Seismic Vulnerability of RC Bridge Piers Designed as per Current IRC Codes including Interim IRC:6-2002 Provisions

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Synopsis

The paper presents a review of seismic strength design provisions for reinforced concrete (RC) bridge piers given in Indian codes. In the earlier IRC codes, the seismic design force for bridges was low and the flexibility of the structure was not accounted for in the design force estimate. These deficiencies have been overcome in the Interim IRC:6-2002 provisions. However, the current Indian codes treat RC piers as gravity load carrying compression members, and no provisions are available for their shear design. Analytically obtained monotonic lateral load-displacement relations of RC bridge piers bending in single curvature indicate that the Indian code-designed piers are vulnerable to strong shaking. Also, the longitudinal reinforcement in these bridge piers is also likely to buckle, and the 'nominal' transverse reinforcement requirements of Indian code are shown to be inadequate.

1. Introduction

Bridges are lifeline facilities that must remain functional even after major earthquake shaking; their damage and collapse may not only cause loss of life and property, but also hamper post-earthquake relief and restoration activities. In some major earthquakes in the past, a large number of bridges suffered damages and collapsed due to failure of foundation (structural and geotechnical), substructure, superstructure, and superstructure-substructure and substructure-foundation connections. Bridge foundation is not easily accessible for inspection and retrofitting after an earthquake, and any inelastic action or failure of the superstructure renders the bridge dysfunctional for a long period. Connection failure is generally brittle in nature and hence avoided. Therefore, the substructure is the only component where inelasticity can be allowed to dissipate the input seismic energy and that too in flexural action. In addition, a flexurally damaged pier can be more easily retrofitted.

In an earlier study [1] on strength design of single-column type RC bridge piers, such piers designed as per the earlier IRC codes [2, 3, 4] (namely, IRC:6-2000, IRC:21-1987, and IRC:78-1983) were investigated. The design shear capacities of short piers (of aspect ratio of about 2 to 3) were found to be lower than the corresponding shear demand under flexural overstrength conditions. Further, solid circular piers with single hoops as transverse reinforcement showed the least shear capacity and were found most

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vulnerable, while hollow rectangular piers had relatively higher shear capacity owing to better distributed transverse reinforcement. Also, buckling of longitudinal reinforcement was found to be common in piers resulting in rapid strength loss. Further, increasing the amount of transverse reinforcement, including providing additional radial links in hollow circular piers, was found to enhance the displacement ductility and produce improved post-yield response. This paper conducts a similar investigation on the seismic strength design provisions of the *current* IRC codes, namely the Interim IRC:6-2002 [5], IRC:21–2000 [6] and IRC:78-2000 [7].

2. Performance of Bridges in Past Earthquakes

Poor seismic performance of bridges is recalled from as early as the 1923 Kanto earthquake (M 8.3) in Japan. Masonry piers supporting bridge spans crumbled during the strong shaking. Based on damages to highway bridges sustained during this earthquake, seismic forces were formally recognized in the design of highway bridges in Japan since 1926, and the equivalent static Seismic Coefficient Method was introduced for the analysis of bridge systems subjected to earthquake lateral loads [8].

The 1971 San Fernando earthquake (M 6.6) served as a major turning point in the development of seismic design criteria for bridges in the United States of America. Prior to 1971, specifications for the seismic design of bridges were primarily based on the philosophy of the then existing lateral force requirements for buildings. During this earthquake, piers primarily failed in shear, both outside and within the 'plastic hinge' region, due to insufficient shear strength and lack of adequate confinement from transverse reinforcement, and thereby showed inadequate flexural ductility. Inadequate transverse reinforcement also ed to crushing of concrete in the core of the cross-section on reaching the unconfined concrete strain and to buckling of longitudinal steel, resulting in rapid strength degradation. In addition, transverse reinforcement opened up at lap splicing locations accelerating the failure process. Pullout failure of column reinforcement occurred due to inadequate development length into the footing and straight-bar anchorage detailing. Further, span collapses exposed the inadequate seat width provisions to accommodate the large relative movements at top of piers. Failure of horizontal restrainer bolts across the movement joints also led to collapse of spans. The lessons learnt from this earthquake and the subsequent major earthquakes, coupled with extensive research and design experience, prompted the development of new and refined design specifications for bridges in USA. As a result, today USA has two state-of-the-art documents for seismic design of bridges, namely the *AASHTO LRFD Bridge Design Specifications* [AASHTO, 1998] [9] by the *American Association of State Highway and Transportation Officials* and the *Seismic Design Criteria* [CALTRANS, 2004] [10] by the *California Department of Transportation*.

The 1989 Loma Prieta earthquake (M 7.1) in California caused widespread damage to the region's highways and bridges. The major contributor to the collapse of over a length of a viaduct is generally understood to be due to insufficient anchorage of cap beam reinforcement into the columns, coupled with improperly designed joint shear reinforcement. In addition, inadequate lap-splice lengths of longitudinal bars caused bond failure in columns, and underestimation of seismic displacements resulted in inadequate clearance between structural components causing pounding of structures.

In the 1995 Hyogo-Ken Nanbu (Kobe) earthquake (M 7.8) in Japan, highway structures were severely affected, particularly the single-column-type RC piers [11]. Most concrete piers failed due to insufficient shear strength caused by insufficient transverse reinforcement, inadequate confinement, and large unsupported lengths of longitudinal bars. Premature curtailment of longitudinal reinforcement caused a number of columns to develop flexure-shear failures at mid-height. Superstructures were mostly simply supported over steel pin bearings, and with short seat lengths; dislodging of girders off the bearings was common. Stiff tension-link restrainers failed and unseated a number of spans. At some locations, lateral spreading of weak soil aggravated the relative displacement of piers, again resulting in unseating of spans. Bridges with multiple-column frame type substructures generally performed better than single column type ones. The *Specifications for Highway Bridges and Commentary, Part V: Seismic Design* published in 1990 by *Japan Road Association* was revised in 1996 in view of these extensive damages, and is available as a design standard, the *Design Specifications of Highway Bridges, Part V-Seismic Design* [PWRI 9810, 1998] [12].

Over the past two decades, India has experienced many moderate earthquakes that caused damage to highway and railway bridges [13]. These earthquakes include the 1984 Cachar earthquake (M 5.6), the 1988 Bihar earthquake (M 6.6), the 1991 Uttarkashi earthquake (M 6.6), the 1993 Killari earthquake (M 6.4), the 1997 Jabalpur earthquake (M 6.0), the 1999 Chamoli earthquake (M 6.5) and the recent 2001 Bhuj earthquake (M 7.7) [14]. Also, during 1897 – 1950, India had experienced four great earthquakes

(M > 8), namely the 1897 Assam earthquake (M 8.7), the 1905 Kangra earthquake (M 8.6), the 1934 Bihar-Nepal earthquake (M 8.4) and the 1950 Assam-Tibet earthquake. Today, over 60% of the country lies in the higher three seismic zones III, IV and V (Figure 1). Thus, India has potential for strong seismic shaking, and the large number of existing bridges and those being constructed as a part of the ongoing National Highway Development Project, as per the existing design specifications, will be put to test.

3. Indian Code Provisions

IS:1893 (Part 1)-2002 [15] provides the seismic loading criteria for structures in India. However, loads and stresses (including those due to seismic effects) for the design and construction of road bridges in India are governed by the Indian Road Congress specification IRC:6-2000 [2]. The seismic design criteria in this has been superseded by the interim provisions in IRC:6-2002 [5]. Additional design provisions specifically for concrete structures are specified in Indian Road Congress specification IRC:21-2000 [6] (earlier in IRC:21-1987 [3]) and for bridge foundations and substructures in IRC:78-2000 [7] (earlier in IRC:78-1983[4]).

In IRC:6-2000 [2], the horizontal design earthquake load on bridges is calculated based on a *seismic coefficient*. The equivalent static horizontal seismic load on the bridge is specified (vide Clause 222.5 in IRC:6-2000) as

$$F_{eq} = \alpha\beta\lambda W , \tag{1}$$

where α is horizontal seismic coefficient (Table 1), β is soil-foundation system factor (Table 2), λ is importance factor (1.5 for important bridges, and 1.0 for regular bridges), and W is the seismic weight of the bridge. The seismic weight, acting at the vertical center of mass of the structure, includes the dead load plus fraction of the superimposed load depending on the imposed load intensity; effects of buoyancy or uplift are ignored when seismic effects are considered. From above, the design seismic force comes out to be only 8% of its seismic weight for a normal bridge on hard soil with individual footing in seismic zone V. This was also the level of design force for normal buildings under similar conditions. But, buildings have more redundancy than bridges. Thus, it seems that Indian bridges would be under-designed as per IRC:6-2000. The AASHTO and PWRI specifications set this design force level for bridges at 20-30% of their seismic weight in their most severe seismic zones. In addition, in the IRC:6-2000 design procedure, the flexibility and dynamic behaviour of the bridge were not considered in calculation of design seismic force for bridges. Further, IRC:6-2000 (vide Clause 222.5) recommends horizontal seismic force estimation by dynamic analysis only for bridges of span more than 150 m.

The seismic design philosophy in the Indian codes primarily covers elastic strength design. Thus, the design force is same for all elements of the bridge and does not consider the difference in ductility of the elements. As per IRC:21 [3, 6], RC members are designed by Working Stress Method with a 50% increase in permissible stresses for seismic load combinations (as per IRC:6 [2, 5]). The code prescribes a modular ratio of 10 to be used in design irrespective of the concrete. This causes smaller calculated stresses in concretes of higher grade. The analysis for forces and stresses are based on gross cross-sectional properties of components, although under seismic shaking, section rigidity reduces with increase in cracking resulting in higher deformability. Such increased deformability, especially of the substructures, can also lead to unseating of the superstructure and/or impounding of adjacent structural components as has been observed in a number of past earthquakes. Hence, when the resultant tension at any section due to the combined action of direct compression and bending is greater than a specified permissible tensile stress, IRC:21 recommends cracked section analysis by working stress design with no tension capacity to be done.

In Clause 304.7 of IRC:21-1987, the general provisions for shear design of RC beams are stated. The code attributes the design shear wholly to the transverse reinforcement. Only, the average shear stress calculated is checked against a maximum permissible shear stress that is a function of the grade of concrete and subject to a maximum value of 2.5 MPa. In the 2000 version of IRC:21 [6], unlike in the 1987 version, contributions of both concrete and shear reinforcement are acknowledged. This is a forward step following the worldwide research on shear strength of reinforced concrete (for example, refer [16]).

However, in IRC:21 [3, 6], the design provisions for columns and compression members (vide Clause 306) do not include shear design even under lateral loading conditions such as during earthquakes. However, detailing provisions are included for transverse reinforcement (vide Clause 306.3). The minimum diameter of transverse reinforcement (*i.e.*, lateral ties, circular rings or helical reinforcement) is required to be the larger of one-quarter of the maximum diameter of longitudinal reinforcement, and 8 mm. The maximum centre-to-centre spacing of such transverse reinforcement along the member length is required to be the lesser of (a) least lateral dimension of the compression member, (b) 12 times the diameter of the smallest longitudinal reinforcement bar in the compression member, and (c) 300 mm. Further, there are no provisions on the need for confinement of concrete in vertical members. Also, possible buckling of longitudinal reinforcement is not considered. The incomplete treatment of shear design and of transverse reinforcement questions on the performance of such Indian bridge piers under the expected strong seismic shaking.

IRC:78 [4, 7] specifies an additional requirement for transverse reinforcement in walls of hollow RC piers. The minimum area of such reinforcement (vide Clause 713.2.4) is given as 0.3% of the sectional area of the wall. Such reinforcement is to be distributed on both faces of the wall: 60% on the outer face and the remaining 40% on the inner face. Again, here also, there are no provisions on additional intermediate ties or links to hold together the transverse hoops on the outer and inner faces of the hollow RC pier.

In IRC:21-2000, the minimum and maximum areas of longitudinal reinforcement for *short* columns are specified to be 0.8% and 8%, respectively, of the gross cross-sectional area of the member. IRC:21 requires that every corner and alternate longitudinal bar have lateral support provided by the corner of a tie having an included angle of not more than 135°, and that no longitudinal bar be farther than 150 mm clear on each side along the tie of a laterally supported bar. When the bars are located on the periphery of a circle, a complete circular tie is to be used. No other special seismic design aspects are addressed. Thus, the Indian codes advocate only flexural strength design; ductility design is not addressed at all; it is not ensured that the shear capacity of the pier section exceeds the shear demand when plastic moment hinges are generated during strong shaking.

3.1 Interim IRC:6-2002 Provisions

After the devastating 2001 Bhuj earthquake, one of the important changes was the revision of the seismic zone map of the country. The country is now classified into four seismic zones (Figure 1). In this, the old Zone I is merged with Zone II with significant changes in the peninsular region; some parts in Zones I and II are now in Zone III. Further, the Indian Road Congress came up with interim measures [5] to be read with the revised zone map (Clause 222.2). As per Clause 222.1 of this interim provision, now all bridges in Zones IV and V are required to be designed for seismic effects, unlike in IRC:6-2000 wherein only in Zone V, all bridges were required to be designed for seismic effects. Clause 222.3 of the interim provisions makes it mandatory to consider the simultaneous action of vertical and horizontal seismic forces for all structures in Zones IV and V. Clause 222.5 of this interim provision recognizes that for bridges having spans more than 150 m, the seismic forces are to be determined based on site-specific seismic design criteria.

One of the most important and welcome changes enforced through the 2002 interim provisions is with regard to the procedure for seismic force estimation. Now, the design horizontal seismic force F_{eq} of a bridge is dependent on its flexibility, and is given as

$$F_{eq} = A_h W \,, \tag{2}$$

where the design horizontal seismic coefficient A_h is given by

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)} . \tag{3}$$

In Eq.(3), Z is the zone factor (Table 3), I is the importance factor (same as in IRC:6-2000), R is response reduction factor taken to be 2.5, and S_a/g is the average response acceleration coefficient for 5% damping depending upon the fundamental natural period T of the bridge (Table 4). The S_a/g value depends on the type of soil (namely rocky or hard soil, medium soil and soft soil) and the natural period T of the structure. Appendix A of the interim provisions gives a rational method of calculating the fundamental natural period of pier/abutment of bridges. But, the interim provisions recommend a single value of 2.5 for the response reduction factor R. This factor is to be used for all components of the bridge structure. However, the bearings do not have redundancy in them and are expected to behave elastically under strong seismic shaking. Therefore, designing the bearings for a much lower seismic force than that it should carry from superstructure to piers is not desirable. In advanced seismic codes, the R factor for design of connections is generally recommended to be 1.0 or less [17, 18]. This interim provision needs to be revised immediately from the point of view of safety of bridge bearings.

With the enforcement of the interim provisions, the prescribed seismic hazard of structures in the country has changed significantly. As an example, consider a single span RC National Highway bridge (importance factor $I = 1.5$) on Type II (medium) soil with well foundation (β =1.2 as per IRC:6-2000). For single pier bridge vibration unit (BVU), for most of the normal construction practice in India, piers tend to be slender in the direction of traffic or the longitudinal direction (L), and stiffer in the direction (T) transverse to that of the traffic (Figure 2). Thus, in general, the natural period of piers is different in the longitudinal and transverse directions. Foe example, the single pier BVU under consideration has natural period of 1.5 sec in the longitudinal direction and 0.3 sec in the transverse direction. Thus, as per the interim provisions, the S_a/g values for the longitudinal and transverse directions are 0.91 and 2.5, respectively. The design seismic coefficient for the bridge in different seismic zones in the country calculated as per the IRC:6-2000 and the Interim IRC:6-2002 provisions are as given in Table 5. In general, the design lateral force on piers in their transverse direction as per the Interim provisions is about twice those as per IRC:6-2000.

Now, consider bridges in the two metropolitan cities, namely Delhi and Madras. Delhi is in Zone IV in both the old and the new zone maps of India, while Madras, originally in Zone II, is now placed in Zone III. Thus, the design seismic coefficient for the single pier BVU in Delhi changes from 0.090 to 0.066 (L) and 0.180 (T), *i.e.*, the seismic force increases by 100% in the transverse direction for such a pier. For bridges in Madras, the design seismic coefficient for the single pier BVU changes from 0.036 to 0.044 (L) and 0.120 (T). Here, the seismic force increases by about 22% and 233% in the longitudinal and transverse directions, respectively. Hence, bridges in Madras become deficient as per the Interim provisions.

In addition, there are special mandatory and recommended measures in the 2002 Interim provisions for better seismic performance of bridges. These include ductile detailing, dislodgement prevention units, and isolation units. However, these are beyond the scope of this paper and hence not discussed.

4. Capacity Design for Bridge Piers

The capacity design philosophy warrants that desirable ductile modes of damage (*e.g.*, ductile under-reinforced flexural damages) precede undesirable brittle ones (*e.g.*, brittle shear failure and bond failure). Under strong shaking, inelasticity in bridges is admissible only in the piers. Further, for strong seismic shaking, since it may not be economically viable to design a structure for elastic response, this inelasticity is deliberately introduced in piers but with adequate ductility. This inelastic action under displacement loading caused by the earthquake in RC piers is associated with large overstrength. Under these overstrength conditions, if the shear demand on the pier exceeds its design shear capacity, undesirable brittle failure for the whole structure may result. Thus, if capacity design of bridge piers is conducted, the piers are designed for shear corresponding to the overstrength flexural capacity of the pier.

In the capacity design of piers, the important items that come into play are design transverse reinforcement, concrete confinement by transverse reinforcement, shear strength of confined concrete, and stability (buckling) of longitudinal reinforcement. In countries like Japan, New Zealand and USA, the design of the bridge pier for seismic conditions is a paramount step in the entire process of bridge design practice. The American highway specifications (AASHTO), California Transportation Department specification (CALTRANS), and New Zealand Standard specifications (NZS) recommend c*apacity design* for shear design of bridge piers [9, 10, 19]. The Japanese specifications (PWRI) [12] explicitly identify piers satisfying Eq.(4) (with $\phi = 1$) as one of "flexural failure type", *i.e.*, they will not fail in brittle shear.

In the capacity design approach, the following procedure is adopted in the above mentioned international codes, in general. First, through an elastic analysis under the specified loads, the bending moments and axial loads at all critical sections are determined, and the members designed for the combined effects of axial load and bending moment. Second, the potential plastic hinge locations and the preferred collapse mechanism are identified. The overstrength flexural capacities of the plastic hinges are determined based on the actual reinforcement provided and the properties of actual material used. This is often done by a moment-curvature analysis considering the "cracked" [10] cross-sectional properties of the member. Third, the structure is re-analysed assuming all potential plastic hinges to have developed their overstrength flexural capacities. The associated axial load, shear force and bending moment in all structural components other than those with the plastic hinges are determined; these members are designed for these forces. The members with plastic hinges (piers) are designed for the shear V_o corresponding to the state when flexural hinges are formed, such that

$$\phi V_n \geq V_o , \tag{4}$$

where

$$V_n = V_c + V_s . \tag{5}$$

In Eqs.(4) and (5), V_n is the nominal shear capacity (calculated using the nominal specified material strengths), V_o is the flexural overstrength-based seismic shear demand (calculated using actual material properties [10] or by multiplying the nominal shear capacity by an overstrength multiplier Ω [9]), ϕ is a resistance factor (less than unity), and V_c and V_s are shear strengths offered by concrete and reinforcing steel respectively. It is clear that this capacity design approach for shear design of substructures may not be possible for substructures of the wall-type; it is not possible to generate the flexural hinge even under the extreme seismic shaking. Detailed studies are required to address the seismic design of wall-type substructures.

The flexural overstrength of the structure, which in turn results in higher flexural overstrength-based shear *demand*, should be based on realistic properties. The flexural overstrength is caused due to many factors. One of them is due to the materials used in construction having strengths higher than the nominal strengths employed in design. For instance, the actual tensile yield strength of steel is higher than its *characteristic* yield strength used in design f_y, and the actual compressive strength of concrete is higher than the *characteristic* compressive strength f_{ck}. Hence, the most likely material properties/strengths have to be used as such while estimating the flexural overstrength-based demands on concrete components resisting seismic effects [10], *i.e.,* without using any factors of safety or partial safety factors on actual values. On the other hand, seismic shear *capacity* is to be conservatively determined based on the nominal material strengths only [10], *i.e.*, by employing strengths smaller than the actual values.

In resisting shear, concrete carries significant part of the total shear force, particularly in large concrete cross-sections and those carrying vertical compressive loads, such as those of bridge piers. In general, the shear force capacity V_c offered by a concrete section depends on the shear strength of both concrete and longitudinal steel; shear strength improves with concrete grade and amount of tension steel (though through dowel action). The shear strength of concrete itself depends on the level of confinement provided by transverse reinforcement, and on the imposed curvature; it increases with increase in volumetric ratio of transverse steel and with decrease in curvature [20]. Also, the average concrete shear strength in plastic hinge region decreases with increase in number of loading cycles and with increase in effective depth of the section. These are considered in the PWRI specifications in calculating the shear capacity of RC sections [12].

In RC structures, the actual constitutive stress-strain relations of concrete and steel significantly affect the seismic response of the structure. Transverse reinforcement causes a confining pressure on concrete resulting in an enhancement of its strength and strain capacities [21, 22, 23]; this, in turn, causes an increase in the load carrying capacity of member. In capacity design, since the maximum flexural overstrength-based shear demand decides the ductile response of the structure, the actual constitutive relations of cover and core concretes must be used considering the confinement action of transverse reinforcement in the analysis [10, 12]. Under confinement, the maximum strain in concrete may be as high as 15 to 20 times the maximum strain of 0.0035 normally used in design, and the peak compressive strength may be 4 times the 28-day characteristic compressive strength f_{ck} (Figure 3 [24]).

Transverse reinforcement in RC piers serves a three-fold purpose, namely for (a) providing shear strength, (b) confining the core concrete and thereby enhancing its strength and deformation characteristics, and (c) controlling the stability of the longitudinal reinforcement bars. The first two functions have been discussed earlier. Regarding the third one, literature reports that inelastic buckling of longitudinal reinforcement in compression can be prevented by limiting the maximum spacing of transverse reinforcement bars to within six times the nominal diameter of longitudinal reinforcement [25, 26, 21]. This limit is generally recommended within the potential plastic hinge region (Table 6). However, the limit is relaxed outside the potential plastic hinge region, only if design calculations are made in line with design lateral force obtained as per Eq.(4).

Also, different codes prescribe minimum amount of transverse reinforcement in plastic hinge zones. For example, for circular piers, the American highway specifications (AASHTO [9]) recommend that volumetric ratio of spiral reinforcement be at least the greater of

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\left(\frac{f_c^{'}}{f_{yt}}\right) \text{ and} \tag{6}$$

$$\rho_s = 0.12\left(\frac{f_c^{'}}{f_{yt}}\right). \tag{7}$$

Likewise, the AASHTO recommendation for non-circular hoop or tie reinforcement is that the total effective area in each principal direction within spacing s in piers is to be at least the greater of

$$A_{sh} = 0.30sD'\left(\frac{A_g}{A_c} - 1\right)\left(\frac{f_c'}{f_{yt}}\right) \text{ and} \tag{8}$$

$$A_{sh} = 0.12sD'\left(\frac{f_c'}{f_{yt}}\right). \tag{9}$$

The NZS [19] specifications recommend that in potential plastic hinge region of circular piers, the volumetric ratio of spiral reinforcement be at least the greater of

$$\rho_s = \frac{(1.3 - \rho_l m)}{2.4}\left(\frac{A_g}{A_c}\right)\left(\frac{f_c'}{f_{yt}}\right)\left(\frac{P^*}{\phi f_c' A_g}\right) - 0.0084 \text{ and} \tag{10}$$

$$\rho_s = \frac{A_{sl}}{110D'}\left(\frac{f_{yl}}{f_{yt}}\right)\left(\frac{1}{d_b}\right); \tag{11}$$

and for non-circular hoop or tie reinforcement, the area A_{sh} in each principal direction within spacing s be greater than

$$A_{sh} = \frac{(1 - \rho_l m)sD'}{3.3}\left(\frac{A_g}{A_c}\right)\left(\frac{f_c'}{f_{yt}}\right)\left(\frac{N^*}{\phi f_c' A_g}\right) - 0.0065sD'. \tag{12}$$

A detailed discussion on the international practice of seismic design of bridges and RC bridge piers is available elsewhere [27, 28, 29].

5. Pushover Analysis

The review of the Indian code provisions for RC pier design in light of the international seismic design practices, and importance of employing the capacity design concept in bridge design necessitates checking the seismic safety of piers designed as per the existing Indian standards. The lateral strength and deformation characteristics of such piers can be determined by conducting, monotonic displacement-controlled experiments on prototype or model specimens. However, in India, the infrastructure required to perform experimental studies is still limited and expensive. Thus, an analytical tool providing sufficient data regarding the pier response is required not only for checking the performance of the designed piers, but also for development of improved design standards; "pushover analysis" is one such tool. Thus, a *displacement*-based pushover scheme is developed that would provide sufficient insight into the *full* response, *i.e.*, till failure, of the most commonly used piers, the single column piers bending in single curvature.

5.1 Geometric Model

Most analytical studies on RC bridge piers, including those with large cross-sections, still idealise the member by its centroidal axis and define the inelastic action of the whole cross-section in a lumped sense. This does not accurately model the spread of inelasticity both along the member length and across the cross-section. Hence, a distributed plasticity model is required, which is described below.

5.1.1 Model Description

In the present analytical model, the pier is discretised into a number of segments along the length, and each segment into a number of fibres across the cross-section (Figure 4). As an RC section is composed of both concrete (of two types, namely the *confined* and *unconfined*) and longitudinal reinforcing steel, the section is further discretised into separate concrete and steel *fibres* (Figure 5). Such a general approach of discretising RC sections into a number of discrete fibres was long adopted to accommodate general geometric irregularities and geometric and material non-linearities, and to capture the complex stress distribution across the cross-section under any loading condition [30, 31]. Also, procedures for obtaining tangent stiffness matrix of a segment discretised into such discrete fibres was presented earlier [32, 31]. For analysis involving material and geometric nonlinearity, incremental equilibrium equations between incremental stress resultants and incremental deformations, *i.e.*, the *incremental* or *tangent load-deformation relations*, are derived for all the fibres. These incremental equations are combined to form the incremental equilibrium equation of a segment. Finally, the incremental equilibrium equation of the entire pier is obtained by assembling those of its segments. *Large displacements* and *small strains* are considered in the analysis.

Each fibre is treated as a two-nodded axial member with no flexural property. Thus, for a segment of length *L*, made of material of Young's modulus *E* and shear modulus *G*, inclined at an angle α to the global axes (Figure 6), the segment tangent stiffness matrix relating the nodal force increments to the nodal displacement increments in global coordinates is given by

$$[K]_t^s = {}^{ab}[K]_t + {}^{sh}[K]_t \,, \qquad (13)$$

where

$$ {}^{ab}[K]_t = \sum \begin{bmatrix} [\Lambda] & -[\Lambda] \\ -[\Lambda] & [\Lambda] \end{bmatrix}, \text{ and} \qquad (14)$$

$$
{}^{sh}[K]_t = GA_s\left(\frac{\beta}{1+\beta}\right)\begin{bmatrix} \frac{b^2}{L} & -\frac{ab}{L} & \frac{b}{2} & -\frac{b^2}{L} & \frac{ab}{L} & \frac{b}{2} \\ & \frac{a^2}{L} & -\frac{a}{2} & \frac{ab}{L} & -\frac{a^2}{L} & -\frac{a}{2} \\ & & \frac{L}{4} & -\frac{b}{2} & \frac{a}{2} & \frac{L}{4} \\ & & & \frac{b^2}{L} & -\frac{ab}{L} & -\frac{b}{2} \\ & & & & \frac{a^2}{L} & \frac{a}{2} \\ Sym. & & & & & \frac{L}{4} \end{bmatrix}, \tag{15}
$$

in which

$$
[\Lambda] = \frac{E_t A}{L}\begin{bmatrix} a^2 & ab & ay \\ ab & b^2 & by \\ ay & by & y^2 \end{bmatrix} + \frac{P}{L}\begin{bmatrix} b^2 & -ab & 0 \\ -ab & a^2 & 0 \\ 0 & 0 & 0 \end{bmatrix}; \tag{16}
$$

$$
a = \cos\alpha \text{ and } b = \sin\alpha. \tag{17}
$$

In Eq.(9), y, A and L are the distance of each fibre (concrete or steel) center from the gross cross-section centroidal axis of the section, its cross-sectional area and its length (equal to the segment length), respectively. P is the axial load (positive for tensile load) and E_t is the tangent modulus of elasticity of the material at the prevailing strain level. In Eq.(6), the total stiffness of a segment, modeled as a general frame member, comprises two sets of actions, namely the combined axial and bending action ${}^{ab}[K]_t$, and the shear action ${}^{sh}[K]_t$. The suffix '*t*' represents the tangent modulus at a given strain level. Here, the shear response was assumed to be uncoupled from the axial load and bending effects, and hence, the linear superposition was conducted even under non-linear and inelastic conditions. In bridge piers of large cross-sections, shear deformation contributes significantly to the overall deformation response of the pier. Hence, it is important to include the same. The stiffness matrix derived is applicable for a general frame member that may be a slender one with predominant flexural behaviour, or a stocky one with significant shearing behaviour. The factor β, which is the relative ratio of flexural lateral translational stiffness and shear stiffness of the segment [33], *i.e.*,

$$
\beta = \frac{12EI/L^3}{GA_s/L}. \tag{18}
$$

helps achieve this.

The complete incremental equilibrium equation of a segment in global coordinates (Figure 6) is given by

$$[K]_t^s \{\dot{d}\} = \{\dot{f}\}, \tag{19}$$

$$\{\dot{d}\} = \langle \dot{d}_1 \quad \dot{d}_2 \quad \dot{d}_3 \quad \dot{d}_4 \quad \dot{d}_5 \quad \dot{d}_6 \rangle^T \text{, and} \tag{20a}$$

$$\{\dot{f}\} = \langle \dot{f}_1 \quad \dot{f}_2 \quad \dot{f}_3 \quad \dot{f}_4 \quad \dot{f}_5 \quad \dot{f}_6 \rangle^T ; \tag{13b}$$

where $\{\dot{d}\}$ and $\{\dot{f}\}$ are the incremental segment end-displacement and end-force vectors. The incremental equilibrium matrix equation of the whole pier is formed by assembling those of all its segments. Symbolically, if $[K]_t$ is the complete global tangent stiffness matrix of the pier, $[K]_t^s$ is the global tangent stiffness matrix of the segment s from Eq.(6), then

$$[K]_t = \sum_{s=1}^{N_s} \{[K]_t^s\}, \tag{21}$$

where $\sum$ is the assembly operator and N_s is the number of segments in the member.

5.2 Material Models

The load-deformation relationship of each fibre is derived using material constitutive laws. In RC structures, the two different materials, namely reinforcing steel and concrete, require two different material constitutive law models. Moreover, core concrete fibres are *confined* and the cover concrete *unconfined*. Also, the *longitudinal* and *transverse* steels can be of different grades and amounts. Transverse steel affects the confinement of the core concrete and influences the axial stress-strain relation of the core concrete. On the other hand, longitudinal steel plays a direct role in the axial, bending and shear resistance of the section. In India, the most widely used reinforcing steel, both for longitudinal and transverse steel, is of HYSD steel conforming to IS: 1786-1985 [34]. A model representing the virgin stress-strain curve for HYSD bars, developed through regression analysis of experimental data from uniaxial tensile tests is used [35].

Brief descriptions of some of the constitutive law models of concrete available in literature are discussed elsewhere [35]. Of the different constitutive models available, the analytical model that is applicable to hollow sections also is used in this study [35]; this model is an extension to an earlier model [36]. Hollow sections address a new situation, wherein the outer and inner hoops are tied by links leading to two distinctly different confining actions, namely hoop action and the direct action of links. The falling branch as defined by the original single equation [37, 36] is too flat and is seen to be above the experimental uniaxial stress-strain data. Hence, the equation is modified [33] in the strain range beyond the strain ε_1 corresponding to the peak stress as

$$f_c = \frac{f'_{cc} x r_o}{r_o - 1 + x^{r_o}}\, ;\ \varepsilon_c > \varepsilon_1 \tag{22}$$

where

$$r_o = r^{(1+1/r)}, \tag{23}$$

$$r = \frac{E_c}{E_c - E_{sec}},\ E_{sec} = \frac{f'_{cc}}{\varepsilon_1},\ E_c = 3320\sqrt{f'_c} + 6900, \text{ and} \tag{24}$$

$$x = \frac{\varepsilon_c}{\varepsilon_1}. \tag{25}$$

In Eqs.(15) to (18), the unit of both the compressive stress *f* and the modulus *E* is MPa.

During pushover analysis of the pier, initially all the fibres are in compression under the action of gravity load. As the pier tip is displaced horizontally, the curvature at any section is gradually increased; the compressive strain in some fibres increases, while in others, it decreases and eventually becomes tensile (unloading in compression and subsequent loading in tension). At a certain curvature, spalling of cover concrete occurs, which results in redistribution of stresses within the section. There is possibility of unloading and reloading of both concrete and steel fibres. However, for the purpose of a monotonic pushover analysis, exhaustive hysteretic models for material stress-strain curves may not be required; simple loading, unloading and reloading rules are therefore used. The following are the salient features of the hysteretic stress-strain model of steel used in this study:

a) All unloading and initial reloading slopes, upto yield, are equal to the initial elastic modulus E_s; there is no stiffness degradation.
b) There is no strength deterioration.
c) As the material unloads from the virgin curve, the whole stress-strain curve translates along the strain axis with the total translation being dependent on the plastic strain history; a kinematic hardening approach is utilized, wherein the stress-strain path translates with accumulation of plastic strain, but without any change of size or shape (as a consequence of (a) and (b)).

Likewise, a simple hysteretic stress-strain model of concrete is used in this study.

The salient features of this model are:

a) Linear unloading and reloading occur with tangent modulus equal to the initial modulus.
b) The residual strain capacity is calculated from the accumulated plastic strain.
c) The tensile strength of concrete is neglected.

The load-carrying capacity of compression reinforcement in RC compression members is significantly affected by the unsupported length of the longitudinal bars between the transverse ties that are expected to provide lateral support and thereby prevent buckling of longitudinal bars. In the present study, longitudinal bars are considered to have buckled if the axial compressive stress in them exceeds the critical stress $\sigma_{cr,b}$, given by [33]

$$\sigma_{cr,b} = Min\left[\sigma^1_{cr,b}\,;\sigma^2_{cr,b}\right]\ . \tag{26}$$

In Eq.(19), $\sigma^1_{cr,b}$ is the critical elastic buckling stress of the longitudinal bar under clamped-clamped condition between the transverse ties, given by

$$\sigma^1_{cr,b} = \frac{\pi^2 E_s}{4\left(s / d_b\right)^2}\ . \tag{27}$$

Further, $\sigma^2_{cr,b}$ is the inelastic critical buckling stress, given by

$$\sigma^2_{cr,b} = \begin{cases} f_u & \text{for} \quad \left(\dfrac{s}{d_b}\right) < 5 \\ f_y + \dfrac{(f_u - f_y)}{5}\left(\dfrac{s}{d_b} - 5\right) & \text{for} \quad 5 < \left(\dfrac{s}{d_b}\right) < 10\ . \\ f_y & \text{for} \quad \left(\dfrac{s}{d_b}\right) > 10 \end{cases} \tag{28}$$

6. Displacement Based Pushover Analysis Procedure

An analytical procedure is developed to assess the inelastic drift capacity of cantilever (circular and square, solid and hollow) RC piers bending in single curvature. The pier is subjected to a monotonically increasing *displacement* (in increments) at its tip in one transverse direction until its final collapse. The force required to sustain the specified displacement is calculated considering the strength of the material, the deformation of the pier and the progression of internal cracking. From this, the overstrength shear demand, drift capacity and displacement-ductility of the RC cantilever pier bending in single curvature, are extracted. Thus, the full lateral load-deformation response is traced.

6.1 Algorithm

To begin with, the gravity load is applied at the top of the pier and the strains and stresses in all fibres of all segments are obtained; the stresses developed in the cross-section are ensured to be in equilibrium with the external gravity load. Then, a *small* displacement increment is imposed at the tip of the cantilever pier. Corresponding to this tip displacement, an initial deformed profile is assumed. Usually, the deformed shape of an elastic cantilever with only bending deformations considered under the action of a concentrated load at the tip, is a good first approximation. Thus, the initial lateral transverse displacement $x(z)$ and rotation $\theta(z)$ at a distance z from the bottom support, for a first displacement increment Δ_o at the tip of the cantilever of height (length) h (Figure 7) are given by

$$x(z) = \frac{z^2}{2h^3}(3h - z)\Delta_o \text{, and} \tag{29a}$$

$$\theta(z) = \frac{3z}{2h^3}(2h - z)\Delta_o . \tag{22b}$$

The change in length of the cantilever is considered while estimating the internal resistance of the pier. For this assumed displacement profile along the height h of the pier, the internal resistance vector along the degrees of freedom $\{p\}$ is calculated (as discussed later). The external load vector $\{f\}$ consists of vertical concentrated load at the top of the pier from the gravity load of the superstructure and vertical dead loads at all intermediate nodes from the dead load of the pier segments.

Pushover analysis involves iterative computations due to the nonlinearities in the constitutive relations of the materials and due to geometric effects. Modified Newton-Raphson Method is used for the iterations. Thus, at the global iteration level, at a general displacement step r and iteration level k, the force unbalance $\{f_u\} - {}^r\{p_u\}_k$ till iteration (k-1) is computed. From this, the incremental deformation vector along the unknown displacement directions ${}^r\{\dot{x}_u\}_k$ is obtained from

$${}^{r-1}[K_{uu}]\,{}^r\{\dot{x}_u\}_k = \{f_u\} - {}^r\{p_u\}_k , \tag{30}$$

where ${}^{r-1}[K_{uu}]$ is the iterating matrix corresponding to the unknown degrees of freedom extracted from the partitioned tangent stiffness matrix $[K]_t$ of the pier, obtained from Eq.(14), based on the cracked section properties at the end of the last displacement step (*r-1*). The net incremental deformation vector ${}^{r}\{\dot{x}\}_{k}^{net}$ in the displacement step *r* and up to iteration *k* is then obtained as

$$ {}^{r}\{\dot{x}\}_{k}^{net} = {}^{r}\{\dot{x}\}_{k-1}^{net} + {}^{r}\{\dot{x}\}_{k}\,, \tag{31} $$

where ${}^{r}\{\dot{x}\}_{k}$ is the incremental deformation vector along all, known and unknown degrees of freedom. ${}^{r}\{\dot{x}\}_{k}^{net}$ is then decomposed to form the global incremental end-deformation vector ${}^{r}\{\dot{d}\}^{s}$ for each segment *s*. Hence, if $\prod^{s}$ is the decomposition operator that depends on the connectivity array of the degrees of freedom at the ends of the segment *s*, then

$$ {}^{r}\{\dot{d}\}^{s} = \prod^{s} {}^{r}\{\dot{x}\}_{k}^{net}\,. \tag{32} $$

Based on the new deformation profile updated using the node coordinates at the end of the displacement step *r*, the net global incremental end-deformations and the coordinate transformation $[T]$ (as in Eq.(34)) are updated. The net incremental deformation vector ${}^{r}\{\dot{u}\}^{s}$ in local coordinate (Figure 8) for each segment *s*, is obtained as

$$ {}^{r}\{\dot{u}\}^{s} = [T]\ {}^{r}\{\dot{d}\}^{s}, \tag{33} $$

where

$$ {}^{r}\{\dot{u}\}^{s} = \{\dot{u}\} = \langle \dot{u}_1 \quad \dot{u}_2 \quad \dot{u}_3 \quad \dot{u}_4 \quad \dot{u}_5 \quad \dot{u}_6 \rangle^{T}\,. \tag{34} $$

Using this, the net incremental axial strain $\dot{\varepsilon}^{f}$ in fibre *f* at a normal distance *y* from the centroidal axis of the gross cross-section of the segment before deforming, is calculated as

$$ \dot{\varepsilon}^{f} = \frac{(\dot{u}_1 - \dot{u}_4) + (\dot{u}_3 - \dot{u}_6)y}{L}\,. \tag{35} $$

Given the state of the fibre at the end of the previous displacement step (*r-1*) and the net incremental axial strain $\dot{\varepsilon}^{f}$, the new stress state σ^{f} of the fibre is obtained using the cyclic constitutive laws of steel and concrete described previously. From the stresses of all the fibres in the cross-section of the segment, the total internal resistance, namely the axial resistance P_c and the bending moment M_c resisted by the section (segment *s*), are calculated from

$$ P_c^{s} = \sum_{i=1}^{N_{fc}} \sigma_i^{c} A_i^{c} + \sum_{j=1}^{N_{fs}} \sigma_j^{s} A_j^{s}\text{ , and} \tag{36} $$

$$M_c^s = \sum_{i=1}^{N_{fc}} \sigma_i^c A_i^c y_i + \sum_{j=1}^{N_{fs}} \sigma_j^s A_j^s y_j, \tag{37}$$

and the total shear ${}^r V_c^s$ resisted by the segment *s* in the displacement step *r* from

$${}^r V_c^s = {}^{(r-1)} V_c^s + GA_s \left(\frac{\beta}{1+\beta}\right) \left\{ \frac{(\dot{u}_2 - \dot{u}_5)}{L} - \frac{(\dot{u}_3 + \dot{u}_6)}{2} \right\}, \tag{38}$$

where ${}^{(r-1)} V_c^s$ is the segment shear force at the end of the previous displacement step. Thus, the components of the end-force vector ${}^r \{r\}^s$ in local coordinate for the segment *s* are obtained as

$$r_1 = P_c^s \tag{39a}$$

$$r_2 = {}^r V_c^s \tag{32b}$$

$$r_3 = M_c^s - \frac{{}^r V_c^s L}{2} \tag{32c}$$

$$r_4 = -P_c^s \tag{32d}$$

$$r_5 = -{}^r V_c^s \tag{32e}$$

$$r_6 = -M_c^s - \frac{{}^r V_c^s L}{2} \tag{32f}$$

Using this, the segment end-force vector in global coordinate, $\{p\}^s$ is computed as

$${}^r \{p\}^s = [T]^T \, {}^r \{r\}^s, \tag{40}$$

where

$$[T] = \begin{bmatrix} a & b & 0 & 0 & 0 & 0 \\ -b & a & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & a & b & 0 \\ 0 & 0 & 0 & -b & a & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}. \tag{41}$$

The assembly of these $\{p\}^s$ vectors of each segment results in the updated complete member residual force vector $\{p\}$. Collecting the forces along the unknown degrees of freedom $\{p_u\}$, the residual force vector $\{r_s\}$ is then computed as

$$\{r_s\} = \{f_u\} - \{p_u\}. \tag{42}$$

The above procedure is reiterated until the residue $\{r_s\}$ is within specified tolerance. Upon convergence, the global coordinates of the nodes and the segment end forces are updated. The target deformed geometry for the next displacement step (r+1) is computed based on the next lateral increment at the tip of the cantilever pier (Figure 9). The above internal resistance calculation procedure is repeated with additional displacement increments until the pier reaches failure. Thus, the full lateral load-lateral displacement response is traced. From this, the flexural overstrength-based shear demand $V_{\Omega}^{\max}$ on the RC pier bending in single curvature is extracted as

$$V_{\Omega}^{\max} = H_{\max}, \tag{43}$$

where $H_{\max}$ is the maximum internal resistance of the pier at its tip during the entire displacement loading history (Figure 10).

7. Numerical Study

The adequacy of strength design provisions as per Interim IRC:6-2002 is investigated for most commonly used solid and hollow RC piers of circular and rectangular cross-sections. Piers of typical 5 m height are designed as per the strength design methodology outlined in IRC:21-2000. The approximate initial choice of section size (cross-sectional area) and probable load on the piers are taken from field data of existing bridge piers. In this study, a 2-lane superstructure is considered. The weight of the superstructure is taken as 162.5 kN/m. Hence, for a span of 40 m, the piers are subjected to a superstructure gravity load of 6500 kN. The lateral and vertical seismic loads on the piers are calculated as outlined in IRC:6-2002 for seismic zone V, with importance coefficient of 1.5 on rocky or hard soil sites.

The nomenclature used to designate bridge piers studied is described as follows. The first character (*i.e.*, 'C' or 'R') indicates piers of *circular* or *rectangular* cross-section. The second character (*i.e.*, 'S' or 'H') indicates *solid* or *hollow* sections. The third character (*i.e.*, 'W' or 'S') indicates piers *without* and *with* shear design. The fourth character (*i.e.*, 'G', 'L' or 'P') indicates type of investigation undertaken on the piers, namely effect of *geometry, slenderness* or *axial load*. The fifth set of numbers in the investigation on effect of slenderness (*i.e.*, '2' or '6') indicates slenderness of the piers, while that in the investigation on effect of axial load level (*i.e.*, '05', '10', '30') indicates the axial load ratio.

Because there is no provision for shear design of piers or compression members in IRC:21-2000, only nominal transverse reinforcement as required by IRC:21-2000 is provided in first set of four piers (one each of solid circular, solid rectangular, hollow circular and hollow rectangular cross-section). These are named as CSWG, RSWG, CHWG and RHWG. However, provisions for shear design in beams and slabs are outlined in IRC:21-2000. Hence, a second set of four more piers (namely CSSG, RSSG, CHSG and RHSG) is designed for shear in lines with these shear design provisions. The overstrength based shear demands of these eight piers are estimated from their monotonic lateral load-displacement responses. Also, the nominal design shear capacities of the sections are computed as per IRC:21-2000 wherein both concrete and transverse steel are considered to contribute to the design shear strength.

Next, the effect of pier slenderness on overall response is investigated. A set of eight piers is designed for two slenderness ratios, namely 2 and 6. The piers (namely CSWL-2, CSWL-6, RSWL-2, RSWL-6, CHWL-2, CHWL-6, RHWL-2 and RHWL-6) are designed for the same superstructure gravity load of 6500 kN, and a transverse load in accordance with Eq.(2) with nominal transverse reinforcement as per IRC:21-2000. In all piers, the cross-sectional area is kept at approximately 4.6 m^2, giving a compression force of about $0.044 f_c^{'} A_g$. Pushover analysis is performed for all the twelve piers to compare the overstrength shear demand with the nominal shear capacity at the critical sections.

Then, the effect of level of axial load on the overall response of piers is investigated. For this, a 10 m long solid circular pier of diameter 2 m is designed for superstructure gravity load of 5050 kN and lateral load of 1032 kN. The pier has nominal transverse reinforcement in the form of circular hoop of diameter 8 mm at 300 mm centres. The pier is then subjected to axial compression loads of $0.05 f_c^{'} A_g$, $0.10 f_c^{'} A_g$ and $0.30 f_c^{'} A_g$ and lateral pushover analysis is performed (analysis cases CSWP-05-1, CSWP-10-1 and CSWP-30-1). The circular hoops in the pier are the enhanced to 12 mm diameters at 100 mm centres, and the lateral load-deformation response for the three axial load levels are obtained for these additional transverse reinforcement type also (analysis cases CSWP-05-2, CSWP-10-2 and CSWP-30-2).

In all numerical studies, concrete cover of 40 mm and concrete grade of 40 MPa are used. All studies are performed for major axis bending, *i.e.,* in the transverse direction (normal to traffic flow). For all the piers, since the resultant tension due to direct compression and bending under design loads exceeds permissible stress given in IRC:21-2000, cracked section analysis was carried out to arrive at the amount of longitudinal steel as required by IRC:21-2000. The permissible stresses used in design are increased by 50% while using seismic load-combinations, as per recommendation of IRC:6-2000.

8. Results

The results of the pushover analyses are shown in Figures 11 to 13. In these, the flexural overstrength based shear demands in the piers are normalised with respect to the design shear force and are plotted against the percentage drift capacity of the piers. The investigation with different cross-section shapes or geometries shows that in all cases, the flexural overstrength based shear demand is more than (2.2 to 3.8 times) the design shear. This is primarily due to the safety factors used in the design. Also, in all cases, the shear demand is more than the shear capacity of the sections (Table 7), implying possible shear failure in these piers. Further, short piers (with slenderness ratio of 1.7 – 2.5) with solid sections and shear reinforcement perform better than the piers with hollow sections with approximately same cross-sectional area, and height (Figure 11).

Hollow sections have larger section dimension and therefore draw more lateral force. In piers with circular cross-section, this increases the overstrength-based seismic shear demand without any appreciable increase in deformability. In piers with rectangular cross-section, the pier with hollow cross-section shows increased deformability, apart from the expected increased shear demand (Figure 11). This is due to the IRC:21-2000 requirement that, in rectangular sections, every corner and alternate longitudinal bar be laterally supported by the corner of a tie, and that no longitudinal bar be farther than 150 mm from such a laterally supported bar. This forces *additional* intermediate ties in both directions in the hollow rectangular sections, which enhance the effective confinement of concrete (compare volumetric ratio of transverse reinforcement in Table 8) and therefore increase the maximum strain that concrete can sustain. This also results in increased deformability of the hollow rectangular section compared to the solid rectangular section with only nominal transverse reinforcement. On the other hand, piers with solid cross-sections with design transverse shear reinforcement have better post-yield behaviour in the form of enhanced deformability and displacement ductility. This signifies the importance of transverse reinforcement on the overall response of piers.

The shear capacities of circular and rectangular sections, both solid and hollow, with nominal transverse reinforcement as recommended by IRC:21-2000 are insufficient for the shear demands due to flexure for these short piers (Tables 7 and 8). Premature brittle shear failure of piers is expected before the full flexural strength is achieved. Of the four types of piers having same height and similar cross-sectional area, and subjected to the same axial compression, the solid circular piers have the least shear capacity. This is attributed to the presence of only a single circular hoop in solid circular piers. In rectangular sections, the intermediate ties in both the directions enhance the shear capacity. Thus, the ratio of transverse reinforcement required (to prevent shear failure) to that provided is maximum (15.69) in pier with solid circular section and least (1.87) in pier with hollow rectangular section (Table 8). Further, the minimum volumetric reinforcement ratio as required in the current international practice (as per AASHTO, NZS and PWRI codes) is much higher than the nominal reinforcement requirement specified in the IRC code (Table 9); the IRC requirement is at least 2-20 times smaller.

Also, in hollow sections, the IRC:78-2000 requirement of minimum area of transverse steel of 0.3% of wall cross-section exceeds the IRC:21-2000 reinforcement requirements. However, even this transverse steel is inadequate to resist the overstrength moment-based shear demand in short piers (Table 8).

In most piers, especially where only nominal transverse reinforcement is provided, buckling of longitudinal reinforcement occurred (Table 7), resulting in sudden loss of load carrying capacity. This is due to the large spacing of transverse reinforcement adopted along the member length; the spacing adopted is as per IRC:21-2000 which is the minimum of (a) 12 times the diameter of smallest longitudinal reinforcement bar, and (b) 300 mm (because the least lateral dimension is always much larger than 300 mm).

The investigation on the effect of pier slenderness reveals that the nominal transverse reinforcement requirements are inadequate for short piers (slenderness ratio of 3), except for pier with rectangular hollow section. On the other hand, for slender piers (slenderness ratio of 6), the nominal design shear capacity is higher than the demand (Tables 10 and 11). Thus, slender piers exhibit a ductile behaviour. In large hollow rectangular piers, better distribution of longitudinal steel and enhanced concrete confinement due to intermediate links result in superior post-yield response than in the other three types of sections considered in this study (Figure 12). Also, with increase in slenderness, the shear demand reduces and the deformability increases. This is due to greater flexibility of piers with increased slenderness. Thus, the target deformability of a pier seems to be a function of its slenderness. However, as in the first study, failure is primarily initiated by buckling of longitudinal steel (Table 10).

The investigation on effect of axial load shows that with increase in axial load level, ductility reduces while the shear demand increases (Figure 13). With increase in axial load, tension yielding of steel is delayed increasing the yield displacement, while the ultimate displacement is reduced due to lesser residual flexural strain capacity of the fibres. This causes a reduction in ductility. In addition, with increase in axial load level, flexural cracking of concrete fibres is delayed, thereby increasing the net uncracked section area. This increases the section rigidity and thus draws in more lateral shear, thereby increasing the shear demand. In addition, with increase in amount of transverse reinforcement, the deformability increases (Figure 13). This is again due to increase in confinement of concrete and corresponding increase in ultimate strain capacity. These observations suggest that with increase in axial load level, for an expected drift capacity, higher amount of transverse reinforcement is required to prevent shear failure (Tables 12 and 13).

9. Observations

A number of important points are brought to attention through the review of IRC:21-2000, IRC:78-2000 and IRC:6-2000 seismic bridge design provisions in light of some international practices relating to capacity design approach and the Interim IRC:6-2002 provisions, and through the numerical investigation of single-column type RC piers based on design methodologies in existing Indian standards. These are:

a) The extreme low values of seismic force as per IRC:6-2000 are eliminated in the Interim provisions and the new provisions provide a more rational basis for seismic force calculation including the effect of structural flexibility.

b) The Interim provisions account for response reduction factor in seismic design. Thus, in essence it advocates nonlinear response of piers. However, the recommended response reduction factor of 2.5 may not be used in the design of connections.

c) For piers designed for design force level as per IRC:6-2000, the design longitudinal reinforcement is insufficient to resist the effects of increased horizontal force level as per Interim provisions, if working stress design philosophy enumerated in IRC:21-

2000 is used.

d) Increase in concrete strength and stain capacities due to confinement by transverse reinforcement and strain hardening of longitudinal steel is not accounted for in shear design of RC piers as per IRC:21-2000; this results in much higher flexural overstrength based shear demand than design shear level and hence makes the bridge vulnerable to brittle shear failure.

e) Possibility of plastic hinge formation in an extreme seismic event is not accounted for in the design procedure outlined in IRC codes; capacity design is not performed.

f) Nominal transverse steel requirements as given in IRC:21-2000 are inadequate in preventing brittle shear failure in short piers (of slenderness ratio of 3) under force levels as per Interim IRC:6-2002.

g) Piers with hollow sections show enhanced deformability and have higher shear capacity compared to the solid ones with approximately same cross-sectional area owing to presence of larger nominal transverse reinforcement.

h) Due to presence of additional intermediate ties, piers with rectangular sections have larger shear and deformability capacity compared to those with circular sections.

i) Buckling of longitudinal reinforcement is not prevented by the existing provision on spacing of transverse reinforcement; buckling is common in piers resulting in rapid strength loss.

j) Increasing the amount of transverse reinforcement (from 2.43 to 15.69 times the current amounts) increases displacement ductility of piers and produces improved post-yield response.

k) Piers under higher axial compression require more transverse reinforcement for expected displacement ductility; transverse reinforcement requirement in IRC code can be made a function of the probable maximum axial load on the pier and the required displacement ductility.

10. Conclusions

This study on the seismic design of RC bridge piers designed as per the current Indian code provisions suggests that the following changes be urgently brought into the IRC provisions:

a) The design of RC members as given in IRC:21-2000 needs to be revised in line with the design philosophy of inelastic action of piers intended in the Interim IRC:6-2002 provisions. Currently, the IRC uses two different design philosophies, the inelastic behaviour philosophy for calculating the seismic load using a response reduction factor (greater than unity) implying *nonlinear* response, and the elastic behaviour philosophy for designing piers by the *elastic* working stress method. The two need to be calibrated for each other, else a consistent *inelastic* design approach may be adopted.

b) The design for shear of slender vertical RC bridge members needs to be based on capacity design concepts. Also, a formal design basis is required for calculation of *design* shear strength V_c of concrete depending on the confinement, level of axial load, and imposed ductility under cyclic loading.

c) The contribution of transverse reinforcement in confining the core concrete and preventing buckling of longitudinal bars, should be included.

Table 1: Horizontal seismic coefficient α as per IRC:6-2000

Seismic Zone	**I**	**II**	**III**	**IV**	**V**
Horizontal Seismic Coefficient α	0.01	0.02	0.04	0.05	0.08

Table 2: Soil-foundation system factor β as per IRC:6-2000

Type of Soil mainly constituting the foundation	**Soil-Foundation System Factor β**			
	Bearing Piles resting on Soil Type I, or Raft Foundations	**Bearing Piles resting on Soil Type II & III, Friction Piles, Combined Footings or Isolated RCC Footings with Tie Beams**	**Isolated RCC Footings without Tie Beams, or Unreinforced Strip Foundations**	**Well Foundations**
Type I :: Hock or Hard Soils *(for N>30)*	1.0	1.0	1.0	1.0
Type II :: Medium Soils *(for 10<N<30)*	1.0	1.0	1.2	1.2
Type III :: Soft Soils *(for N<10)*	1.0	1.2	1.5	1.5
Note: *N* = Standard Penetration Test Value				

Table 3: Seismic Zone factor Z as per Interim IRC:6-2002

Seismic Zone	**II**	**III**	**IV**	**V**
Seismic Zone Factor Z	0.10	0.16	0.24	0.36

Table 4: Average Response Acceleration Coefficient S_a/g (for 5% damping) as per Interim IRC:6-2002

Soil Type	**Average Response Acceleration Coefficient** S_a/g
Rocky or Hard Soil	$2.50 \; ; \; 0.00 \le T \le 0.40$
	$\frac{1.00}{T} \; ; \; 0.40 \le T \le 4.00$
Medium Soil	$2.50 \; ; \; 0.00 \le T \le 0.55$
	$\frac{1.36}{T} \; ; \; 0.55 \le T \le 4.00$
Soft Soil	$2.50 \; ; \; 0.00 \le T \le 0.67$
	$\frac{1.67}{T} \; ; \; 0.67 \le T \le 4.00$

Table 5: Design Seismic Coefficients as per IRC:6-2000 and Interim IRC:6-2002 for seismic shaking in the transverse and longitudinal directions of the bridge

Seismic Zone	**IRC:6-2000** α_{2000} **(Longitudinal and Transverse)**	**Interim IRC:6-2002** $\alpha_{Interim}$		**Ratio** $\frac{\alpha_{Interim}}{\alpha_{2000}}$	
		Longitudinal	**Transverse**	**Longitudinal**	**Transverse**
V	0.144	0.098	0.270	0.68	1.88
IV	0.090	0.066	0.180	0.73	2.00
III	0.072	0.044	0.120	0.61	1.67
II	0.036	0.027	0.075	0.75	2.08
I	0.018			1.50	4.17

Table 6: Maximum recommended spacing of transverse reinforcement sets in piers.

Specification	Maximum Spacing	
	Outside Potential Plastic Hinge Region	Within Potential Plastic Hinge Region
AASHTO	*Min* [b; 300 mm]	*Min* [$b/4$; 100 mm]
CALTRANS	---	*Min* [$b/5$; $6d_b$; 220 mm]
NZS (fully-ductile)	*Min* [$b/3$; $10d_b$]	*Min* [$b/4$; $6d_b$]
NZS (partially-ductile)	*Min* [$b/3$; $10d_b$]	*Min* [$b/4$; $10d_b$]
PWRI	> 150 mm	150 mm

Note: b = Least cross-sectional dimension of the pier
d_b = Least nominal diameter of longitudinal reinforcement

Table 7: Results of analyses of four types of 5 m long piers comparing shear capacity and demand, and showing final form of failure.

Pier (L=5 m)	Section (m)	Area (m^2)	Reinforcement		Shear		Failure Mode
			Longitudinal	Transverse	Capacity (kN)	Demand (kN)	
CSWG	2.00 ϕ	3.14	54Y28	Y8@300	1778	3884	Buckling of long. steel
CSSG	2.00 ϕ	3.14	54Y28	Y12@190	2137	3904	Buckling of long. steel
RSWG	2.6×1.2	3.12	46Y28	Y8@300	1969	5319	Buckling of long. steel
RSSG	2.6×1.2	3.12	46Y28	Y8@150	2473	5383	---
CHWG	2.6(OD), 1.6(ID)	3.30	54Y25	Y12@150	2930	4771	Buckling of long. steel
CHSG	2.6(OD), 1.6(ID)	3.30	54Y25	Y12@150	2930	4771	Buckling of long. steel
RHWG	1.4×3.0(OD), 0.5×2.1(ID)	3.15	96Y20	Y10@110	4328	6726	---
RHSG	1.4×3.0(OD), 0.5×2.1(ID)	3.15	96Y20	Y10@110	4328	6726	---

Table 8: Transverse reinforcement requirement to prevent shear failure in 5 m long piers of four types of cross-section in investigation on effect of geometry.

Pier	Shear		Volumetric Ratio of Transverse Reinforcement (10^{-3})		Ratio ρ_s^r / ρ_s^p
	Capacity (kN)	Demand (kN)	Provided ρ_s^p	Required ρ_s^r	
CSWG	1778	3884	0.346	5.43	15.69
CSSG	2137	3904	1.23	5.43	4.41
RSWG	1969	5319	1.26	5.36	4.25
RSSG	2473	5383	2.51	6.11	2.43
CHWG	2930	4771	3.49	8.09	2.32
CHSG	2930	4771	3.49	8.09	2.32
RHWG	4328	6726	6.51	12.16	1.87
RHSG	4328	6726	6.51	12.16	1.87

Table 9: Minimum transverse reinforcement requirement in plastic hinge regions as per Indian and international codes

Pier	Minimum Volumetric Ratio of Transverse Reinforcement (10^{-3})			
	Provided	Required		
	IRC:21-2000	*AASHTO 1998*	*NZS 1995*	*PWRI 1998*
CSWG	0.346	9.25	5.61	2.79
CSSG	1.23	9.25	5.61	2.79
RSWG	1.26	26.44	14.79	10.10
RSSG	2.51	26.44	14.79	10.10
CHWG, CHSG	3.49	9.25	4.46	6.38
RHWG, RHSG	6.51	34.12	28.88	12.22

Table 10: Results of analyses of four types of piers of two slenderness ratio comparing shear capacity with demand, and showing final form of failure.

Pier Name	*L* (m)	Section (m)	Area (m^2)	Reinforcement		Shear		Failure Mode
				Long	Trans	Capacity (kN)	Demand (kN)	
CSWL-3	7.2	2.40 ϕ	4.52	66Y28	Y8@300	2133	3888	Buckling of long. steel
CSWL-6	14.4	2.40ϕ	4.52	76Y28	Y8@300	2271	1958	Buckling of long. steel
RSWL-3	8.7	2.9×1.6	4.64	64Y28	Y8@300	2790	4367	Buckling of long. steel
RSWL-6	17.4	2.9×1.6	4.64	64Y28	Y8@300	2774	1997	Buckling of long. steel
CHWL-3	10.2	3.4(OD), 2.4(ID)	4.56	81Y25	Y12@150	3997	4222	Buckling of long. steel
CHWL-6	20.4	3.4(OD), 2.4(ID)	4.56	99Y25	Y12@150	4194	2216	Buckling of long. steel
RHWL-3	10.8	2.0×3.6(OD), 1.0×2.6(ID)	4.60	128Y20	Y12@150	6008	4664	----
RHWL-6	21.6	2.0×3.6(OD), 1.0×2.6(ID)	4.60	128Y20	Y12@150	5991	2127	----

Table 11: Transverse reinforcement requirement to prevent shear failure in piers in investigation on effect of slenderness.

Pier	Shear		Volumetric Ratio of Transverse Reinforcement (10^{-3})		Ratio ρ_s^r/ρ_s^p
	Capacity (kN)	Demand (kN)	Provided ρ_s^p	Required ρ_s^r	
CSWL-3	2133	3888	0.286	3.14	10.98
CSWL-6	2271	1958	0.286	0.286	1.00
RSWL-3	2790	4367	1.37	3.86	2.82
RSWL-6	2774	1997	1.37	1.37	1.00
CHWL-3	3997	4222	3.49	4.03	1.15
CHWL-6	4194	2216	3.49	3.49	1.00
RHWL-3	6008	4664	6.04	6.04	1.00
RHWL-6	5991	2127	6.04	6.04	1.00

Table 12: Results of analyses of solid circular pier with three axial load ratio and two different circular hoops with percentage lateral drift.

Pier	Section Diameter	Area	Reinforcement		Shear		Axial Load Ratio	Lateral Drift
			Long.	Trans.	Capacity	Demand		
(L=10.0m)	(m)	(m^2)			(kN)	(kN)		(%)
CSWP-05-1	2.0	3.14	70Y32	Y8 @300	1997	2458	0.05	1.80
CSWP-10-1	2.0	3.14	70Y32	Y8 @300	1997	2586	0.10	1.50
CSWP-30-1	2.0	3.14	70Y32	Y8 @300	1997	2862	0.30	0.95
CSSP-05-2	2.0	3.14	70Y32	Y12@100	2874	2569	0.05	3.10
CSSP-10-2	2.0	3.14	70Y32	Y12@100	2874	2664	0.10	2.60
CSSP-30-2	2.0	3.14	70Y32	Y12@100	2874	2937	0.30	1.50

Table 13: Transverse reinforcement requirement to prevent shear failure in piers in investigation on effect of axial load.

Pier	Shear		Volumetric Ratio of Transverse Reinforcement (10^{-3})		Ratio ρ_s^r/ρ_s^p
	Capacity (kN)	Demand (kN)	Provided ρ_s^p	Required ρ_s^r	
CSWP-05-1	1997	2458	0.347	1.04	3.00
CSWP-10-1	1997	2586	0.347	1.36	3.92
CSWP-30-1	1997	2862	0.347	1.96	5.65

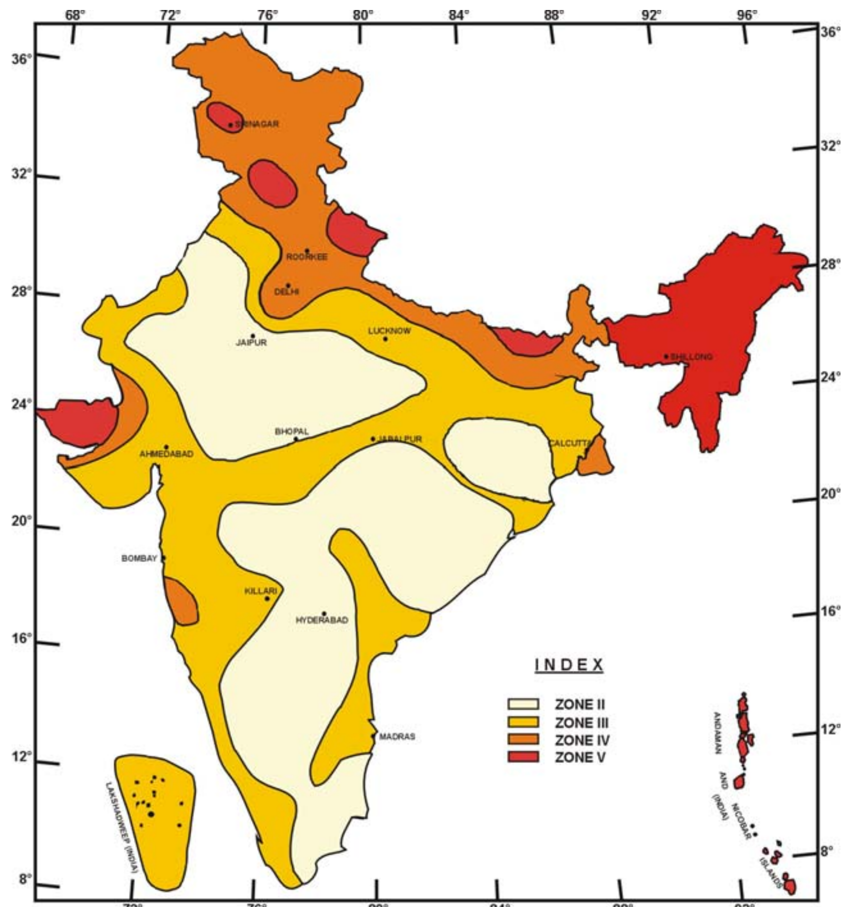

Figure 1: Seismic Zones and Zone Map of India [IS:1893 (Part 1), 2002].

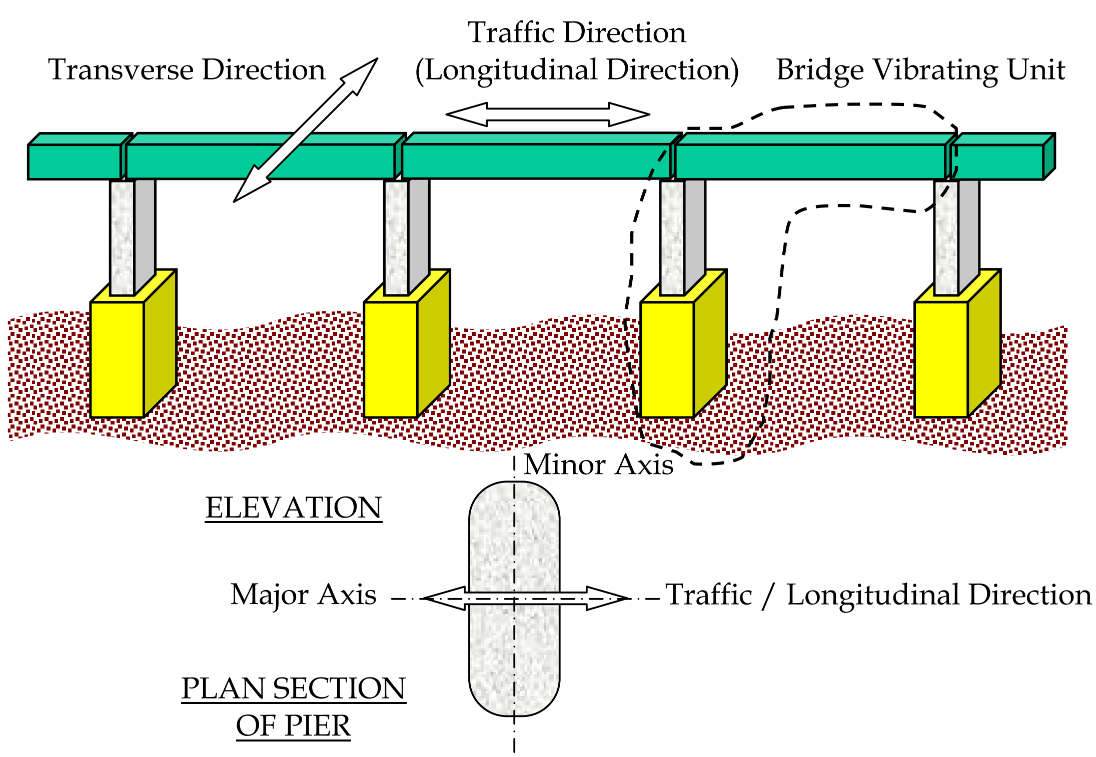

Figure 2: Single pier bridge vibration unit with typical orientation of the pier section.

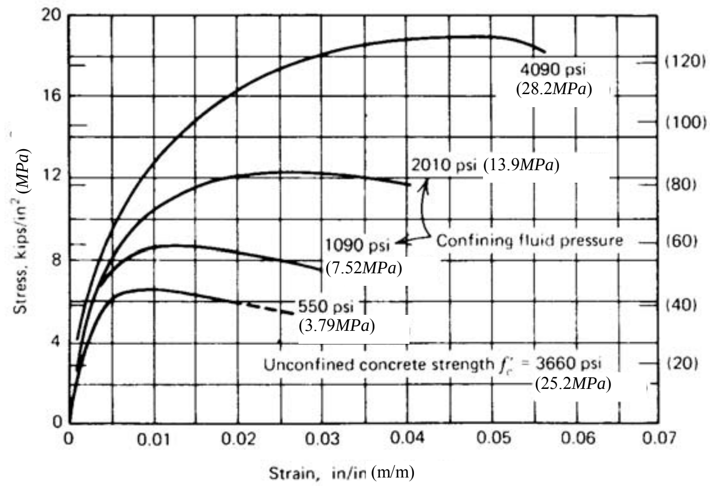

Figure 3: Experimental stress-strain curves of concrete under various confining pressures [Scott *et al.*, 1982].

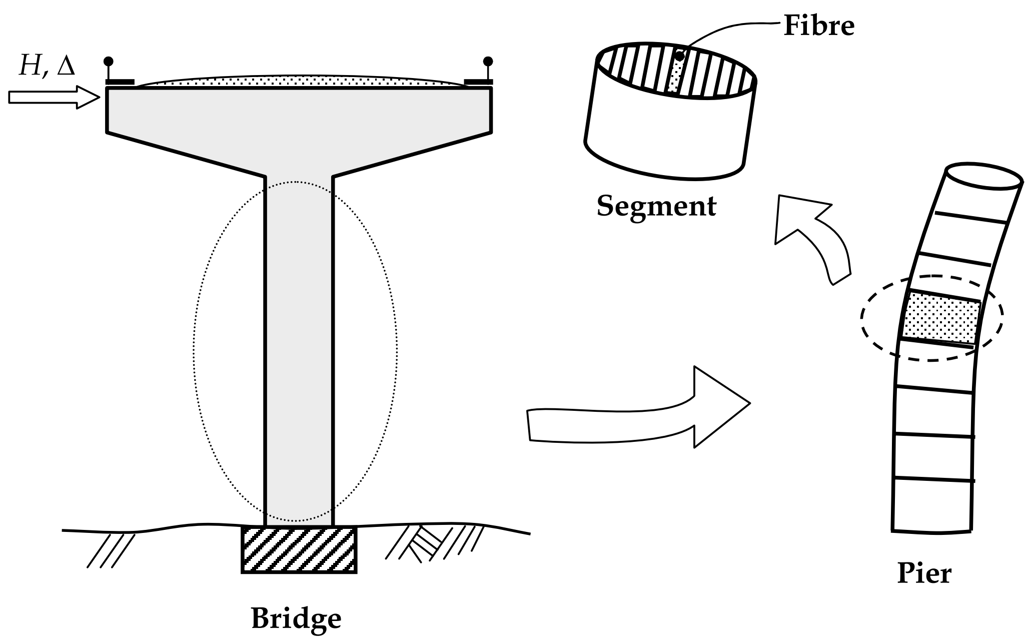

Figure 4: Discretisation of a bridge pier into segments, and further, segments into fibres.

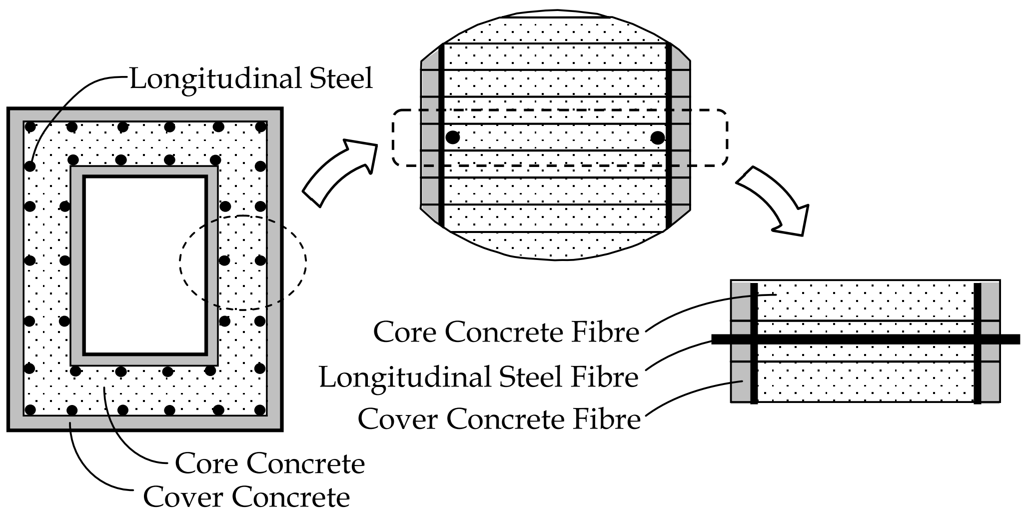

Figure 5: Discretisation of a hollow rectangular RC section into concrete (core and cover) and longitudinal steel fibres.

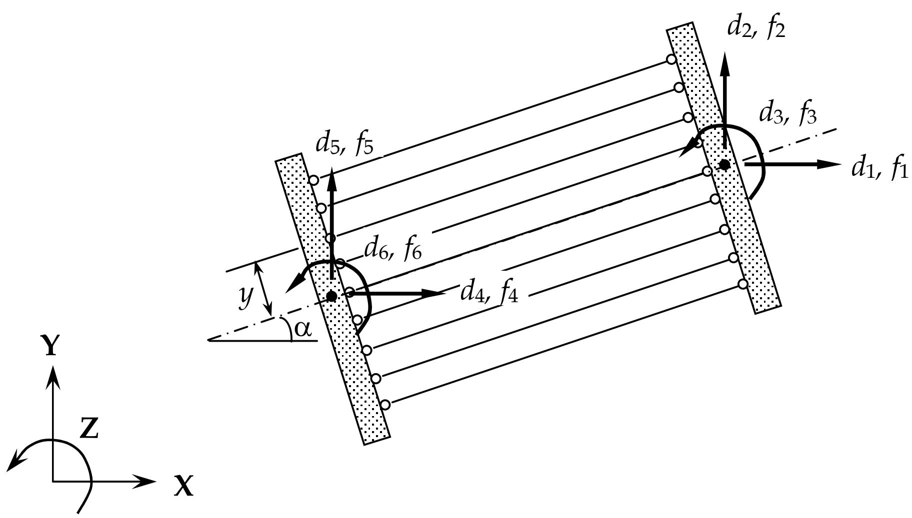

Figure 6: End-forces and displacement quantities on a segment in global coordinates.

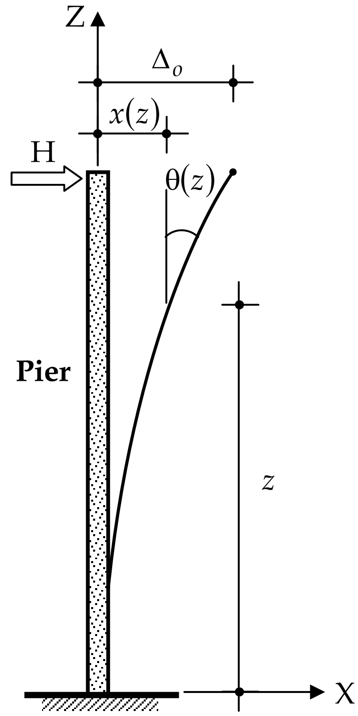

Figure 7: Initial lateral and rotational deformation of cantilever pier section for specified tip displacement.

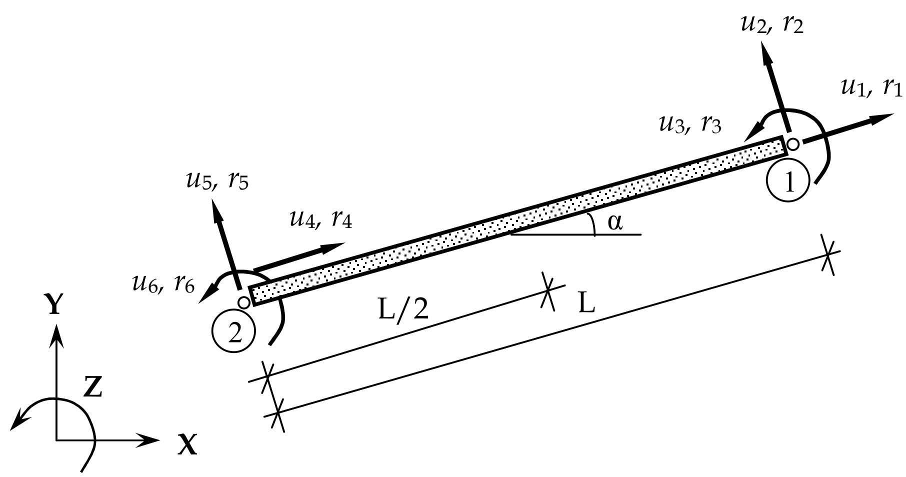

Figure 8: End-forces and displacement quantities on a segment in local coordinates.

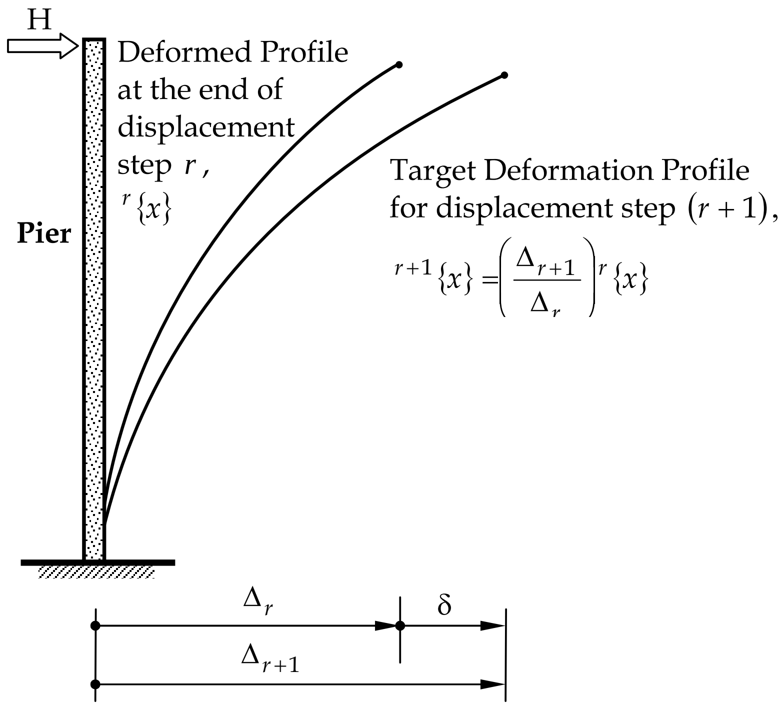

Figure 9: Target displacement profile of pier for next displacement step in pushover analysis.

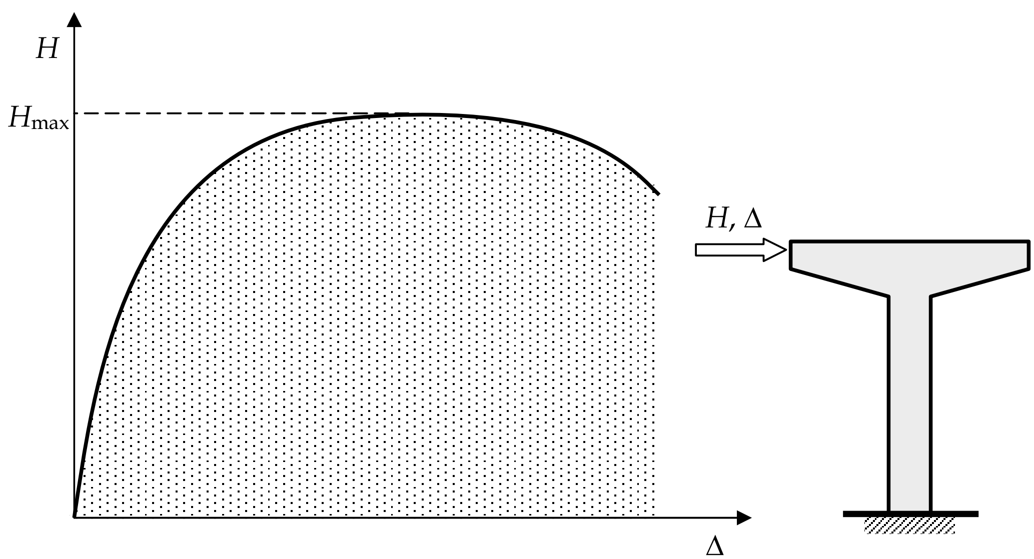

Figure 10: Maximum shear demand on the pier during the entire displacement loading history.

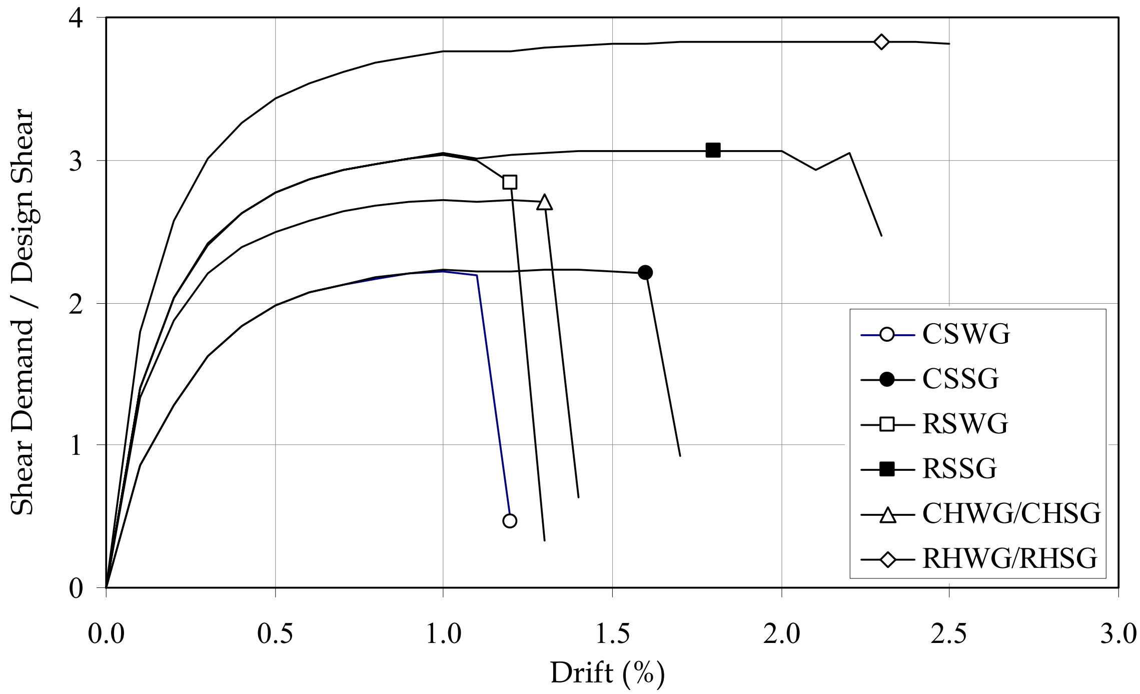

Figure 11: *Effect of Cross-Section Geometry*: Lateral load-deformation response of 5 m tall piers. Piers having solid circular and rectangular cross-sections and with design shear reinforcement have stable post-yield response.

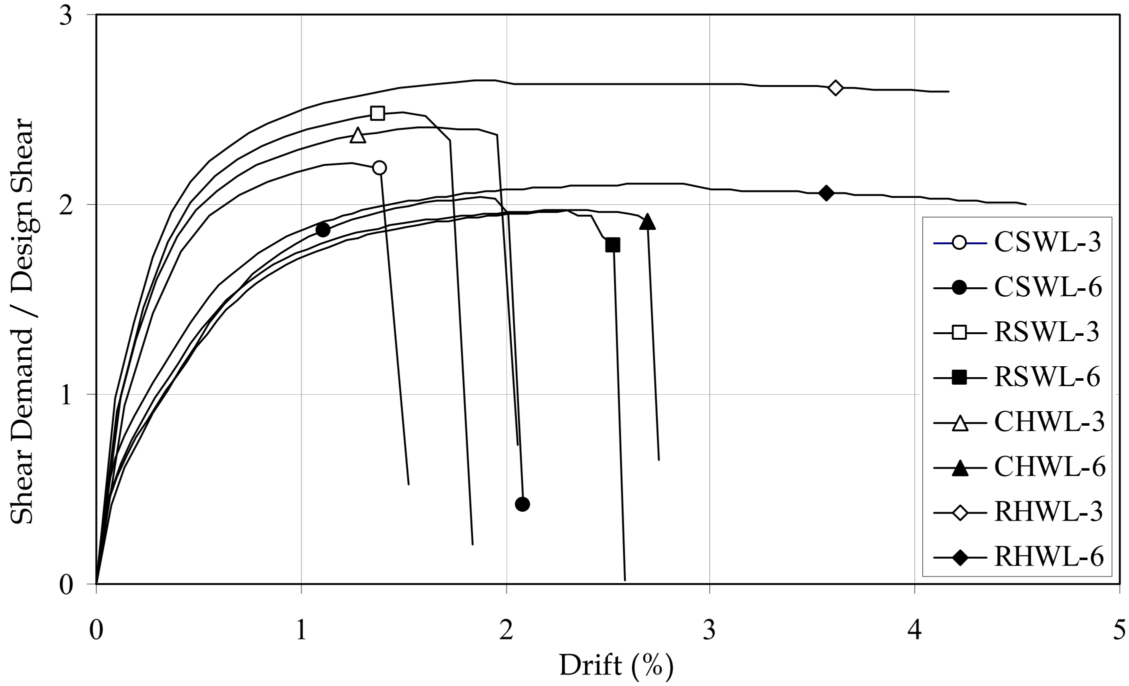

Figure 12: *Effect of Pier Slenderness*: Lateral load-deformation response of solid-circular, solid-rectangular, hollow-circular, and hollow-rectangular piers having slenderness ratios 3 and 6 with corresponding nominal shear capacities. Short piers with slenderness of 3 are vulnerable in shear.

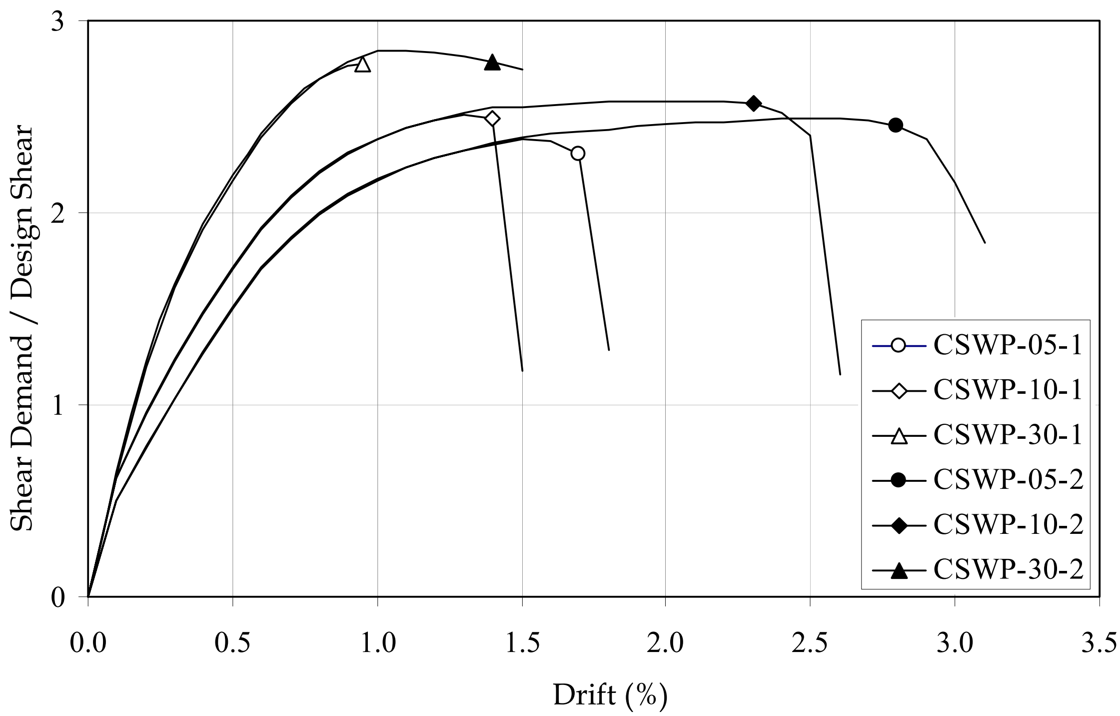

Figure 13: *Effect of Axial Load*: Lateral load deformation response of solid circular piers with axial load ratios of $0.05f_c^{'}A_g$, $0.10f_c^{'}A_g$ and $0.30f_c^{'}A_g$ with nominal and increased transverse reinforcement. Axial load increases overstrength shear demand while transverse reinforcement increases displacement ductility.

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List of Symbols

Symbol	***Description***
A	Area of cross-section
A_c	Area of core of pier
A_g	Gross cross-sectional area of pier
A_h	Horizontal seismic coefficient in Interim provision
A_s	Shear area
D'	Core dimension in the direction under consideration
E	Modulus of elasticity
E_c	Modulus of elasticity of concrete
E_{sec}	Secant modulus of elasticity of confined concrete at ultimate stress
E_t	Tangent modulus of elasticity of material
F_{eq}	Horizontal equivalent seismic force on a structure
G	Bulk modulus of elasticity
$H_{\max}$	Maximum lateral internal resistance of the pier

Symbol	Description
I	Importance factor
${}^{ab}[K]_t$	Tangent stiffness matrix of a segment corresponding to axial and bending effects in global coordinates
${}^{sh}[K]_t$	Tangent stiffness matrix of a segment corresponding to shear effects in global coordinates
$[K]_t^s$	Tangent stiffness matrix of a segment in global coordinates
$[K]_t$	Complete tangent stiffness matrix of member in global coordinates
L	Length of fibre / segment
M_c	Moment resisted by a section at a general iteration level
N_f	Number of fibres in a segment
N_{fc}	Number of concrete fibres in a segment
N_{fs}	Number of steel fibres in a segment
N_s	Number of segments
P	Axial load (positive for tensile load)
P^*	Design axial load on pier at ultimate limit state in NZS 3101
P_c	Axial load resisted by a section at a general iteration level
R	Response reduction factor
S_a	Average response acceleration
T	Fundamental period of vibration
$[T]$	Transformation matrix relating displacements and forces in local and global coordinates
V_c	Shear strength provided by concrete; Total shear resisted by a section at a general iteration level
V_n	Nominal shear strength
V_o	Flexural overstrength based shear demand
V_s	Shear strength provided by transverse reinforcement
$V_\Omega^{\max}$	Flexural overstrength-based shear demand on RC pier
W	Seismic weight (Dead load plus appropriate Live load)
Z	Zone factor
d_b	Nominal diameter of longitudinal reinforcement
$\{d\}$	End-displacement vector of a segment in the Fibre Model in global coordinate
f_c	Stress in concrete
$f_c^{'}$	Characteristic compressive (cylinder) strength of concrete
$f_{cc}^{'}$	Ultimate stress of confined concrete
f_{ck}	Characteristic compressive (cube) strength of concrete
f_u	Ultimate stress of steel
f_y	Characteristic yield stress of steel

Symbol	Description
f_{yl}	Characteristic yield stress of longitudinal steel
f_{yt}	Characteristic yield stress of transverse steel
$\{f\}$	End-force vector of a segment in global coordinates; External load vector on the member in global coordinates
g	Acceleration due to gravity (=9.81 m/s^2)
h	Height of cantilever pier
m	Modular ratio
$\{p\}$	Internal resistance vector on the member in global coordinates
r, r_o	Factors used in concrete confinement model
$\{r\}$	End-force vector of a segment in local coordinate
$\{r_s\}$	Residual force vector in global coordinates
s	Longitudinal spacing of transverse reinforcement
$x(z)$	Initial lateral displacement at height z from pier bottom due to Δ_o
$\{x\}$	End-displacement vector of a segment in global coordinates
$\{u\}$	End-displacement vector of a segment in local coordinates
y	Distance of midpoint of a fibre from gross cross-section centroidal axis
z	Distance of a cross-section of pier from the base
Δ_o	Initial displacement increment of pier top
Ω	Multiplier to convert nominal shear capacity to overstrength based seismic shear demand
α	Basic horizontal seismic coefficient in IS:6-2000; Inclination of a segment in the global coordinate
β	Soil-foundation system modification factor in IS:6-2000; Relative ratio of flexural lateral translational stiffness and shear stiffness of the segment
ε^f	Net incremental axial strain in fibre
ε_1	Strain corresponding to ultimate stress of confined concrete
ε_c	Strain in concrete
ϕ	Resistance factor (less than unity)
ρ_l	Longitudinal reinforcement ratio
ρ_s	Volumetric ratio of transverse reinforcement
λ	Importance factor in IS:6-2000
$\theta(z)$	Initial rotation at height z from pier bottom due to Δ_o
$\sigma^1_{cr,b}$	Elastic buckling stress of longitudinal steel in piers
$\sigma^2_{cr,b}$	Inelastic buckling stress of longitudinal steel in piers
$\sigma_{cr,b}$	Critical stress for buckling of longitudinal steel in piers