4.4 - ALLOWABLE STRESSES AND EARTHQUAKE PERFORMANCE

by

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SYNOPSIS

Determination of allowable stresses in aseismic design is approached with detailed consideration of factors in both the capacity to resist motion and in the demand of ground motion, which are generally ignored. Reconciliation of real capacity and real demand on a probabilistic basis provides the range of safety factors available against major earthquakes. A hypothetical office building and a hypothetical nuclear power plant are considered, and safety factor probabilistic estimates are presented. It is concluded that most conventional buildings are stronger than codes imply but that the possible great demands on them are such as to provide only nominal safety factors at current stress and design coefficient levels. Nuclear power plants, on the other hand, have great resistance and large safety factors at current stress and design levels. Twenty-two items that may account for actual structure performance in major earthquakes are discussed; many of these are viable research subjects.

GLOSSARY OF TERMS

C = adjusted capacity to resist ground shaking; $\breve{C}$ = median value

D = adjusted spectral acceleration (demand); $\breve{D}$ = median value

D' = raw demand prior to adjustment

f_e = the elastic stress assuming no yield point

f_y = the yield point stress

FS = factor of safety

N_C = standard geometric deviation for capacity

N_D = standard geometric deviation for demand

S = normal code working stress under permanent loads; i.e., without increase for ultimate design or for seismic forces

y = standard normal variable with zero mean and unit standard deviation

μ = ductility, or total deformation over yield deformation

σ_{FS} = the standard deviation of the factor of safety

INTRODUCTION

Since the beginning of earthquake-resistant design, it has been customary to increase allowable stresses for earthquake forces combined with other loads and forces if the amount of building material so provided is not less than that determined under nonseismic loading with normal stresses. These increases have amounted to 1/3, 1/2, and more above normal stresses. There have been various reasons for this practice besides the underlying one of not wanting to make the cost of seismic resistance too great. Some

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materials and some types of stress have increased strength under rapid loading, or short-time loading, as compared to strength under permanent static loads. Greater risk, or a lower safety factor, was considered acceptable for an extreme condition of loading with a low probability of occurrence.

The foregoing discussion has introduced such terms as normal stress, rate of loading, duration of loading, acceptable risk, extreme loading, probability, and safety factor. The latter term involves the relationship of demand and capacity and the reliability of our knowledge about each. Obviously, allowable stress must include consideration of these and other matters, such as the consequences of failure and economy; this term must also be extended to the nonlinear range and include the basic subjects of inelastic behavior, ductility, and energy absorption capacity. A ductile material deserves a much different allowable stress as compared to yield than a brittle material. Another related subject is redundancy. If there are different stress paths progressively, or if there are multiple simultaneous stress paths, the allowable stress may be approached in different ways. It is essential to examine current knowledge about these matters and then reconsider the design strategy in the context of the overall picture.

GENERAL CONSIDERATIONS

Allowable stress is an important but not the only part of capacity (C), defined here as the resistance to ground shaking, or demand (D). Both C and D are made up of many random variables, most of which may be considered as independent. Capacity may be defined as the resistance at first yield, at the point of collapse, or anywhere in between. Capacity and demand may be defined for any probability of being exceeded. Thus, for the selected definitions and levels on a deterministic basis:

$$\text{Safety Margin} = SM = C - D \quad (1)$$

$$\text{Factor of Safety} = FS = C/D \quad (2)$$

However, C and D are random variables and may have various values depending on the probabilities associated with them. It has been found that D can be well modeled as a lognormal distribution, and C as a lognormal or Weibull distribution (1). Using the convenient lognormal form for each variable:

$$C = \breve{C}\, N_C^{\,y} \quad (3)$$

$$D = \breve{D}\, N_D^{\,y} \quad (4)$$

Taking the natural logarithms of Equations (3) and (4),

$$\ln C = \ln \breve{C} + y \ln N_C \quad (5)$$

$$\ln D = \ln \breve{D} + y \ln N_D \quad (6)$$

FS involves the joint probability of ln C - ln D, for which joint distribution the mean is $\ln \breve{C} - \ln \breve{D}$ and the standard deviation, σ_{FS}, is

$$\sigma_{FS} = \sqrt{(\ln N_C)^2 + (\ln N_D)^2} \quad (7)$$

Thus

$$\ln FS = \ln \breve{C} - \ln \breve{D} + y\sigma_{FS} \quad (8)$$

and $$FS = e^{\ln \breve{C} - \ln \breve{D} + y\sigma_{FS}} \quad (9)$$

For the special case where $\breve{C} = \breve{D}$,

$$FS = e^{y\sigma_{FS}} \quad (10)$$

Consideration of what Equation (10) shows is often ignored. The real FS value may be more or less than unity; it is subject to variations even though $\breve{C} = \breve{D}$. For example, if $N_C = 1.5$ and $N_D = 2.2$, σ_{FS} is 0.887 and there is a 10% chance of failure even for a safety factor of 3.11. A safety factor of 0.32 provides a 10% chance of not failing. These variations can be greatly reduced with more knowledge about and control of $\breve{C}$, $\breve{D}$, N_C, and N_D.

CAPACITY AND STRESSES

The true capacity of an engineered structure is generally greater, often considerably greater under controlled conditions, than would be indicated by conventional analysis and design procedures. This is indeed fortunate because code-required seismic forces are less than can be expected within the life span of most structures. A more logical procedure of design than the conventional one is to consider real forces and real capacities together with their joint probabilities. Real capacity will be considered in this section.

1. The determination of normal stresses allowed in building codes usually involves empirical test data and judgment. Tests are conducted to determine values for failure of columns; moment resistance of beams; shear value of concrete; joint details; bolt values; wood in flexure; masonry in shear; etc. Constants are developed from the data for use in design formulas. The selected stresses are not the mean failure values but values at or near the level where few if any failures occur. It is the writer's opinion that stresses allowed before safety factors are applied are from one to two standard deviations below the mean test values.

As a hypothetical example, assume that data on specimens tested to failure range from 1200 to 800 and the mean value is 1000 units. The assigned test value is likely to be at or near 800. Safety margins will then be applied, and the code may allow only 500 units in design. The true mean value in the structure is 1000, or twice the normal design value. Should this be an earthquake stress with a 1/3 increase allowed, the design level would be 1.33 x 500, or 667, still only 66.7% of the real mean value.

2. Material specifications call for minimum test values such as yield in structural steel or reinforcing bars, a 28-day compression test for concrete cylinders, etc. The penalty for not meeting the tests and subsequent rejection can be severe, especially if the material is incorporated into the structure. The result is conservatism in specifying and providing the materials. Essentially all rebars test better than called for; the same is true of structural steel and concrete. In designing a concrete mix for a 3,000 psi specified value at 28 days, the ingredients will be selected so that only a small percentage of the test cylinders would fail below 3,000 psi. The mean strength of the concrete may be 15% to 25% above the specified (design) value at 28 days; it will be even stronger as the concrete ages and dries. Thus the mean earthquake resistance is greater than designed for.

3. The yield point and ultimate stress level of materials are considerably greater than normal stresses, and, even with the 1/3 or 1/2 increase under seismic response, the yield point is usually not reached. In addition, the actual yield stress of the materials provided may well exceed the specified minimum yield stress.

4. In many cases the materials actually provided exceed minimum specifications indicated by the design calculations. There may be overall economy in duplicating member sizes rather than having too many variations in size or shape, in using identical wall thicknesses or column sizes even though not required, etc. This factor may be greater in the United States than elsewhere because of its high ratio of labor cost to material cost.

5. Probabilistic reductions of stress in design codes (such as ϕ in reinforced concrete design) should be treated more rationally for seismic analysis for true resistance rather than compound safety factors. The true mean value with its statistical deviations about the mean is a better index of real earthquake resistance.

6. In many members, elements, and joints, the seismic stress may be a small part of the total stress. This is generally the case in beams, girders, columns, and sometimes in bearing walls. It is not true for seismic braces and shear walls. Thus, even great increases in seismic stress may have a nominal effect, or no critical effect, in the member, element or joint. Because of the relatively low normal stresses allowed by code, there may be a great reserve for seismic effects. This should be considered in determining real capacities.

7. Redundancy can greatly increase capacity by passing a local overstress along to other elements, which in turn can pass their overstress along to others. This does work and absorbs energy. It also provides a reserve capacity that would often justify greater allowable stresses. In the process, the natural period may increase and thus limit dynamic amplification. Damping may also increase. A redundant structure with ductility is highly desirable as it increases capacity.

8. Nonessential elements, planned or not, can fail without loss of the structure. They may or may not be of a structural nature, and they may or may not be part of a redundant system. An example of a nonessential element is a spandrel wall section between two wide columns or pilasters. The spandrel cracks and is thereafter unable to transmit much vertical shear between the vertical elements, which then function as vertical cantilevers without moment reversals at the level of the spandrel. If the vertical elements are adequate, the structure has longer periods but is still intact. This is not classified as a redundant element because a major change occurs in the nature of the system. A high stress or crack in the nonessential spandrel element during a major earthquake would not be critical -- the real capacity is greater.

9. There are many nonstructural elements in buildings that tend to resist motion and do much work in the process. They are often quite beneficial and have saved many traditional-type buildings (2, 3). Such elements include filler walls, partitions, stairways, and fireproofing. Although their damage may be very costly, these elements can save the structure. In modern design, the whole system should be integrated to minimize overall

losses. A building with much aid from nonstructural elements may have its basic frame stressed severely only after these elements have failed and the worst of the demand is over. In such a case, higher allowable stresses may be justified. However, situations like partial walls that cause overstress in the adjoining columns must be avoided.

10. The greatest contribution to capacity in many buildings is that of the inelastic range beyond yield where ductility and capacity to absorb energy mean the difference between some damage and collapse. Very few existing buildings except nuclear power plants have the capacity to resist major earthquakes in the elastic range. Thus, allowable stress becomes academic and the criterion is allowable ductility. The Reserve Energy Technique (RET) was developed to solve this problem by taking advantage of all materials and elements through yield to their ultimate values and equating the maximum kinetic energy demand based upon spectral velocity to the strain potential and work capacity of the inelastic element (4, 5). It is not necessary to idealize elastoplastic or any other type of response, but this is often done for convenience. Actually, most inelastic systems have much more resistance than elastoplastic systems. A bilinear softening system is often more appropriate and less conservative. The more resistance provided in the elastic system with greater forces or lower allowable stresses, the less need for severe inelastic excursions and the greater value in the inelastic range should it be needed. We thus come again to the interaction between demand and capacity and the need to consider stresses in a realistic manner as part of the overall system.

RET can be used in a form that relates directly to stress by first computing stress without regard to any yield level. Then, under an assumption of elastoplastic (or any other type of inelastic) behavior, the allowable stress and allowable ductility are reconciled. For the elastoplastic case under the assumption of energy preservation,

$$f_e = f_y \sqrt{2\mu - 1} \qquad (11)$$

Thus, with μ representing the maximum allowable ductility, f_e becomes the allowable stress. If a safety factor is desired, f_e should be reduced accordingly. In addition, the possibility of prior buckling failure must be considered. The better procedure, especially for a complex structure, is to plot realistic shear-deformation diagrams and use RET. This provides a graphical picture of the deformation and the damage to each element.

The real capacity as related to normal code stress, S, is a function of all of the items shown above that are appropriate to each specific case. Most engineered buildings have much more capacity than indicated by codes and normal stress levels. This largely explains why most engineered structures survive earthquakes of much greater intensity than code forces imply. Table I provides estimates of mean capacity factors with the normal code stress, S, as a base for the items discussed above. It would be expected that the first three items would apply to any engineered, controlled building and that even the most brittle structure under such conditions would have an ultimate ductility factor, μ, of 2 for item 10. Assuming unity for items 4 through 9, there would be an adjusted mean capacity of

$$1.25 \times 1.2 \times 1.8 \times \sqrt{4 - 1}\ S, \text{ or } 4.68S,$$

or the mean failure-point capacity of the structure would be 4.68 times the level of normal code stresses without any increase for seismic forces.

It is possible, although unlikely, that all items 4 through 9 would apply to one building, and even more unlikely that they would apply in the greatest degree. They are independent conditions. Let us assign to each the average of the range shown and allow a μ-factor of 4 for item 10. The overall result is 16.6S. This indicates the fallacy of assuming that all code-designed buildings have only code-level capacities when assessing earthquake intensity. The procedure should be to take each item of Table I in turn for a specific building and assign the factors according to actual conditions.

Because capacity and demand are coupled in the probabilistic sense, as shown in Equation (9), the real factor of safety, and therefore allowable stresses, can be estimated only in view of both capacity and demand.

DEMAND

Demand is defined here for convenience as the spectral response acceleration of a single-degree-of-freedom system having the fundamental period and damping of the building in its undamaged state. To get this demand, there must be an earthquake of magnitude M at a hypocentral distance R and a given site condition; then, a suitable spectral diagram must be constructed for these conditions. Factors will be developed, as they were for capacity, that will adjust the raw demand, D', for various considerations, some or all of which may apply in specific situations.

a. Many structures have deep foundations or basement stories, whereas most strong motion records have been obtained at or near the ground surface. It is known that ground motion is greater at the surface than underground.

b. Many structures are very large in plan size as compared to the structures or pads on which most strong motion records have been taken. Conservatism results when such records are used in the design of large structures. The response is greater horizontally, vertically, and in rotation than it should be for very large foundations, which tend to "iron out" some of the most intensive pulses or waves and may cancel the effects of high-frequency waves. A simple analogy is to compare the response of a large ship to that of a small boat in a turbulent sea. The record of large buildings in major earthquakes has been generally good; for example, the Palace Hotel, the St. Francis and Fairmont Hotels, the Flood Building, the Appraisers' building, the Mint, and Fort Point all survived the 1906 San Francisco 8.3M earthquake a few miles away.

c. Peak ground acceleration used to construct spectral diagrams and to normalize time histories of motion is often a poor index of response. It has been shown that 20% to 30% reductions in peak ground acceleration reduce spectral response peaks by only about 5% (6, 7).

d. Free-field motion characteristics may be altered beyond items (a) and (b), above, by structure interaction in a different manner than was the case for the structures in which the strong motion records were obtained.

e. Structure periods may not be constant, as they were for the idealized system assumed in response spectra development. Period variations, even though slight, tend to decrease resonant buildup and dynamic amplification.

f. Damping may be different than assumed.

g. Although it is assumed that horizontal components of motion are equal, one is usually greater than the other. The difference is no doubt greater close in than at long epicentral distances. Thus, a design basis with equal horizontal components is conservative.

h. Soils and some rocky materials behave in a nonlinear manner depending on strain, number of cycles, etc. Thus, linear analyses may be conservative because they use constant periods and damping that are subject to variations that tend to reduce the response from that of a linear system. Nonlinear analysis may or may not reconcile these matters, depending on how well the model represents the prototype.

i. Base shear coefficients are less than spectral acceleration in g units for multilevel systems (8). This is not always reconciled.

j. Ground motion and spectral response (shape) may be much less than the plus one-sigma, or even the mean values, used in analysis.

k. The stiffness of the foundation material may have been under- or overestimated in soil-structure interaction analysis, thus causing more or less response and amplification than for the prototype.

l. Actual spectral response diagrams are not smoothed but highly irregular. Thus, when subjecting a structure or system of modes to a smoothed spectrum, conservatisms are introduced because the real spectral shape would not lie consistently at the plus one-sigma or other level used for the spectrum. A raw spectrum has a negligible probability of falling entirely on or above a plus one-sigma smoothed spectrum; in fact, there is a low probability of one falling entirely on or above a mean smoothed spectrum.

Table II lists the above considerations together with estimates of the ratios of real mean values to conventional[II] levels. Not all of these factors would apply to any one structure. Some of the ratios might be greater than unity but the product of these independent factors would generally be less than unity, thus decreasing the raw spectral demand, D', to a usable value, D. The mean value of D for a nuclear plant could be, for items (a) through (l): (0.9)(0.9)(0.8)(0.95)(0.9)(1)(0.9)(1)(1)(0.65)(1)(0.95)D' = 0.31D'. An office building designed to a seismic code but later exposed to a response spectrum might have its mean value of D = (0.95)(1)(0.8)(.95)(0.8)(1.2)(1)(1)(0.9)(.8)(1)(0.95)D' = 0.48D'.

The procedure to adjust raw demand is to consider each item in Table II and assign factors appropriate for the specific conditions. Good judgment is required, and more research needs to be done on many of these concepts. However, it is clear that using raw demand without adjustment can be quite conservative.

JOINT DEMAND AND CAPACITY

The allowable stress should be established to provide the desired safety factor in view of all considerations. Equation (9) provides the means, but the input values are not easy to obtain in most cases. The

II "Conventional" refers to the unadjusted value that would be developed directly from strong motion records, analysis of spectral shapes, and earth science studies.

sparse data in the literature (1, 9), indicate that N_D is often about 2, and N_C is less (1, 10), say about 1.5. An office building designed to a base shear coefficient of 0.04 was assessed according to Table I with a resulting base shear capacity of 10 times code shear. The building demand, D, is 0.48D'. If the building fundamental period is 1.0 sec and damping is 5%, an expected mean value of D' could be 0.5g, given a major nearby earthquake. Higher modes are ignored here, as is multiaxial motion. Will the building survive the earthquake?

The participation factor related to base shear has already been provided for in the demand adjustment, item (i); thus the spectral acceleration can be associated with the base shear coefficient. The median capacity, $\tilde{C} = (10)(0.04) = 0.40$. The median demand, $\tilde{D} = (0.48)(0.50) = 0.24$. From Equation (7), $\sigma_{FS} = \sqrt{(\ln 1.5)^2 + (\ln 2)^2} = 0.80$. From Equation (9),

$$FS = e^{\ln 0.40 - \ln 0.24 + 0.80y} = e^{0.511 + 0.80y}$$

which results in the safety factors and probabilities shown in Table III. The probability of failure (FS = 1) is only 26%, given the earthquake demand, in spite of the 4% base shear building being subjected to a 0.5g spectral acceleration. The probability would be a little greater if higher modes and/or biaxial motion were used. The most likely condition from Table III is a safety factor of 1.67 against failure.

The hypothetical nuclear plant would be designed much more conservatively than the building. It is estimated using Table I that its capacity would be about 5.6 times its design value with a ductility factor of 2. It would be designed to a spectral value above the mean spectral demand, say 0.50g x 1.8 or 0.9g at a 1-sec period. Thus, $\tilde{C} = (0.9)(5.6) = 5.0$, and $\tilde{D} = (0.31)(0.50) = 0.155$. Using the same N_C and N_D values as for the building in Equation (9),

$$FS = e^{\ln 5.0 - \ln 0.155 + 0.80y} = e^{3.47 + 0.80y}$$

The results are shown in Table III. The probability of failure given the major nearby earthquake and the strong spectral acceleration is 0.000007. The most likely FS for this exposure is 32.1.[III]

CONCLUSIONS

The allowable stresses in seismic design must be considered with the design force requirements, the real earthquake demands, and the consequences of failure. All factors listed in Tables I and II and discussed in the text are involved in the problem of allowable stresses together with probabilities of many other complex events and conditions. Although the examples shown are hypothetical and other specific cases may differ considerably, the safety factors for buildings designed to existing codes are not excessive; for some conditions, they may be considered low. The building example has 3 chances in 4 of surviving the local major earthquake. There seems to be no basis for increasing the allowable stresses with the current level of design coefficients. For special risks, the stresses should be reduced or the design levels increased. The hypothetical nuclear plant designed to a more realistic level of seismic intensity has excellent safety factors. No change in stresses is suggested, although safety factors should not be compounded beyond reason.

III Even if this structural failure should occur, there would also have to be concurrent operating failure to develop a nuclear problem.

No change in allowable stress levels per se is indicated by this study, but a more reasonable and direct approach is recommended. The factors in Tables I and II and in the text should be considered for codes and for major projects as a means of reconciling the performance -- good or bad -- of actual buildings in strong earthquakes. More research is needed in many of the areas listed. The factors provided are the writer's best current estimates based on considerable judgment.

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TABLE III - ESTIMATED FS VALUES FOR HYPOTHETICAL STRUCTURES

y	Probability C/D < FS shown, given earthquake	Factor of Safety, FS	
		Office building	Nuclear plant
- 4.34	0.000007	---	1.0
- 2	0.02	0.34	6.5
- 1.28	0.10	0.60	11.5
- 1	0.16	0.75	14.4
- 0.64	0.26	1.00	---
0	0.50	1.67	32.1
1	0.84	3.72	71.5
1.28	0.90	4.66	89.5
2	0.98	8.31	159.0

TABLE I - CAPACITY FACTORS

Item no.	Description	Estimated ratio of real mean value to conventional value S
1.	Average stress from tests	1.25
2.	Material specifications	1.2
3.	Yield level vs. S	1.8
4.	Materials actually provided	1.0 to 1.1
5.	Probabilistic reductions	1.0 to 1.3
6.	Seismic stress vs. total stress	1.0 to 1.4
7.	Redundancy	1.0 to 1.3
8.	Nonessential elements	1.0 to 1.2
9.	Nonstructural elements	0.7 to 1.6*
10.	Ductility, capacity to absorb energy	$\sqrt{2\mu-1}$ to $1.2\sqrt{2\mu-1}$

*In some buildings of a box type or with heavy walls with few, small openings, the 1.6 estimate could be conservative; ratios less than unity apply to cases where the nonstructural elements would be harmful to the structure, such as a filler wall that does not extend to the tops of adjoining columns.

TABLE II - DEMAND FACTORS

Item no.	Description	Estimated ratio of real mean value to raw value D'
a.	Deep foundations (embedment)	0.80 to 1.00
b.	Large structures	0.80 to 1.00
c.	Peak ground accelerations	0.70 to 0.90
d.	Soil-structure interaction	0.90 to 1.00
e.	Structural period variations	0.80 to 0.90
f.	Damping	0.80 to 1.20
g.	Horizontal components not equal	0.90 to 1.00
h.	Nonlinear soil systems	0.90 to 1.00
i.	Base shear vs. spectral accelerations	0.70 to 1.00
j.	Mean spectral value < analysis level	0.50 to 0.90
k.	Stiffness of foundation materials	0.90 to 1.10
l.	Smoothed vs. actual response spectra	0.90 to 1.00

DISCUSSIONS ON 4.3 AND 4.4

P.K. Kandaswamy (India)

My comments will cover the above two topics under three types of constructions as given below:-

1) Masonry Dam - A non-overflow section is designed with and without earthquake force. While designing without earthquake force, the base width is fixed in such a way that the resultant of all the forces falls within the middle third. The same section is found to be safe against earthquake forces with a marginal tension of 10T/sq.m with the resultant of the forces falling within the middle fourth. A number of cases of dams in Tamil Nadu have been studied in our design office and found to be same in all cases. So, in the case of low dams, we are not providing any extra width in a section for seismic force alone. However, in the case of high dams owing to high hydro-dynamic pressure, greater sectional area may be required to resist earthquake forces.

In the code of practice, it has been stated that it is enough if the resultant falls within the middle three fourths of the base width while designing for earthquake forces. For high dams, there may be large tensile stress when the resultant falls outside middle third but within middle three fourths. No mention has been made in the Indian Standard code about allowing tensile stress and if so, its magnitude in the masonry dams under earthquake conditions. This requires to be codified by the I.S.I.

In our practice in Tamil Nadu, we are allowing a tensile stress of 1T/sq. ft (10 t/sq.m) in the R.R. masonry, if seismic forces are considered, as we mostly come across low dams only.

2) Earthdams - The earth dam embankment stability is analysed by the most familiar swedish arc or slip circle method. The stability analysis is done for the maximum height of section. The U/S slope is checked for sudden draw-down condition and the d/s slope is checked for steady seepage condition. Number of trials are made to locate that slip circle which gives the lowest resistance to shear. The d/s slope alone is checked for earthquake forces, with reservoir full condition. That slip circle which gives the least factor of safety is taken for stability analysis with earthquake forces. The minimum F.S. required against failure is 1 when earthquake forces are

considered for d/s slope alone. Generally, d/s slopes designed without earthquake force is also safe against earthquake force in the case of low earth dams.

3) Buildings - The best type of foundation ensuring resistance to earthquake will be the foundations with grillages on concrete piles reaching solid earth, even when ground conditions are unfavourable. All the other things being equal, rigid structures on soft grounds are less damage prone than rigid structures on hard grounds. Similarly, flexible structures on soft grounds are likely to be affected to a greater extent than flexible structures on hard grounds.

A rational earthquake design must ensure the following:-

i) No damage during mild shocks.

ii) Controlled damage duging severe shocks.

iii) No collapse during maximum shocks.

The I.S.I. makes the following recommendations in the design of buildings against earthquakes.

I. for various loading classes, the horizontal earthquake force shall be calculated for the full dead load and a percentage of live load.

II. for buildings greater than 40m in height and upto 90m, modal analysis is recommended.

III. for buildings taller than 90m, in zones other than I and II, detailed dynamic analysis shall be made based on expected ground motion.

IV. for buildings greater than 40m height, checking for forces like drift and torsion is to be done.

In reckoning the height of the building, especially when the depth of foundation is large, it is for consideration whether a part of the foundation depth may also have to be included to the height of the building above the foundation level, as the whole structure is functioning as monolithic unit under earthquake conditions.

Closure by A.S. Arya

Although the comments of the discussor are directed to the Panelists Dr. A.R. Chandrasekaran and Mr. J.A. Blume of Panel 4, these were sent to me for replies since they pertain to the Indian Standard Criteria for Earthquake Resistant Design of Structures. My views on the various points raised by the discussor are given below:

MASONRY DAM. The main question raised here is about the allowable tension in high masonry dams and whether the resultant could be allowed to act within middle 3/4 of the base width. The recommendation of the resultant lying in the middle 3/4 as contained in the code is applicable to retaining walls and not dams. Since in the case of dams there is uplift pressure of water, the cracking or lifting on the upstream side will not be desirable. The value of allowable tensile stress is still undecided. This value naturally depends upon the richness of the mortar. Experimental research is called for determining the tensile strength of such masonry not only under static loads but also under high rate of strain as encountered under earthquake load. A value of 2/3 of the modulus of rupture may be a good guess for allowable tensile stress under seismic condition.

EARTH DAMS. The upstream slope of earth dams usually shows more critical state under earthquake condition and should also be thoroughly examined. Degrading strength of the earth fill under high alternating stresses should be duly considered for assessing the safety properly. This aspect is not yet covered in the ISI Code.

BUILDINGS. The height of buildings as specified in the code provides guidance as to what type of analysis may be carried out considering the risk involved in case such a building fails under earthquake condition. For these purposes the intention of the code is to measure the height above the general foundation level which in the case of pile foundations will be taken above the pile caps. So far as dynamic analysis taking into account of the soil-structure interaction is concerned, naturally the soil pile system and the building together will have to be considered as one unit.