3.4 - CODES

by

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SYNOPSIS

The main provisions in Earthquake Regulations are briefly stated and compared for a few countries. The similarities and differences are highlighted. Some issues as involved in code writing are raised. Attention is drawn to the need of writing adequate specifications for earthquake resistant construction of unengineered buildings. Also it is considered that greater scientific basis is necessary in the specifications for engineered buildings so that the actual factor of safety against damage or failure in the probable earthquake may be more accurately estimated.

1. INTRODUCTION

The consequences of failure of engineering structures start a chain reaction of other effects like short circuiting and breakage of pipes leading to fire; inundation and flooding in case of dam failures; and disruption of services like water supply, sewage disposal, road traffic, hospitals, education, etc. Due to the suddenness of the direct and indirect effects put together, untold misery is caused to individuals and groups and a sort of helplessness to the community as a whole. Therefore in the whole complex system of earthquake engineering, the structural resistance of buildings, dams and bridges has to be given the highest premium. The aims of earthquake engineering may be stated as follows:

1. Protection of life
2. Continuity of vital services
3. Minimise property damage

Since in the case of most structures it will not be economically feasible to design them to remain damagefree in the probable maximum earthquakes expected during their life-span and in view of the occasional nature of such maximum events, it is usually considered adequate to aim at collapsefree rather than damagefree structures. The objectives of earthquake resistant structural design can therefore be stated as follows (SEAOC Commentary):

1. Resist minor earthquakes without damage.
2. Resist moderate earthquakes without structural damage but with some nonstructural damage.
3. Resist major earthquakes without collapse but with some structural as well as non-structural damage.

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Unfortunately, the terms minor, moderate and major have so far remained rather undefined. Therefore in achieving the above stated objectives, shelter is (vaguely) taken under energy dissipation through inelastic deformations in ductile frames and frictional sliding in brittle structures.

A review of the earthquake codes of various countries (World list 1973, Supplement 1976) shows that much of the information in the codes appears to be empirically based and not theoretically derived. In that respect the recommendations must be subject to continuous review and change as more data become available. Also, since the building systems and finishes are themselves undergoing fast changes, making the available empirical results inapplicable in their cases such updating is very necessary. It is therefore not surprising that the various codes differ a great deal from each other, which is partly due to, as it should be, differences in available materials of construction as well as technological development. The aim of this paper is to bring out the important issues involved in writing the earthquake codes. Since most earthquake resistance codes are in fact concerned with buildings, the other structures being considered as special structures, the scope of the paper is generally limited to building code.

There are certain effects of earthquakes like subsidence or liquefaction of soil, land slides, tsunamis, direct faulting, etc., which lead to severe structural damage or collapse of buildings and should need special treatment. Many times these are left to fate and are assumed not to occur while laying down specifications, thougu it is absolutely necessary that they are duly considered at the time of land use planning by proper soil exploration and other investigations at the site.

2. TYPES OF BUILDINGS

From the code point of view, the buildings could be divided into two main categories a) engineered buildings and b) unengineered buildings, their percentage being quite different in developed, developing and underdeveloped economies. In India, for instance, most dwellings constructed in small towns and villages are built according to tradition, quite unawares of the modern engineering developments and earthquake risks. Even in big towns and cities, although earthquake resistant codes are always followed in designing framed buildings of several storeys but only a few bearing wall buildings are designed or constructed to be earthquake resistant. In most cases, the code conciousness is present in Government and public sector undertakings but not so much in the private sector. The requirements of the National Building Code are still to find a place in the building byelaws of the municipalities. The situation will not be much different in most countries with underdeveloped economy. The earthquake code in such situation should cater for two types of needs. First, simple and relatively inexpensive methods of strengthening of traditional buildings should be laid down which could be followed by the available artisans with little extra effort. These details include a) devices to mechanically connect the the cross walls for integral action like reinforced concrete or timber bands at plinth, lintel and roof levels, or dowel bars every few courses at corners, b) tension-carrying elements like vertical timber posts or reinforcing bars in masonry and c) shearing elements like diagonal braces. IS:4326-1967 specifies such measures in much detail. Secondly, design forces and other requirements for good behaviour are to be specified for engineered buildings. Most codes have addressed themselves to this task and their important provisions will be discussed herein in some detail.

3. SYMBOLS AND NOTATIONS

The following symbols and notations are used in the paper:

B = plan dimension of building in a direction at right angles to the applied horizontal force (m)

C = numerical coefficient for base shear

C_p = numerical coefficient for earthquake force on parts of building

D = dimension of building in a direction of the applied horizontal force (m)

F = subsoil - foundation factor

h_i = height of level i above base of building (m)

I = importance factor

i, j = subscript denoting any level of building

K = structural performance factor

N = number of stories above external grade of building

n = number of levels above base, subscript denoting top level

Q_o = total horizontal shear acting at the base of the building

R = risk or hazard factor

S = seismic response factor, usually a function of fundamental time period of the structure

T = fundamental time period of building vibration (sec)

T_s = characteristic period of sub-soil (sec)

W = total vertical load including fixed dead load and weight of partitions plus a portion of the design live load = $\sum_{i=1}^{n} w_i$

W_p = weight of a part or portion of the building

w_i = dead load, weight of partitions and portion of design live load located or assigned at level i

x_{ir} = deflection at level i in vibration mode r

X_{ir} = any variable quantity (load, shear moment, displacement etc) at level i in mode r

Z = seismic zone coefficient

η_{ir} = modal displacement at level i in mode r

Δ = increment in seismic coefficient with height

ζ = damping ratio to critical damping

4. LATERAL FORCE FORMULA

The trend for the lateral force formula as adopted in most codes now is either to express first the total base shear of a building as a product of many independent factors and then relate the seismic force at any level to the base shear through a distribution formula, or to express the seismic force at any level in terms of the mode shape. The expressions could be written as follows:

(i) Base Shear $Q_o = (ZSKIRF)W$ ... (1)

Force at level i, $q_i = q_i\ (w_i\ h_i, h_n, D, Q_o)$ ... (2)

(ii) Force at level i, $q_i = (ZSKIR\,\eta_i)w_i$ ... (3)

The various factors do not occur explicitly in all codes. In many cases some of the factors are combined in one. Then, the values of the factors as well as expressions depend on other parameters, notably the fundamental time period T and sub-soil predominant period T_s, and vary from one code to the other. The allowable stresses under earthquake condition also vary. Some of the important factors are discussed in the following:

Factor S: This factor in fact replaces the average response spectra shape. Some expressions adopted are as follows:

$S = 0.5/T^{1/3}$ S not to exceed 1.0 (Canada) ... (4)

$S = 0.9/T$ S not to exceed 3.0 nor be less than 0.6 (Cuba) ... (5)

$S \dfrac{1}{15\sqrt{T}}$ S not to exceed 0.12 (Australia USA) ... (6)

In the Indian code, a curve S vs T is instead given which is based on the 5% damping average response spectra given by Housner. The main drawback of the expressions or a single flexibility curve is that the structural or radiation damping of the material and soil is not properly considered. It should be more rational and realistic to adopt a set of average spectra shapes for a range of damping values such as produced by Blume and Associates (1973) or others instead of the approximate expressions as above.

Factor Z: For specifying the relative seismic intensity at various places, most countries have prepared seismic zoning maps based on Modified Mercalli Intensity scales. The high seismicity zone corresponds to Intensity IX or more and two other zones are usually related to MMI VIII and VII. Areas likely to have maximum intensity VI or less are usually left out of seismic design requirement except for very important and hazardous structures like storage dams and atomic power plants. The zone factor Z is either indicative of the seismic coefficient for rigid structures, like 0.1, 0.05, 0.025 for the three zones, or their relative seismic value is indicated like 1.0, 0.5, 0.25 and the actual value is built into factor S. Thus for comparison of seismic coefficients the product SZ will be appropriate. Values of SZ for some of the countries are compared in Fig. 1 for the high seismicity zone.

For the other two zones the values usually bear the ratio of 1/2 and 1/4 to that of the severe zone, but in some cases higher ratios are adopted such as the following:

India 5/8 and 1/2; Japan 0.9 and 0.8; New Zealand 5/6 to 2/3.

The main issue involved here is howfar the seismic zoning carried out on the basis of a single parameter describing the severity of ground motion is reliable without consideration of the time element. How do we define minor, moderate and major earthquakes for a seismic zone ? The seismic zoning maps will be enhanced in their value if they could either incorporate the probability of occurrence of certain acceleration values or conversely could specify probable accelerations over periods of say 200, 100 and 50 years.

Factor K: Past experience of building damage during earthquakes such as in Caracas, Venezuela has demonstrated the superiority of certain structural systems over the others in the matter of post-cracking and post-yielding behaviour. The factor K is specified to take this performance into account, putting a higher premium on the brittle structural systems. More and more attention is being given to this factor in the revisions of the earthquake codes. Some of the values specified are shown in Table 1. Moment resistant ductile space frames have the best energy dissipation particularly where the plastic hinges form in beams. Coupled shear walls designed for ductility by hinging in the spandrel beams are as efficient, but single cantilever shear walls have limited ductility. Likewise buildings with a box system and frames with infill masonry panels will be stiffer and less ductile. Unreinforced masonry is heavy, rigid and brittle, hence K-value specified as high as 4.0 against 0.67 for the ductile space frames. A reduction factor of about 6.0 below the elastic response value is envisaged for the ductile frames. Using the allowable stress 33% in excess of normal stresses, a load factor of about 1.4 is available upto yield, the reduction due to ductility alone being about 6.0/1.4= 4.3. To achieve this reduction factor, a ductility ratio of 4 to 5 is indeed called for to make the energy capacity adequate.

Factors I and R: Many codes now specify an importance cum risk factor I. Some specify a risk factor R separately from importance factor I. Larger coefficients ($I > 1.0$) are specified for structures of post earthquake importance like hospitals, police stations, fire stations, telephone centres and emergency relief stores and buildings of large assembly like schools, cinemas, etc. Similarly risk factor $R > 1.0$ is applicable to buildings the failure or collapse of which may lead to hazardous conditions like containers of inflammable or poisnous gases, dams and atomic power plants. For the latter two types of structures, of course, special seismicity studies are called for and the factors are only for preliminary design. An idea of the values of these factors can be obtained from Table 2. In view of higher seismic coefficient, the important structure are expected (though not certain) to remain functional after the earthquake their ductility demand being comparatively less than other buildings.

Soil Foundation Factor F: There are two aspects of the subsoil effects to be considered. The first is the experience that buildings standing on soft subsoil have usually suffered greater damage than others standing on rock presumably because of magnificatiin of the ground motion passing through soft soils. The second is the consideration of predominant period of ground at a site and its relation with the fundamental period of the building so as to determine if quasi-resonance effects are likely to be caused. Another aspect is the type of structural foundation chosen for the structure since it has been observed (Niigata Earthquake) that structures founded on point-bearing piles have shown better performance under conditions of water bearing loose soil deposits. In view of these varying aspects, codes are seen to vary greatly in the specification of the soil factor. A few examples are given here below:

a) The soils are usually classified as hard, medium and soft. The Indian Standards specify the soil stiffness in terms of standard penetration value N. Thus hard soil is defined to have $N > 30$, and soft soil $N < 15$. The factor F increases with softness of soil as shown in Table 3.

b) The factor F is specified as a function of ratio T/T_s, some of the expressions used are (Australia, U.S.A.)

$$F = 1.0 + \frac{T}{T_s} - 0.5 \left(\frac{T}{T_s}\right)^2 \text{ for } \frac{T}{T_s} \leq 1.0 \qquad \ldots (7)$$

or

$$F = 1.2 + 0.6 \frac{T}{T_s} - 0.3 \left(\frac{T}{T_s}\right)^2 \text{ for } \frac{T}{T_s} > 1.0 \qquad \ldots (8)$$

with the provision that $0.5 \leq T_s \leq 2.5$,
and that $F = 1.5$ if T_s is not properly established; T is to be taken equal to or more than 0.3 sec.

In Chilean code, the factor F is combined with factor S as follows:

$$SF = \frac{2T/T_s}{1+(T/T_s)^2}, \quad T > T_s \qquad \ldots (9)$$

and T_s = 0.2, 0.3, 0.9 sec for hard, medium and soft soils.

The equations (7,8) have the merit that the resonance of structure with ground wave is given credance indicating that rigid structures on hard ground and flexible structures on soft ground are particularly vulnerable to damage. The values have been so adjusted that the maximum value of F is 1.5. The only problem will be to establish T_s for the subsoil and T for the structure. The Chilean code recommends the values of T_s. Values of T are recommended by most codes based on building period measurements in California as follows:

For buildings with lateral load resisting walls or diagonal braces or box system,

$$T = 0.09\, h_n/\sqrt{D}\,,\ h_n \text{ and D are measured in metres} \qquad \dots (10)$$

For buildings having a ductile space frame only with moment resistant joints without more rigid resisting elements,

$$T = 0.1N \qquad \dots (11)$$

Alternatively, T has to be established by calculations taking all lateral load resisting elements into account.

<u>Weight W:</u> All codes specify to consider the dead load of the structure and the superimposed load of partitions for calculating the horizontal earthquake force on the structure. But there is considerable variation in the amount of live load to be taken. Several codes like that of Australia, Canada and U.S.A. donot take any live load for finding earthquake force, though while considering combined stresses in members full live load is considered in the vertical direction. Several other countries like Cuba, Chile, India, Japan and New Zealand consider a reduced proportion of the live load. The concept here is that the design live load on floors consists partly of stored material like furniture, stationery, equipment, books etc and partly of moving load like human beings and the impact caused by them. The proportion of design live load specified is more where the weight of stored material is estimated to be proportionately high. Thus the proportions considered are: Chile and India 0.25 and 0.5; Japan 1/2 and 2/3 and New Zealand 0, 1/2 and 2/3.

<u>Distribution of Earthquake Force along Height</u>. All codes provide for the increase of the effective seismic coefficient at higher elevations of the building. Most codes take the distribution proportional to $w_i h_i$ with a concentrated additional force Q_n at the top floor level in the case of more flexible structures, as follows:

$$q_i = \frac{(Q_o - Q_n)\, w_i h_i}{\sum_{i=1}^{n} w_i h_i} \qquad \dots (12)$$

There are however variations in the specifications for Q_n.

$Q_n = 0.07 \; TQ_o \not> 0.25Q_o$ but $Q_n = 0$ for T 0.7 sec (Australia, USA)

$Q_n = 0.004(h_n/D)^2 Q_o \not> 0.15Q_o$ but $Q_n = 0$ for $h_n/D \leq 3.0$ (Canada)

$Q_n = 0.1Q_o$ for $h_n/D \geq 3.0$ and zero for $h_n/D < 3.0$ (New Zealand)

The Indian Standard specifies the distribution proportional to $w_i h_i^2$ which make-s the shear more or less uniform in the lower 40% height of the building. This has been done with a view to achieve better ductile behaviour during severe shocks (Arya 1973b). The Japanese code specifies the seismic coefficient with an increment, like $0.2+\Delta$, where Δ is 0.01 for every 4m above 16m height.

The other approach specified by most East European Countries is the use of the fundamental mode shape and the distribution specified at any level is given by

$$\eta_{ir} = \frac{x_{ir} \sum_{j=1}^{n} w_j x_{jr}}{\sum_{j=1}^{n} w_j x_{jr}^2} \qquad \text{(U.S.S.R., Yugoslavia)} \qquad \ldots (13)$$

For more flexible systems ($h_n/D \geq 5$ or $T \geq 0.5$ sec) consideration of second and third modes is also recommended. But the approach is still pseudo-static since the basic seismic coefficient is not based on the response spectrum and time period and damping.

For buildings of height more than 40m, the Indian Standard specifies the dynamic (modal analysis) approach for design, wherein the first three modes are to be considered, the average acceleration response spectra are to be used for the time periods and appropriate damping value, a zone factor takes care of the seismicity level and the resulting modal forces are super-imposed by the following expression

$$X_1 = (1-\gamma)\,\Sigma X_{ir} + \gamma\sqrt{\Sigma X_{ir}^2} \qquad \begin{bmatrix} h_n = 20 \text{ or less} & 40 & 60 & 90 \\ \gamma = 0.4 & 0.5 & 0.8 & 1.0 \end{bmatrix} \qquad \ldots (14)$$

5. OTHER ASPECTS

The other aspects of earthquake codes are earthquake force on parts of buildings, combination of earthquake force with other loads, combination of earthquake components, determination of overturning and torsional moments and specifications for drift and separation of buildings. The specifications are in most cases based on judgement supported by some theoretical investigations.

Earthquake Force on Parts of Buildings. The earthquake force on some parts of a building may be much higher than the structure as a whole and also the secondary resistance due to ductility may not be available. Higher seismic coefficients are required for cantilever parapets and smoke chimneys above roof level, for checking the stability of partition walls and for designing the connections of exterior filler or facia elements with the structural framing. The seismic force for design the parts can generally be written as

$$Q_p = ZIC_pW_p \qquad \ldots (15)$$

C_p gives the ratio in which the design seismic coefficient should be increased for parts. Table 5 gives the values of C_p in certain codes.

Combination of Earthquake Components. The earthquake motion of the ground consists of two horizontal and one vertical component acting simultaneously. Their amplitudes change randomly with time and their peaks are seldom in phase. Also the vertical component maximum peak is usually smaller than the other two. For design, the codes usually specify one horizontal component to be considered at a time and the vertical component is not usually considered except where stability becomes the criterion for design. However since in epicentral tracts the vertical acceleration may be relatively high, and there could be amplification of vertical motion in upper storeys of tall buildings due to high frequency content of vertical motion as well as that of axial vibrational modes, proper consideration of vertical accelerations needs greater attention.

The values of vertical seismic coefficient specified by codes vary from 1/2 to 3/4 of the horizontal seismic coefficient.

Combination of Earthquake Force with other Loads. The earthquake force makes the occasional or abnormal combination with other loads. In all codes wind and earthquake forces are not taken together. The Indian Standard does not also consider the combination of earthquake force with maximum design flood for bridges or dams and the maximum wave heights for off-shore and on-shore structures. In view of the the occasional nature of the combination, codes permit higher allowable stresses in working stress design or reduced load factor in ultimate load design. The usually specified increase in working stresses is 33 1/3% and the reduction in load factor is in the ratio 1/1.333. The Japanese code permits higher ~~sx~~ increase which goes well with the higher design seismic coefficient specified.

Overturning Moments. Once the floor level earthquake forces are known, the overturning moment at any level could be directly determined statically. Most codes are therefore silent about this calculation. However, since the seismic force distribution is an empirical fit with the seismic force distribution computed on the basis of first three modes, the moments determined by the statical equivalent forces would be higher than the actual combination. For this reason, the Canadian codes specify a reduction factor J to be applied to the statically computed moments. At base

$$J = (1.1-0.2T) \text{ but not to exceed } 1.0 \text{ nor less than } 0.8 \text{ (Canada)} \quad \ldots(16)$$

For other level i,

$$J_i = J + (1-J)(h_i/h_n)^3 \quad \ldots(17)$$

Current thinking, however, is to consider full moment without reduction. For instance the U.S.A. code of 1970 had the provision of $J=0.6/T^{1/3}$ which has been omitted in the 1974 revision.

Torsional Moment. Seismic codes recommend symmetrical plans and elevations for buildings for use in severe seismic zones. However for several reasons perfect symmetry may be difficult to achieve and eccentricity of the centre of applied lateral forces relative to centre of rigidity of the structure may occur. The torsional effects are then to be considered in the aseismic design. To account for errors in the estimation of rigidities or variations in the dead and live loads, the calculated eccentricity is recommended to be increased in many codes by 50%. In some codes a minimum eccentricity is considered even for symmetrical buildings or in addition to the computed value. This is taken as a fraction of the width of the building at right angles to the direction of the earthquake force. The most common value is 0.05B. Thus some recommendations are

Canada: $e_1 = 1.5e + .05B$ and $0.5e - .05B$ but if

$e_1 > 0.25B$, use dynamic analysis or double the torsional effects ... (18)

India: $e_1 = 1.5e$ ... (19)

Australia:
U.S.A. $e_{min} = 0.05B$ ... (20)

Drift or Lateral Deflection. Control of drift or inter-storey displacement is intended to restrict the damage to infill panels, glass panels, door and window frames etc, to check the P-Δ effect from assuming dangerous proportions and check the discomfort to occupants from motion sickness. There appears to be no agreement on the limiting values from these considerations. The values recommended in terms of storey height h_i in some codes are; Australia and Canada $.005h_i$, India $.004h_i$, Turkey $.0025h_i$, USA $.005h_i$. Another question is about the calculation of elastic drift for comparison with the allowable value. The best approach will of course be dynamic. The above drift limitations are perhaps intended to be used for statically calculated drift using the seismic coefficients.

Another question related with lateral deflections is the separation between adjacent buildings or dissimilar parts of the same building. The values recommended by some codes are: 1cm for each storey height with a minimum of 4cm; 3cm for 5m height + 2cm for each 5m addition.

6. DISCUSSION

The above description of the codes shows clearly an evolutionary process that has gone on from the first thumb-rule of uniform seismic coefficient of arbitrary magnitude to the present day empirical specifications partially backed by theoretical investigations. Having designed a structure according to the present codes, it remains any body's guess as to what is the realistic factor of safety of the structure in the probable maximum earthquake at the site, what will be the ductility demand on it, to what extent will it be damaged or deformed. What is expected to be ensured is that following these minimum standards, collapse will be avoided. But whether the post earthquake facilities expected to remain functional will remain so or not is still subject to doubt since the importance factor provided for this purpose may be inadequate.

Another important point which remains to be settled is regarding the provision of ductility in structural systems. No doubt some codes have given the details of reinforcement etc. to achieve ductility in frames, a question can be asked whether this is adequate and would ensure the necessary reduction in the seismic force. For instance the elastic response to Koyna earthquake accelerogram is compared with code based design seismic coefficient in the area in Fig. 2(Arya 1973a). The gap between the two is large and the doubt is whether the available ductility could meet the energy demand in such cases.

The effect of damping is not directly considered in the present codes and the soil-structure interaction effects are rather arbitrarily provided. A more scientific approach is called for in rewriting the codes wherein dynamic analysis using standardised spectra shapes and seismic zone factors including probability of peak accelerations or velocities should find a place along with reduction factors related with achievable ductilities.

7. CONCLUSION

Two types of code specifications are required. One for non-engineered traditional constructions and the other for engineered buildings. The present code provisions are pseudo-static in nature, empirically based not theoretically derived. There is a need to make the specifications for engineered buildings more scientifically based so that better estimate of factor of safety against functional and structural failure could be obtained.

REFERENCES

1. Arya A.S. (1971), "Rational Seismic Coefficient for Earthquake Resistant Design", Bull. Indian Society of Earthquake Technology, Vol. 8, No. 4, March, pp. 178-187.

2(a). Arya A.S. (1973), "Inelastic and Reserve Energy Analysis of Multi-storeyed Buildings", Fifth World Conference on Earthquake Engg., Rome, June 1973.

2(b). Arya A.S. (1973), "Seismic Behav-iour and design of Multi-storeyed Steel Buildings" National Conference on Tall Buildings, New Delhi, Jan. 22-24, 1973.

3. Blume J.A. and Associates, Recommendations for Shape of Earthquake Response Spectra , U.S.A.E.C., Washington D.C., Wash 1254, Feb. 1973.

4. Australia (1976), Draft Code for the Design of Earthquake Resistant Buildings, Standards Association of Australia, August, 1976.

5. Canada (1975) National Building Code of Canada and Commentaries on Part 4, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, 1975.

6. Cuba (1964), Standards for Structural Design, Paragraph 214-Seismic Loads, Ministerio de la Construccion, Cuba, 1964, (in Earthquake Resistant Requlations, A World List 1973).

7. India (1967), Indian Standard Code of Practice for Earthquake Resistant Construttion of Buildings. IS:4326-1967, Indian Standards Institution, New Delhi.

8. India (1975), Indian Standards Criteria for Earthquake Resistant Design of Structures, IS:1893-1975 (Third Revision), Indian Standards Institution, New Delhi.

9. Japan, Building Standard Law, Building Standard Law Enforcement Order, Ministry of Construction Notification No. 1074.

10. New Zealand (1976), New Zealand Standard Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1976 Standards Association of New Zealand, 1976.

11. USSR (1969), Construction in Earthquake Regions Design Code SN&P II-A,12-69, translated from Russian by Building Research Station, Garston, England (in Earthquake Resistant Regulations, A World List 1973).

12. USA (1970), Uniform Building Code, 1970 Edu, Vol. 1, International Conference of Building Officials, USA (in Earthquake Resistant Regulations, A World List 1973). Also Uniform Building Code, 1976 edition, USA as referred in Commentary on Draft Code of Australia.

13. Earthquake Resistant Regulations - A World List 1973 and Supplement 1976, Compiled by International Association for Earthquake Engineering, 1973 and 1976.

Table 1 - Typical Values of Factor K*

Type or arrangement of Resisting Elements	Australia, U.S.A.	Canada	New Zealand
1. Buildings with a ductile moment-resisting space frame capable of resisting specified horizontal force	0.67	0.7	0.8-1.0
2. Buildings with dual bracing system consisting of a ductile moment-resisting space frame and shear walls or braced frames	0.80	0.7-0.8	0.8-1.0
3. Building with a box system or buildings with ductile frame having masonry infills	1.33	1.3	1.66
4. All other building framing systemse except as defined above	1.00	1.00	-
5. Buildings of Non-ductile material or construction including unreinforced masonry	4.00 (Australia)	2.00	-

* Cases are more detailed and varied, all are not listed here.

Table 2 - Importance Factor I

Country	Temporary	Ordinary	Public Assembly	Post EQ importance	Hazardous
Australia	1.0	1.0	-	1.5	-
Canada	1.0	1.0	1.0	1.3	-
Cuba	0.0	0.75	1.0	1.0	-
Chile	0.8	1.0	-	1.2	-
India	1.0	1.0	1.5	1.5	2.0 to 6.0
New Zealand	1.0	1.0	1.3	1.6	2.0, 3.0
U.S.A.	1.0	1.0	1.25	1.5	-
U.S.S.R.	0.5	1.0	1.5	2.0	-

Table 3 - Comparison of Factor F

Country	Factor F for soil		
	Hard	Medium	Soft
Canada	1.0	1.3	1.5
Cuba	1.0	1.0	1.5
India*	1.0	1.3	1.5
New Zealand	1.0	1.0	1.1, 1.25, 1.3 for zones
U.S.S.R.	0.5	1.0	2.0

* The factor is further dependent on the type of foundation element, less for raft and piles and more for inditual or strip footings.

Table 4 - Values of F as function of T/T_s

T/T_s	0.1	0.3	0.5	1.0	1.5	2.0	2.5
F(Eq.7,8)	1.1	1.26	1.38	1.5	1.425	1.2	0.825
F(Eq.9)	1.0	1.0	1.0	1.0	$\frac{3}{3.25}$	0.8	$\frac{5}{7.25}$

Table 5 - Sample Values of C_p

Part	Value of C_p in Code			
	Australia and U.S.A	Canada	India	New Zealand
1. Walls, exterior or interior, filler or partitions, or bearing, force acting normal to wall face	1.67	2.0	-	2 to 12
2. Cantilever smoke chimneys above building	8.33	2.0-3.0	5.0	4 to 8
3. Parapet walls and ornamentations	8.33	10.0	5.0	4 to 8
4. Connections of exterior wall panels with structural framing	16.67	25.0	-	6.7 to 13.3

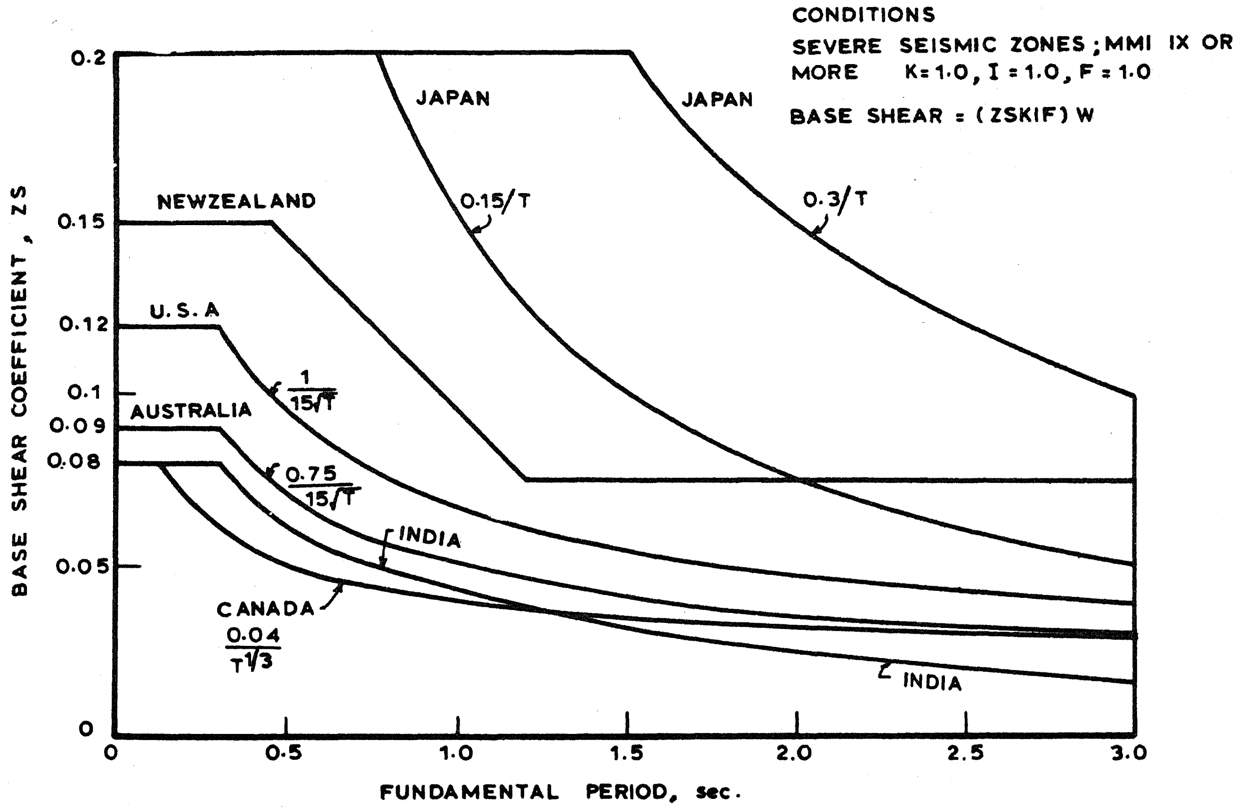

FIG. 1 _ BASE SHEAR COEFFICIENT FOR DESIGN

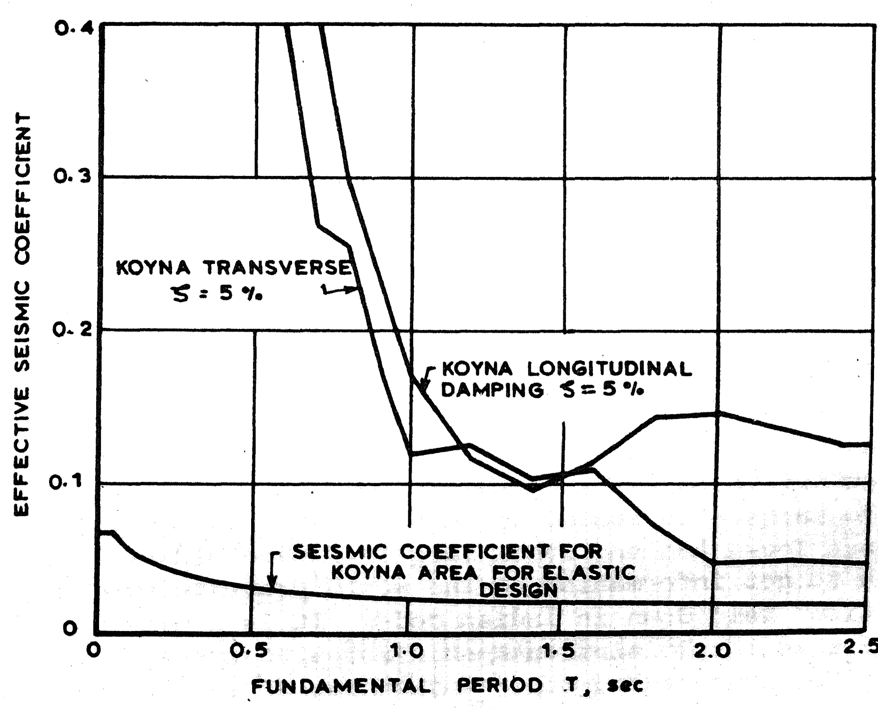

FIG. 2 _ ACTUAL RECORDED VS CODE SEISMIC COEFFICIENT

DISCUSSIONS

Recardo Duarte (Portugal)

In the last draft of the Portuguese Code (as in presentation of paper 5-55), the design eccentricities are computed according to:

$$e_1 = 1.5e + 0.05B \quad \text{(Same notations as in panel paper)}$$

$$e_2 = e - 0.05B$$

These expressions are very similar to those referred by you. However, they were introduced in the Code for different reasons:

The variation of e to 1.5e is intended to cover uncertainties in the dynamic amplification factor of torsional eccentricity, and not to account for errors in the estimation of rigidities or variations in the dead and live load (and I think codes shall not provide for errors).

The quantity $\pm$ 0.05B is an additional eccentricity to take care of the effects of the rotational component of ground motion. I think this philosophy is not particular to the Portuguese Code.

I will appreciate your comments on the subject.

M.G. Joseph (India)

The peak acceleration of Koyna (long. & Trans.) earthquake is of the order of 360 gals. The Indian Code however lays down seismic coefficients less than 0.1g. What is the philosophy behind this ?

The scale factors in Indian code for use with Housner's spectrum are also too low and do not compare well with Koyna accelerogram (Peak value). But for numerical integration, often Koyna accelerogram is used. This requires reconciliation.

I wish to get information on the world practice on "crumple section" laid down in Indian code. It is a wide expansion joint with special treatment. For a 20-storey building, the gap required to be provided is about 20 cm. Codes of other countries do not appear to specify crumple sections. What procedure do other countries follow ? Is there a substitute in place of crumple sections ? More practical of course!

Y.C. Das (India)

Dr. Arya did not touch upon drift of tall structures in his presentation. Should not the Code put limits on drift ?

Gostu Venkatesulu (India)

My comments pertain to the Codes for the design of Road Bridges:-

1. Page 3.4-0.2 states "much of the information in the codes appears to be empirically based and not theoretically derived". I consider that the codes have been based on study of the effects (which may or may not include failures). We have recently many earthquakes, like in Italy etc. I am not sure whether any study was made for the effects on the structures (separately for minor and major) located in the areas subjected to earthquakes and any inferences drawn regarding any changes in the existing codes available for bridge design.

2. Of particular interest to me is the horizontal force acting on earth retained by an abutment or a wing wall and horizontal forces acting on water surrounding pier, I feel we should derive inspiration from Japanese Codes in this connection.

3. Could the author explain if he has attempted the design of (a) an abutment, (b) a pier using well foundation in a seismic region like Assam for (a) a minor bridge and (b) a major bridge like the existing road bridge across the Brahmaputra.

Has any analysis been carried out for the safety of the existing bridge structure across the Brahmaputra in Assam?

Whether the author considers that we have to consider the horizontal force on earth behind a abutment, return wall and water round a pier for a minor bridge of 3 spans of 12 metres span with height of pier above the bed level assuming say 3.5 metres.

In a major bridge of the magnitude of the existing rail-cum-road bridge across the Brahmaputra what would be the increase in the sections and consequently in cost for the abutments, wells and piers in case horizontal force on the earth retained by an abutment in the design of abutment/wing wall is considered and horizontal force on the water surrounding a pier is also considered.

I considered that in the process of the evolution of designs from the first thumb-rule to the present day design codes (specially the IS Codes) we are making the bridge structures massive. In some cases like abutments, it may not be possible to design for the other horizontal forces specified by the I.R.C. for the design of road bridges. For a developing country like India where large amounts are spent on road bridges in the various sectors, in every five year plan, the authors of the codes for India will have to survey the sections adopted for the existing bridges and the forces experienced in earthquake and arrive at the stipulation in the various codes. You will agree we cannot be a camp follower. Codes of other countries can be taken as a guide and the brains of the country will have to put in great effort in hammering codes of design of bridges particularly horizontal forces on earth on abutments and water surrounding piers in minor and major bridges.

O.S. Srivastava (India)

On pages 02 and 03 some devices for strengthening the traditional buildings with reference to I.S. Code have been mentioned which include R.C.C. bands at plinth, lintel and roof levels. Some time back Earthquake School, Roorkee had performed some model tests which indicated that the bands at plinth and roof levels are not very effective. At roof level, the R.C.C. slab, in case of slab roof, serves to connect the cross walls. The author may kindly give his views.

According to the provisions in the revised Indian code on earthquake resistant designs, modal analysis is required not only for buildings higher than 40 metres but also in case of buildings with storey heights more than 5 metres, irregular storey heights, basements attached to part of the buildings etc. Thus most of the buildings in an Industrial complex would need to be designed by modal analysis, if provision of this code are followed. This will be a very time consuming process and would need computer analysis in each case. The author may kindly elucidate with reference to provisions in codes of other countries also.

With certain basic facts already known such as, taller the building, more its natural period and hence lesser the acceleration as per response curves, cannot certain other factors be introduced to determine coefficients for acceleration more accurately rather than resorting to Modal analysis to avoid going to computor frequently?

The I.S. Code provides determination of base shears due to first three modes of vibrations. In one of the papers read earlier in this conference, the need for considering effects of higher modes has also been pointed out. Will the author kindly throw some light on the subject based on other studies which may have been carried out ?

In this paper as well as in other references, the damping coefficient for concrete has been taken as 5% taking into account energy dissipation through inelastic deformations also. Will the author, clarify the damping coefficient to be adopted for structures where only elastic deformations can be permitted such as those located in the vicinity of quarry areas where blasting is done as frequently as daily or twice a week or so ?

Author's Closure

The author thanks the discussors for their interest in the paper and valuable comments for clarifying certain points or for supplementing the subject matter. The author's views on the various points raised are given in the following:

Mr. Duarte Ricardo's comments on the torsional eccentricities are valid. The use of the word 'uncertainties' instead of 'errors' is indeed appropriate and correct. The additional eccentricity of 0.05B in indeed explained on the basis of rotational component of ground motion.

Mr. Joseph has raised two questions. The first one seeks the reasons for the gap between the code based design seismic coefficients and the actual accelerations recorded in some past earthquakes, say Koyna in India, Dec. 11, 1967. Figure 2 of the paper also draws attention to this. The reconciliation between the two is possible on the basis of ductility and reserve energy capacity in structures. Reference in this regard is invited to Reference No. 1 at the end the paper and to the paper "Structural Dynamics in Earthquake Resistant Design" by J.A. Blume, Trans. ASCE, Vol. 125, 1960 pp 1086-1139. Regarding crumple section, there is wide agreement that adequate separation between dissimilar buildings is required to avoid damage due to pounding. The provision of weak material, that will easily crush or buckle in the event of an earthquake, is optional. Also the amount of gap to be provided varies among many countries as stated in the paper under 'Drift'. The values specified in the Indian code are based on rational dynamic deflections calculated for building of several storeys and represent probable values.

Dr. Y.C. Das is requested to refer to the section on Drift in the paper. Limits are certainly called for to avoid non-structural damage, P-Δ effects and discomfort to occupants.

Mr. Venkatesulu is mainly concerned with earthquake effects on bridges. The scope of the paper being limited to building codes, information on bridges could not be included. His querries are briefly answered here. After most major damaging earthquakes, scientific studies of the causes of damage to bridges and methods for improvement to avoid distress in future earthquakes are always made. This continuing process has led us to the present state of knowledge wherein many improvement have been possible. The Indian Standard IS:1893-1976 and earlier versions have duly recognised the dynamic effects of earthquakes on earth retained behind abutments and wing walls and water surrounding piers and provided relevent rational methods of calculating these effects. The author has had the occasion of computing such effects and designing aqueduct bridges as well as highway bridge substructures and will be most happy to repeat the exercise for bridges across Brahamputra whenever an occasion arises. Since the occurrence of the above effects is a scientific reality, they should be considered appropriately in all bridges. It is a different matter if for expediency, these effects are neglected for small bridges, but this could be done only if trial calculation show them to be negligible. There is a need to take up such studies in bridge design offices, particularly by the Ministry of Transport.

Mr. O.S. Srivastava has raised several points regarding reinforced concrete bands, complex industrial buildings, number of modes and damping in reinforced concrete. Though some what outside the actual scope of the paper, an attempt is made here to briefly answer them. The plinth and roof level bands are also useful in certain situations, the former in the case where the base soil is soft and the latter where the roof is other than solid slab. Generalised simple rules for industrial buildings are not possible at this stage in view of their complexity and lack of many analyses. Therefore, a rational modal analysis must be used. For small buildings of course the seismic coefficient method may be used for simplicity. Experience of analysis of multistory buildings of usual heights in India shows that first three modes are adequate. The actual sufficient number of modes depends on type of structure. For a masonry dam first two modes should be enough but for an earth dam even six modes may not suffice. It also depends on the forces required. For deflections even one mode may be good enough but for determining shears, many more than three modes may be needed. Five percent damping in reinforced concrete is infact achieved even in the elastic range after the concrete develops hair cracks and therefore represents a realistic value. Under plastic strain condition, a damping of even 12-15 percent could be achieved. If however crack-free concrete is desired, a damping value of 2-3% will be reasonable.