A review of IS:1893-1984 provisions on seismic design of buildings

[redacted]

This paper presents a comprehensive review of the Indian seismic code (IS:1893-1984) provisions on building systems. Proposals are made to upgrade the code based on some of the recent research findings and to bring it at par with the seismic codes of some of the countries with advanced seismic provisions. Inconsistencies in some of the Indian codal provisions are identified and recommendations made to overcome them.

Seismic codes around the world[1] are under constant review and revision, and the Indian code is no exception. *Indian Standard Criteria for Earthquake Resistant Design of Structures* (IS:1893), first published in 1962, has been revised in 1966, 1970, 1975 and 1984[2]. Such frequent revisions are necessitated by the continually-improving understanding of the seismic response of structures.

In India, the design seismic forces are specified by IS:1893, while the specifications for detailing of earthquake-resistant construction are given in IS:4326-1976[3], which has been revised very recently[4]. The latest version of the former, that is, IS:1893-1984[2], has incorporated changes, which remove some of the deficiencies in its previous edition. This paper contains a comprehensive review of IS:1893-1984 provisions on buildings and recommends modifications and improvements. The provisions of IS:1893-1984 regarding water tanks and chimneys have been reviewed elsewhere[5,6].

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Zone map and basic seismic coefficient

The seismic zone map of the country adopted by IS:1893-1984 was developed based on the epicentral distribution of all known past earthquakes with Richter magnitude greater than 5.0 and their iso-seismal contours. Geological and geo-physical data obtained from the tectonic maps and the aero-magnetic and gravity surveys, have also been considered subsequently. The zone map divides the country into five seismic zones, namely I to V, associated with Modified Mercalli intensity (MMI) V (and below), VI, VII, VIII, and IX (and above), respectively. This map is based on expected intensity of ground shaking in a region and does not consider the frequency of occurrence of such shaking. Hence, the map does not divide the country into areas of equal seismic "risk". For instance, while both Delhi and Mungher (Bihar) appear in zone IV, the fact that Mungher is prone to more frequent shaking is not acknowledged. Hence, the zone map needs to be revised so as to reflect the zones of similar seismic risk considering appropriately the frequency of shaking. For instance, IS:875(Part 3)-1987[7] on wind loads does consider the concept of mean return period while specifying the design wind speed in different zones.

The basic seismic coefficients α_0 recommended by the code for zones I to V are 0.01, 0.02, 0.04, 0.05, and 0.08, respectively. While observations on performance of buildings during the past earthquakes seems to have formed the basis for assigning the value of 0.08 for zone V, the value for other zones was fixed more or less arbitrarily. A number of relations to correlate the intensity of shaking (MMI) with peak ground acceleration have been proposed in the literature[8]. While these relations disagree on the actual magnitude of peak ground acceleration for a given intensity, they show a definite trend on the relative magnitudes at two consecutive intensities, *Table* 1.

Table 1 : Average peak ground acceleration (horizontal component, in cm/sec^2) as a function of Modified Mercalli intensity (MMI) from different empirical relations

Indian seismic zone	MMI	Empirical relations 1	2	3	4	5	6
I	V	15	32	31	21	22	32
II	VI	32	64	61	46	53	56
III	VII	68	130	122	104	126	100
IV	VIII	146	265	243	232	302	178
V	IX	314	538	485	519	724	316

Empirical relations[8] :

1.	Gutenberg and Ritcher, 1956	log a = 0.333 I - 0.500
2.	Neumann, 1954	log a = 0.308 I - 0.041
3.	Trifunac and Brady, 1975	log a = 0.300 I - 0.014
*4.	Trifunac and Brady, 1977	log a = 0.350 I - 0.435
*5.	Neumann, 1977	log a = 0.380 I - 0.560
6.	Murphy and O'Brien, 1977	log a = 0.250 I - 0.250

a = average peak ground acceleration (horizontal component)

I = Modified Mercalli Intensity

* Revised by Murphy and O'Biren, 1977

As the MMI increases by one, the peak ground acceleration increases by a factor of about two. Thus, the seismic coefficient for zones I to V should generally be in the ratio of 1:2:4:8:16; while at present it is 1:2:4:5:8. A recent study[9] on the acceleration time histories recorded during the 1991 Uttarkashi earthquake corroborates this observation. In general, it appears that the seismic coefficients for zones IV and V are on the lower side.

While the code defines the "basic seismic coefficient" α_0 for the seismic coefficient method, it provides a "seismic zone factor" F_0 for the response spectrum method. In fact, F_0 is simply five times α_0. Thus, there is no need to specify two different terms for each seismic zone; this sometimes causes confusion.

Shape of design spectrum

Specifying a design spectrum is one of the most important steps in the seismic design procedure[10]. In earlier years, when no strong motion records of the Indian earthquakes were available, the design spectrum suggested by Housner[10] based on response spectra of four earthquakes of southern California in U.S.A. was adopted in IS:1893. The study[9] of the strong motion records of the 1991 Uttarkashi earthquake suggests that the shape of the response spectrum of the strong ground motions in India is quite different from the code-specified design spectrum, *Fig* 1. The average acceleration spectrum from these records shows significantly greater concentration of energy in the short period range and also lesser energy in the long period range, than the code spectrum. Understandably, the attenuation rates in the Himalayas are different from those in the southern California area. Therefore, the shape of the design spectrum in the code must be revised keeping in view the ground motion characteristics of Indian earthquakes.

The philosophy of earthquake resistant design is that a structure should

(i) resist minor levels of earthquake ground motion without damage,

(ii) resist moderate levels of earthquake ground motion without structural damage, but may possibly experience some non-structural damage, and

(iii) resist severe earthquake ground motion having an intensity equal to the strongest shaking, either experienced or forecast at the site, without collapse of the structural framework, but possibly with some structural as well as non-structural damage.

In accordance with this philosophy, a structure is designed for much less base shear force than what would be required if it were to remain elastic during the most severe shaking likely at that site. The design base shear is usually established by reducing the elastic base shear demand by a response reduction factor R, which primarily consists of two factors.

The first factor is to reduce the demand force to the level of maximum yield strength of the structure. This reduction basically depends on ductility of the structure μ, its energy dissipation capacity, natural period T, and type of structure. This factor is called the ductility reduction factor R_μ, which is not constant over the whole range of natural periods[11]; it is smaller for the short period range than for the intermediate and long period ranges. A bilinear model best represents R_μ as a function of T. *Fig* 2 shows the derivation of inelastic spectra corresponding to yield condition from elastic spectra. The other factor, called the overstrength Ω, is due to the

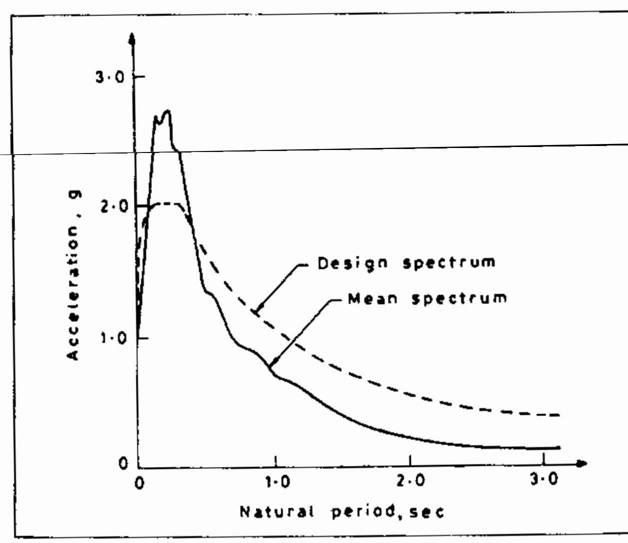

Fig 1 Comparison of the mean response sepctrum from ground motions recorded during 1991 Uttarkashi earthquake with the IS:1893-1984 design spectrum (Scaled to 1.0 g peak ground acceleration) (Ref. 9)

Tabel 2 : Response reduction factor R_w for different building systems as per UBC 1991

Sr. no.	*Building system type*	R_w
1.	Bearing wall systems with shear walls	6
2.	Frame systems with shear walls	8
3.	Frame systems with concentric bracings	8
4.	Frame systems with steel eccentrically braced frames	10
5.	Special moment-resisting frames (SMRF) - steel	12
6.	Special moment-resisting frames (SMRF) - concrete	12
7.	Intermediate moment-resisting frames (IMRF) - concrete	8
8.	Ordinary momet-resisting frames (OMRF) - steel	6
9.	Ordinary moment-resisting frames (OMRF) - concrete	5
10.	Concrete shear walls with SMRF	12
11.	Concrete shear walls with steel OMRF	6
12.	Concrete shear walls with concrete OMRF	9
13.	Massnry shear walls with SMRF	8
14.	Masonry shear walls with concrete IMRF	7

reserve strength inherently introduced in the code-designed structures, through the partial safety factors for materials and load factors amongst others, between the maximum lateral strength of the structure and the code prescribed unfactored design base shear force. A recent study[12] has shown that Ω is strongly dependant on the seismic zone and mildly dependent on the type of structure. Hence,

$$R = R_\mu \, \Omega \tag{1}$$

The value of response modification factor for a few of the structural systems as recommended by UBC 1991[13] is given in *Tables* 2 and 3.

Since non-linear behaviour is expected in the structure under strong shaking, the averaged elastic acceleration spectrum from the strong motion data from the Indian earthquakes need not be specified as the design spectrum. The detailed procedure to arrive at the inelastic response spectra for specified ductility is well established and documented in the literature. The code must replace the current spectrum by a design (inelastic) spectrum incorporating the above factors.

Attention is drawn to the shape of the design spectrum of the code for response spectrum method in the period range of $T < 0.1$ sec, *Fig* 3. While this shape is in accordance with the linear elastic response spectrum, the design spectrum for ordinary buildings usually ignores the reduction in spectral ordinates as period reduces from 0.1 sec to zero, *Fig* 4. One reason for increasing the design spectrum in the range of 0 to 0.1 sec is that R_μ is lesser in the short period range; in fact, R_μ is equal to 1 at $T = 0$. In 1975 edition of the code, the shape of the design spectrum to be used in the response spectrum method is as shown in *Fig* 5, while it was revised to the one in *Fig* 3 in 1984 edition. Further, for the seismic coefficient method, the code specifies the spectrum shown in *Fig* 6. In effect, the code is specifying two different shapes of the design spectrum for the two methods; this is not justified. It is recommended that the code specify only one spectrum and preferably one similar to *Fig* 4.

Calculation of natural period

Computation of the seismic base shear requires the fundamental natural period of vibration T of the building. However, for the building configuration adopted and the construction material chosen, it is not always possible to exactly determine T from theoretical considerations, that is, through detailed dynamic analysis. Hence, empirical formulae, obtained through experimentally-observed behaviour of buildings, are utilized. The stiffness contribution of many non-structural elements, such as in-fill masonry panels, to the building varies for structures built in different countries, due to types of construction and material. For this reason, the empirical expressions for T may be specific to each country.

The earthquake-resistant regulations practiced in the world[1] clearly reflect the expertise available on the estimation of T. In general, a steel building has been observed to possess a longer period than an identical concrete building. Also, a moment-resisting frame (MRF) building has a longer period than a shear wall building. The height of the building, or the number of storeys if the storey heights are standard, and the width of the building in the direction of the applied seismic force, are the primary parameters in the empirical representation of T. Some countries use refined expressions, which also incorporate in them relative proportions of walls and frames (Argentinean code), relative proportions of steel and concrete structural frames (Japanese code), or stiffness of the foundation structure (German code).

In the past, most countries used to allow estimation of T by any "properly" substantiated method of analysis. This may,

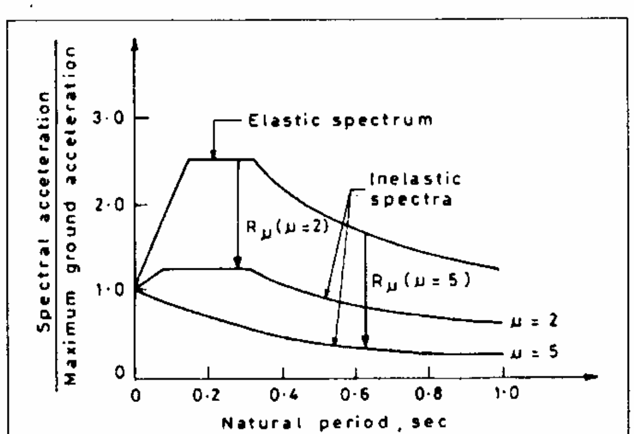

Fig 2 Derivation of inelastic spectra from elastic spectrum (Ref. 11)

Table 3 : Response reduction factor R_w for non-building structures as per UBC 1991

Sr. no.	*Structure type*	R_w
1.	Tanks, vessels or pressurised spheres on braced or unbraced legs	3
2.	Cast-in-place concrete silos and chimneys having walls continuous to the foundation	5
3.	Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels	4
4.	Trussed towers (free-standing or guyed), guyed stacks and chimneys	4
5.	Inverted pendulum-type structures	3
6.	Cooling towers	5
7.	Bins and hoppers on braced and unbraced legs	4
8.	Storage racks	5
9.	Signs and billboards	5
10.	Amusement structures and monuments	3
11.	All other self-supporting structures not otherwise covered	4

in some situations, lead designers to arrive at unusually large estimates of T from dynamic analysis of bare skeletal frames ignoring non-structural elements, such as partitions and un-reinforced brick filler walls. A higher estimate of T will lead to a lower design force. To safeguard against this, seismic codes in the world now adopt one of the following three strategies:

(i) enforce an upper bound, either a uniform bound (for example, Canadian and Turkish codes) or a zone based bound (for example, NEHRP[14]), on fundamental period that can be used in design, based on empirical T,

(ii) place a lower bound on the design seismic force based on empirical T (for example, UBC 1991 and Egyptian code), or

(iii) remove dynamic analysis as a specific option for determining base shear (for example, 1985 edition of the Canadian code[15]).

Currently, IS:1893-1984 allows the estimation of T by any one of the following methods:

(i) experimental observations on similar buildings (which almost never happens in practice),

(ii) any rational method of analysis (referring to dynamic analysis), or

(iii) using the empirical expression $T = 0.1$ N for moment-resisting frames and $T = 0.09 \frac{H}{\sqrt{d}}$ for other building systems.

It is recommended that the code should specify a lower bound on the design base shear based on empirical value of T, that is, if dynamic analysis gives a lower value of design base shear than the base shear obtained using empirical estimate of T, then the forces obtained by the dynamic analysis should be scaled-up appropriately.

The important issue then is what should be the appropriate empirical expressions for T. The two expressions presently given in the code are based on studies conducted on buildings in California, U.S.A., in the sixties and their applicability to buildings in India is doubtful[16]. In fact, these expressions are no longer included even in the American codes, which have revised the expression to $T = C_t (h_n)^{0.75}$, based on acceleration records obtained from a number of multi-storey buildings during strong earthquake motions. Here, C_t is a constant which depends on type of the structure, and h_n is height of the building. Some ambient vibration tests[16] conducted on buildings in India, indicate that the above expression recommended by NEHRP is more reasonable for predicting the time period of Indian buildings, but with a different value of C_t than what has been used in the US codes. Hence, extensive vibration tests need to be performed on a wide range of buildings in India to obtain realistic empirical expressions for T.

Soils and foundations

There are four main aspects pertaining to the influence of soils and foundations on the response of a building during an earthquake. These are:

(i) effect of local soil-type on input ground motion,

(ii) soil-structure interaction,

(iii) effect of global soil-types along the path of seismic waves from the hypocentre to the structure, and

(iv) differential settlement.

The local soil conditions in the immediate neighbor-

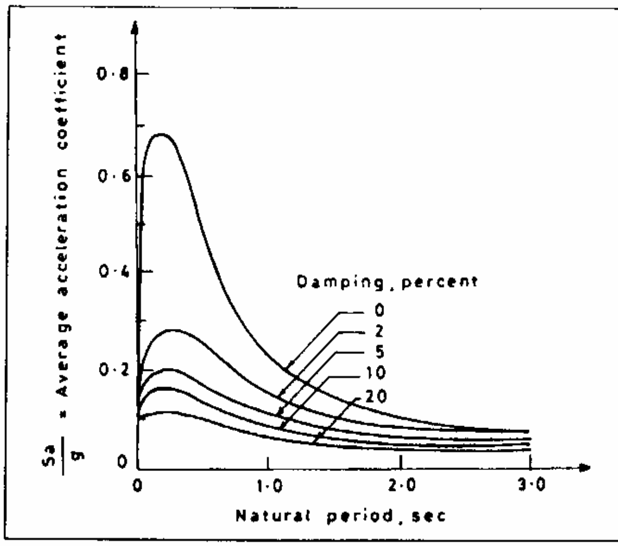

Fig 3 Average acceleration spectra as per IS:1893-1984 (Ref. 2)

Table 4 : IS:1893-1984 recommended methods for seismic design of buildings

Sr no.	*Building height*	*Seismic zone*	*Recommended method*
1.	Greater than 40 m	III, IV and V	Detailed dynamic analysis (either modal analysis or time history analysis based on expected ground motion for which special studies are required). For preliminary design, modal analysis using response spectrum method may be employed
2.	Greater than 90 m	I and II	Modal analysis using response spectrum method
3.	Greater than 40m and upto 90 m	All zones	Modal analysis using response spectrum method. Use of seismic coefficient method permitted for zones I, II and III
4.	Less than 40 m	All zones	Modal analysis using response spectrum method. Use of seismic coefficient method permitted in all zones

hood of the structure play a vital role. In general, soft soils amplify the ground motion more than stiff soils. Also, the frequency content in the ground motion gets significantly altered by the local soil medium. Acceleration spectra for different soil conditions under the same earthquake are shown in *Fig* 7[17]. Clearly, in the long period range, the design seismic force is higher for soft soils than for stiff soils, for a given time period *T*. Also, the flat portion of the spectrum is extended for softer soils. The attenuation with period *T* is much faster in case of hard soils. The acceleration spectrum of the ground motion at the base of the structure to be used in the lateral force computation is, therefore, a function of the local soil type. UBC 1991 considers this effect while specifying a separate design spectrum for each soil-type, *Fig* 8.

Soil-structure interaction leads to a higher natural period and a higher damping of the building, and thus, generally, to a reduced seismic force. Hence, it is normally conservative to ignore the effect of soil-structure interaction in the seismic design of buildings, as is done in most seismic codes in the world. NEHRP allows the designer an option to use soil-structure interaction.

Seismic waves of short and long periods experience different levels of attenuation along their path depending on the source-to-site travel path characteristics. This affects the shape of design spectrum. For instance, NEHRP[14] accounts for this effect by specifying the "effective peak acceleration" and the "effective peak velocity-related acceleration", the values of which are based on attenuation characteristics of the region. Chinese code also incorporates this effect but in a simplistic manner by distinguishing near-field and far-field ground motions. Considering lack of information about the attenuation characteristics of different regions in our country, it may not be possible to incorporate this effect in the Indian code at the present time.

Differential settlements lead to additional undesirable stresses in the structure, causing its premature failure. Buildings, whose foundation systems behave as a single entity with minimum differential settlement, behave better in earthquakes. Most countries prohibit differential settlements by an appropriate choice of foundation. Only two countries namely India and Austria, place a penalty on buildings resting on foundations vulnerable to differential settlement; that is, the building has to be designed for a larger lateral load if it is resting on a foundation susceptible to differential settlements.

The current global trend in seismic design practice is to consider the local soil conditions, to ignore, or to consider as an option, the soil-structure interaction, to neglect the variation in effect of global soil conditions along the seismic wave path, and to avoid differential settlements by a suitable choice of foundation. The present approach of IS:1893 is to ignore the first three effects, and to consider the fourth effect through soil-foundation system factor β in the expression for base shear. The value of β depends on the vulnerability to differential settlement; it is higher for soil-foundation systems which give high differential settlement. For example, a building on soft soil with isolated-untied footings has to be designed for 50 percent higher seismic force than an identical building supported on a raft foundation. While it is recognized that vulnerable foundations demonstrate poor earthquake performance due to higher differential settlement, designing the building for larger earthquake force is not a remedy. Thus, the code seems to penalize for differential settlements, rather than avoid it. In fact, vulnerable foundations are not effective in poor soils even under non-seismic conditions; thus restricting their use in good soils only.

The following strategy is suggested for adoption in the IS:1893.

(i) Include the effect of local soil conditions through different design spectra for different soil-types, *Fig* 8. Seismic codes of countries with advanced seismic provisions now incorporate this feature.

(ii) Include soil-structure interaction as an option.

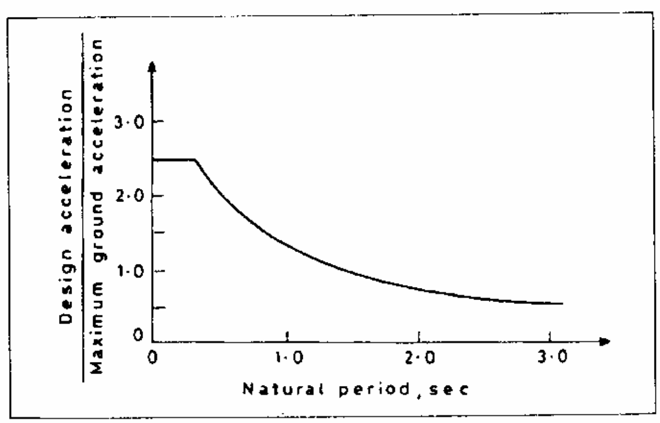

Fig 4 Shape of typical design spectrum used for buildings

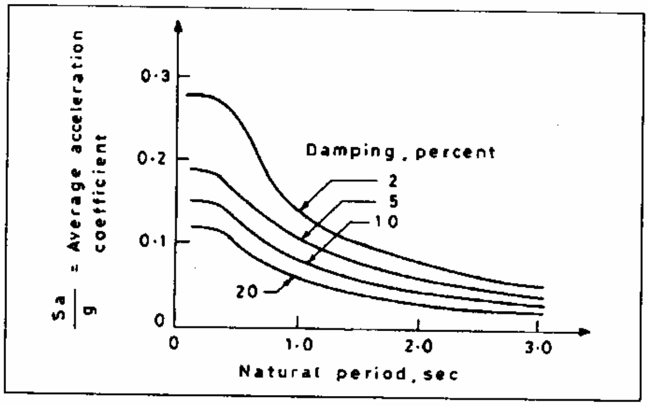

Fig 5 Average acceleration spectra as per IS:1893-1975

(iii) Due to lack of adequate information about attenuation characteristics in different regions of the country, neglect the effects of source-to-site travel path characteristics

(iv) The factor β may be dropped. Instead, a general clause may be introduced prohibiting the use of foundations liable to differential settlement, particularly for buildings in higher seismic zones.

Performance factor and ductility

The seismic design philosophy is to accept damage in a building during a severe earthquake. Hence, the code-specified design seismic force for a building is only a fraction of the seismic force that it will experience if it were to remain linear elastic during the severe ground motion. The codes, thus, rely on the inelastic response of the structure to withstand the severe ground motions. Thus, structures in severe seismic zones should necessarily be ductile. Originally, the seismic codes used to prescribe formulae for the design seismic force without any ductility factor; many users mistook this to be the maximum expected seismic force, since it does not clearly reflect the ductility contribution. However, the current trend is to recognize the varying ductility capabilities of different structural systems by appropriately reducing the computed elastic force to obtain the design seismic force. This issue has been discussed in detail earlier in this paper.

IS:1893-1984 recognizes the ductility of the building through performance factor K, given in *Table* 5 of the code, which ranges from 1.6 for non-ductile constructions to 1.0 for ductile constructions. This factor K is an unqualified and normalised value, and, hence, does not give the designer a physical feel of real ductility of the structure. Hence, it will be more appropriate for the Indian code to first prescribe the elastic seismic force, which has to be reduced to the design seismic force by using realistic values of response reduction factor.

Ductile detailing

In IS:4326-1976, the requirement for ductile detailing is evaluated based on product of the basic seismic coefficient α_0, the soil-foundation system factor β, and the importance factor I. If $\alpha_0 \beta I$ is 0.05 or more, ductile detailing is required, else ordinary detailing is considered sufficient. This implies that ductile detailing is necessary for all structures in zones IV and V, and some important structures in zone III. However, IS:1893-1984 seems to allow the option to the designer to choose either of the two types of detailing in all the zones. The choice, however, is reflected in the design seismic force through the performance factor K. It takes the values 1.6, if ordinary detailing is adopted, and 1.0, if ductile detailing is chosen. Two points arise for discussion due to this.

Firstly, $K = 1.6$ for buildings with ordinary detailing leads to an increase in the design seismic force for low "risk" buildings in zones I and II, though this was not intended. In light of the fact that seismic overstrength Ω is more in building frames designed for zones I and II[12], there is no need for either special ductile detailing for frames in these zones or for an increase in design seismic force. The ACI code, the Australian code, and the New Zealand code currently prescribe three types of detailing for buildings, depending on the seismic zone. The Indian code must relax the ductility requirements for buildings in low risk seismic zones.

Secondly, the mandatory requirement of ductile detailing in strong seismic zones is not spelt out explicitly by IS:1893, though stated in IS:4326. One could possibly argue on the basis of IS:1893 that ordinary detailing for buildings in zones IV and V is acceptable as long as $K = 1.6$ is adopted. Designing the structure for a higher seismic force does not necessarily guarantee its better seismic performance, and sometimes does not even guarantee an economical structure. Hence, IS:1893 should prohibit non-ductile constructions in severe seismic zones, as is done in the UBC 1991. This will ensure that structures, which may experience severe ground motions, are necessarily ductile.

It may be important to reconsider the IS:4326 requirement that ductile detailing is required if $\beta I \alpha_0$ is 0.05 or more. This leads to difficulties in the construction practice. For cities located in zone III, builders and engineers may have to prac-

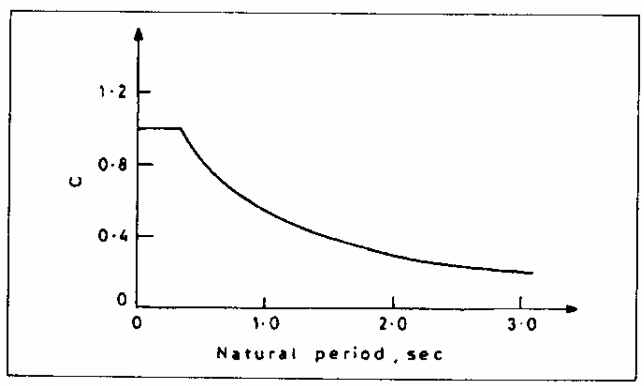

Fig 6 Coefficient *C* versus natural period *T* as per IS:1893-1984 (Ref. 2)

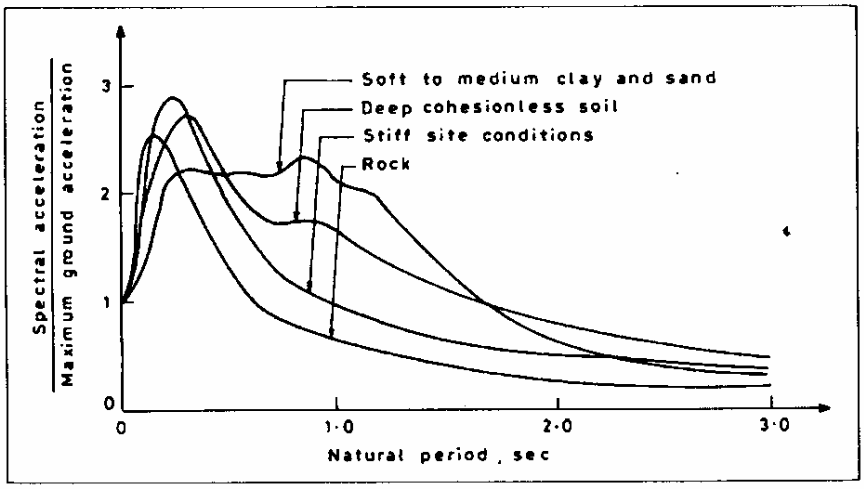

Fig 7 Normalised average acceleration spectra for different soil conditions (Ref. 17)

tice different detailing schemes. Understanding the nascent stage of seismic construction practice in India, it may be prudent for the code to provide requirements that could be based on seismic zone alone.

Lateral force procedure, design criteria and damping

The code provides two procedures for calculation of design seismic force on buildings, namely the seismic coefficient method and the response spectrum method. The design spectrum in the seismic coefficient method is given by $KC\beta I\alpha_0$, where C is given in *Fig* 6, and that for each mode in the response spectrum method is $K\beta IF_0\left(\frac{S_a}{g}\right)$, where $\left(\frac{S_a}{g}\right)$ is given by *Fig* 3. For 5 percent damping, the design spectrum in the two methods is essentially the same, except in the range of $T < 0.1$ seconds. Most seismic codes do not specify two separate spectra for estimation of the design lateral force. Codes usually specify a single design spectrum; the calculation of the design lateral force is then allowed either by the seismic coefficient method or by the response spectrum method. The same approach may be adopted by the Indian code.

No seismic design criterion is complete without specifying the damping ratio along with the design spectrum[10]. In Article F.3.1, the code gives ranges of damping coefficients for various types of structures. For reinforced concrete (RC) buildings, a range of 5 percent to 10 percent is given. The designer is free to choose any value of damping in this range; this may result in the design force to vary by as much as 20 percent. Such a variation in design force specification renders vagueness to the code. The code indicates specific damping values to be used for many structures other than buildings, for example, 2 percent for steel tanks; 5 percent for concrete tanks; 5 percent for concrete and masonry dams; and 10 percent for earth and rockfill dams. Similarly, the code should specify a specific value of damping coefficient to be used for buildings. A damping coefficient of 5 percent of critical is suggested for buildings.

The code recommends the seismic coefficient method, modal analysis method using response spectrum, and time history analysis based on expected ground motions, depending on building height and seismic zone, *Table* 4. Also, for buildings with irregular configuration, response spectrum method is recommended by the code. For buildings higher than 40m in seismic zones III, IV and V, the response spectrum method is recommended. In fact, the same article recommends the seismic coefficient method for buildings between 40m and 90m in height, in seismic zones I, II and III. These two recommendations, which contradict each other for buildings in zone III, are not justified. It appears to the authors that there is a typographical error in this regard. For instance, if in *Table* 4, serial no. 1, "Greater than 40 m" is replaced by "Greater than 90 m," then this contradiction goes away and the table appears to be more in order.

At present, the two methods (viz., seismic coefficient method and response spectrum method) could give significantly different design force for the same building; this is not expected from the code. Since the primary aim of the code is to provide a minimum design specification for the structure, the code, therefore, should really specify only one method.

Hence, for buildings, the seismic coefficient method with empirical T, is sufficient as a minimum design criteria. Other methods, for example, dynamic analysis, could be used by the designer but not to reduce the overall design seismic force. The dynamic analysis (that is, response spectrum method or time history analysis) has an edge over the seismic coefficient method for load distribution with height and to different lateral load-resisting elements, such as frames, walls, and their combinations; this is indeed useful and important for irregular and tall buildings. Therefore, for irregular and tall buildings, the code could recommend dynamic analysis procedures to be used with the same design spectrum as that used in the seismic coefficient method. However, there should remain a lower bound on design base shear on the basis of empirical T. As explained earlier in this paper, even if the dynamic analysis yields smaller design base shear, still the design base shear by seismic coefficient method should govern, but the distribution of that force should be in accordance with the relative proportions obtained by dynamic analysis.

The time history analysis is not always practicable. Though the intention for doing time-history analysis seems to be site-specific ground motion studies, which is burdened with numerous uncertainties, it is impractical for most building design projects. But, even if such studies were possible, a code-based minimum specification is still needed.

Seismic force on planar frames and floor diaphragms

The lateral force on the building is obtained by the product of its mass and the acceleration. Similarly, at each floor level, the total force acting at that floor is given by the product of its mass and acceleration. Since the acceleration can be in any direction, total mass of the floor has to be considered in either direction. This force at the floor level is then distributed to the various planar frames supporting the floor by the in-plane floor diaphragm action.

Many a time, design is carried out by analyzing two-dimensional frames individually. It has been seen by the authors that many designers in the country calculate the design seismic force directly for individual two-dimensional frames; in the process they consider only the tributary mass on the frame. This grossly under-estimates the design seismic force in both principal directions because only a part of the inertia is considered in each direction. The title of *Fig* 5A of the code sometimes adds to this confusion; it seems to suggest that seismic lateral loads should be computed frame-by-frame, instead of the whole building. To remove such a confusion, the code may clearly specify that the design base shear must be calculated for the building as a whole, and then distributed to the individual planar frames. Also, the title of *Fig* 5A of the code should read "Building" instead of "Frame".

To distribute the total seismic force to the individual planar frames of the building, IS:1893-1984 provides in article 4.2.1 (a) that if the floors are capable of providing rigid diaphragm action, the total shear in any horizontal plane shall be distributed to the various elements of the lateral force-resisting system assuming the floors to be infinitely rigid in the horizontal plane. However, in article 4.2.1 (b) the code states that if the floors are not able to provide the rigid diaphragm action, the building frames may be considered to behave independently, and may be analyzed frame-by-frame using tributary masses; this amounts to treating the moment of inertia I of the floor for bending in its own plane to be zero. In effect, it states that if I is not infinity, it is zero. It is obvious that if I is not infinity, it may be finite and far from zero, which will lead to a substantially-different distribution of seismic load among the different frames than what can be expected by assuming either $I = \infty$ or $I = 0$[18,19]. The floor slabs act as beams in their own plane, supported by the frames and/or shear walls. This in-plane floor flexibility results in unequal deformations of the different frames of a building. It is this floor-diaphragm action that dictates the distribution of lateral loads to the different frames. This needs to be recognized appropriately by the code.

The provision in article 4.2.1 (b) should therefore be replaced by a statement such as: "If the floors do not provide the rigid horizontal diaphragm action, the in-plane floor flexibility must be considered and the lateral load be distributed accordingly to the plane frames."

Performance evaluation of code-designed buildings

The seismic zone map of the code classifies the country into five zones I, II, III, IV and V, in which one may reasonably expect, in future, earthquake shocks of more or less same intensity, that is, MMI V (and less), VI, VII, VIII, and IX (and above), respectively. The code states that structures properly designed and detailed according to good construction practice will not suffer serious damage.

For the post-earthquake performance evaluation of the buildings as well as the codes, it will be very useful to know what level of damage is to be expected in code-designed buildings. For example, consider a building built in seismic zone IV which is designed as per the codal provisions. In the event of an earthquake shaking of the design intensity (MMI VIII), what level of damage should be expected ? Should there be no damage, light damage, serious damage or not-repairable damage ? Such a performance criteria would go a long way in calibrating the code after severe earthquake motions in our country.

Poor clausing in the code

The sequence and numbering of some clauses in IS:1893-1984 cause a certain amount of confusion. These need to be taken care of in the next revision.

Horizontal seismic coefficient, α_h

Articles 3.4.2.3(a) and 3.4.2.3(b) give expressions for the computation of α_h by seismic coefficient method and response spectrum method, respectively. These two expressions for α_h are completely different in form and substance,

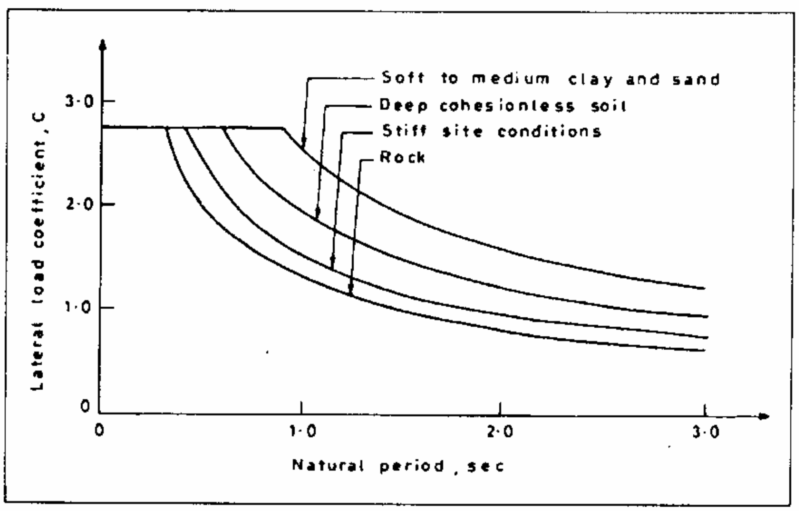

Fig 8 Lateral force coefficient for different soil types as per UBC 1991 (Ref. 12)

and hence misleading. While α_h for the former method does not include the flexibility of the structure, the α_h for the latter method does. Further, the code does not say that α_h in the response spectrum method must be calculated separately for each mode of vibration. In fact, the very need to define the variable, α_h, is not clear. It is recommended that the expression for α_h may be absorbed in the expression for base shear, and thereby simplify the code by avoiding the definition of α_h and eliminate the confusion.

Sequencing

The horizontal seismic coefficient α_h computed as per article 3.4.2.3 (a) is intended for use in the seismic coefficient method, while α_h computed as per article 3.4.2.3 (b) is intended for use in the response spectrum method. The two methods and the corresponding expressions for α_h are introduced in article 3 titled "General Principles and Design Criteria." Then, article 4 is given, titled "Buildings" containing provisions specific to buildings. Article 4.2.1.1 gives the design base shear V_B, while article 4.2.1.2 gives the load distribution with height. The code nowhere says or even indicates that article 4.2.1.1 and article 4.2.1.2 are applicable only to the seismic coefficient method, even though that is the intention. This sometimes leads a designer to use the α_h of the response spectrum method into the expression for V_B of the seismic coefficient method, and get incorrect design forces, which happen to be considerably low. Further, designers sometimes use article 3.4.2.3 (b) for load distribution with height even in the response spectrum method.

Article 4.2.2 is titled "Modal Analysis;" this refers to the response spectrum method. Both of these mean the same, but at times it is not clear to the users. In view of this dual titling, some designers associate the response spectrum method with the calculation of α_h only and then plug this value in the expression for V_B intended for seismic coefficient method.

These clauses need to be reorganized.

Modal combination and number of modes

IS:1893-1984, in article 4.2.2.2, recommends that the modal storey shear forces be combined as per the formula

$$V_i = (1-\gamma)\sum_{r=1}^{3} V_i{}' + \gamma\sqrt{\sum_{r=1}^{3}\{V_i{}'\}^2}$$

γ is a factor between 0 and 1, and depends on the height. The first term represents absolute sum of modal storey shears (good for closely spaced modes), and the second term represents SRSS (square root of sum of squares) of modal storey shears (good for well-separated modes). For buildings with torsional coupling, the natural periods may be closely spaced; in that case this formula is not adequate. However, when the natural periods are not closely spaced, the above equation is not only conservative but puts additional burden on the analysis because most commercially-available computer programs provide only for SRSS combination and do not have provisions for modal superposition which is a combination of absolute sum and SRSS. Hence, it is recommended that the code should provide for

(i) modal combination by SRSS when natural periods are well-separated, say by more than 15 percent as recommended by New Zealand Code[20], and

(ii) absolute sum of those modes whose natural periods are within 15 percent of each other and SRSS with the rest of the modes.

In the same article, the code recommends that only the first three modes need be considered. Instead, the code should state that all modes having a significant contribution to the total structural response should be included. In fact, UBC 1991 and New Zealand Code recommend that the number of modes considered should be such that at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction. The same may be adopted in the IS:1893.

Eccentricity

In article 4.2.4, IS:1893-1984 recommends that 1.5 times the computed eccentricity be taken to incorporate torsional effects. Considering the approximate nature of calculations for centre of mass and centre of stiffness and considering the probabilistic nature of the location of the centre of mass, many codes also suggest a lower bound on eccentricity that must be used. In fact, such a provision currently exists in the UBC 1991.

The lower bound is usually kept in the range of 10 percent-15 percent of the plan dimension of the building in the direction of the computed eccentricity[21]. A similar criterion may be included in IS:1893.

Incorrect solved example in the code

Article F.4, titled "Method of Using the Spectra," shows the way to use response spectrum method for calculating the design force; unfortunately, this procedure is incorrect. In response spectrum method, the idea is to use different modes. The natural period of each of these modes is used to get the corresponding $\frac{S_a}{g}$. This aspect is completely overlooked. Also, while arriving at the total horizontal seismic force (base shear) in a given mode, the modal mass is to be applied and not the total mass. The solved example seems to use the fundamental natural period and the total mass. This is essentially the same as seismic coefficient method. Further, the performance factor K has been ignored in this example. The dynamic analysis of buildings is a sophisticated procedure and requires adequate expertise. The codes cannot, and usually do not, try to teach dynamic analysis procedure. In its attempt to explain the dynamic analysis procedure, the code gives an oversimplified, and unfortunately incorrect, view of the method. Hence, it is suggested that the code must remove this example; instead a detailed example showing all important aspects of the dynamic analysis should be included in the explanatory handbook[22].

Summary and conclusions

The various provisions of IS:1893-1984 pertaining to the seismic design of buildings have been reviewed in detail. Issues have been raised at various levels - basic philosophy of design, design criteria, mechanics of design force computation, inadequacies and errors in the present codal provisions, and the stand that the code must take on some critical issues.

Some of the issues raised in this paper also affect the design of structures other than buildings.

The following is a brief summary of the recommendations of this paper.

1. The seismic zone map and the basic seismic coefficients must be revised.
2. A single design acceleration spectrum, for both seismic coefficient method and response spectrum method, consistent with the inelastic response of structures must replace the two spectra currently used.
3. The concept of ductility be brought into the code explicitly, by eliminating the performance factor and introducing the response modification factors. The overstrength of structures must also be explicitly recognized.
4. A more appropriate empirical formula for estimating the fundamental natural period, may replace the existing one.
5. A lower bound be specified for the design base shear, based on empirical estimates of the fundamental natural period. The code should not allow the dynamic analysis to reduce the design base shear. However, it may be used to obtain a more refined lateral load distribution.
6. The soil-foundation system factor be dropped. Instead, a clause should be introduced to prohibit the use of foundations vulnerable to differential settlements in severe seismic zones.
7. Damping ratio to be used for buildings be stated explicitly.
8. The code may specifically mention that seismic lateral load is first to be computed for the building as a whole and then distributed to individual lateral load resisting vertical elements depending on the floor diaphragm action.
9. For a better appraisal of the design criteria, the code may indicate the maximum level of damage expected in code-designed buildings when shaken by ground motions of design intensity.
10. Definition of the horizontal seismic coefficient α_h may be eliminated by absorbing the various factors in the formula for design seismic force.
11. Sequence and title of some of the clauses need to be revised to remove some ambiguities in the present edition.
12. Clauses on number of modes to be considered in dynamic analysis, on modal combination method, and on minimum eccentricity requirement need to be reconsidered.

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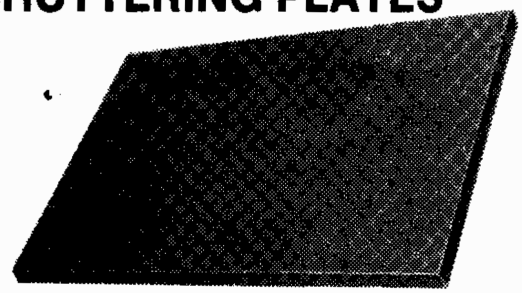

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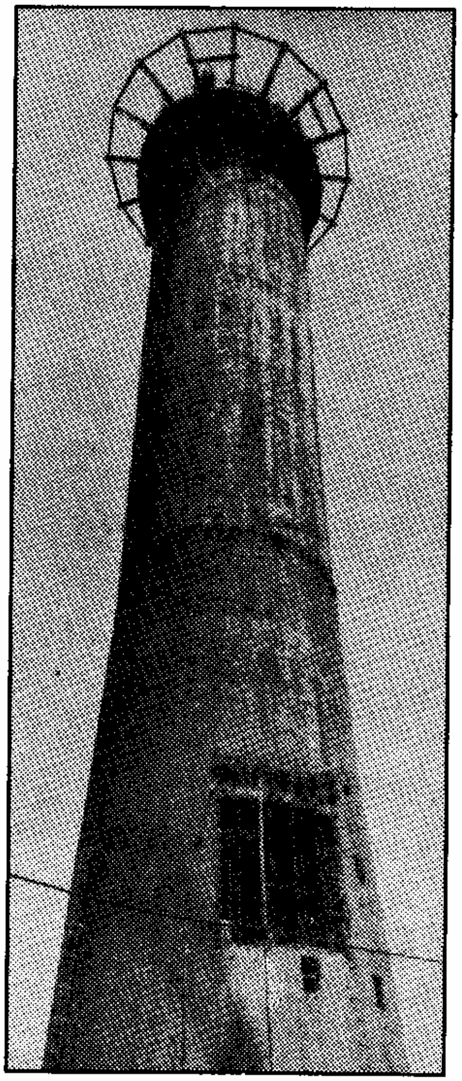

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