UNCERTAINTIES IN EARTHQUAKE RESISTANT DESIGN

Jai Krishna(I)
Brijesh Chandra (II)
Presenting Author: Jai Krishna

SUMMARY

Uncertainities arise in earthquake resistant design of structures at various stages - in assuming design earthquake parameters like magnitude of expected earthquake, its hypocentral distance and the effective peak accelerratation as also in the parameters of the structure like its effective stiffness,damping characteristics and other parameters relating to the presence of live-loads at the time of earthquake. This papaer attempts to put down in quantitative terms the possible errors in design seismic forces due to variation in some of the design parameters. It is emphasized that unless the design data are refined, the usefulness of sophistication in the methods of analysis would be limited.

INTRODUCTION

Earthquake engineering research has made significant advances with availability of strong ground motion data and high speed computer leading to refinements in the computation of dynamic response of structures and systems subjected to seismic excitation. This enabled better understanding of the system behaviour, hitherto not feasible. This progress has,however, not resulted in removing many uncertainties associated with estimating future ground motion or the dynamic properties of the materials of a structure at the time of an earthquake. Designs are, therefore, based on assumptions made in these respects, which quite often make material error in predicting the behaviour of a structure in a real earthquake as evidenced by surprises that have been observed in the recent years. This raises the question as to whether the refinements in analytical procedures are consistent with the reliability of data selected for the system and the site. An attempt has been made in this paper to quantify some of these uncertainties and to focus the attention of designers on the relative importance of refinements and assumptions in the actual design.

PARAMETERS FOR A SEISMIC DESIGN

For the design of an important structure,two sets of engineering decisions are made - one pertains to the structural system and the other to the seismic loading. Obviously the choice of the parameters does not offer a unique value situation. The problems involved in making the choices are discussed in the following paragraphs:

Ground Motion Parameters

For estimating ground motion at a site following steps are taken (Ref.1,2):

(I) Professor Emeritus)Department of Earthquake Engineering,
(II)Professor)University of Roorkee,Roorkee, INDIA

(a) Collection of historical data of earthquake occurrence in the region(a radius of about 150 km from the site may be called effective region)

(b) Identification of faults and their activity around the site/region of interest through geological and seismological observations.

(c) Deciding upon one or more design earthquakes corresponding to the acceptable damage in each event.

(d) Choosing a shape for design spectra based on these derived for recorded shocks under similar geological and foundation environment.

(e) Selecting an effective peak ground acceleration and duration of shock expected.

(f) Determination of time history of accelgrogram based on above decisions.

Geological, magnetic and gravity surveys, quite often locate discontinuities and give an idea of the stratigraphy,structural trends, faults and their recent movements. Although use of high magnification seismographs recording microtremors over a sufficiently long period of time may indicate active areas, yet it is difficult to determine with any certainty as to which source will produce an earthquake of consequence in the life time of the structure at a site. It is also not certain whether the microtremors are an indication of the possibility of occurrence of large size ones(Ref. 3). This leads to adopting a conservative approach in identifying the possible active sources around the site, and assuming that the largest size of earthquake, based on historical data, would occur from the source nearest to the site. Similarly, the depth of the centre of mass rupturing to cause the shock will have to be assumed on very doubtful grounds particularly because energy is released haphazardly over a fairly large area. Thus a considerable uncertainty, difficult to quantify, exists in the choice of design earthquake parameters.

Having chosen the parameters for design basis earthquake viz.Magnitude (M), focal depth (h) and epicentral distance(D), one falls back on one of the several empirical relationships correlating these with peak acceleration (Ref.1,4). It is also known that all such relationships have a large scatter for most sets of parameters mainly because these have been derived from statistical studies of recorded accelerograms in different geological and foundation environment. Examples of actual accelerograms, recorded in an earthquake in the same city being considerably different, are numerous (Ref.5,6).

Choosing an expression for correlation between earthquake parameters and effective peak ground acceleration(Ref.7), a_{max}, and realising that there could always be variations in the choice of parameters M, D and h,the computed values of a_{max} are shown in Table 1.It is interesting to note that the usual variations (or errors) in choosing these parameters make considerable error in the value of a_{max}.

No effort has been made here to estimate the cumulative effect of errors in choosing the parameters on a_{max}, but it can be seen that it is

Table 1 - Effect of Variations in M,D, h on a_{max}

Parameter	Range of variation	Difference in a_{max}
Depth of focus	15 km - 30 km	43%
Magnitude	$\pm$ 0.5	37%
Epicentral Distance	5 km - 10 km	15.5%
	5 km - 15 km	17.5%

likely to be larger than any precision which the refinements in procedures for analysing a structure may affect.

Parameters of Structures

Earthquake response of a structure of a certain form is largely influenced by its natural period of vibration, damping characteristics,mass distribution and foundation interaction. Naturally, therefore, the parameters which control these need be chosen carefully. A number of investigators have improved upon the procedures for determining the period of vibration and dynamic response more precisely considering the effects of non-uniform mass distribution, foundation interaction, rotation of joints, eccentricities and consequent torsion and $P - \Delta$ effect, with the help of high capacity computers. For example, the effect of joint rotation on response with normally stiff beams is of the order of 5% to 10%(Ref.8). Consideration of axial deformation in columns could elongate the period by about 15% which may reduce the seismic acceleration by about 8%(Ref.9). The bending effect of gravity loads($P - \Delta$ effect) may increase displacements by approximately 10%(Ref.10). In relation to torsion, it is observed that if eccentricity is within about 5% of building width, the response is more or less unaffected(Ref.11).The provisions of code in this regard as also for interstorey drift are reasonable safeguards in as much as they limit the effects of torsion and gravity effects. From the above it would be seen that if a building is carefully designed the effects of other parameters like those discussed above would not be too large.

Response spectra for a large number of earthquakes indicate that the structures having their period between 0 and 1 second quasi-resonate with the ground motion, unless situated on particularly soft ground away(at a distance of more than about 100 kms.) from the centre of shock. In the latter case only very flexible structures having a period of more than 1 second tend to amplify ground motion considerably. Spectra also indicate that damping more than 10% of the critical reduce the earthquake forces greatly. The reduction in response between 1% and 15% damping is of the order of 66% in short period range ($<$0.5 sec) to 57% in long period range ($>$1.5 sec),and between 0.5 sec and 1.5 sec the decrease is 60% (Ref.12). That shows how accurately one should know this parameter to be able to estimate the response of a structure reliably. In the absence of accurate estimates of this parameter, there is bound to be large errors in seismic response of structures and this could explain some of the anamolous situations in their performance.

Assumptions Regarding Materials

Elastic modulus E, moment of inertia I and the distribution of effective mass of the system need to be ascertained to compute the natural period of the structure. It is well known that the elastic modulus varies considerably with time and strain particularly for concrete, masonry and earth structures. In concrete the reduction in E value could be 50% with time compared to its value at the time of construction and could further reduce with increased strain. This alone could result in elongation of time period of the structure by about 25% which may result in reduction of seismic acceleration by as much as 10% particularly in the short period range.

Moment of inertia of a reinforced concrete structural member is a parameter which does not have a precise definition and is generally taken on the basis of gross-cross-section mainly for convenience. This is however, not correct since it is a function of vertical load intensity as well as lateral loads which together control the neutral axis. On the basis of effective section, it is seen that the value of I for a column could be about 32% less for a section having 0.8% steel and nearly 200% more for columns having 8% steel. These could mean elongation of period by 21% in the former case and shortening of period by 43% in the latter case. Consequently the response would be about 9% less in the former case and 125% higher in the second case.

Effective mass of structure inclusive of live loads present at the time of earthquake is another important factor which controls not only the period but also the seismic shear (Ref.13). In case of buildings, many designers choose to reduce the live loads (Ref.14) on the system since design live loads are not likely to be present during an earthquake(usually not even in non-seismic condition). Any reduction in this and consequent change in period, and eventually shear, is therefore, arbitrary but desirable from point of view of economy in design. However, in certain types of structures like water towers supporting large capacity tanks, the quantum of live load can be estimated reasonably well since 'tank-full' condition is quite normal and can be adopted for design. However, there are some other aspects in such structures which are explained in the following section through a case study.

A water tower approximately 18 m in height and weight 465 kips supporting a 100,000 gallon capacity tank (weight 454 kips) was studied experimentally under various conditions of fillings (Ref.15). Free vibration behaviour of the structure showed that period of the structure varied from 0.70 sec in 'tank-empty' condition to 0.78 sec. for 'tank-full' condition, giving the ratio as 1.11. However, assuming that EI of the supporting structure to remain constant, this ratio should be 1.62. This clearly indicates that EI does not remain constant and in fact increases considerably with the intensity of vertical load. This aspect is generally not realized in practice but can lead to serious discrepancies in estimation of period and consequently the seismic shear as mentioned earlier. For the above example, if the stiffness is worked out corresponding to 'tank-empty' condition the design seismic shear would be underestimated by about 30% compared to the realistic value (for 'tank-full' condition) if average spectra are used with 5% damping. So far no attempt has been made to provide for such effects in practice.

Damping in Structures:

Damping characteristics are usually assumed for seismic design based on experience of the designer. Actually,there is a wide range in which it falls for most structures particularly, concrete,masonry and earth structures. In the elastic range, even an earth dam or an earth and rockfill dam exhibits small damping(say 2% of the critical),but as vibrations loosen out the material,damping increases and may reach a value as high as 20-30%. Therefore, taking a constant value(say 10%)could not be considered as rational in the analysis. Unfortunately, there is no method by which this value can be estimated at the time of design since it can only be obtained experimentally. Being largely a matter of assumption, the uncertainity in computation of response is inevitable. The ratio of response values for 2% and 5% damping in the period range 0.2-2.5 sec lies between 1.25 and 1.50 while for 5% and 10% this ratio varies between 1.18-1.37. This can easily introduce an error of 18-50%.

OTHER CONSIDERATIONS

Differential Settlement:

In design of framed structures,apart from dead, live and seismic forces, one aspect,which is difficult to take into account, is the load resulting from differential settlements and this has been the cause of failure/collapse of a large number of structures in the past. Differential settlements of the order of about 1% of the beam spans should not be unexpected in seismic condition in any region where sound rock is not available to 'found' the structure. A simple example was worked out to estimate the forced caused in a frame(Fig.2)due to such an action. Table 2 shows the maximum bending moments caused in the frame due to vertical, seismic and settlement loads. It can be seen that in a system designed for normal loads and seismic coefficient of 0.10,a settlement of 1 cm would generate as much as 72% of the design moments. And these are normally not considered in structural design.

Table-2 Maximum Values of Bending Moment in Frame, kg-m

Vertical load moment(including live loads)	1928
Seismic moment	625
Settlement moment	1846

It would thus be clear that continued efforts to refine the procedures to estimate seismic forces alone would not be very fruitful unless other accopanying forces like settlement moments are also estimated.

Bearing Pressure for Foundation Soil

There are questions about the possible increase in the strength of foundation soil mostly because different practices consider this aspect differently. Most designers,however, accept that permissible loads may be increased by 33.1/3% or more when seismic actions are considered in view of the infrequent nature of this load(consequently permitting lesser factor of safety in this situation)and also because there is a real increase in strength if loading rate is high(except in loose materials). In countries where basic seismic forces recommended for design are high, the increase in strength is also specified to be higher and vice-versa. However,this factor of 33.1/3% or 50% is again an arbitrary figure which affects the design of a system.

MAXIMUM POSSIBLE ERRORS

The possible deviations in anticipated values of seismic response corresponding to design basis earthquake have been indicated in quantitative terms in the preceeding sections. Considering that the variations in the parameters occur on the unsafe side of design conditions, the total error(e) in response can be estimated from the following expression:

$$e = \frac{a_m - a_o}{a_o} \qquad ..(1)$$

in which a_m is the modified value of seismic acceleration due to various factors and a_o is the design seismic acceleration. a_m is given by,

$$a_m = \theta_g.\theta_s.a_o \qquad ..(2)$$

in which θ_g is the ratio of modified ground acceleration and the design value, and θ_s the ratio of modified structural response and the design value.The θ_g and θ_s values in turn are products of modification factors due to magnitude, focal depth, epicentral distance in the former case and elastic modulus, moment of inertia, damping in the latter. Taking the maximum values as indicated in the preceding sections,

$\theta_g = 1.37 \times 1.43 \times 1.175$
$= 2.30$
and $\theta_s = 1.1 \times 2.25 \times 1.50$
$= 3.71$ (for heavily reinforced sections)
$\theta_s = 1.1 \times 1.20 \times 1.50$
$= 1.98$ (for moderately reinforced sections)
Thus,$e = (2.30 \times 3.71 - 1)$
$= 7.53$ (for heavily reinforced sections)
and $e = (2.30 \times 1.98 - 1)$
$= 3.55$ (for moderately reinforced sections)

The errors could, therefore,be as high as 753% or 355% as indicated above. This and other effects like settlement of foundation could perhaps explain how even the cumulative factor of safety built into structures could fall short of the requirements. This may also explain some of the many surprises found in history of earthquake performance of structures in the past. For practicel design work, the usefulness of refinements in analytical procedures may be viewed in the light of the above uncertainties.

CONCLUSIONS

The uncertainties associated with earthquake resistant design of structures are numerous. Refinements and sophistication in methods of analysis are desirable but not adequate without corresponding refinements in the data fed into the analysis. In the absence of this judgment plays a major role.

REFERENCES

1 Krishna Jai,Chandra B.and Singh V.N.,"Seismic Data for Structural Design",Central Board of Irrigation & Power Golden Jubilee Commemorative Volume,1977.

2 Chandra B.,"Seismic Data for Design of Structures",Jai Krishna Sixtieth Birth Anniversary Commemorative Volume on Earthquake Engg.,Sarita Prakashan,Meerut,1974.

3 Krishna Jai,Chandra B and Paul D.K.,"Earthquake Design Criteria",(unpublished),Deptt.of Earthquake Engg.,University of Roorkee,Roorkee,1981.

4 Paul D.K.,Singh V.N.,"A Guide to Acceleration Velocity and Displacement Relationships with Distance and Magnitudes",Bull. of Indian Society of Earthquake Technology, Vol.15, No.4, 1978.

5 Housner G.W.,"State of the Art Report on Earthquake Ground Motion",Proc. of International Conference on Planning and Design of Tall Buildings, Lehigh University, Bethelehem,Pennsylvania, 1972.

6 Seed H.B.,"State of the Art Report on Dynamic Characteristics of Soil-Structure Systems",Proc.of Int. Conf.on Planning and Design of Tall Buildings,Lehigh University,Bethelehem,Pennsylvania,1972.

7 McGuire R.K.,"Seismic Design Spectra and Mapping Procedures using Hazard Analysis Based Directly on Oscillator Response",Earthquake Engineering and Structural Dynamics, Vol.5, No.3, 1977.

8 Chandrasekaran A.R.,"Dynamic Study of Multistorey Frames",Ph.D. Thesis, University of Roorkee, Roorkee.

9 Arya A.S. and Thakkar S.K.,"Seismic Behaviour of Multistoreyed Buildings",Jai Krishna Sixtieth Birth Anniversary Commemorative Volume on Earthquake Engineering,Sarita Prakashan,Meerut, 1974.

10 Carr A.J. and Moss P.J.,"The Effects of Large Displacements on the Earthquake Response of Tall Structures",Proc. Seventh World Conf. on Earthquake Engineering,Istanbul,1980.

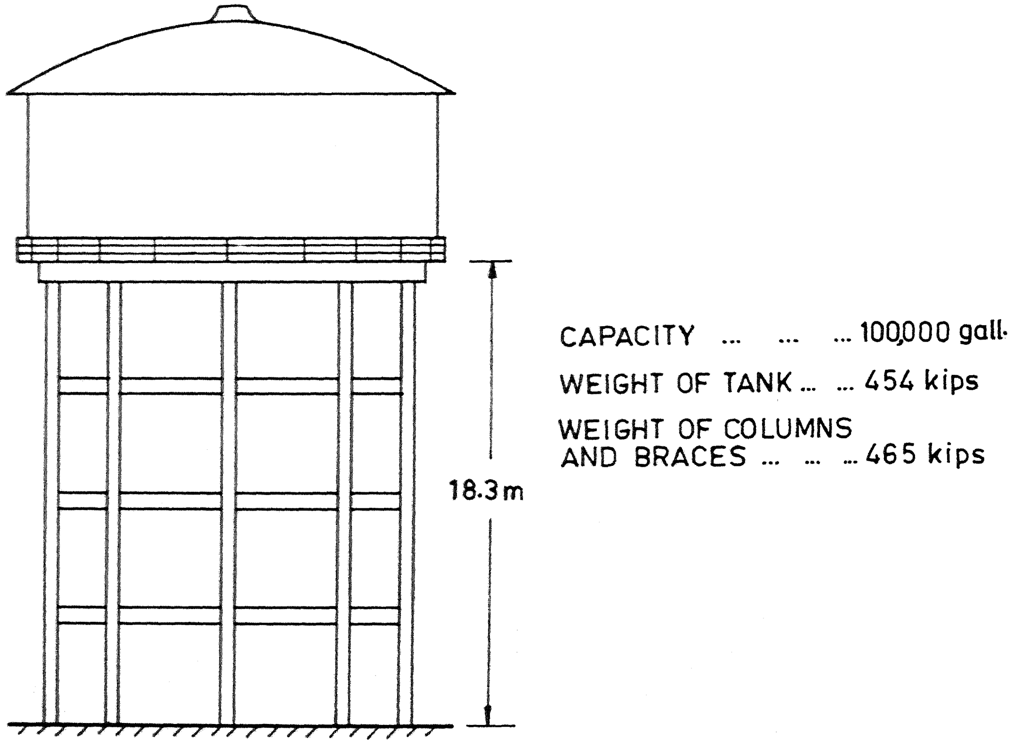
11 Sakurai J.,"Vibrational Characteristics and Earthquake Response of Asymmetric Multistorey Buildings",Proc.Seventh World Conf.on Earthquake Engineering,Istanbul,1980.

12 IS:1893-1975-"Criteria for Earthquake Resistant Design of Structures", Indian Standards Institution,New Delhi,1976.

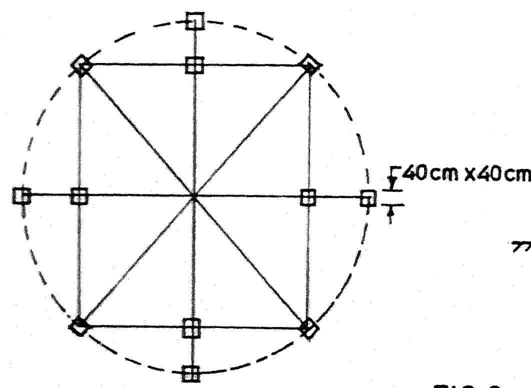
13 Chandrasekaran A.R. and Saini S.S.,"Live Load Effect on Dynamic Response of Structures",Proc. ASCE, Vol. 95, No.ST4, 1964

14 SP:22(S&T),"Explanatory Handbook on Codes for Earthquake Engineering", IS:1893-1975 & IS:4326-1976",Indian Standards Institution,New Delhi,1982.

15 Chandrasekaran A.R. and Krishna Jai,"Water Towers in Seismic Zones",Proc Third World Conf on Earthquake Engineering,New Zealand,1965.



(a) ELEVATION



(b) PLAN

FIG.1_ THE WATER TOWER

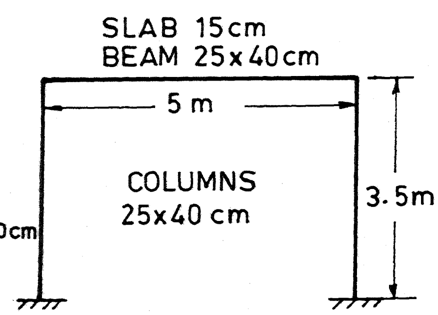


FIG. 2 _ THE FRAME USED FOR EXAMINING EFFECT OF UNEQUAL SETTLEMENT