Review of Indian seismic code, IS 1893 (Part 1) : 2002

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The Indian seismic code IS 1893 has now been split into a number of parts and the first part containing general provisions and those pertaining to buildings has been released in 2002. There has been a gap of 18 years since the previous edition in 1984. Considering the advancements in understanding of earthquake-resistant design during these years, the new edition is a major upgradation of the previous version. This paper reviews the new code; it contains a discussion on Clauses that are confusing or vague and need clarifications immediately. The typographical and editorial errors are pointed out. Suggestions are also included for next revision of the code.

With rapid strides in earthquake engineering in the last several decades, the seismic codes are becoming increasingly sophisticated. The first Indian seismic code (IS 1893) was published in 1962 and it has since been revised in 1966, 1970, 1975 and 1984[1]. More recently, it was decided to split this code into a number of parts, and Part 1 of the code containing general provisions (applicable to all structures) and specific provisions for buildings has been published[2].

Considerable advances have occurred in the knowledge related to earthquake resistant design of structures during the 18 years interval between the two editions of the code[3]. Some of these new developments have been incorporated in the 2002 version of the code, while many others have been left out so that the implementation of the code does not become too tedious for Indian professional engineers. For example, in the United States, the codes are revised every three years, and hence, a typical building code in the United States has acquired sophistication gradually over about six revisions during these 18 years. Since the Indian code has had to make a quantum jump with respect to many of the provisions, it still requires considerable effort for an average professional engineer to fully appreciate the new code and to be able to implement it correctly.

In the above scenario, the following steps are urgently needed:

(i) careful review of the new code to remove any deficiencies, errors, or scope for misinterpretation

(ii) development of explanatory handbook on the code to explain the new code with solved examples

It is not uncommon to have errors or omissions in the codes. However, it is important to quickly correct these errors or omissions. This paper reviews the code and the suggestions for changes in the next revision are listed. Also listed are Clauses that are confusing or vague and need clarifications immediately. Finally, the typographical and editorial errors are pointed out.

Philosophical changes in the new code

As compared to the previous version, several major modifications have been incorporated in the new code. Some of the important modifications include the following.

- The seismic zone map now contains only four zones as compared to the five zones earlier, and the relative values of zone factors are now different.
- The code now provides realistic values of acceleration from which the design forces are obtained by dividing the elastic forces by a response reduction factor; this enables a clear statement of intent to the designer that the design seismic force is much lower than what can be expected in the event of a strong shaking.
- The design spectrum shape now depends on the type of soil and the foundation-soil factor (β) has been dropped.
- The code now requires that there be a minimum design force based on empirical fundamental period of the building even if the dynamic analysis gives a very

high value of natural period and thus low seismic force.

Comments and suggestions

Other parts of IS 1893 (Foreword, page 2)

The fourth revision of the code (IS 1893 : 1984) covered buildings, water tanks, stacklike structures, bridges, dams and retaining walls. When carrying out the fifth revision, it was decided to split the code into five parts.

Part 1: General provisions and buildings

Part 2: Liquid retaining tanks – elevated and ground supported

Part 3: Bridges and retaining walls

Part 4: Industrial structures including stack like structures

Part 5: Dams and embankments

While Part 1 of the code has been released, the other parts are still in various stages of development. To address the situation that other parts of the code are not yet released, *Note* on page 2 of the code states:

> "Pending finalization of Parts 2 to 5 of IS 1893, provisions of Part 1 will be read along with the relevant Clauses of IS 1893 : 1984 for structures other than buildings".

This is problematic in many situations, for instance, let us consider the case of overhead water tanks. In the 1984 code seismic design force for water tanks depends on parameters β, I, and α_o. In the new code, terms β and α_o do not exist. Instead, one now needs the response reduction factor (R) for water tanks which is supposed to be provided in Part 2 of code. Clearly, there is no way one can combine Part 1 of the 2002 version with the 1984 code for water tanks and there is bound to be dispute and possibly litigation in case of lump sum contracts for water tanks. Similarly, it is not possible to implement the new provisions for bridges, stacks and dams. However, the problem is most serious for water tanks: bridges are designed as per the provisions of IRC or Indian Railway codes, and the design of industrial structures is usually done by established structural consultants.

To obviate this situation, the process for finalisation of remaining parts of the code must be completed at the earliest. In the meanwhile, a *model code* is urgently needed for the water tanks which could be adopted by the government departments for their contracts.

Earthquake intensity (Foreword, page 3, last para)

A number of intensity scales are used for qualitatively describing the intensity of earthquake shaking. Most common are the modified Mercalli scale, and the MSK (Medvedev-Sponhener-Karnik) scale. In last para of page 3, the code refers to this scale as "Comprehensive Intensity Scale (MSK64)" while in Clause 3.15 a mention is made of MSK scale. The term "Comprehensive Intensity Scale" is not common and to avoid confusion it is more appropriate that the code should simply use MSK or MSK64 scale. The same also holds for the heading of *Annexure* D of the code.

Risk level

Para 5 on page 3 of the code states

> "The seismic hazard level with respect to ZPA at 50 percent risk level and 100 years service life goes on progressively increasing …".

This statement is made in the context of earthquake geology of the country. However, it may give a false impression that the values of ZPA (denoted by Z) given in the code are for 50 percent risk level and 100 years service life. Such a confusion needs to be avoided by modifying this statement as "The seismic hazard level goes on progressively increasing….".

Peak ground acceleration

Item (b) on page 2 of the code uses the term "Effective Peak Ground Acceleration" (EPGA). This term is also defined in Clause 3.11. For the purposes of the code it is not important to differentiate between EPGA and "Peak Ground Acceleration" PGA. Similarly, the code also uses the term "Zero Period Acceleration" (ZPA) at several places. Since the stiff structures (having natural period of zero) experience same acceleration as the ground acceleration, the ZPA value is same as PGA. To avoid confusion, it is best to just use the term "Peak Ground Acceleration" (PGA), and the terms ZPA and EPGA should be dropped from the code.

Service life of structure (Item (b) on page 2, Clause 3.33, and Clause 6.4.2)

Item (b) on page 2 states that the values of seismic zone factor reflect more realistic values of EPGA considering "Maximum Considered Earthquake" (MCE) and service life of structure in each seismic zone. A similar mention of the service life is made while defining Z in Clause 6.4.2. This confuses the user since he then asks questions such as:

(i) what value of service life should be considered for his structure

(ii) if he is willing to reduce the service life of his structure say from 100 years to 50 years, how much reduction in the seismic design force would be allowed by the code.

The fact remains that the values of Z specified in the code were arrived at empirically based on engineering judgment and no explicit calculations were done or envisaged for service life. Hence, it is best to drop the mention of "service life". This suggestion is consistent with the fact that in the definition of Z in Clause 3.33 also, the code makes no mention of service life.

References (page 4, Foreword)

A list of four references is provided on page 4 of the code. However, these references are obsolete and newer versions of some of these were available and used in the development of the code. For instance, Uniform Building code[4] has been revised in 1997 and later replaced by the International Building Code 2000 and 2003[5]. NEHRP documents too are being revised every three years and the 2000 version is available[6].

It is best to mention later versions of the references. Further, a considerable part of the code is based on two published articles of the authors[7,8]. These two articles could provide additional background materials to the engineer and hence it is appropriate to add these to the list in the code.

Response spectrum (Clauses 3.5, 3.27, 3.30, 6.4, …)

In the code, different terms are used for response spectrum, for example, "Design Acceleration Spectrum" (Clause 3.5); "Response Spectrum" (Clause 3.27); "Acceleration Response Spectrum" (used in Clause 3.30); "Design Spectrum" (title of Clause 6.4); "Structural Response Factor"; "Average response acceleration coefficient" (see terminology of S_a/g on p. 11), etc. It is best to use one single term consistently to avoid confusion. It is suggested that the term be "Design Acceleration Spectrum" for the plot of response spectrum with natural period, and the term be "Response Acceleration Coefficient" for the value of S_a/g for a given value of natural period.

Maximum considered earthquake (MCE) and design basis earthquake (DBE)

This edition of the code introduces two new terms:

> *"Maximum Considered Earthquake" (MCE)*: Defined in Clause 3.19 as "The most severe earthquake effects considered by this standard", and
>
> *"Design Basis Earthquake" (DBE)* : Defined in Clause 3.6 as "It is the earthquake which can reasonably be expected to occur at least once during the design life of the structure."

Both these definitions are quite incomplete and do not tell anything specific to the user. For instance, what is meant by "reasonable expectation"! Also, the design life of different structures may be different and yet the code specifies the same PGA value regardless of the design life of a structure.

Let us consider the use of these terms in the International Building Code (IBC). The IBC 2003 defines MCE as corresponding to 2 percent probability of being exceeded in 50 years (2,500 year return period), and the DBE as corresponding to 10 percent probability of being exceeded in 50 years (475 year return period). Clearly, there is no ambiguity in IBC on this account.

Since the seismic zone map in Indian code is not based on probabilistic hazard analysis, it is not possible to deduce the probability of occurrence of a certain level of shaking in a given zone based on this code. Therefore, use of terms such as MCE and DBE do not add any new information, and can sometimes cause confusion and disputes. For instance, someone may argue that the value of Z=0.36 for MCE in zone V of the code implies that the PGA value in zone V can not exceed 0.36g, which is not the intention of the code. For instance, during 2001 Bhuj earthquake, ground acceleration ~0.6g has been recorded at Anjar located at 44 km from epicentre.

Clause 6.1.3 implies that DBE relates to the "moderate shaking" and MCE relates to the "strong shaking". This is at variance with the definitions of MCE and DBE given in Clauses 3.19 and 3.6 as mentioned above. Again, it clearly shows that there is an element of confusion about the definition and implications of these two terms. Considering that these terms do not add any substantial value to the codal provisions, the two terms may be dropped from the code.

Centre of stiffness and centre of rigidity

In Clause 4.5, centre of stiffness is defined, but in Clause 4.21 while defining static eccentricity, the term centre of rigidity is used. Both centre of stiffness (CS) and centre of rigidity (CR) are the same terms for purposes of the code and hence to avoid confusion, it is best to use only one term consistently. It is proposed that centre of stiffness be replaced by the term centre of rigidity wherever it appears in the code.

Clause 4.5 defines centre of stiffness as "The point through which the resultant of the restoring forces of a system acts." This definition is incomplete. For single storey buildings it may be defined as:

> "If the building undergoes pure translation in the horizontal direction (that is, no rotation or twist or torsion about vertical axis), the point through which the resultant of the restoring forces acts is the centre of stiffness".

For multi-storeyed buildings, centre of rigidity (stiffness) can be defined in two ways.

> *All floor definition of centre of rigidity*: Centre of rigidities are the set of points located one on each floor, through which application of lateral load profile would cause no rotation in any floor, *Fig* 1*(a)*. As per this definition, location of CR is dependent on building stiffness properties as well as on the applied lateral load profile.
>
> *Single floor definition of centre of rigidity:* Centre of rigidity of a floor is defined as the point on the floor such that application of lateral load passing through that point does not cause any rotation of that particular floor, while the other floors may rotate *Fig* 1*(b)*. This definition is independent of applied lateral load.

The two definitions for multi-storey buildings will give somewhat different values of design eccentricity but the

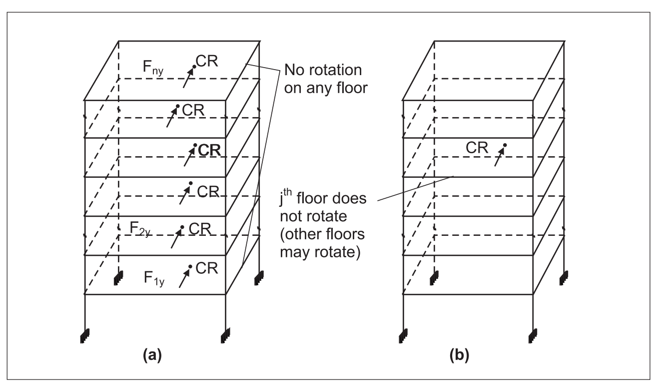

Fig 1 *(a)* All floor definition of centre of rigidity *(b)* Single floor definition of centre of rigidity

difference is not very substantial. Hence, choice of the definition should be left to the designer and the above definitions should be added in the code.

Moment resisting frame (Clause 4.15)

Clause 4.15 defines ordinary and special moment resisting frames (SMRF). Frames specially detailed for ductile behaviour are termed as SMRF while those designed following the routine non-seismic codes (for example, IS 456[9]) are termed as ordinary moment resisting frames (OMRF). Ductile structures perform much better during earthquakes, and therefore are designed for lower seismic forces than the ordinary structures, located in the same seismic zone.

IS 13920 : 1993[10] deals with provisions on ductile detailing of RC structures for seismic performance. As of now, IS codes do not have ductility provisions for detailing of steel structures in high seismic regions. However, for the sake of completeness, reference to SP6(6)[11] on plastic theory in design of steel structures has been made in the code which may not be adequate. Hence, it is important that a code on ductile steel frames for high seismic regions is developed early. The steel design code IS 800[12] is in the process of revision and it includes ductility provisions; it is important that the same be finalised early.

Clause 4.15.2 also mentions IS 4326 : 1976[13] for specifications of ductile detailing of RC frames. However, ductile detailing provisions for RC buildings were dropped in 1993 edition of IS 4326 since a separate code (IS 13920) was developed in 1993. Hence, mention of IS 4326 should be dropped from Clause 4.15.2.

Number of storeys (Clause 4.16) and building height (Clause 4.11 and Clause 7.6)

Prior to the 2002 version, IS 1893 provided an empirical equation for natural period as $T = 0.1n$, and hence the definition of number of storeys was important. In the new code, this empirical equation has been dropped. Instead, the empirical equations for natural period of the building now require the term "building height (*h*)" which is defined in Clause 7.6.1. On the other hand, in Clause 4.11 the term "height of structure (*h*)" is defined which is incomplete and inadequate. It is suggested that in Section 4, definition 4.11 be dropped, and a definition in lines with Clause 7.6 be added for "Building Height".

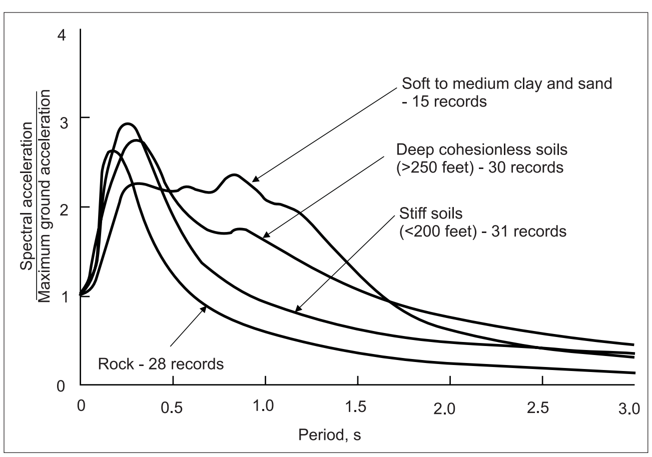

Fig 2 Effect of local soil profile on response spectrum[14]

Soft storey buildings

Clause 4.20 defines soft storey, while *Table* 5 of the code defines soft storey and extreme soft storey. Soft storey is defined as one with lateral stiffness less than 70 percent of that in the storey above, or less than 80 percent of the average lateral stiffness of the three storeys above. Extreme soft storey is defined when these numbers are 60 percent (in place of 70 percent) and 70 percent (in place of 80 percent), respectively.

This is in line with the US codes which separately define soft storey buildings and extreme soft storey buildings. However, in the US codes, extreme soft storey buildings require more stringent treatment in analysis and design as compared to soft storey buildings. In IS 1893, there is no difference between the treatment for soft and extreme soft storey buildings. Moreover, there is not much of a difference between soft storey and extreme soft storey buildings as defined in the code. Hence, it is suggested that the term "extreme soft storey" be dropped from *Table* 5.

Most Indian buildings will be soft storey buildings as per codal definition simply because the ground storey height is usually different from that in the upper storeys. Hence, the definition of soft storey needs a review. We should allow more variation between stiffness of adjacent storeys before terming a building as a "soft storey building". For instance, IS 1893 allows for more variation in the weight of the adjacent floors, as compared to the NEHRP code, before terming a building as having mass irregularity. A similar approach is needed for definition of soft storey buildings.

Definition of V_{roof}

On page 11 of the code, V_{roof} , is defined as peak storey shear force *at the roof* due to all modes considered. It is better to define it as *peak storey shear in the top storey due to all modes considered*.

Load combination 0.9DL ±1.5EL

Seismic loads are reversible in direction; in many cases, design is governed by effect of horizontal load minus the effect of gravity loads. In such situations, a load factor higher than 1.0 on gravity loads will be unconservative, and hence, in Clause 6.3.1.2, a load factor of 0.9 is specified on gravity loads in the combination 4) for RC buildings. A similar load case (0.9DL ±1.7EL) should be added in Clause 6.3.1.1 for steel structures.

Treatment for different types of soils (Table 1)

There are serious problems with *Table* 1 — in the way different types of soils are defined. Some of the soil types mentioned in this *Table* are not standard soil types as per geotechnical engineering conventions or codes. Moreover, not all types of soils are covered, leaving scope for disputes. This needs to be corrected.

It is now well established that the local soil type affects the ground motion, *Fig* 2. The code now specifies the design spectrum in terms of Type I, II, and III soils which are indirectly defined in Table 1 of the code. Note 4 of *Table* 1 mentions that the value of N is to be taken at the *founding level*. This is not consistent with the concept of site effects. The ground motion depends not just on the soil type at the founding level, but on the general soil profile at the site. For instance, the International Building Code (IBC2003) classifies the soil type based on weighted average (in top 30 m) of soil shear wave velocity, or standard penetration resistance, or soil undrained shear strength. It is best if IS 1893 also specifies the criteria based on average soil profiles. Further, "founding level" is open to dispute in case of pile or well foundations.

Seismic intensity (*Table* 2)

The seismic zone map in Indian code has been originally developed based on anticipated intensity of shaking. This is clearly outlined in the last para of page 3 of the code as: Zones II to V are associated with seismic intensity of VI (or less), VII, VIII, and IX (and above), respectively. However, *Table* 2 of the code gives "Seismic Intensity" as *Low, Moderate, Severe* and *Very Severe* for zones II to V, which is vague and contradicts a more specific mention of intensity on page 3. Hence, the row for seismic intensity in *Table* 2 should be removed.

Response reduction factor

As per seismic design philosophy, the structure is expected to sustain damage in the event of severe shaking, and hence, the seismic design force is much less than what is expected under strong shaking if the structure were to remain linear elastic. Earlier edition of the code just provided the required design force and it gave no direct indication that the real force may be much larger. The current code provides for realistic force for elastic structure corresponding to the design basis earthquake (DBE) as $A_h = (Z/2)I(S_a/g)W$ which is then divided by (R) to obtain design force. Recall that zone factor is Z for maximum considered earthquake (MCE) and it is divided by factor 2 to arrive at design basis earthquake (DBE). In other words, the maximum elastic force for the structure corresponding to maximum considered earthquake (MCE) is $ZI(S_a/g)W$ which is then divided by ($2R$) to obtain design seismic force. As suggested earlier, the terms maximum considered earthquake (MCE) and design basis earthquake (DBE) are quite confusing to the designer and it is best to drop these terms. Hence, it is recommended that the design force be given with respect to Z values directly by enhancing the values of R by a factor of 2, and by dropping the factor of 2 in the equation for A_h. That is, the maximum elastic force be given by $ZI(S_a/g)W$, and the design seismic force as $ZI(S_a/g)W/R^*$, where R^* is the new response reduction factor which is simply twice the R values given in the code at present.

Definition of R on page 14 contains the statement, "However, the ratio (I/R) shall not be greater than 1.0 (*Table* 7)". It is recommended to drop this statement. For buildings, I does not exceed 1.5 and the lowest value of R is 1.5 in Table 7 and therefore this statement does not become effective for buildings. For other structures, there could be situations where (I/R) will need to exceed 1.0, for instance, for bearings of important bridges.

Value of A_h for stiff structures

Clause 6.4.2 specifies the value of A_h as $(ZIS_a)/(2Rg)$ and adds, *"Provided that for any structure with $T \leq 0.1s$, the value of A_h will not be taken less than $Z/2$ whatever be the value of I/R"*. This statement attempts to ensure a minimal design force for stiff structures. Note that this statement is valid only when the first (fundamental) mode period $T \leq 0.1$ sec even though the code does not specify so. For higher modes, this restriction should not be imposed and this needs to be corrected in the code.

The Bureau of Indian Standards has issued a draft amendment to change the above provision from $(Z/2)$ to $(Z/4)$. This seems to have been necessitated when one considers a SMRF (Response Reduction Factor R = 5.0) with T less than 0.1 second versus an SMRF with T greater than 0.1 second. Assuming importance factor of 1.0, and zone IV (Z=0.24g): building with T=0.11 second will be designed for A_h as 0.06g, while a building with T= 0.09 second will be designed for $(Z/2)$ as 0.12g.

However, the problem is more complex than just changing $(Z/2)$ to $(Z/4)$. For instance, what happens for buildings with R-value different from 5.0, say an OMRF building (R=3.0) located in seismic zone II (Z=0.10). If importance factor is 1.0, a building T=0.11 second will be designed for a coefficient of 0.042, while a building with T=0.09 second will be designed for 0.05g or 0.025g depending on whether $Z/2$ or $Z/4$ is used, respectively. Hence, it appears to the author that the replacement of $(Z/2)$ by $(Z/4)$ is not the correct approach.

The codes have traditionally followed a different approach for very stiff buildings: they simply disallow the use of rising part of the spectrum curve between T=0 second to T=0.1 second for static analysis, and for first mode of the dynamic analysis. *Fig* 3 illustrates the suggested provision that takes care of the stiff structures adequately without any ambiguity.

Design spectrum

The variation of S_a/g with natural period (T) for different soil types is given in second para of Clause 6.4.5. However, this para and the equations for design spectrum do not go well

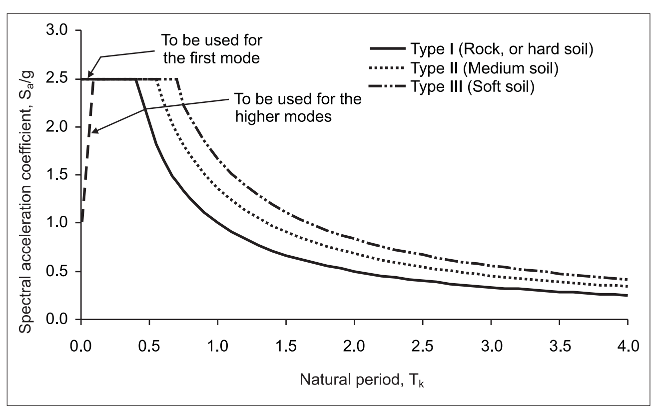

Fig 3 Suggested treatment of stiff structures with fundamental period less than 0.1 second

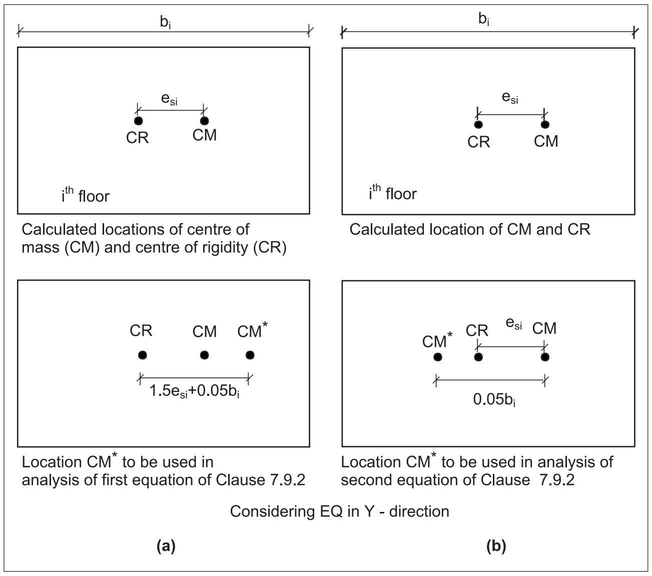

Fig 4 *(a)* First equation for design eccentricity *(b)* Second equation for design eccentricity

with first para of Clause 6.4.5 which talks of vertical coefficient. It is better to give a separate Clause number to second para including equations for design spectrum.

Response spectrum shapes in *Fig* 2 of the code are for 5 percent damping. *Table* 3 gives multiplying factors to obtain design spectrum for other values of damping. Note that the multiplication is not to be done for zero period acceleration (ZPA) and this needs to be clarified in the code.

The word "proposed" in second para of Clause 6.4.5 is misleading and should be deleted.

Table 7 for response reduction factor

Note 6 in *Table* 7 prohibits ordinary RC shear walls in zones IV and V; there are two problems with this.

(i) As per IS 13920, all structures in zones III, IV and V should comply with ductile detailing (as per IS 13920). Hence, ordinary RC shear walls are prohibited in zone III also.

(ii) There are a number of other systems that are prohibited in high zones and those are not specifically mentioned in a similar manner in this table. For instance, ordinary moment resisting frames (OMRFs) are also not allowed in zones III, IV and V as per IS 13920, and load bearing masonry buildings are required to have seismic strengthening (lintel bands, vertical bars) in high zones as per IS 4326.

In the present form Note 6 causes confusion and it is sometimes argued on the basis of this note that the code allows ordinary moment resisting frames in higher zones. Hence, it is best to drop Note 6 from *Table* 7 and in its place, a general note be added that some of the above systems are not allowed in high seismic zones as per IS 4326 or IS 13920.

Design imposed load (Clause 7.3.3)

Clause 7.3.3 requires the designer to use reduced imposed load (25 percent or 50 percent, as the case may be) for load combinations involving imposed load and seismic loads simultaneously. As a result, the load combination [1.2 DL + 1.2LL + 1.2LL] effectively reduces to [1.2 DL + 0.3LL + 1.2LL] when imposed load is 3.0 kN/m^2 or less, and to [1.2 DL + 0.6LL + 1.2LL] when the imposed load is more than 3.0 kN/m^2. This is unjustified and hence the Clause 7.3.3 should be dropped.

Damping value

In Clause 7.8.2.1, the code specifies damping to be used for steel buildings as 2 percent of critical, and for RC buildings as 5 percent of critical. This leads to a steel building being designed for about 40 percent higher seismic force than a similar RC building. While it is true that in a RC building the damping may be higher due to development of micro cracks in concrete; however, steel buildings have inherent advantages of better seismic performance. Moreover, both an RC building and a steel building may have the same types of partitions and other non-structural elements which will contribute same material damping. Hence, it cannot be justified that steel buildings should be designed for 40 percent higher design forces than similar RC buildings. Clearly, specification of damping has a direct bearing on the seismic design force level and considering this, it is recommended that the code should require both steel and RC buildings to be designed for 5 percent damping.

The code at present does not specify damping for masonry buildings; it may be specified as 5 percent of critical for masonry buildings also.

Clause 7.8.2.1 is located within the section 7.8 on dynamic analysis. Hence, the damping values specified in Clause 7.8.2.1 do not technically become applicable for buildings being analysed as per Clause 7.5.3 (static method, wherein dynamic analysis is not performed). It is appropriate if Clause on damping is inserted as a separate Clause between Clause 7.5.2 and Clause 7.5.3 so that it is applicable both for static and dynamic analyses.

Design eccentricity (Clause 7.9)

In the new edition of the code, the provisions for torsion have been changed considerably. The design eccentricity is now given as:

$$e_{di} = 1.5 e_{si} + 0.05 b_i$$

$$= e_{si} - 0.05 b_i$$

Notice that the first equation has 1.5 times the computed eccentricity, plus additional term due to accidental eccentricity (which is *5 percent of plan dimension).* The second equation does not have factor of 1.5, and sign of accidental eccentricity is different.

In the first equation, the intention is to add the effect of accidental eccentricity to 1.5 times calculated eccentricity

(*Fig* 4*(a)*) and hence, the first equation should be taken to mean having + and - sign for the second term, whichever is critical:

$$e_{di} \;=\; 1.5e_{si} \pm 0.05b_i$$

In second equation, it is expected that there is accidental eccentricity in the opposite sense, that is, it tends to oppose the computed eccentricity. Hence, factor 1.5 is not applied to the computed eccentricity, *Fig* 4*(b)*. Again, this equation also should have + and - sign for second term, and whichever is critical should be used.

$$e_{di} \;=\; e_{si} \pm 0.05b_i$$

In Clause 7.9.1, the following statement should be deleted, "However, negative torsional shear shall be neglected." This statement is needed only when second equation of design eccentricity is not specified.

The above provisions on treatment of torsion in a building require very considerable extra computations in the design of buildings. This additional effort is not commensurate with the importance of this problem and hence it is best to simplify the codal provisions on torsion.

Clause 7.9.3 says, "In case of highly irregular buildings analyzed according to 7.8.4.5, additive….". However, Clause 7.8.4.5 says that it is applicable only for regular or nominally irregular buildings. Indeed, Clause 7.8.4.5 is not applicable to buildings highly irregular in plan and hence, Clause 7.9.3 should be dropped.

Buildings with soft storey (Clause 7.10)

In this edition of the code, section 7.10 has been added for treatment of "buildings with soft storey". Note that *Table* 5 defines "soft storey" and "weak storey" buildings. Most of the time, a soft storey building is also a weak storey building, and section 7.10 really pertains to both types. This should be clarified in Clause 7.10 title and text inside.

This section gives two approaches for treatment of soft storey buildings. First approach, as per 7.10.2, is a very sophisticated approach requiring non-linear analysis (usually push over analysis). The code provides no specifications for applying this approach. In view of this, and the fact that with current state of the practice of structural engineering in India, this approach cannot be applied for routine design applications. On the other hand, the second approach of Clause 7.10.3 is quite empirical. Considerable back up research is needed to develop simple and reliable design methodology for soft storey buildings.

Foundations

In Table 1, Note 7 has been introduced which states that, *"Isolated RCC footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with N<10."* This is applicable for all seismic zones. On the other hand, Clause 7.12.1 also addresses the same issue by stating, *"In seismic zones IV and V, individual spread or pile caps shall be interconnected with ties except when individual spread footings are directly supported on rock"*. Note that the requirement of Clause 7.12.1 is applicable for all sites other than rock regardless of the N value. It will be better if Note 7 of *Table* 1 is moved to Clause 7.12.1 so that the entire issue is discussed only at one location.

As per Clause 7.12.1, ties are to be designed for an axial load (in tension and in compression) equal to $A_h/4$ times the larger of the column or pile cap load. This specification appears on the low side and needs to be reviewed for next revision of the code. Recall that many structural engineers traditionally design the ties for 5 percent of the larger of the column or pile cap load.

Compound walls

Clause 7.12.3 requires the compound walls to be designed for design horizontal coefficient A_h. Clearly, the value of A_h has to be based on the wall properties and not on the basis of the building properties. Calculation of A_h for the wall requires not only the natural period of the wall but also the response reduction factor for the wall; code does not provide these. Hence, this provision needs to be modified.

Regular and irregular configuration

In this edition of the code, irregular configuration of buildings has been explicitly defined in *Tables* 4 and 5; these tables have been adapted from similar Tables in NEHRP Code. *Figs* 3 and 4 of the code illustrate some of these irregularities. These figures are taken from NEHRP and from Jain[8]. Unfortunately, there are errors in these figures and these need to be corrected. *Fig* 5 shows the incorrect figures of the code and the correct figures from the original sources.

In *Fig* 3B of the code, irregularity due to reentrant corner is defined as $A/L > 0.15$-0.20, while in *Table* 4, it is defined as 15 percent. It will be best if the figure also shows only $A/L >$ 0.15.

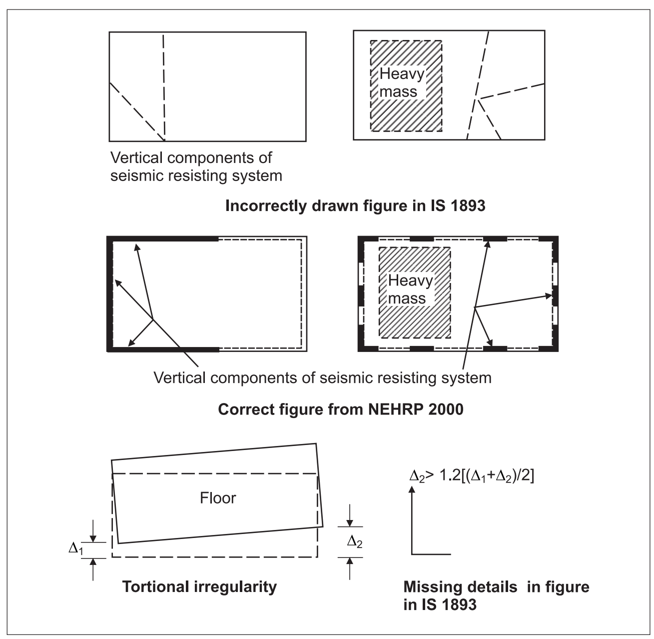

Fig 5 Corrections in *Fig* 3 A of IS 1893 (part 1) : 2002[2]

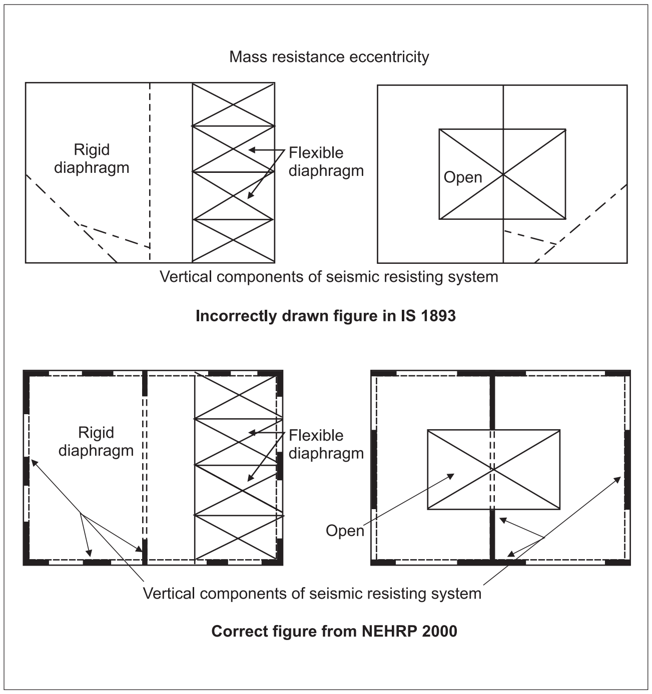

Fig 6 Corrections in *Fig* 3 C of IS 1893 (part 1) : 2002[2]

Treatment for irregular buildings

Depending on the type of irregularity, a different approach to treat the irregularity is required. For instance,

(i) in buildings with vertical irregularity, load distribution with building height is different from that in Clause 7.7.1. Hence, dynamic analysis may be required.

(ii) in buildings with plan irregularity, load distribution to different vertical elements is complex. Floor diaphragm plays an important role and needs to be modelled carefully. A good 3-D analysis is particularly important for such buildings.

(iii) in an irregular building, there may be concentration of ductility demand in a few locations and hence special care may be needed in detailing.

(iv) buildings with non-parallel systems require consideration of seismic excitation in two horizontal directions by 100 percent + 30 percent rule (Clause 6.3.2.2).

Clearly, just dynamic analysis may not solve the problem for all types of irregularities. However, the code seems to address the problem of irregularity by just requiring dynamic analysis (Clause 7.8.1). In the next revision of the code, it is important to develop more specific provisions for treatment of different types of irregularities.

Minor editorial and typographical errors

Modal participation factor (Clause 3.21)

In definition of Modal Participation Factor (Clause 3.21), there is a typographical error in the statement, "Since the amplitudes of 95 percent mode shapes can be scaled arbitrarily, ..". Phrase "amplitudes of 95 percent mode shapes" should be replaced by "amplitude of mode shapes".

Direction of horizontal ground motion in design (Clause 6.3.2.1)

A minor typographical error in this Clause needs to be corrected by replacing "direction at time" by "direction at a time".

Treatment for different types of soils (*Table* 1)

There are a number of typographical and editorial errors in *Table* 1 of the code:

- The sub-table within *Table* 1 gives values of desirable minimum values of *N*. This sub-table pertains to Note 3 and hence should be placed between Notes 3 and 4, and not between Notes 4 and 5 as printed currently.
- Caption of first column in this sub-table should read "Seismic Zone" and not "Seismic Zone level (in metres)"
- Caption of second column in this sub-table should read "Depth Below Ground Level (in metres)" and not "Depth Below Ground"
- Note 1 is also repeated within Note 4 and hence, Note 1 should be dropped.

Fundamental natural period (Clause 7.6.2)

In first printing of the code, in equation $T = 0.09h/(\sqrt{d})$, the term "h" was missed and this should be corrected.

In this Clause "brick infill panels" should be replaced by "masonry infill panels".

Clause 7.8.1

There is a typographical error in section (b) in Clause 7.8.1. *"All framed buildings higher than 12m…."* should be replaced by "*All buildings higher than 12m….*".

Number of modes (Clause 7.8.4.2)

There is a typographical error in the first sentence which reads as: *"The number of modes to be used in the analysis should*

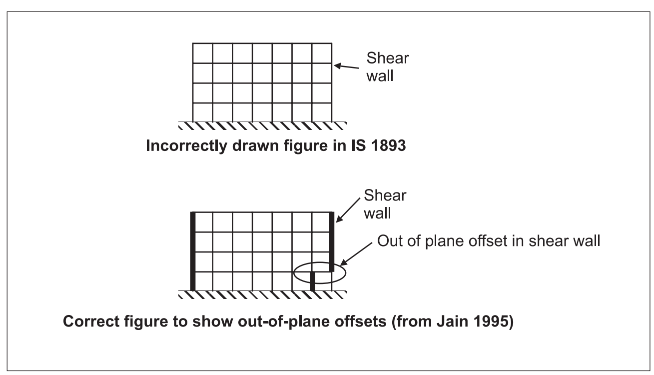

Fig 7 Corrections in *Fig* 3 D of IS 1893 (part 1) : 2002[2]

be such that the sum total of modal masses in all modes considered is at least 90 percent of the total seismic mass and missing mass correction beyond 33 percent." From this sentence, "*and missing mass correction beyond 33 percent*" should be deleted.

Last sentence of this Clause reads as, "*The effect of higher modes shall be included by ….*" It should read as: "*The effect of modes with natural frequency beyond 33 Hz shall be included by….*"

Modal combination (Clause 7.8.4.4)

In the equation for CQC method, there is a typographical error in the first printing of the code. The equation should be as follows:

$$\rho_{ij} = \frac{8\zeta^2(1+\beta)\beta^{1.5}}{(1-\beta^2)^2 + 4\zeta^2\beta(1+\beta)^2}$$

Seismic zone map

In first printing of the code, some errors got introduced in the seismic zone map.

(i) Locations of Allahabad and Varanasi have been interchanged in the map. Varanasi should be in zone III and Allahabad in zone II.

(ii) Kolkata is shown in zone IV, it should be in zone III.

Summary and conclusions

The current code is a significant upgradation of the earlier version. Several issues have been rationalised in this edition, and some of the newer concepts of earthquake resistant design introduced. However, the code needs some further improvements in both editorial issues and in some details; these are listed in this paper.

Major areas requiring improvement in the code for its next edition are as below.

- The provisions on torsion in buildings have become too cumbersome for the design office. These need to be simplified.
- The provisions for treatment of soft storey buildings are quite ad-hoc and a rational framework needs to be developed for such buildings.
- The code still lacks adequate provisions on treatment of buildings with masonry infill walls. These need to be developed.
- Different types of building irregularities require different treatment in terms of analysis and design methodology. The code currently takes a simplistic approach of requiring dynamic analysis for all irregular buildings. This needs to be addressed.

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