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RATIONAL SEISMIC COEFFICIENTS FOR EARTHQUAKE RESISTANT DESIGN

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INTRODUCTION

Since the introduction of response spectrum technique in earthquake resistant design of structures[12-15], considerable progress has been made in the earthquake regulations of the world to include, in one form or the other, the dynamic properties of earthquakes as well as those of the structures. Important differences, however, still exist in the approach for determining the seismic coefficients for design not only from one code to the other in different countries[8], but from region to region or organisation to organisation within the same country[7] and from one structure to the other in the same code[17] also. Usually base seismic coefficients are specified for different seismic zones, and certain modifying factors are included for taking other parameters into account. The response spectrum technique or dynamic analysis of structures is permissible as an alternative.

The aim of this paper is to state the various parameters affecting the design seismic acceleration at any point of structure, examine, if such parameters are given any weightage in the present codes and suggest a unified approach to obtain rational seismic coefficients for designing various structures for earthquake motions. Reference will be mainly made to 'Recommended Lateral Force Requirements and Commentary' by Seismology Committee of Structural Engineers Association of California 1968[22]; 'Earthquake Resistant Design for Civil Engineering Structures, Earth Structures and Foundations in Japan' compiled by the Japan Society of Civil Engineers, 1968[7]; and 'IS:1893-1970, Indian Standard Criteria for Earthquake Resistant Design of Structures' by Indian Standards Institution, 1970[17]. These will be briefly referred to as SEAOC, JSCE and IS: 1893 Code respectively in the discussions that follow.

EARTHQUAKE PARAMETERS

For determining motion at a given site, the following earthquake parameters are important:

(i) Magnitude of earthquake as a measure of energy release, M;
(ii) Depth of focus of earthquake, h;
(iii) Distance of site from the causative fault, D; and
(iv) The soil on which the structure will stand. This is the intervening medium between the base rock and the structure.

Several expressions are proposed in the literature for computing the peak ground acceleration at site for given M, h and D values[10, 20]. One such recent expression[20] is as follows:

$$a = a_0\, e^{-\alpha (D/h)^{1 \cdot 5}} \qquad \ldots(1)$$

where a = peak ground acceleration expected at site
a_0 = peak ground acceleration expected in epicentral area
α = attenuation factor depending upon the geological conditions in the region including the intervening medium such as soil.

From studies in Koyna area after the Koyna earthquake of Dec. 11, 1967[11], the attenuation factor was found to be 0.1 which is very low as compared with a probable

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value of $\alpha=0.26$ for several earthquakes[20]. The reason is very different geological condition in the Koyna area where more energy is transmitted through stiff rocks so that the attenuation is small.

In the resulting ground motion at site, besides the peak acceleration, the predominant frequency or number of zero crossings per second as well as the total duration of the strong motion are also important for the effective seismic intensity or damage potential of the earthquake. These two factors are however still indeterminate and can at best be estimated by comparing the conditions at the site with some recorded accelerogram in similar geological conditions. The basic data of earthquake engineering are the accelerograms which represent the net product of all the earthquake parameters[16].

STRUCTURAL RESPONSE

The response of a structure to a given accelerogram depends upon the following factors:

(i) Mass distribution in the structure;

(ii) Stiffness distribution in the structure in the elastic range;

(iii) Dissipation of energy by internal friction, that is, damping; and

(iv) The complete resistance vs deflection diagram of the structure under repetitive loading upto ultimate, changes of stiffnesses with load level, hysteretic energy absorption, and deterioration of energy reserve under cyclic loading.

The interaction of foundation soil with the structure including feed-back of energy from the structure to the ground could also be an important factor in determining structural response. Thus the foundation soil has two effects: first, modifying the ground motion and, second, interacting with the structure in its response to the ground motion.

In the ultimate behaviour of a structure, absorption or dissipation of envergy by inelastic deformations plays an extremely important role. A brittle structure like unreinforced masonry building, masonry retaining wall or a concrete or masonry dam, will depend only upon its strength in flexural and diagonal tension for resisting earthquake and other forces. But a ductile structure of reinforced concrete, prestressed concrete or steel, in which all factors leading to premature failure have been eliminated by proper design details, can depend to a great extent on its deformability during severe ground shocks.

APPROACH TO DESIGN

From the facts stated in the preceding paragraph, it is seen that there are many uncertainties as well as difficulties in determining the true ground motion and the true structural response up to ultimate stage. For design purposes, therefore, simplifying assumptions have to be introduced. Other factors which should be included in the design criteria are: the probability of occurrence of strong ground shaking, the nature of building damage which may be sustained without collapse or functional deterioration and the cost of repairing damage as compared with additional cost for making the structure earthquake resistant. The design criteria must however lead to approximately a uniform factor of safety for different structures and for different parts of the same structure unless otherwise desired due to the vulnerability of the structure or its part.

The problem of structual design may be divided into two stages, first, design based on elastic stage, and second, design based on inelastic stage permitting ductile deformations. The design approach in the two stages is given below:

(*a*) *DESIGN BASED ON ELASTIC STAGE ONLY*

This stage will apply only to brittle structures where no plastic deformations are intended to be permitted for the expected earthquake motion.

Fortunately the structural response in the elastic stage can be appropriately represented by displacement velocity and acceleration spectrum curves[14] drawn for the given earthquake accelerogram. Since choosing an appropriate accelerogram is also a tedius task, the response spectra may be idealised for general design purposes such as the average spectra[12, 21] presented in IS:1893–1970 on arithmetic scale and shown in Fig. 1. While using such average design spectra, the expected seismic intensity at a site may be accounted for by using multiplying factor F. If this approach is used, all the earthquake parameters stated earlier reduce to the choice of this multiplying factor only.

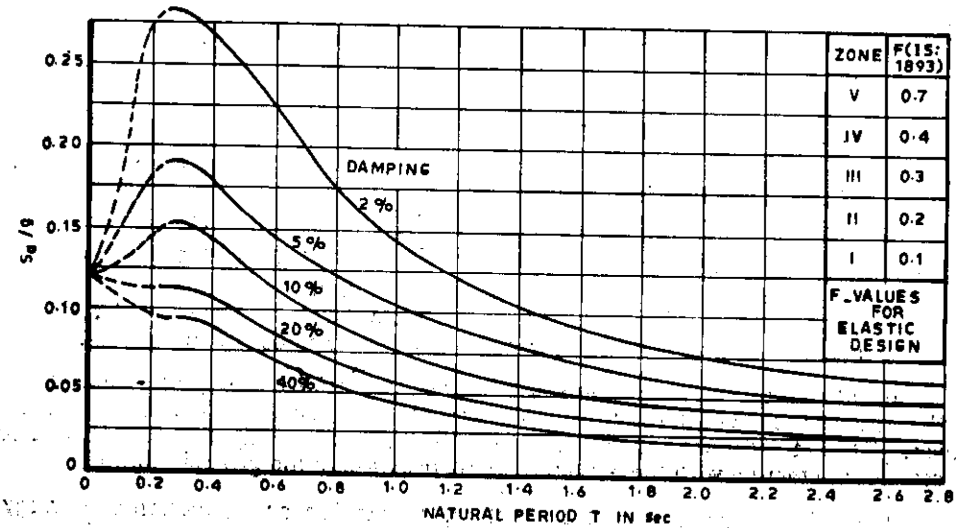

Fig. 1: Average acceleration spectra and Zone-Factors

For designing brittle structures in the elastic range, the *maximum probable earthquake* at site should serve as the basis of the factor F. If the record of such an earthquake is not available, the factor F may be chosen by comparison. For instance for Zone IV of IS Code, which is based upon Modified Mercalli Intnesity VIII, the factor F may be taken to be that for the Koyna Earthquake of Dec. 11, 1967 (Longitudinal Component). Its value is 1.35. For Zone V of IS Code, based upon M.M. IX or more, the factor F may be chosen as 2.7 which is equal to that for EI-Centro Earthquake of May 18, 1940, N-S component.

For making use of the elastic response spectra either the normal mode shapes and natural periods of vibration should be worked out and distribution of forces, shears and moments at various points may be obtained by the super-position of adequate number of modes. Alternatively an appropriate approximate method[3] may be used to obtain an equivalent single degree of freedom system. Also an appropriate value of damping has to be assumed (see Ref. 17 App. C).

The allowable stress for such structures must be based upon the cracking strength. If no factor of safety is desired over and above the probable maximum earthquake, stresses may be permitted equal to the cracking strength itself. However, since there is also the probability of occurrence of an abnormal earthquake exceeding the expectations, it will be desirable to adopt a factor of safety with regard to cracking strength.

Now it may be argued that a given seismic zone will neither have the same potential nor the same probability of occurrence of a given seismic intensity at all points in the zone. Moreover, all structures may not require the same immunity from earthquake damage in view of their purpose, expected life etc., and economic consideration also will not permit the adoption of equal strengthening measures against earthquakes. Like insurance, the premium to be paid for earthquake resistance will depend upon such factors as importance of the structure to the community, its performance during shaking, life expectancy, consequences of its failure etc. These points merit some discussion.

Since earthquake is an occasional not an every day phenomenon, structures expected to have long life like dams, large bridges, monumental buildings etc., should be designed to have greater factor of safety. Structures which are important to the community before the earthquake as well as in post earthquake period like schools, hospitals, water towers, water reservoirs etc., should be stronger. Finally, structures whose failure may result in a catastrophy such as a dam or nuclear power house or too much dislocation in the public life such as a major bridge or too long closure of a major power house, will need greater provision against risks. On the other hand, temporary structures used for storage etc. may not involve much risk requiring much less or no safety provision against earthquakes.

Rational design for earthquake motion must take the above factors into account.

(*b*) *DESIGN BASED ON DUCTILE DEFORMATION*

For single degree of freedom systems it is shown[5, 18] that when ductile deformations are permitted at constant strength (elasto-plastic) or somewhat increasing strength (bilinear), the system can resist with the given yield strength much larger ground shocks by undergoing larger plastic deformations under increasing intensity of ground motion. Inversely it could be stated that for a given large earthquake, an inelastic system will require less yield strength to resist the earthquake as compared with an elastic system having the same natural period and damping as the inelastic system in its elastic range. Hence when ductile deformations are permissible, a reduction factor may be used to calculate the required strength of the plastic system from that of an equivalent elastic system. In other words, the multiplying factor F of the elastic system could be reduced to take the ductility into account, or, inelastic response spectra could be constructed.

For multi-degree systems, however, the situation is more complex. Since a structure behaves non-linearly during inelastic deformations, the principle of superposition cannot be used. The structure must be investigated for ductility at various points by direct integration of the equation of motion[6]. Where enough analyses have been carried out such as for multistoreyed buildings, a reasonable distribution of shear force based on good inelastic behaviour may be adopted. Alternatively, the problem may be analysed by the reserve energy technique[4].

Other concepts in the approach to design are the same as for elastic design. The point mainly to be decided in this approach is 'for what ductility ratio the structure should be designed?'. This question could be posed in a different way as well. What should be the minimum earthquake as a fraction of the maximum probable earthquake for which the structure should be designed elastically ? When this is ensured, the ductility requirement can be worked out for the maximum probable as well as the abnormal earthquake and details of the structure designed accordingly.

DESIGN SEISMIC COEFFICIENT

From the discussion of various factors presented in the foregoing paragraphs, it becomes evident that besides the zone factor F, other factors are also to be chosen to arrive at the design seismic forces. For most structures, the quantities of interest in design are the shear forces and bending moments at various sections. In a very large variety of structures for which accurate elastic and inelastic analyses have been made in the literature, it is convenient to express the values of forces and moments at other sections as ratios of their values at the base of the structure whereas the base shear and base moment are determined by using appropriate design seismic coefficients. Taking into account the various factors described, the design seismic coefficient may be expressed as

$$\alpha_h = C_D\ C_P\ C_F\ (F\ S_a/g) \qquad \text{...(2)}$$

where

C_D = Reduction factor due to ductility

C_P = Performance factor to account for structural behaviour, importance, life expectancy and economic consideration

C_F = Soil-Foundation factor

F = Seismic intensity factor depending upon seismic zone

S_a = Spectral acceleration from average Response Spectra for given fundamental time period and appropriate damping

g = Acceleration due to gravity

LATERAL FORCE FOR DESIGN

Knowing the overall design seismic coefficient α_h, the complete forces and moments in a single degree of freedom system are determined. However, in the case of multidegree or continous systems, the internal forces depend upon mode participation factors, mode shapes and spectral accelerations corresponding to higher modes. The design could, however, be simplified by adopting average ratios of forces as determined by dynamic analysis for each structural type taking a wide range of parameters. In that case only the fundamental time period is needed for finding S_a in the expression of α_h. Then using the design seismic coefficient α_h, the base shear and base moment may be found as follows:

$$V_B = C_{VB}\ \alpha_h\ W \qquad \text{...(3)}$$

$$M_B = C_{MB}\ \alpha_h\ W\bar{H} \qquad \text{...(4)}$$

where

C_{VB} = coefficient for shear at base depending on the structure

C_{MB} = coefficient for moment at base depending on the structure

W = total weight of structure including appropriate proportion of live load

$\bar{H}$ = height of centre of gravity of the weight of structure above its base.

Finally the values of shear and moment along the height can be expressed in terms of the corresponding values at the base as follows:

$$V_x = C_{Vx}\ V_B \qquad \text{...(5)}$$

$$M_x = C_{Mx}\ M_B \qquad \text{...(6)}$$

where C_{Vx} and C_{Mx} are coefficients for shear and moment respectively at any point X of the structure.

The above approach has already been used in IS:1893 for designing multistoreyed buildings and stacklike structures and it could be extended to cover other structures like earth and masonry dams by analysing a wide range of these structures.

VALUES OF VARIOUS FACTORS

The values of various factors in the expression for design seismic coefficient α_h are discussed in the following and suggested values given. The other factors C_{VB}, C_{MB}, C_{Vx} and C_{Mx} which depend on the type of structure are beyond the scope of the present paper and will not be considered.

(*a*) *Seismic Force Factor* (*F*)

As stated earlier, one approach of fixing the seismic force factor F is with reference to maximum recorded accelerograms in areas of different Intensity Zones. Another approach could be to adopt the factor F for the most severe Intensity Zone and reduce it in other Zones approximately in relation to accelerations associated with various Intensities. Two such approximate scales are shown in Fig. 2[9, 11]. The ratios of accelerations for lower intensities to that of Intensity IX according to Gutenberg and Richter scale are higher than those according to Hershberger scale. Adopting the former conservatively, the ratios of average values of acceleration are given in Col. (4) in Table 1. The ratios of F values adopted in IS:1893-1970 are given in Col. (5) for comparison. It can be seen that the ratios in IS:1893 are on the conservative side particularly for zones I to III. Adopting F=2.7 for Zone V and using the ratios as per IS:1893 given in Col. (5), the values of factor F for various Zones are obtained as shown in Col. (6) which may be used for design.

The value of F for zone IV on the basis of Koyna earthquakes of Dec. 11, 1967 works out as 1.35 which compares well with the values given in Table 1.

The ratios of coefficients in various Zones in IS, SEAOC and JSCE codes are compared in Table 2. It appears that whereas IS and SEAOC are comparable, ratios in JSCE are on the high side particularly in view of the acceleration ratios shown in Col. (4) of Table 1.

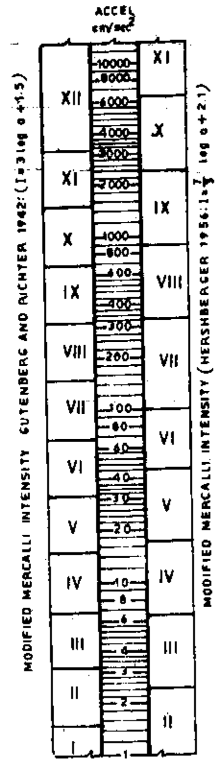

Fig. 2. Intensity vs Acceleration

TABLE 1
VALUES OF SEISMIC ZONE FACTOR (F)

Zone	M. M. Intensity	Range of acceleration cm/sec² (Fig. 2)	Ratio of Average Accn. of Zone to that in Zone V	Ratio of F values in IS: 1893–1970	Factor F using IS code ratios
(1)	(2)	(3)	(4)	(5)	(6)
V	IX (or more)	300–700	1.00	1.00	2.7
IV	VIII	150–300	0.50	0.56	1.6
III	VII	70–150	0.25	0.42	1.2
II	VI	30–70	0.10	0.28	0·8
I	V (or less)	15–30	0.05	0.14	0.4

TABLE 2
COMPARISON OF RATIOS OF SEISMIC COEFFICIENTS

Seismic Intensity	IS: 1893		SEAOC		JSCE	
	Zone	Ratio	Zone	Ratio	Zone	Ratio
IX or more	V	1.0	3	1.0	High Intensity	1.0
VII–VIII	III, IV	0.42,0.56	2	0.5	Medium Intensity	0.67 to 0.9*
V–VI	I, II	0.14,0.28	1	0.25	Low Intensity	0.5 to 0.8*

*The ratio depends upon the code—Buildings and Foundations : 1.0, 0.9, 0.8 ; Railways and Harbours 1.0, 0.67, 0.5.

(*b*) *Soil-Foundation Factor* (C_F)

To take the influence of soil-foundation system on the structural response into account, the factor C_F should increase the seismic coefficients for poor soils and poor foundations. The numerical values of C_F are not perhaps possible to find at this stage. The effect of soil has been considered in IS:1893 Code since its first formulation in 1962. The values are now given more rationally in Cl. 3.4.2, Table 2 of the 1970 Revision taking into account the effect of type of foundation also.

It may be noted that SEAOC code does not consider the effect of soil-foundation system on the seismic coefficients. But the design values of seismic coefficients in JSCE show a dependence on soil type. The multiplying factors adopted are 0.8 for hard soil, 1.0 for medium soil and 1.2 for soft soil. Here also, type of foundation is not taken into account. Since the observed behaviour of structures having different types of foundation during past earthquakes has clearly shown that pile and raft foundations are better, the coefficients in IS: 1893 are more rational and may be adopted.

(*c*) *Performance Factor* (C_P)

This factor involves structural performance along with importance, life expectancy as well as economic considerations. The value of this factor is intimately linked with the reduction factor due to ductility and will vary from one structure to the other. For those structures which are designed elastically for the probable maximum earthquake, using a factor of safety of about 1.4 over the flexural tensile strength, no increase on account of importance or long life of the structure is necessary. For ordinary structures on the other hand, like masonry retaining walls, brick buildings etc., an economy factor less than 1.0 may be used representing a reduction in the value of design seismic coefficient. That is, such structures may be designed for an earthquake much less than the probable maximum, thus accepting some damage during a severe shock.

For those structures, which can undergo plastic deformations without failure, the design factor will be reduced due to ductility ratio, C_D, discussed in (d).

IS Code specified an importance factor of 1.5 for buildings, and 2.0 for dams to take the importance of structure into account. According to JSCE factors 0.5, 1.0 and 1.5 are used for unimportant, important and very important structures of ports and harbours. For rail-bridges the corresponding factors are 0.9, 1.0 and 1.1 respectively. For other structures, no specification is laid down to cover importance. In the SEAOC code, the importance factor is not used as such, but it is covered in an overall behaviour factor K. The values in Table 3 could perhaps be adopted for C_P.

TABLE 3
VALUES OF COEFFICIENT C_P

Structure	Brittle*	Ductile**
Involving Risk (e.g. Dams, Reservoirs, Refinery Columns, Containers for inflamable gases or liquids, Atomic Power Plants)	1.0	2.0
Important (e.g. Hospitals, Water Towers and Tanks, Schools, Major Bridges, Monuments, Major Power Houses)	0.75	1.5
Ordinary	0.5	1.0
Unimportant (e.g. storage structures, temporary structures, minor retaining walls along roads, small culverts where diversion easily possible, electric or telephone poles, etc.)	0.25	0.5

* A factor of safety of 1.4 to be used with respect to tensile strength.
** A load factor of 1·4 and maximum ductility ratio of 4·0 is assumed.
In both the cases, probable maximum earthquake represented by (FS_a/g) is to be used with these values.

(*d*) *Ductility-Reduction Factor* (C_D)

It is found[5] that this factor is mainly a function of the time period of the structure and the ductility ratio. Based on the studies with regard to ElCentro and Taft earthquakes and eleato-plastic and bilinear resistance-deflection characteristics, the values given in Table 4 are suggested for determining the yield strength of the structure at which ductile deformation begins[1, 2]. For equivalent elastic design permitting $33\frac{1}{3}\%$ increase in allowable stresses or an ultimate load design using 1.4 load factor, the design seismic coefficient may further be reduced in the ratio 1/1.4.

TABLE 4
REDUCTION FACTORS FOR DUCTILITY, C_D

Ductility Ratio	Reduction Factor for Time period T equal to				
	0.5 sec. or less	1.0 sec.	1.5 sec.	2.0 sec.	2.5 sec. or more
1.0	1.0	1.0	1.0	1.0	1.0
2.0	0.35	0.45	0.45	0.59	0.33
3.0	0.21	0.32	0.34	0.42	0.20
4.0	0.16	0.26	0.28	0.28	0.14
5.0	0.12	0.22	0.22	0.20	0.10
6.0	0.11	0.20	0.20	0.16	0.08

It is possible to achieve a ductility ratio of about five in reinforced concrete and steel structures by proper design against diagonal tension or buckling failure[2]. This value could be adopted for determining the reduction factor for ductile structures.

(*e*) *Shear and Moment Coefficients* (C_{VB}, C_{MB}, C_{VX}, C_{MX})

As stated earlier, these coefficients depend mainly upon the type of structure, and its stiffness and mass distribution. These will be discussed in a subsequent paper.

ILLUSTRATIVE EXAMPLES

Example 1. Masonry dam in Zone IV having T=0.4 sec., damping 10% of critical, Rock foundation.

Since the structure is brittle, no ductility will be considered. The following values are obtained for the structure from the relevant sources:

$S_a = 145$ cm/sec^2	(Fig. 1 of paper)
$F = 1.6$	(Table 1 of paper)
$C_F = 1.0$	(Table 2 of IS: 1893)
$C_P = 1.0$	(Table 3 of paper)
$C_D = 1.0$	(Table 4 of paper)

Hence from Eq. (2), the design seismic coefficient is given by

$$\alpha_h = 1.0 \times 1.0 \times 1.0\ (1.6 \times 145/981)$$
$$= 0.236$$

Using this seismic coefficient, tension may be allowed in the masonry upto $(1/1.4) \times$ flexural tensile strength.

Example 2. Reinforced concrete multistoreyed building for hospital in Zone V having T=0.6 sec., damping 5% of critical, resting on individual footings with tie beams on Type II soil.

It is assumed that the frames are suitably designed to given a ductility ratio of 4.0 only. The following values are obtained for various factors.

$S_a = 148$ cm/sec^2	(Fig. 1 of paper)
$F = 2.7$	(Table 1 of paper)
$C_F = 1.2$	(Table 2 of IS: 1893)
$C_P = 1.5$	(Table 3 of paper)
$C_D = 0.18$	(Table 4 of paper)

Therefore, design seismic coefficient

$$\alpha_h = 0.18 \times 1.5 \times 1.2\ (2.7 \times 148/981)$$
$$= 0.132$$

When this seismic coefficient is used, critical sections in the structure should be designed to have their ultimate load capacity for the forces caused by the earthquake. If it is desired to use the elastic design method with $33\frac{1}{3}$% increase in permissible stresses or use the ultimate load method of design adopting a load factor of 1.4 as recommended in IS:1893, the seismic coefficient will be further reduced by a factor 1.4. That is, the seismic coefficient for design should be

$$\alpha_h = \frac{0.132}{1.4} = 0.094$$

CONCLUSION

It has been shown that the design seismic coefficient for structures can be expressed as a product of five factors, namely, spectral acceleration S_a for appropriate natural period and damping, seismic zone factor F, soil foundation factor C_F, performance factor C_P and ductility-ration factor C_D. Their values are closely related with permissible ductility in the structure and the factor of safety or load factor specified for design. Suitable values for the various factors are suggested as based upon past experience and current knowledge.

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