SEISMIC DATA FOR THE DESIGN OF STRUCTURES

By

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Many countries are faced with the problem of establishing standards for the earthquake-resistant design without having available adequate precise instrumental data on the destructive ground motions that have occurred in the past. Despite this it is necessary to estimate the intensity of ground motion that might be expected at various locations in the future even if this requires rather drastic assumptions. It is proposed to discuss in this paper such assumptions and to present an approach for estimating possible future ground motions for which structures may be designed.

The only region where considerable number of strong motion records have been obtained is California. Elsewhere, particularly in Japan, strong motion instruments have only been recently installed in some number and there is a similar move on in India. Future shocks in these regions will perhaps make more information available regarding the ground motion and the corresponding behaviour of different structures. For the present, the data necessary to estimate the ground motion which a particular location may experience in a future shock is scanty.

With whatever data is available about the past shocks, a designer is still faced with the question as to whether at a particular site the future shocks would cause the ground motion of the same intensity as the past one did or bigger, and whether the future shocks would originate from the same faults or some other faults nearer the location of the structure. Obviously, the nearer the origin of the shock and the bigger its size, the more intense will be the ground motion. (It is only when the distance from the fault becomes very small that this general statement may not be true.) A structure designed for bigger ground motion will also cost more. A designer, therefore, has to decide upon the size of a future shock, occurring in the nearest zone of seismic activity for which the structure should be designed. In deciding this, the importance of the structure, its expected life, and probability of occurrence of the shock during the life time of the structure will have to be considered. At the same time, it will be necessary to examine how the decrease in the cost of structure by designing it for a smaller shock matches with the increased risk. There are certain structures, such as a dam or a retaining wall designed primarily to withstand horizontal forces, in which an increase in the seismic coefficient beyond a certain limit may result in an increase of their weight to such an extent that the addition to factor of safety is negligible, although the increase in cost may be considerable. Thus taking all these factors into account, the size of the design shock has to be decided upon.

There are two scales that can be used to define this shock--"magnitude" to represent the total energy released at the fault, and "intensity"

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to represent the damage that may be expected at the site. For qualitative study of the relative structural behaviour in the past shocks the "intensities" allotted to the site are of great value, but since these are based on damage reports, while damage was a function of materials, workmanship, method of construction, foundations, besides geology of the region, size and location of the shock, it is difficult to make quantitative use of this information in the design. Further, an attempt to estimate ground motion from actual failures brings in several uncertain factors regarding the structure itself so that a graph plotted between "intensity" and recorded "ground acceleration" shows a very big scatter (1). On the other hand "magnitude" by virtue of its definition is an instrumental measure of the "intensity" of ground motion at the site of the instrument and is a comparatively more precise estimate. Its definition correlates "distance", "ground motion" and total "energy" released. Although these relations are also some-what empirical yet they give the information in the form a designer needs it, and eliminates many other factors which are included in the "non-instrumental" estimation of the size of an earthquake. It is, therefore, suggested that a suitable "magnitude" of the expected shock may be chosen in the light of considerations in the previous paragraph.

Having fixed the size of the shock, it would be necessary to make certain assumptions in order to estimate the ground acceleration at the site of the structure. (At this stage it may be stated that recent studies have shown that "velocity" of the ground motion seems to have a more direct relation with damage to a structure, but, since the use of "velocity" is made in the "limit design" methods which for the present are understood well only in some simple cases, "acceleration" continues to be utilized for design purposes). The assumptions are:-

(1) The radiation of energy released from the disturbed mass in uniform in all directions, and the source of energy may be considered to be the centre of this mass from which all distances are measured.

(2) The design shock has the centre of the disturbed mass approximately at a depth of 15 miles below surface (bulk of the past shallow shocks which have caused major damage had their centre at a depth of 15 miles as an average.) For shocks of the large magnitude it is conservative to consider the release of the entire energy at the centre of the mass as the fault length may be several miles.

(3) The intensity of ground motion decreased with the square of the distance from the centre of the disturbed mass and increases with the cube root of the energy released as suggested by Gutenberg and Richter (2).

The distance-ground motion relationship has been borne out by the results of the blasts recorded by Carder and Cloud (3) and also the spectrum curves presented by Housner (4), shown as Fig. A. The relationships adopted from the work of Professors Gutenberg and Richter are:

$$\mathrm{Log}_{10} E = 9.4 + 2.14 M - 0.054 M^2 \qquad (1)$$

and

$$a = C(E)^{1/3} \frac{h}{D^2 + h^2} \qquad (2)$$

Where 'M' is the "magnitude" of the shock; 'E' the energy released in ergs; 'a' the acceleration expected at a point situated at a distance 'D' from the centre of the disturbed mass, which is 'h' below ground level. The constant C included the effects of the ground conditions etc.

Comparing with the El Centro earthquake of May 18, 1940, for which $h \triangleq 15$ miles; $D \triangleq 30$ miles; $M = 7.1$; and the strong motion seismograph recorded the maximum ground acceleration equal to 0.33 g, the constant of proportionality in Eq.2 can be eliminated.

Putting the right hand side of Eq.1 equal to A, we have

$$E = 10^{A}$$

Substituting this in Eq.2, and comparing with El Centro shock mentioned above, we get.

$$a = (0.33\,g)\,\frac{h}{15}\,\frac{30^2 + 15^2}{D^2 + h^2}\,10^{\frac{1}{3}(A-21.87)}$$

where the value of A for El Centro shock is 21.87.

$$\text{or,} \qquad a = \frac{24.75\,h}{D^2 + h^2}\,10^{\frac{1}{3}(A-21.87)}\cdot g \qquad (3)$$

Giving different values to 'M' and 'D' and assuming $h \triangleq 15$ miles, the values of 'a' have been worked out and shown in Fig. 1. Table 1 shows a comparison of the values thus obtained with those actually recorded in certain past California shocks.

It will be seen from Table 1 that, except for the Taft and Seattle earthquakes there is considerable agreement between the actual records with those obtained from Eq. 3 or from Fig. 1. The shocks of Taft and Seattle were perhaps produced by somewhat different type of slipping or the soil played a part in damping out waves before they reached the instruments. That is why the actual records are exceptionally low in magnitude compared with what would be expected for shocks of this size. It may be stated here that estimation of D,h and M have some margin of error and thus the estimated acceleration will represent average values that may be expected. There could be some special circumstances in which these results can be wide off the mark. For example, an instrument housed on the top of very loose soil which gets compacted quickly under vibratory force transmitted by an earthquake will record something quite different from what one would expect. The soil may exaggerate the motion due to low elastic modulus or it may filter the high frequency violent part of the shock and may record only the less violent part. Thus the geological and foundation conditions may produce exceptional results. It may, however, be mentioned that a study of blast tests carried out in Sweden, Canada and U.S.A. has shown that the general pattern of relationship between ground motion, distance from the blast and the quantity of charge is very similar even though the soil conditions were not the same, and also that ground motion record and its characteristics are similar to those of an earthquake. Thus for any region unless there is specific data available, the approach suggested above for estimating the ground motion will offer reasonable guidance.

It will be noted that an effort has been made to assess the maximum ground acceleration, although it is well understood that the design of a structure is not a function of the peak acceleration only. It is necessary to take into account the duration of the shock and the variation of motion with time. In fact it is the integrated effect of the ground motion over the period of time the shock lasts, such as is given by a response spectrum analysis, that will give a true picture of the forces exerted on a structure. A study of the different earthquake records has shown that many features of the ground motion records are the same, and if their maximum ordinate and duration were the same, the overall results will be comparable. Thus it is proposed here that the shape of the record of the design shock may be assumed to be the same as that of El Centro Shock May 18, 1940, which is the strongest motion so far recorded, with the ordinates having the same ratio as the maximum values. The duration of the shock may also be the same. This approach or set of assumptions enables the designer to use the spectrum curves worked out at the California Institute of Technology, reproduced here as Fig. 2 even though they have been drawn for California shocks.

As a specific numerical example, using Fig. 2 and the multiplying factors proposed by Housner (4), we find that for a structure having a period of one second damping equal to 10 % the ordinate on the response spectrum is 3.5 ft/sec.^{2}. If the structure is located at 50 miles from the nearest active fault region, the design shock has a magnitude of 7.5 having the centre of disturbed mass 15 miles below the ground surface, and the point on the fault surface trace nearest to the structure is immediately above the centre of the disturbed mass, the maximum ground acceleration expected may be of the order of 0.20 (read from Fig. 1). Comparing with El Centro shock May 18, 1949, the multiplying factor for the spectrum will be (0.20/0.33) x 2.7 = 1.64. Thus the maximum acceleration response of a single degree of freedom structure may be 1.64 x 3.5 = 5.75 ft/sec. If the structure is water tower having a concentrated mass supported at the top of a flexible structure the entire mass may be subjected to this acceleration. On the other hand, if, the structure is more or less uniform such as a frame building, the variation of the force and shear will be as shown in Fig. 3.

The actual shape of the force diagram will be given by the following equations:-

$$\ddot{Y} = \sum_{i=1}^{N} \omega_i C_i \int_0^t a\, e^{-n_i \omega_i (t-\tau)} \sin \omega_i (t-\tau)\, d\tau$$

Where $\ddot{Y}$ is the acceleration response of the structure, $(2\pi/\omega_i)$ the period of vibration for the ith mode, 'Gi' the factor taking into account the vibrating masses, the stiffness of the structure, and the location of point where $\ddot{Y}$ is determined; 'a' the ground acceleration, n_i the damping factor, 'N' the degrees of freedom and 't' the period of time upto the moment when $\ddot{Y}$ is determined. In actual practice, the use of this equation could be tedious and a study is being made to find approximate shapes for specific structures generally designed. Some of the building codes use the force diagram to be a triangle such that the shear diagram is parabolic.

The method suggested above will make a design shear diagram available on the assumptions that the future shock affecting the site of the structure to be designed will have general characteristics similar to those of California shocks and the behaviour of similar structures will be comparable.

References

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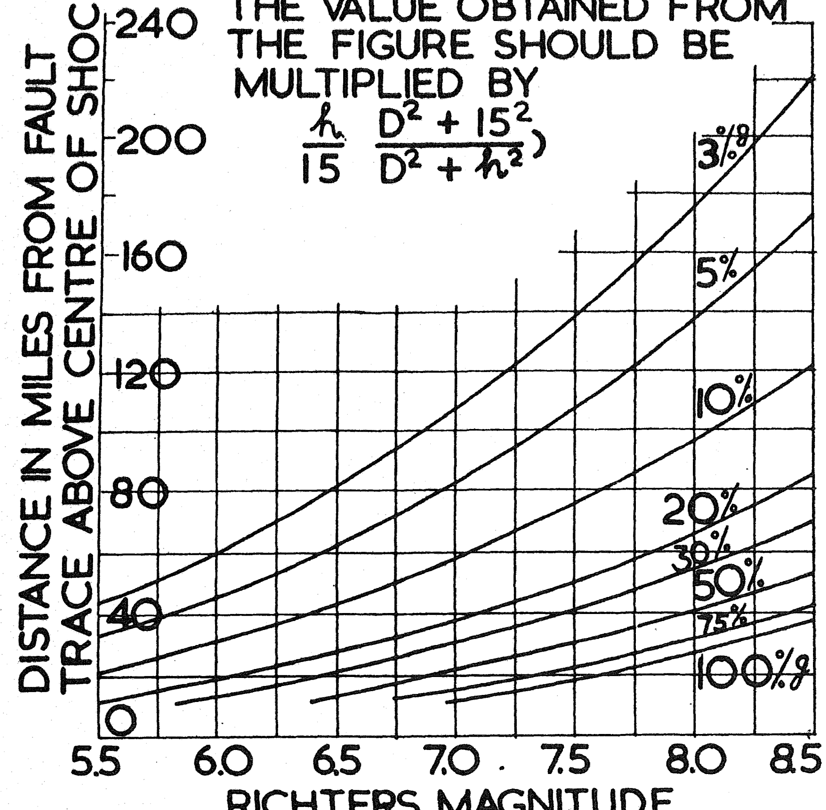
FIG.1-VARIATION OF GROUND ACCELERATION WITH DISTANCE DUE TO EARTHQUAKE SHOCKS OF DIFFERENT MAGNITUDES. (DEPTH OF CENTRE OF SHOCK ASSUMED 15 MILES BELOW GROUND LEVEL. FOR OTHER VALUES, THE VALUE OBTAINED FROM THE FIGURE SHOULD BE MULTIPLIED BY

$$\frac{h}{15}\ \frac{D^2 + 15^2}{D^2 + h^2})$$



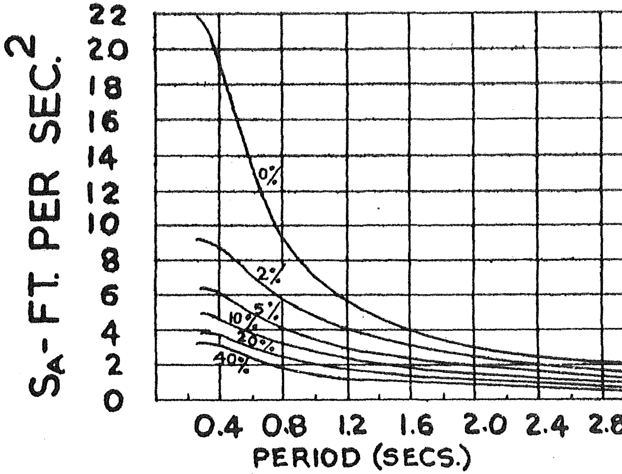


FIG.2 AVERAGE ACCELERTION SPECTRUM CURVES.

MULTIPLYING FACTORS.

1.	El CENTRO, 18 MAY 1940	2.7
2.	El CENTRO, 30 DEC. 1940	1.9
3.	OLYMPIA, 13 APRIL 1949	19
4.	TAFT, 21, JULY 1952	1.6
5.	VERNON, 10 MARCH 1933	1.5

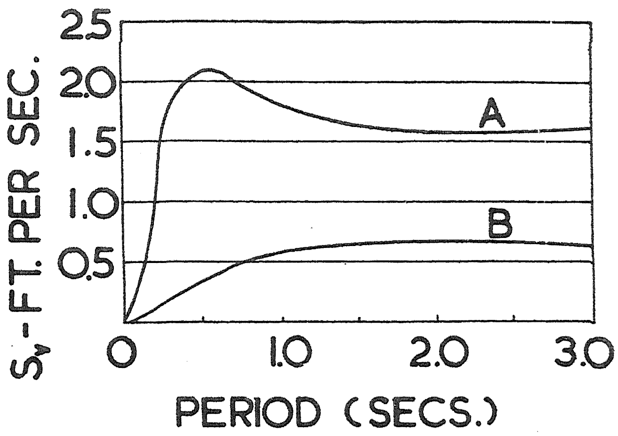


FIG. A - UNDAMPED VELOCITY SPECTRA.
A. ± 25 MILES FROM CENTRE OF LARGE SHOCK.
B ± 70 MILES FROM CENTRE OF LARGE SHOCK.

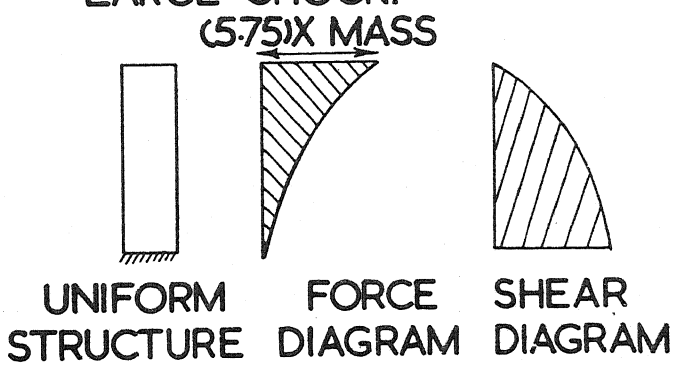


FIG. 3.

Table 1

Comparison of Recorded and Calculated Accelerations

(D 15 miles)

No.	Location	D	h	M	Maximum Acc. Recorded		Acc. as calculated
					(Multiples of		'g')
		(Miles)			(both comp.)		
1.	El Centro, May 18, 1940	30	15	7.1	0.33	0.23	0.33
2.	El Centro Dec. 30, 1934	35	15	6.5	0.26	0.20	0.16
3.	Olympia, Wash April 13, 1949	45	45	7.1	0.31	0.18	0.28
4.	Taft, July 21, 1952	40	15	7.7	0.18	0.17	0.40
5.	Vernon, March 10, 1933	28	15	6.3	0.19	0.13	0.16
6.	St. Barbara, June 30, 1941	15	19	5.9	0.24	0.23	0.22
7.	Ferendale, Oct. 3, 1941	50	15	6.4	0.13	0.12	0.075
8.	L. A. Subway Terminal, May 10, 1933	33	15	6.25	0.065	0.04	0.11
9.	Seattle, Wash April 13, 1949	55	45	7.1	0.075	0.058	0.20
10.	Hollister, Cal March 9, 1949	10	15	5.3	0.23	0.11	0.18
11.	Helena, Oct. 31, 1935	15	25	6.0	0.16	0.14	0.19
12.	Ferendale, Feb 9, 1941	75	15	6.6	0.075	0.040	0.04
13.	Vernon, Cal. Oct. 2, 1933	17	15	5.3	0.12	0.085	0.10
14.	Ferendale, Cal. Feb 9, 1941	75	15	6.6	0.075	0.04	0.04
15.	L. A. Subway Terminal, Oct. 2, 1933	22	15	5.3	0.065	0.06	0.07