic acceleration spectrum in line with the recommendations of draft code for IS:1893[11]:

(a) The basic horizontal seismic coefficient α_0 is replaced by the seismic zone factor Z, and the soil-foundation system factor β has been replaced by a soil-profile factor S. The terms representing the importance factor I and the structure flexibility factor C are retained. While the values for I have been retained the same, the expression for C has been revised.

(b) The term Z now reflects relastic values, as fraction of the acceleration due to gravity, of the expected peak ground acceleration in different seismic zones. For instance, the draft code specifies zone IV for areas which are likely to sustain shaking of intensity VIII on the Modified Mercalli Intensity (MMI) scale. The value of Z (= 0.24) for zone IV gives the value of peak ground acceleration as 0.24g which may be reasonably expected in shaking intensity VIII. The seismic zone factor Z has rationalised the relative values of design seismic force for different seismic zones. Data from past earthquakes show that, as the intensity of shaking goes up one level on the MMI scale (from VI to VII, or from VII to VIII), the peak ground acceleration almost doubles. In the existing Indian Codes[1,2], this is not reflected since the seismic force in different zones vary in the ratio 1:2:4:5:8. The draft code uses a factor of about 1.5, resulting in the ratio 1:1.6:2.4:3.6.

(c) Another change introduced in this draft is that, the soil-foundation system factor β, depending on the type of soil and the type of foundation, was intended to increase the design force for systems which are more vulnerable to differential settlements. However, in real earthquake situations, bridges do not experience higher earthquake-induced inertia forces on account of vulnerability to differential settlement. Also, the problem of differential settlement cannot be addressed by increasing the design seismic force on the bridge; instead it has to be addressed by a proper choice of the foundation. On the other hand, records obtained from past earthquakes clearly show that the average acceleration spectrum tends to be different for sites with different soil profiles. The soil-profile factor S considers this variation. The classification of soil as given in IS: 1893-1984[2] is used in this draft code. The values of S are taken from AASHTO code[10].

(d) The product of terms C and S shown in Figure 3 of the draft code represents the shape of the design spectrum with peak ground acceleration scaled to the value of 1.0. This shape is same as the average shape of the acceleration response spectrum, except in the range 0-0.1 sec. In this range, the value of CS is constant as against the response spectrum which varies from 1.0 to the maximum value (2.5) at a period of about 0.1 sec. The shape of the response spectrum in this range is modified for design purposes in view of the fact that ductility does not help in reducing the maximum forces on the stiff structures with fundamental period in the range 0.-0.1 sec. In developing this C versus T spectrum, 5% damping is implicitly assumed.

(e) The fundamental natural period T of the bridge, along the considered direction of lateral force, is required to obtain the bridge flexibility factor C. The expression proposed for C is taken from the AASHTO code[10].

In case of buildings, experimental measurements are made on existing buildings and empirical expressions are arrived at for the fundamental natural period T. However, in case of bridges, there is a significant variation in the parameters of the bridges even within the same structural system. Thus, an empirical expression for natural period cannot be arrived at. Hence, recourse to analytical methods becomes essential.

(C3.1.1) For the purposes of the seismic coefficient method, a simple procedure based on static analysis is recommended to obtain the fundamental natural period. The bridge is assumed to behave like a single degree of freedom system in the considered direction of shaking, and the natural periods is obtained by the expression.

$$T = 2\pi\sqrt{\frac{m}{k}}$$

Here, the mass m of the bridge is obtained from its dead load D (kN) by dividing with the acceleration due to gravity. Also, in order to obtain the stiffness k in kN/mm, a force F is applied in the direction of the considered lateral force at the centre of mass of the bridge system, such that the displacement along that direction is 1 mm (Fig. C1). Thus, $k = F/1 = F$. And the expression for T modifies to:

$$T = 2\Pi\sqrt{\frac{D/g}{F}}$$

To keep the units consistent, g has to be in mm/sec^2 (9810 mm/sec^2). Thus, the equation reduces to:

$$T = 2\pi\sqrt{\frac{D}{9810F}}$$

Simplifying,

$$T = 2.0\sqrt{\frac{D}{1000F}}$$

where,

D Dead load reaction of the bridge in kN, and

F Lateral force in kN required to be applied at the centre of mass of the superstructure for one mm horizontal deflection of the bridge along the considered direction of lateral force.

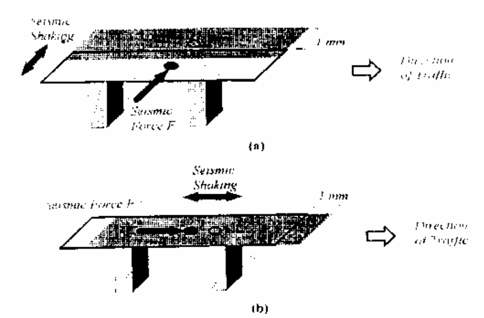

FIG.C1 DEFORMED GEOMETRY OF BRIDGE DECK WHEN EARTHQUAKE FORCE IS (a) Normal to the direction of traffic and (b) Along the direction of traffic. Full circle shows the location of the center of mass of the bridge system before application of the force f and the hollow circle that after application of the force f.

Maximum Elastic Forces and Deformations (C3.2)

The inertia force is generated at the locations of the mass. This clause suggests that the entire inertia force generated in a bridge component be applied as a concentrated load at its centre of mass. Clearly, when the mass is distributed along the dimension of the bridge component, the above approach may result in the incorrect estimation of force resultants due to inertia forces. Designers may require to subdivide such bridge components into smaller segments, and evaluate the inertia force for each of these segments separately. In such a case, the inertia force generated by the mass of each segment may be proportionally distributed at the end nodes of that segment. In fact, this is already recommended in the AASHTO code[10], which requires that

(a) The superstructure should be modelled as a series of plane frame members with nodes at quarter span points, and joint elements. The lumped mass inertia effects should be properly distributed at these locations; and

(b) The substructure should be modelled as a series of plan frame members and joint elements. In case of short stiff columns having lengths less than one-third of either of the adjacent span lengths, intermediate nodes are not necessary. However, long flexible columns should be modelled with intermediate nodes at the third points.

The criteria for earthquake resistant design is complete only when recommendation on all the following are included: (i) load factors and allowable stresses, (ii) design acceleration spectrum, including the method of obtaining the natural period T, (iii) damping ratio, and (iv) method of analysis. The response reduction factors R, to reduce the maximum elastic forces to the design forces, are calibrated keeping in mind these factors. Thus, this clause specifies that linear elastic analysis be used to obtain the bending moment, shear and axial forces at different locations in the bridge.

Inertia Force Due to Mass of Each Bridge Component (C3.2.1)

The inertia force due to the mass of a bridge component under earthquake ground shaking, in a particular direction, depends on the elastic seismic acceleration coefficient computed for shaking along that direction. Clearly, this acceleration coefficient will be different along different directions, for the same mass, owing to different natural periods along those directions.

Elastic Seismic Acceleration Coefficient for Portions of Foundations below Scour Depth (C3.2.2)

The propagation of waves within the body of the earth is modified at the surface of the earth, owing to the wave reflections at the boundary surface. For this reason, it is generally accepted that the shaking is relatively more violent at the surface. Hence, the draft code permits reduction in the elastic seismic acceleration coefficient A for portions of foundations below scour depth.

Seismic Weight (C3.2.3)

The dead load of the superstructure also includes the superimposed dead load, that is permanently fastened or bonded with its structural self weight. Since there is a limited amount of friction between the live load and the superstructure, only a part of the live load is included in the inertia force calculations.

It is clear that the seismic force on a bridge component is generated due to its own mass, and not due to the externally applied forces. The presence of buoyancy and uplift forces does not reduce its mass. Thus, the clause requires that buoyancy and uplift forces be ignored in the seismic force calculations.

Design Seismic Force Resultants for Bridge Components (C3.3)

The basic philosophy of earthquake resistant design is that a structure should not collapse under strong earthquake shaking, although it may undergo some structural as well as non-structural damage. Thus, a bridge is designed for much less force than what would be required, if it were to be necessarily kept elastic during the entire shaking. Clearly structural damage is permitted, but should be such that the structure can withstand the large deformations without collapse. Thus, two requirements come into picture, namely (a) ductility, i.e., the capacity to withstand deformations beyond yield, and (b) overstrength. Overstrength[12] is the total strength including the additional strength beyond the nominal design strength considering actual member dimensions and reinforcing bars adopted, partial safety factors for loads and materials, strain hardening of reinforcing steel, confinement of concrete, presence of masonry infills, increased strength under cyclic loading conditions, redistribution of forces after yield owing to redundancy, etc., The response reduction factor R, used to reduce the maximum elastic forces to the design forces, reflect these above factors.

Clearly, the different bridge components have different ductility and overstrength. For example, the superstructure has no or nominal axial load and hence its basic behaviour is that of flexure. However, the substructure which is subjected to significant amount of axial load undergoes a combined *axial load-flexure* behaviour. It is well-known that the latter system is less ductile than the former. Also, the damage to the substructure is more detrimental to the post-earthquake functioning of the bridge than damage to the superstructure. In the second case, the span alone may have to be replaced, while the first requires replacement of the entire bridge. Thus, the R factors for superstructures are kept at a higher value than those for substructures. Similar argument can be given for the R values of foundations, which are even lower values than those for substructures.

An important issue is that of connections, which usually do not have any significant post-yield behaviour that can be safely relied upon. Also, there is no redundancy in them. Besides, there is a possibility of the actual ground acceleration during earthquake shaking exceeding the values reflected by the seismic zone factor Z. In view of these aspects, the connections are designed for the maximum elastic forces (and more) those are transmitted through them. Thus, the R factors for connections are recommended to have values less than or equal to 1.0.

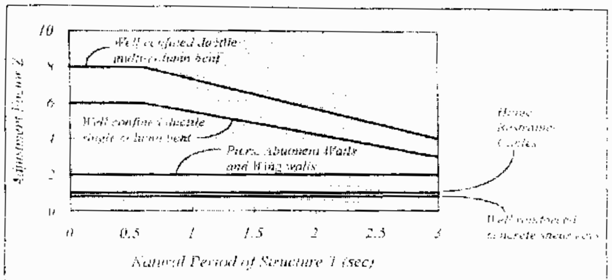

FIG. C2 CALTRANS ADJUSTMENT FOR DUCTILITY AND RIS ASSESSMENT FACTOR Z

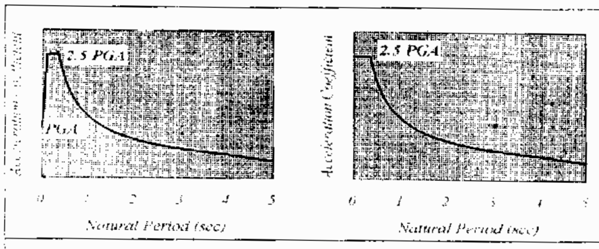

FIG.C3 ACCELERATION RESPONSE SPECTRUM
(a) Actual (but smoothened), and (b) Idealised for design purposes

For quite sometime now, countries with advanced seismic provisions have been using this approach of obtaining the design forces from the elastic maximum forces. For example, the CALTRANS code[13] uses the *adjustment factor* Z (similar to the response reduction factor R ; values of the same are shown in Fig.C2. Similarly, the AASHTO code uses a factor R, called the response *modification factor*, whose values are shown in Table C1.

TABLE C1
RESPONSE MODIFICATION FACTOR R AS PER AASHTO CODE[9]

Substructure[1]	R	Connections	R
Wall-Type Pier[2]	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents	3	Expansion Joints Within a Span of the Superstructure	0.8
a. Vertical Piles only b. One or more Batter Piles	2	Columns, Piers or Pile Bents to Cap Beam or Superstructure[3]	1.0
Single Columns	3	Columns or Piers to Foundations[3]	1.0
Steel or Composite Steel and Concrete Pile Bents a. Vertical Pile Bents b. One more Better piles	 5 3	Columns or Piers to Foundtions	2.0

1. The R-factor is to be used for both orthogonal axes of the substructure.

2 A wall-type pier may be designed as a column in the weak direction of the pier, provided all the provisions required for ductile detailing are followed. The R-factor for a single column can then be used.

3 For bridges classified as SPC (Seismic Performance Category) C and D, it is recommended that the connections be designed for the maximum forces capable of being developed by plastic hinging of the column bent as specified in the code. These forces will often be significantly less than those obtained using any R-factor of 1.

RESPONSE SPECTRUM METHOD (C4.0)

Elastic Seismic acceleration Coefficient A_k in Mode k (C4.1)

Typical shape of the acceleration response spectrum when plotted with natural period on the x-axis, is shown in Figure C3(a). It starts at the value of Peak Ground Acceleration (PGA) at zero period, rises to about 2.5 times (for 5% damping) the PGA value at a period of about 0.1 sec. and then remains at that value upto about 0.3 sec. period. However, seismic design codes usually assume the design spectrum shape to be horizontal for the range from 0.3 sec. upto zero period codes ignore the fact that the spectrum has lower values of acceleration in 0-0.1 sect. range, as shown in Figure C3(b)). There are several reasons for this conservative approach. For instance, ductility does not help in reducing the maximum forces if natural period is in this range of 0-0.1 sec[14] Hence, it is necessary to raise the level of spectrum in this range. Also, since the acceleration response spectrum has a very steep slope in the range 0-0.1 sec., any small underestimation of the natural period T may lead to a significant reduction in the seismic force.

However, in multimode analysis, this draft allows the designer to use the ascending part of the spectrum in the range 0-0.1 sec, but only for the higher modes of vibration. Since, the fundamental mode makes the most significant contribution to the overall response and the contribution of higher modes is relatively small and this is now permitted by several codes[8,10].

Inertia Force due to Mass of Bridge at Node i in Mode k (C4.2)

The above expression is part of the routine solution procedure for analysis of elastic structures subjected to seismic ground motion represented by its pseudo-acceleration response spectrum. The mathematical model of the bridge structure should properly account for all stiffnesses and masses. A suitable number of intermediate nodes are required for each bridge component, to properly estimate the stress resultants caused by the seismic inertia forces. In doing so, it will be advantageous to follow the current AASHTO code practices. Rotational moment of inertia of certain masses in the bridge structure may become important particularly in case of joint elements; the same may be incorporated in the matrix of seismic weights, as mass moment of inertia times acceleration due to gravity.

Seismic Mass Matrix (C4.2.1)

The seismic weight of each bridge component is proportionally distributed to its end and intermediate nodes as lumped masses considering its geometry. These lumped masses are used to form the matrix of seismic weights, keeping in mind that the mass lumped at a node contributes to all the translational degrees of freedom at that node.

Number of Modes to be considered (C4.2.2)

This clause indirectly requires that all modes those contribute significantly to the response be included in the analysis. And, the book-keeping is done through the modal masses. Clearly, the modes with low participation in the dynamics of the bridge for earthquake shaking, along a chosen principal direction, will have very small modal mass and the dynamic force carried by these modes would also be small. The clause suggests that at least 90% of the total seismic mass (clauses 4.2.1 and 3.2.3) shall be included through the modes that are considered.

Maximum Elastic Forces and Deformations (C4.3)

The modal response quantities (Bending moment, Shear and Axial forces, Displacements and Rotations) at any location of the bridge in each mode need to be combined to obtain the maximum response due to all modes considered. Studies on modal response combinations show that, when modal frequencies are well-separated the ''Square Root of Sum of Squares (SRSS) Method'' provides reasonable estimates. If two modal frequencies are separated from each other upto or equal to 10% of the smaller one, then the two modes may be termed as closed-spaced modes. The Complete Quadratic Coefficient (CQC) method provides reasonably good estimates of the overall response, irrespective of whether the modal frequencies are closely-spaced or separated. However, the CQC method as stated in clause 4.3.1 assumes that the modal damping ratio is same for all the modes of vibration. In case it is not so, reference shall be made to literature for suitable expressions for modal response combination[15].

Design Seismic Force Resultants in Bridge Components (C4.4)

As discussed in the commentary under 3.3, the various components of the bridge do not enjoy the same level of ductility and overstrength. Hence, the level of design seismic force vis-a-vis the maximum elastic force that will be experienced by the component, if the entire bridge were to behave linearly elastic, varies for different bridge components. The values of the response reduction factor R given in Table 5 reflect the same.

Site-Specific Spectrum (C4.7)

To ensure at least the minimum strength in the bridge structure, this clause prevents the designer from using a site-specific spectrum, that results in unduly small design force resultants in comparison with those given by this draft code.

SUPERSTRUCTURE (C5.0 & C5.2)

Since the supporting width of the span in the transverse diection is relatively small in comparison with that in the longitudinal direction, overturning of superstructure (those are resting on the substructure and without monolithic connections) in the transverse directions may be possible under the combined action of seismic forces along transverse and vertical directions. Of course, in these calculations, the direction of vertical seismic force shall be taken so as to produce the worst effect.

(C5.3) This clause makes it mandatory in high seismic regions to have suitable linking devices provided between the superstructure and substructure, and between the suspended spans (if any) and restrained portion of the superstructure.

(a) Vertical hold-down devices to prevent the superstructure from lifting off its supports atop the substructure, particularly under vertical seismic forces combined with the transverse seismic forces, and

(b) Horizontal linkage elements to prevent excessive relative deformation between portions of the superstructure, or between the superstructure and substructure.

Vertical Hold-down Devices (C5.4)

Vertical hold down devices are considered essential in the draft provisions to minimise the potential of adverse effects of vertical seismic excitation. The provisions for design force of vertical hold-down devices have been adapted from the AASHTO code[10].

Horizontal Linkage Elements (C5.5)

The design seismic force for each bridge component is only a fraction of the maximum elastic force that can be sustained if it were to remain completely elastic during earthquake shaking. However, the deformations calculated from the linear analysis of the bridge subjected to these design forces are much smaller than the actual deformations experienced during seismic shaking.

Unseating of superstructure from the substructure from the substructure or the suspended span from the restrained portion are the possible consequences, if the actual deformations are not accounted for in the design of the supports at these interface locations. Sometimes, the two portions that may move relative to each other are securely fastened by *positive horizontal linkage elements*, usually either high tensile wire strand ties, cables or dampers. For the design of these devices, the recommendations from the AASHTO code[10] are used. The design forces specified are conservative to provide increased protection at a minimum additional cost.

SUBSTRUCTURE (C6.0)

Hydrodynamic Force (C6.4)

This clause is retained as given in IS:1893-1984[2], except that A replaces α_h. As stated earlier, A is different from α_h. Hence, the hydrodynamic forces calculated as per this code will be higher than those estimated as per IS:1893-1984.

FOUNDATIONS (C7.0)

Seismic Zones IV and V (C7.3)

Damage to foundations have very serious implications from structural safety considerations. Also, foundation repairs are very expensive as it is very difficult to access. Hence, it is *required* to ensure that these are not damaged. This clause is intended to achieve the objective that, in case of severe ground shaking, the foundation is not damaged. This is done first by requiring a much lower value of response reduction factor for foundation than for the substructure, i.e., a much higher design seismic coefficient for foundation, than that for the substructure. However, this is qualified through the *concept of capacity design*[3,16].

Since the seismic forces are inertia induced, the foundation can never experience a seismic force higher than what the substructure is capable of transmitting. The attempt is to obtain this upper-bound force that can be transmitted by the substructure, by calculating its overstrength plastic moment capacity. The lower of (a) and (b) is recommended to be used in design of the foundation.

CONNECTIONS (C8.0)

Design Force for Connections within Superstructure and between Superstructure and Substructure (C8.1)

Seismic Zones I, II and III (C8.1.1)

In low seismic regions, the effort in the seismic design of the bridges is reduced to some extent by this clause, by requiring only a simple design force calculation for the restrained supports (Rocker or Elastomeric bearings). The clause, same as that in the AASHTO code, is considered to provide a little overestimate of the design force.

Seismic Zones IV and V (C8.1.2)

The most common cause for earthquake disasters in case of bridges is the failure of connections, particularly those between superstructure and the substructure. Hence, extra caution is needed to ensure the safety of connections. This is done by requiring the value of response reduction factor for bridges as 0.8 or 1.0 (Table 5), which implies that the design force for connections is equal to (or more than) the maximum expected elastic force. However, by allowing the designer to use the lower value from (a) and (b) above, the code brings in the capacity design concept. Force obtained by (b), provides an upper-bound on the inertia force that can be developed in the superstructure before the substructure becomes plastic. Once the substructure becomes plastic, the bridge will not be able to sustain higher inertia forces.

Provisions to account for Displacements at Connections where Motions are Permitted (C8.2)

Separation Between Adjacent Units (C8.2.1)

When two adjacent units are designed such that the relative movement between them is expected to occur at their separation joint, then adequate clearance is necessary between them to avoid pounding and the consequential damage. To provide the cumulative sum of the displacements of the two units at the separation, would be too conservative. Thus, this clause proposes that the square root of the sum of squares of the calcualted displacements of the two units under the earthquake forces may be provided as the clearance.

Minimum Width of seating at Supports of Superstructure on Substructure, or of the Suspended Span Portion on the Restrained Portion of the Superstructure (C8.3)

The connections between superstructures and substructures are designed for forces specified under clause 8.1. Even though these values are conservative, there still will remain possibilities of the actual seismic force exceeding the actual strength of the connections. Also, in bridges the substructures are liable to undergo large displacements due to dynamic earth-pressures. Under these conditions, it is possible that the superstructure span may get separated at the connection. At this instance, if adequate width is available on top of the substructure, the superstructure span is prevented from being dislodged off its support. Clearly, if the superstructure is still resting atop the substructure, the cost of repairing the connection and restoring the superstructure to its desired position is far more economical than having to rebuild the superstructure.

Hence, this clause attempts that even under maximum expected deformations, possibility of collapse or loss of span are minimised through conservative provision of minimum seating widths. The values of seating width recommended for high seismic regions are about 66% higher than those for low seismic regions; The above provision for minimum seating widths W_b (mm) is similar to that adopted in the AASHTO code[10], given by:

$$W_b = \begin{cases} 203 + 1.67L + 6.66H & \text{for low seismic performance categories} \\ 305 + 2.5L + 10H & \text{for high seismic performance categories} \end{cases}$$

where L and h are as defined in the draft code. Clearly, this draft code recommeds a higher seating width than the American practice. This is motivated by the Japanese Code[17], which requires that the seat length S_E (mm) from edge of superstructure to the edge of the substructure shall be longer than the value estimated by the expression

$$S_E = \begin{cases} 700 + 5L & L \leq 100m \\ 800 + 4L & L > 100m \end{cases}$$

where L represents the span length (in mm)

CAPACITY DESIGN OF BRIDGE COMPONENTS (C9.0)

This clause requires some additional provisions, which ensure that brittle failure do not precede the ductile failure. In a structure having both brittle and ductile elements, if it can be ensured that the ductile elements will yield prior to failure of brittle elements, the post-yield behaviour of the structure will be ductile. The concept of *capacity design* is used to ensure post-yield ductile behaviour of a structure having both ductile and brittle elements. In this method, the ductile elements are designed and detailed for the design forces. Then, an upper-bound strength of the ductile elements is obtained. It is then expected that if the seismic force keeps increasing, a point will come when these ductile elements will reach their upper-bound strength and become plastic. Clearly, it is necessary to ensure that even at that level of seismic force, the brittle elements remain safe. This procedure is referred to as the capacity design procedure[16]. In India steel substructures are not yet prevalent. Hence capacity design provisions provided herein, are meant only for *bridges with reinforced concrete substructures. In case* steel substructure is to be adopted for high seismic zone, one must consult relevent provisions for codes of other countries.

Design Force for Substructure (C9.1)

The clause is meant to ensure ductile behaviour of the substructure. In R.C. members, flexural failure can be ductile if the member is detailed appropriately. On the other hand, shear failure is brittle. Hence, the columns are designed and detailed for flexure first. Then, using the principle of capacity design, one calculates how much is the maximum possible earthquake force that this column can sustain in the event of strong shaking. Since the shear failure is a brittle failure, shear design for columns is carried out for this upper bound load. A similar provision (clause 7.2.5 in IS:4326-1976[18] and clause 6.3.3 in IS: 13920-1993[7]).

Overstrength Plastic Moment Capacity (C9.3)

Limit State Method of Design (C9.3.1)

The factor 1.4 is derived as following: The partial safety factor used in limit state design[8] for reinforcing steel is 1.15, i.e., the yield stress in steel used in design is $0.87f_y$. Further, in estimating the overstrength capacity, it is assumed that 25% increase in steel stress is possible owing to its strain-hardening behaviour; thus the maximum stress in reinforcing bars for design can be up to $1.25f_y$, as against $0.87f_y$, used in calculating ultimate moment capacity. Since sections are necessarily under-reinforced for ductile behaviour, the ultimate moment carrying cpacity is influenced primarily by the stress in steel, and only very marginally by the grade of concrete. Thus, the ultimate moment capacity of the section can be scaled-up proportional to the ratio of the maximum stress in steel in the two cases, to obtain the plastic moment hinge capacity. Thus, $1.25 \times (1/0.87) = 1.437$, is rounded off to 1.4. Similar factor of 1.4 is in practice in the ductile detailing provisions of IS: 13920-1993 for reinforced concrete members.

Working Stress Method of Design (C9.3.2)

The entire framework of capacity design is really applicable only to limit state design. It is difficult to extend it to the working stress method of design. However, considering that the Indian professionals will continue to use working stress method for design of bridges, this clause provides an approximate method for the capacity design concept for such situations.

The overstength moment capacities of reinforced or prestressed concrete sections those are usually designed to be under-reinforced, are governed by the stress in reinforcing steel. Considering the permissible stress design in direct tension of $0.6f_y$ to increase to $1.25f_y$ owing to strain-hardening, the strength increase is given by $1.25/0.6 \approx 2.1$.

Similarly, for structural steel sections, the permissible stress in bending of $0.67f_y$ may increase to $1.25f_y$ owing to strain hardening. Considering the shape factor of around 1.15 for rolled steel sections, the strength increase when the entire cross-section is yielding in bending, is given by $(1.25/0.67) \times 1.15 \approx 2.1$.

DISCUSSION AND CONCLUSION

The proposed draft code includes significant improvements over the existing provisions in the Indian codes, namely IRC:6-1966[1] and IS:1893-1984[2], for seismic design of bridges. Many of the issues raised in earlier literature on the performance of bridges in India during past earthquakes[5] and on the state-of-the-art reviews of Indian seismic code provisions[3,4] have been incorporated.

However, there are still a number of areas where the Indian codes need to be further improved; these include detailed clauses on the design and detailing of individual components of foundations and abutments, of all structural steel bridge components, and of all reinforced concrete bridge components.

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