A state-of-the-art review on seismic design of bridges - Part I : Historical development and AASHTO code

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The basic philosophy of seismic design is the same for all structures. However, there are certain significant and necessary differences in the design calculations for bridges as against those for buildings. For instance, the American codes employ different response reduction factors for different bridge components. The distinctly different calculations have arisen after evaluating the performance of the bridges during past earthquakes in the USA, Japan and other countries. In this paper, the historical development of the American seismic code provisions for design of bridges is reviewed to highlight the departure from the method of calculations usually adopted for buildings and the origin of the special calculations for bridges. Further, the paper shows how the seismic performance observed in the bridges in USA has been translated into code provisions in the AASHTO code of USA.

Rather poor performance of bridges in the 1971 San Fernando earthquake in California, USA, and the 1978 Miyaji-Ken Oki earthquake in Japan clearly revealed that the usual seismic design procedures applicable to bridges cannot be applied to bridges. Bridges pose their own unique problems vis-a-vis seismic performance. As a result, in the last twenty years, the state-of-the-art of earthquake-resistant design of bridges has undergone significant changes and major modifications have taken place in the bridge codes of USA, Japan and New Zealand.

On the other hand, the provisions on the seismic design of bridges in the Indian codes[1,2] continue to remain rather simplistic and in line with what is perhaps adequate for buildings. This is reflected in the poor performance of bridges in India[3,4]. In this paper, the current provisions in AASHTO code[5], USA are reviewed. A companion paper[6] presents a review of provisions in another American code, namely CALTRANS code[7] together with those in the draft New Zealand code[8] and the current Indian codes.

Background on earthquake-resistant design

Earthquake-resistant design is fundamentally very different from the design for other dynamic effects, such as wind loads and vehicle loads. This section reviews some of the basic issues involved in seismic design.

Since the size of a future earthquake and shaking intensity expected at a particular site cannot be determined accurately, the seismic forces are difficult to quantify for the purposes of design. Further, the actual forces that can be generated in the structure during an earthquake are very large, and designing the structure to respond elastically against these forces makes the structure too expensive. Therefore, in the earthquake-resistant design, post-yield inelastic behaviour is usually relied upon to dissipate the input seismic energy. Thus, the design earthquake force may be only a fraction of the maximum (probable) forces generated if the structure is to remain elastic during the earthquake. For instance, the design seismic force may at times be, say, 8 percent of the maximum elastic seismic force. Thus, earthquake-resistant design and construction does not aim to achieve a structure that will not get damaged in a strong earthquake having low probability of occurrence; it aims to have a structure that will perform appropriately and without collapse in the event of such a shaking.

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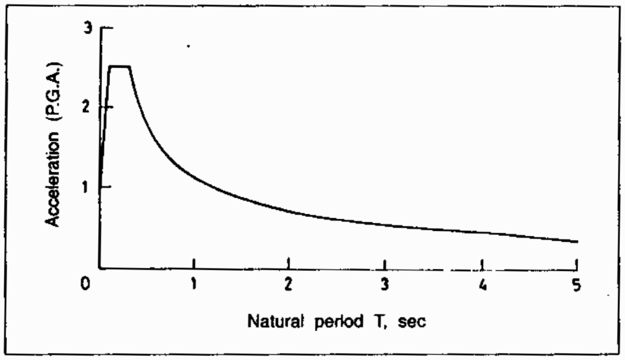

Fig 1 A typical average acceleration response spectrum (5 percent damping) of recorded ground motions (in units of Peak Ground Acceleration, PGA).

In seismic design, 5 percent-damped elastic acceleration response spectrum S_a is often used. A typical average shape of acceleration response spectrum is shown in *Fig 1*. The spectral value S_a corresponding to zero natural period $T = 0$ is the peak ground acceleration (PGA). The horizontal PGA may be about 0.3 g in area sustaining ground shaking of intensity VIII on the Modified Mercali Intensity (MMI) scale. Also, the maximum S_a value (for natural period of about 0.1 s to 0.3 s) is about 2.5 times PGA in case of 5 percent damped spectrum. Thus, a building, located in seismic zone IV (which corresponds to MMI VIII) and having natural period of 0.2 s may sustain maximum lateral force (if it were to remain elastic) of about 75 percent of its self weight. As against this, Indian seismic code provides the design coefficient as 0.05 g for a typical building with fundamental period of 0.2 s and located in seismic zone IV. The difference in the two numbers, which involves a factor of about 10 to 15, is accounted for by overstrength and ductility in the building.

Overstrength is the actual strength of the structure which is usually much higher than the design strength; the difference is inherently introduced in the code-designed structures. Numerous factors contribute to this. For example, load factors and strength reduction factors used in design, lower gravity loads present at the time of the earthquake than assumed in the design, actual strengths of materials which are often larger than characteristic values used in design, larger member sizes and higher reinforcement provided than required from strength considerations, material strengths under cyclic earthquake conditions being higher than under static conditions, and contribution of non-structural and structural non-seismic elements to lateral resistance. The value of overstrength in buildings varies widely. For instance, values in the range of 2.8 to 15.0 have been reported for one type of reinforced concrete (RC) moment resisting frames[9].

Ductility is the capacity of a structure (or a member) to undergo deformation beyond yield without losing much of its load carrying capacity. Higher the ductility of the structure, more is the reduction possible in its design seismic force over what one gets for linear elastic response. In well-designed buildings, a ductility reduction factor of upto 4-5 can be achieved.

One can now define the response reduction factor R as the product of overstrength and ductility reduction factor. The design seismic force for the structure can be taken as the maximum seismic force expected, if the structure responds elastically, divided by the response reduction factor. Thus, with an overstrength of 3.0 and the ductility reduction factor of 4.0, one could design the building for one-twelfth of the maximum elastic force.

The design horizontal force of the order of five to twelve percent of the weight of the building, originally came from the actual performance of buildings during the damaging earthquakes in Japan in the early part of this century. The actual ground motions, caused by strong earthquake shaking, were recorded only later. Only recently it has become possible to explain why design based on such a small fraction of the maximum elastic forces was sufficient in buildings. As will be seen subsequently, the earlier bridge codes adopted seismic design criteria similar to that for buildings: this required that the bridge as a whole be designed for 5-12 percent of its weight acting in the horizontal direction. Failures of bridges in the US and Japan in the seventies clearly showed that this was not sufficient. The bridges do not have same amount of ductility and overstrength in all parts of the structure as in case of buildings. For instance, the overstrength available at the connections between the superstructure and substructure is only nominal and there is hardly any ductility available at the connections. Therefore, the connections in bridges are to be designed for much higher levels of seismic force. The concept of capacity design is extensively used in the design of individual bridge components. A brief review of the principal of capacity design is given in the next section.

Capacity design

Consider a structure having both brittle and ductile elements. As load on this structure is increased, if the brittle elements fail while the ductile elements are still below yield, the structural failure will be brittle. However, 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.

The concept can be explained with the example of a chain under tensile load, *Fig 2*. One obvious way to ensure ductile behaviour of the chain is to simply ensure that all the links in the chain are ductile. Let us say that it is too expensive, or simply infeasible, to have all the links as ductile. Assume one link is made of a ductile material (say, mild steel) and the rest are of a brittle material (say, cast iron). The conventional design of the chain is as follows.

- The most reasonable assessment of required

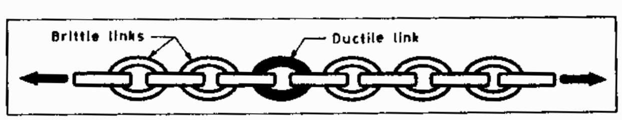

Fig 2 A chain under tensile loading

strength of the chain is made.

- Some factors of safety are applied on this force and on strength of the two types of links, and the links are designed.

The actual strength of the individual links may be different from the calculated strength. As load on the chain goes beyond the design load, there is a possibility that failure may occur in any of the links and once any one link fails, the entire chain fails. If one of the brittle links fail first, the chain will fail in a brittle manner and vice-versa.

However, to ensure that the chain behaves in a ductile manner, it can be designed as follows:

- assess the required strength of the chain
- apply suitable factors of safety on this load and material properties, and design the ductile link carefully so that it will behave in a ductile manner
- assess the upper bound on load that the ductile link can sustain before failure, considering factors of safety, overstrength in the link, its strain hardening, etc.
- design the brittle links for the upper-bound load calculated in the previous step.

This design will ensure that the brittle elements will remain elastic at all loads prior to the failure of the chain. Thus, the brittle failure mode has been prevented. This procedure is referred to as the capacity design procedure[10].

Historical developments of seismic bridge codes of USA

It is of interest to review the development of bridge codes in the United States. There are two major US codes of interest for seismic design of bridges, namely, Standard Specification for Highway Bridges by the American Association of State Highway and Transport Officials (AASHTO), and the Bridge Design Specifications of the California Department of Transportation (CALTRANS). The following review is based on an unpublished note[11] and section 1.2 "background" in the AASHTO code[3].

The first provisions in USA for considering aseismic design of bridges appeared in the AASHTO 1958 standard specification for highway bridges. These required that irrespective of the location of the bridge and seismic risk, the design earthquake force V be taken as:

$$V = kW, \tag{1}$$

where

W = total weight of the structure, and

k = 0.02 for bridges supported on spread footings on soils having an allowable bearing capacity greater than 3.5 tons/ft^2,

= 0.04 for the spread footings on soils having an allowable bearing capacity less than 3.5 tons/ft^2, and

= 0.06 for pile footings.

On the other hand, prior to the 1971 San Fernando (California) earthquake, the California State Division of Highways specified that all structures, except underground structures and retaining walls, shall be designed to resist earthquake forces EQ applied horizontally at their centre of gravity. Further, this force shall be distributed to supports according to their relative stiffness. Here, the design earthquake force EQ was specified as :

$$EQ = KCD \geq 0.02D, \tag{2}$$

where

K = 1.33 for bridges where a wall with a height-to-length ratio of 2.5 or less resists horizontal forces applied along the wall.

= 1.00 for bridges where single columns or piers with a height-to-length ratio of 2.5 or less resist horizontal forces.

= 0.67 for bridges where continuous frames resist horizontal forces applied along the frame.

$C = \frac{0.05}{\sqrt[3]{T}} \leq 0.10$, and

$T = 0.32\sqrt{\frac{D}{P}}$ (for single storey structures).

In the above, coefficient K reflects the energy absorption capability of the structure depending on its substructure; C is a coefficient representing the structures stiffness; T is the natural period of the structure; D is the dead load reaction of the structure; and P is the lateral force required for one inch horizontal deflection of the structure. Further, it was also added that special consideration be given to structures founded on soft materials capable of large earthquake movements, and to large structures having massive piers.

The poor performance of bridges constructed with the above criteria during the San Fernando (California) earthquake of 1971 caused so much concern that as an interim and immediate measure the California State Division of Highways increased the design seismic load for bridge columns by 200-250 percent pending a more rational design approach. This interim measure required bridge columns to be designed for seismic forces as follows

$EQ = 2.0\ KCD$ for frames on spread footings

$$EQ = 2.5\ KCD \text{ for frames on pile footings} \tag{3}$$

In 1973, the California Department of Transportation (CALTRANS) introduced a new design criteria for bridges. It considered the relationship between the bridge site and active faults, the seismic response characteristics of soils at the bridge site, and the overall dynamic response characteristics of the bridge. In 1975, these CALTRANS provisions were slightly modified and enforced in all parts of the USA by AASHTO.

In 1977 the Federal Highway Administration (FHWA) funded the Applied Technology Council (ATC) to identify the research studies related to highway bridges, to develop and recommend new seismic design guidelines for bridges, and to evaluate the probable impact of these recommended guidelines on bridge design, construction and cost. In 1978, CALTRANS revised its design criteria. The risk and ductility factors were removed from the design spectrum; these factors, however, were included in the design on a member component basis. In 1981, the ATC publication "seismic design guidelines for highway bridges, ATC-6"[12] became available. In 1983, AASHTO adopted this document as a guideline specification.

Since then, a lot of developments have taken place in this field and codes are being revised regularly. In addition to lessons from 1971 San Fernando earthquake, the bridge failures in other earthquakes in California, Alaska, Japan, Guatemala and New Zealand have significantly contributed to the development of improved seismic design in several countries, including USA, New Zealand, Japan and China.

Review of AASHTO code (USA)

Design philosophy

The elastic forces in the members and connections generated under the maximum probable earthquake are first obtained. Then, different reduction factors are used to arrive at the design forces for each component, whether a member or a connection. The reduction factor used is different for different members/connections. Thus, the superstructure may be designed for altogether different "design seismic coefficient" than, say, the substructure; and, the connections between the superstructure and the substructure may be designed for altogether different forces. In the design of foundations, the concept of capacity design is used, wherein the foundations are designed to withstand the smaller of :

(*i*) the maximum elastic forces, and

(*ii*) the forces resulting from the formation of plastic hinges in the columns of the substructure.

The code provides the minimum support length of girders over substructure (abutments/columns) to avoid loss-of-span type of failures. It also requires, where appropriate, the vertical hold-down devices and positive linkage elements.

Design force level and reduction factors

Acceleration coefficient

The seismic risk at a site is represented by the acceleration coefficient A at the location of the bridge. This is given by a contour map, as against the more commonly used zone map. The term acceleration coefficient here represents the effective peak velocity-related acceleration coefficient A_v as defined in the ATC-3-06[13]; it basically represents the value of expected peak ground velocity in the units of acceleration. The recommended values are arrived at on the basis that there is a 90 percent probability that these values will not be exceeded during a 50-year period. The seismic coefficient A takes values as large as 0.80 for the most severe seismic areas; that is, it is recognised that peak ground acceleration can be as high as 0.80 g.

Importance of bridge

The code accounts for importance of bridge by providing for more complexity and sophistication in the analysis and design, and not by increasing the design seismic force. All bridges located in area having acceleration coefficient A greater than 0.29 are assigned an importance classification IC as follows:

$$IC = I \text{ for essential bridges}$$
$$IC = II \text{ for other bridges} \tag{4}$$

Essential bridges are those that must remain functional after an earthquake. This is to be decided based on social/survival and security/defence requirements as well as on average traffic on the bridge. For bridges located in area; having acceleration coefficient A equal to or less than 0.29, the importance of the bridge does not affect the seismic analysis and design procedures.

Seismic performance categories

Depending on the acceleration coefficient A and the importance classification IC, the bridge is assigned a seismic performance category (SPC) denoted by A,B,C, or D, *Table 1*. Category D denotes the most stringent seismic requirements.

Table 1: Seismic performance categories (SPC)[3]

Acceleration coefficient, A	*Importance classification IC*	
	I	*II*
$A \le 0.09$	A	A
$0.09 < A \le 0.19$	B	B
$0.19 < A \le 0.29$	C	C
$0.29 < A$	D	C

Soil classification

The effects of site conditions on the basis of the elastic response spectrum are included through the site coefficient S, *Table 2*. These coefficients apply irrespective of the type of foundation. In the absence of sufficient information on the soil profile at the site, or when the soil profile does not match the ones listed here, the site coefficient for soil profile *II* may be used.

Response modification factor

Using the value of acceleration coefficient and the site coefficient, the maximum elastic force on the structures is calculated by considering the typical shape of design spectrum. The maximum elastic force thus arrived at is then divided by the response modification factor, *Table 3*, to obtain the design force. It is noted that R is different for different components of the same bridge (as against buildings where the building as a whole is assigned a value of R, and this factor remains same for all components of that building). Also note that R is as low as 0.8 for the connections, meaning thereby that the connections are to be designed for more than the maximum expected elastic design force.

Table 2 Site coefficient S (AASHTO, 1992)[3]*

	Soil profile type		
	I**	II***	III****
S	1.0	1.2	1.5

Note : *AASHTO LRFD Bridge Design Specifications (S.I. Units), First Edition, 1994, provides another soil profile type (*Type IV*) with S factor of 2.0. A profile with soft clays or silts greater than 12,000 mm in depth shall be taken as *Type IV*. These materials may be characterised by a shear wave velocity of less than 550 km/hr and might include loose natural deposits or man-made, non-engineered fill.

**Soil profile Type I is a profile with either

(i) rock of any characteristic, either shale-like or crystalline in nature [such material may be characterised by a shear wave velocity greater than 2500 ft/s (762 m/sec, or by other appropriate means of classification]; or

(ii) stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, and stiff clays.

***Soil profile type II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

****Soil profile type III is a profile with soft to medium stiff clays and sands, characterised by 30 ft (9 m) or more of soft to medium stiff clays with or without intervening layers of sand or other cohesionless soils.

Method of analysis

The code provides two analysis procedures, namely the "single mode spectral method" (procedure 1) and the "multi-mode spectral method" (procedure 2) as given in *Table 4*. Procedure 1 is acceptable for regular bridges with two or more spans, while the latter is required for irregular bridges in seismic performance categories C and D. Detailed seismic analysis is not required for a single-span bridge or for bridges classified as *SPC A*.

Table 3: Response modification factor *R* (AASHTO, 1992)

*Substructure**	*R*	*Connections*	*R*
Wall-type pier**	2	Superstructure to abutment	0.8
Reinforced concrete pile bents		Expansion joints within a span	
(a) Vertical piles only	3	of the superstructure	0.8
(b) One or more batter piles	2		
Single columns	3	Columns, piers or pile bents	
		to cap beam or superstructure***	1.0
Steel or composite steel		Columns or piers to	
and concrete pile bents		foundations***	1.0
(a) Vertical piles only	5		
(b) One or more batter piles	3		
Multiple column bents	5		

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

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

***For bridges classified as *SPC C* and *D*, it is recommended that 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 an *R*-factor of 1.0.

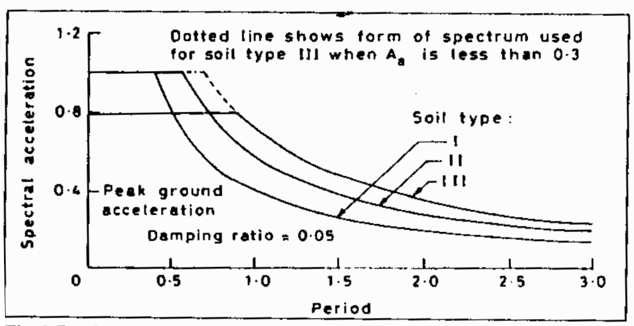

Fig 3 Design spectrum used in Procedure 1 for $A = 0.4$ as per AASHTO 1992

Elastic response coefficient and spectrum

The code provides the maximum elastic force using the standard shapes of the response spectrum for the three types of soil profiles. These shapes basically correspond to 5 percent damping.

Procedure 1: When only one mode of vibration is being considered (procedure 1), the elastic seismic response coefficient C_s is given by:

$$C_s = \frac{1.2AS}{T^{2/3}} \leq 2.5A \tag{5}$$

$\leq 2.0\ A$ when soil type is III and $A \geq 0.30$

The plot of C_s versus fundamental natural period, T of the bridge for A=0.40 is shown in *Fig 3*.

Procedure 2 : When more than one mode of vibration are being considered in analysis (procedure 2), the elastic seismic response coefficient C_{sm} for the m^{th} mode of vibration is given by:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \tag{6}$$

Where T_m is the natural period of the m^{th} mode of vibration. However, the following conditions are applicable:

- The value of C_{sm} need not exceed 2.5 A.
- For soil type III in areas where $A \geq 0.3$, C_{sm} need not exceed 2.0 A.
- When soil type is III, for modes other than the fundamental mode, which have periods less than 0.3 s, C_{sm} may be determined in accordance with the following formula:

$$C_{sm} = A(0.8 + 4.0T_m) \tag{7}$$

- For structures in which any T_m exceeds 4.0 s, the value of C_{sm} for that mode may be determined in accordance with the following formula:

$$C_{sm} = \frac{3AS}{T_m^{4/3}} \tag{8}$$

Fig 4 shows the response spectrum shapes for different soil conditions as per the above.

Combination of orthogonal seismic forces

The elastic seismic forces and moments resulting from analyses in the two perpendicular direction are combined by the "100 percent + 30 percent rule".

Design forces for *SPC A*

The connections of the superstructure to the substructure are to be designed to resist in the restrained directions a horizontal seismic force equal to 0.20 times the dead load reaction force. Thus even though the bridges in *SPC A* do not require detailed seismic analysis, their connections are still to be designed for 0.2 g coefficient.

Table 4: Analysis procedure AASHTO 1992[3]

Seismic performance category	*Regular* bridges with 2 or more spans*	*Irregular** bridges with 2 or more spans*
A	-	-
B	1	1
C	1	2
D	1	2

Note *A "regular" bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). For example, a bridge may be considered regular if it is straight or describes a sector of an arc not exceeding 90° and has adjacent columns, or piers, that do not differ in stiffness by more than 25 percent. (The percentage difference is to be based on the lesser of the two adjacent quantities as the reference.)

**An "irregular" bridge is any bridge that does not satisfy the definition of a regular bridge.

Design forces for *SPC B*

All elements of the bridge, except the foundation, are to be designed for force obtained by dividing the maximum elastic force by the response modification factor for that element. Foundations (except the pile bent) are to be designed for the maximum elastic force divided by half the R factor for the substructure (column or pier) attached to the foundation. This means that design seismic force for the foundation is twice the seismic force for the column/pier that it supports. For pile bents, the foundation is to be designed for the same seismic force as the column or the pier that it is supporting.

Design forces for *SPC C* and *D*

For *SPC C* and *D*, two sets of design forces are specified. The first set is based on the maximum elastic force divided by the response modification factor for the concerned element (except for foundations for which the response modification factor is taken as 1.0 for this calculation; that is, maximum elastic force is taken for the foundation). And, the second set is based on the maximum seismic force that can be developed in the element considering the capacity design principles. The code then specifies either of the two for design of a particular component. Usually, the capacity design forces are lower than the alternative forces, and are recommended for design. These two sets are first described followed by the specifications for different components.

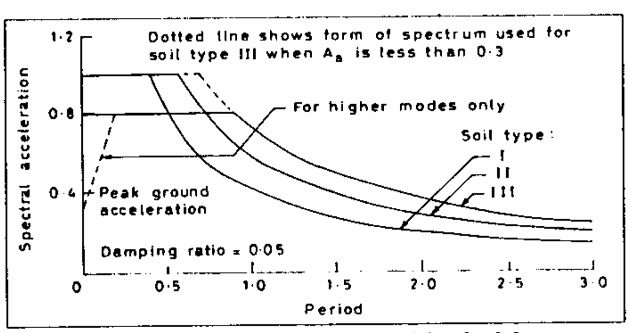

Fig 4 Design spectrum used in Procedure 2 for $A = 0.4$ as per AASHTO 1992

***(i)* Design force set 1**

For all the elements of the bridge, except for the foundation, this force is obtained by dividing the maximum elastic force by the response modification factor for that element. For the foundation, the design force for this set is calculated using R=1, that is, maximum elastic force.

***(ii)* Design force set 2**

The design force for this set is that resulting from plastic hinging at the top and/or bottom of the column (capacity design concept). The code provides detailed procedure for both single column/pier situations and bents with two or more columns.

Single column/pier situations: The capacity design force is to be calculated for the two principal axes of a column and in the weak direction of a pier or bent.

(i) The overstrength plastic moment capacity of the column is determined.

(ii) The shear force in column corresponding to the overstrength plastic moment capacity is calculated.

(iii) The axial force in the columns is the unreduced maximum and minimum seismic axial load plus that due to the dead loads.

Multiple columns/piers situations: For bents with two or more columns, forces are to be calculated both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent, the forces shall be calculated as for single columns discussed above. In The plane of the bent, the forces are to be calculated in the following manner.

(i) Overstrength plastic moment capacity of the columns is determined. This is assessed assuming that the columns are carrying only the dead loads and there is no axial load due to seismic condition.

(ii) The shear force in individual columns of the bent is calculated corresponding to the overstrength plastic moment capacity.

(iii) Column shears calculated above are summed to obtain the maximum shear force that the bent can take.

(iv) The bent shear force calculated above is applied to the top of the bent (that is, at the centre of mass of the superstructure above the bent). For this condition, the axial force in the columns is determined.

(v) Using the above axial force in the columns plus the axial load due to dead loads, a revised overstrength moment capacity of the columns is calculated. Now steps (ii) to (v) are repeated

until the bent shear force value has converged (to say within 10 percent)

(vi) Now the forces in the individual columns in the plane of the bent corresponding to column hinging are:

1. Axial force in the columns is that due to the dead loads plus the converged values under seismic loads as calculated above.
2. Moment in the columns are given by the column overstrength plastic moments corresponding to the axial force in (1) above.
3. Shear in the columns is calculated corresponding to the moment obtained in (2) above.

Column and pile bent design force

(i) **Axial force**: The maximum and minimum design force is either (i) maximum elastic design values plus that due to dead loads, or (ii) value corresponding to plastic hinging calculated in design force set 2 plus that due to dead loads. Lower of these two can be used; the latter values will generally be lower.

(ii) **Moments**: Design moments in columns will be as per design force set 1, that is, maximum elastic moment divided by the response modification factor.

(iii) **Shear**: Design shear will be either (a) maximum elastic shear force (calculated taking response modification factor of 1.0), or (b) that corresponding to plastic hinging of the column as calculated in design force set 2. Lower of the two values can be used in design; usually the latter value will be lower.

Note that the column is being designed for the reduced moment, but the axial and shear forces on the column are being calculated by the capacity design principles. This is because shear failure is to be prevented as it is brittle failure. Similarly, column failure under axial load is brittle, and must be avoided.

Pier design forces

Design forces for the piers will be as per the design force set 1; that is, maximum elastic forces divided by the response modification factor. However, if the pier is being designed as a column in its weak direction, then all design requirements of the columns discussed above will be used for the weak direction.

Connection design forces between superstructure and columns, and between columns and foundations

These will be lower of (i) those as per design force set 1, and (ii) forces developed at the top and bottom of the columns

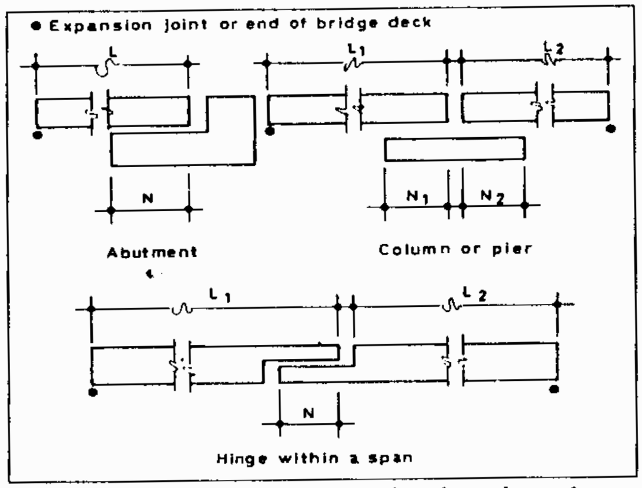

Fig 5 Dimensions for minimum support length requirements (AASHTO, 1992)

due to column hinging as determined in design force set 2. Of course, the forces in (ii) here are to be calculated only after the column design is complete and the overstrength moments are calculated. Here, while forces as per (i) are higher than the maximum elastic force (because the response modification factor is less than 1.0 for the connections), those in (ii) are as per capacity design. Usually those by the capacity design being lower will govern.

Longitudinal linkage forces

Positive linkage is to be provided by means of ties, cables, dampers or an equivalent mechanical means between the adjacent sections of the superstructure at supports and expansion joints within a span. Friction is not to be considered a positive linkage. Where linkage is provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans. Linkages are to be designed for a minimum force of acceleration coefficient *A* times the weight of the lighter of the two adjoining spans or parts of the structure.

Design force for hold down devices

In continuous structures, hold-down devices are to be provided at all supports or hinges if the vertical seismic force due to longitudinal horizontal seismic load opposes and exceeds 50 percent of the dead load reaction. The minimum design force for the hold down device is greater of (i) 10 percent of the dead load reaction that would be exerted if the span were simply supported, and (ii) 1.2 times the net uplift force (that is, vertical upward seismic force minus the dead load reaction)

Foundation design forces

The foundation (including footings, pile caps and piles) are to be designed for the either (i) forces calculated as per design force set 1 (which is equal to the maximum elastic force), or (ii) forces that develop at the base of the column corresponding to column plastic hinging (calculated as per capacity design principle in design force set 2). The lower of the two can be used, usually the latter forces will be lower.

Relative displacements and seating widths

The structural configuration of bridges in the USA fall into two general categories, namely monolithic systems and girder bearing systems. While both systems appear to be equally seismically resistant, engineers accept that monolithic superstructures are preferable to reduce the joint pull apart and subsequent collapse. In girder bearing systems, relative displacements between superstructure and substructure require three aspects to be carefully attended to.

Firstly, under seismic forces acting transverse to the longitudinal girders, uplift forces may be generated at the supports. Special vertical hold down devices are necessary to increase stability of the superstructure against overturning.

Secondly, under seismic forces acting along the longitudinal girders, adjacent superstructure units at supports and at expansion joints within the span may move away from each other by undesirable amounts. Special horizontal linkage elements are necessary to keep these units together.

Thirdly, under seismic forces acting along the longitudinal girders, the relative longitudinal motion between the superstructure and substructure may be larger than the available width of support on top of the substructure (that is, pier or column). At least, a minimum seating width atop the substructure for the superstructure must be ensured in accordance with the actual displacements envisaged during the maximum credible earthquake, and not in accordance with the displacements calculated under the design loads which could be smaller. In girder bearing systems, the need for minimum seating width specifications draws importance from numerous loss-of-span type of failures experienced in the past earthquakes. The code requires that at the expansion ends of the girders, at least a minimum support length (in *mm*) measured from the end of the girder to the face of the pier or abutment, *Fig 5*, shall be provided, given by

$N = 203 + 1.67\ L + 6.66\ H$ for low seismic performance categories A and B

$N = 305 + 2.50\ L + 10.00\ H$ for high seismic performance categories C and D (9)

where L is the total length of superstructure between expansion joints (in *m*) and H is the height of the column or pier (in *m*).

Conclusions

It is necessary to recognise that redundancy is rather low in bridges unlike in buildings. For this reason, the seismic design lateral force is kept higher for bridges than that for buildings. Further, the experiences of bridge failures during earthquakes show that strength criteria alone is insufficient in assuring good seismic performance; deformational aspects are as important as the strength criteria. Since bridges are composed of a set of components that are serially connected to each other with relatively very few support points on the ground, a strength hierarchy is required to be developed in these components based on the concept of capacity design. It is ensured that ductile modes of failure precede the brittle modes of failure. The force transfer from superstructure to substructure through the connections (bearings) are accounted for keeping in mind that bridge components resist the forces during an earthquake through inelastic action, and hence they are designed for only a fraction of the actual forces appearing on them during seismic shaking. However, connections do have their responsibility of transferring the actual forces generated in the superstructure to the substructure. Thus, the connections have to be designed for the actual forces (or more) and not reduced forces. The AASHTO code has very nicely translated the experiences of performance of bridges during past earthquakes into practical design provisions.

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(To be continued)

References

1 ______ IS:1893-1984, *Indian standard criteria for earthquake resistant design of structures*, Bureau of Indian Standards, 1984, New Delhi.

2 ______ IRC:6-1966, *Standard specifications and code of practice for road bridges: Section II loads and stresses*, Indian Roads Congress, 1985, New Delhi

3 MURTY, C.V.R., and JAIN, S.K., Seismic performance of bridges in India during past earthquakes, *Report submitted to Ministry of Surface Transport*, New Delhi, March 1996.

4 MURTY C.V.R., and JAIN, S.K., Seismic performance of bridges in India during past earthquakes, *The Bridge and Structural Engineer*, Journal of ING-IABSE, New Delhi, Vol. 27, no 4, December 1997, pp 45-79.

5 ______ AASHTO, *Standard specifications for seismic design of highway bridges*, American Association of State Highway and Transportation Officials, Washington, D.C., USA, 1992.

6 JAIN, S.K., and MURTY, C.V.R., A state of the art review of seismic design of bridges - Part II: CALTRANS, TNZ and Indian codes, a companion paper under publication in *The Indian Concrete Journal*, 1998.

7 ______ CALTRANS, *Bridge design specifications*, California Department of Transportation, Sacramento, CA, USA., 1991.

8\. ______ TNZ Bridge Manual, *Bridge manual : Design and evaluation*, Transit New Zealand, Wellington, New Zealand, 1991.

9\. JAIN, S.K. and NAVIN, R., Seismic overstrength in reinforced concrete frames, *Journal of Structural Engineering*, ASCE, Vol 121, No 3, 1995, pp 580-585.

10\. PAULAY, T., and PRIESTLEY, M.J.N., *Seismic design of reinforced concrete and masonry buildings*, John Wiley and Sons, New York, 1992.

11\. IMBSEN, R.A., *Seismic design of bridges - history*, unpublished note.

12\. ______ ATC-6, *Seismic design guidelines for highway bridges*, Applied Technology Council, 555 Twin Dolphin Drive, Suit 550, Redwood City, CA 94065-2101, USA, 1981.

13\. ______ ATC-3-06, *Tentative provisions for the development of seismic regulations for buildings*, Applied Technology Council, 555 Twin Dolphin Drive, Suit 550, Redwood City, CA 94065-2101, USA, 1978.

14 ______ IS:456-1978, *Code of practice for plain and reinforced concrete*, Bureau of Indian Standards, New Delhi, 1978.

15 ______ IS:13920-1993, *Indian standard code of practice for ductile detailing of reinforced concrete members subjected to earthquake forces*, Bureau of Indian Standards, New Delhi, 1993.