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GROUND ACCELERATIONS DURING EARTHQUAKES

Jai Krishna* and Brijesh Chandra†

Abstract

For computation of structural response in seismic regions, the maximum anticipated accelerations and the form of an accelerogram are required. Since the programme of installing strong motion instruments over the entire seismic zone takes time and it takes many years to get data from such instruments, it is necessary in most countries to take advantage of the data already collected and establish relationships expressing accelerations as functions of the magnitude, focal depth of anticipated earthquake and the distance of the site from the known epicentres or active faults. This paper presents such a relationship based on data obtained in the past earthquakes.

Introduction

Two approaches are generally used for determining design coefficients for engineering structures in seismic zones. One is based on the 'Intensity' concept and the other on 'Energy' criterion. The latter derives the ground accelerations from the magnitude of the shock.

Attempt has been made to relate ground accelerations, worked out by studying the overturning and sliding of objects, with the intensities assigned on the basis of personal feeling and judgment (1,2,3). Gutenberg and Richter (4) have proposed a logarithmic relationship between maximum ground acceleration 'a' and intensity 'I' as follows :

$$\log_e a = \tfrac{1}{3} I - 0.5 \tag{1}$$

Hershberger (2) studied 108 strong motion records from 60 earthquakes in United States during 1947–1954 and gave a relationship

$$\log_e a = \tfrac{3}{7} I - 0.9 \tag{1a}$$

Medvedev (3) gives a table showing ground accelerations against intensities.

All the above studies reveal that variation in ground accelerations for the same intensity is so great that such values are hardly of significance for engineering purposes. Taking an average of such widely different quantities is also not sound.

* Professor and Director, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, India.

† Reader, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, India.

Under the circumstances, it may be more appropriate to use 'magnitude', which is an instrumental measurement of the shock intensity, for relating it to the ground motion.

Gutenberg and Richter (4) assume that intensity of ground motion is proportional to cube root of energy release. This energy is calculated from the relationship,

$$\log_{10}E = 9.4 + 2.14\,M - 0.054\,M^2 \tag{2}$$

(This has now been modified as $\log_{10} E = 11.4 + 1.5\,M$)

Also it is assumed that ground motion intensity is inversely proportional to the square of distance from the centre of disturbed mass. The ground motion-distance relationship has been borne out by results of blasts recorded by Carder and Cloud (5). The relationship results in the following form :

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

where 'a' is the acceleration, 'E' the energy release (obtained from equation 2), 'D' the epicentral distance, 'h' focal depth. 'C' is a contant absorbing the effects of ground conditions The author (6) worked out the values of 'a' for a number of strong motion earthquakes for which instrumental data was available. These values compared very well with the recorded values of accelerations. The ground conditions were assumed similar for all cases.

Eqn. 3 above would give exceedingly large accelerations for earthquake magnitudes greater than 7, specially near the epicentre. Housner (7) believes that there seems to be a limit to the ground motion that can occur during an earthquake however big the shock may be. The bigger is the magnitude of an earthquake, the greater is the area with high intensity of motion and, therefore, the highest intensity of ground motion near the epicentre does not greatly differ from that during moderate shocks. A value of 0.5 g has been suggested by Housner. This value is only a rough estimate of expected accleration. During Koyna earthquake 1967 (Magnitude 6.5), the max. acceleration recorded was 0.63 g. Also in Assam shock of 1897 (Magnitude 8.5) throwing up of boulders from mountain slopes was an indication of accelerations to the tune of 0.6–0.65 g. It is significant that the maximum ground motion in both the shocks was about the same, but the extent of areas that suffered damage was widely different in the two cases. It may therefore be appropriate to assume a certain limiting value of the maximum accelerations. This has been incorporated in the recommendations made in the present paper.

Form of New Relationship

In the present analysis, it is assumed that the soil is firm at the point where acceleration is to be estimated. At sites having loose deposits of alluvial soils, the dominant design factor is "settlement". If the foundation of structure at such sites is taken to firm foundation or supported on piles firmly held by the soil, this approach will be reasonably applicable. In countries where probable epicentres of strong motion earthquakes are in hilly regions, the formula suggested in this paper will be found to give reasonably good estimate of ground accelerations.

The solution of the problem is sought in two parts —

(i) Calculation of epicentral acceleration (a_0) and (ii) attenuation of acceleration with distance.

With regard to (i), good deal of help is taken from reference (6). As indicated in these studies, epicentral accelerations predicted are very high for earthquake magnitudes, greater than 7. It is proposed to have a limiting value of 0.65 g for maximum accelerations resulting from the biggest shock near the epicentre and it is assumed that the focal depth (h) affects accelerations inversely. Using these, and also the acceleration-magnitude relationship near epicentre from reference 6, the following relationship is obtained :

$$\frac{a_0}{g} = \frac{2.925\,\dfrac{10^{(M-5)}}{h}}{1+4.5\,\dfrac{10^{(M-5)}}{h}} \tag{4}$$

with the proviso that the limiting value of 'a_0' in the above eqn. is 0.65 g.

Regarding attenuation of acceleration, it is postulated that shallow focus shocks die out faster whereas attenuation is much slower in deep focus shocks. It is assumed that this attenuation is an exponential function of the $(D/h)^{3/2}$. Expressing acceleration (a) at any distance D,

$$a = a_0 e^{-\alpha\,(D/h)^{3/2}} \tag{5}$$

where 'α' is an arbitrary constant. Parkfield earthquake of 1966 has offered 'a' values for more than one location, and therefore provide useful data for evaluating 'α'. According to this, 'α' works out as 0.26.

Therefore,

$$\frac{a}{g} = \frac{2.925\,\dfrac{10^{(M-5)}}{h}}{1+4.5\,\dfrac{10^{(M-5)}}{h}}\; e^{-0.26\,(D/h)^{3/2}} \tag{6}$$

Equation 6 expresses maximum ground accelerations at a distance 'D', from the epicentre of an earthquake with magnitude 'M' and focal depth 'h'. Fig. 1 shows this relationship in graphical form.

Table 1 lists the strong motion earthquakes of this century recorded in firm ground conditions. Acceleration values as calculated from eqn. (6) are listed against the maximum recorded values. A good comparison can be seen from this table.

Pattern of Accelerogram

Another important point to be considered is regarding the form of accelerogram to be used for design of structures in various regions. Parkfield earthquake 1966 (Magnitude 5.4) and Koyna earthquake of 1967 (Magnitude 6.5) have offered typical records of ground motion in epicentral regions while El Centro earthquake of 1940 (Magnitude 7.1) offers a typical record for ground motion around 30 miles from epicentre. Similarly Ferendale earthquake of Oct. 3, 1941 offers a record at 50 miles away from the epicentre.

Knowing the maximum ground acceleration and the pattern of accelerogram, input data for computations of structural response could be readily prepared.

Table 1

Comparison of Calculated and Recorded Accelerations in Regions with Firm Ground Conditions

S. No.	Particulars		M	D	h	D/h	$(D/h)^{3/2}$	$0.26 \left(\frac{D}{h}\right)^{3/2}$ A	e^{-A}	a_o	a	Recorded Accn.
1.	El Centro	May 18, 1940	7.1	30	15	2.00	2.83	0.735	0.480	0.633	0.304	0.33
2.	El Centro	Dec. 30, 1934	6.5	35	15	2.335	3.56	0.925	0.397	0.588	0.233	0.26
3.	Taft	Jul. 21, 1952	7.7	40	15	2.666	4.35	1.130	0.323	0.646	0.208	0.18
4.	Ferendale	Oct. 3, 1941	6.4	50	15	3.333	6.08	1.580	0.206	0.574	0.118	0.13
5.	Hollister	Mar. 9, 1949	5.3	10	15	0.66	0.544	0.141	0.868	0.243	0.211	0.23
6.	Ferendale	Feb. 9, 1941	6.6	75	15	5.00	11.20	2 910	0.055	0.600	0.033	0.075
7.	Olympia	Apr. 13, 1949	7.1	45	45	1.00	1.00	0.260	0.770	0.602	0.464	0.31
8.	Parkfield	Jun. 27, 1966	5.4	06	03	2.00	2.83	0.735	0.480	0.524	0.251	0.24
9.	St. Barbara	Jun. 30, 1941	5.9	15	19	0.79	0.702	0.182	0.833	0.424	0.354	0.24
10.	Helena	Oct. 31, 1935	6.0	15	25	0.60	0.465	0.121	0.886	0.418	0.370	0.16
11.	Sanfrancisco	Mar. 22, 1957	5.3	7.8	07	1.115	1.18	0.306	0.735	0.38	0.267	0.13
12.	Koyna	Dec. 11, 1967	6.5	03	05	0.60	0.465	0.121	0.886	0.628	0.567	0.63
13.	El Centro	Apr. 9, 1968	6.5	40	12.5	3.20	5.72	1.485	0.226	0.595	0.135	0.14

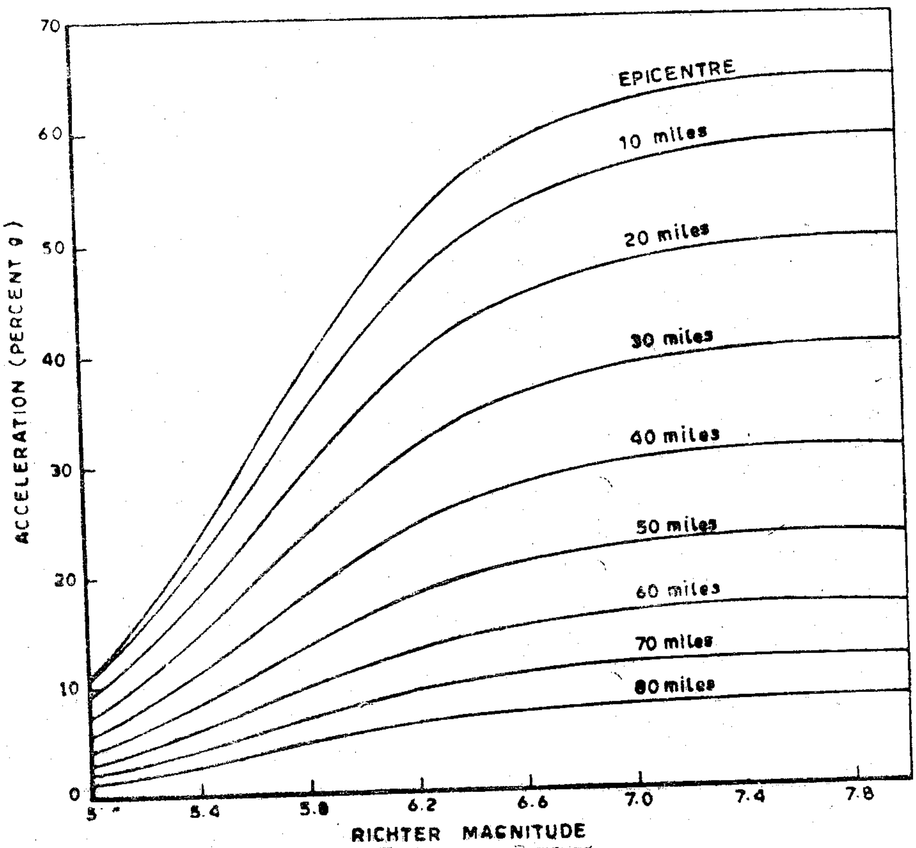

Fig. 1(a). Magnitude-Distance-Acceleration Curves (Depth of Focus=20 miles)

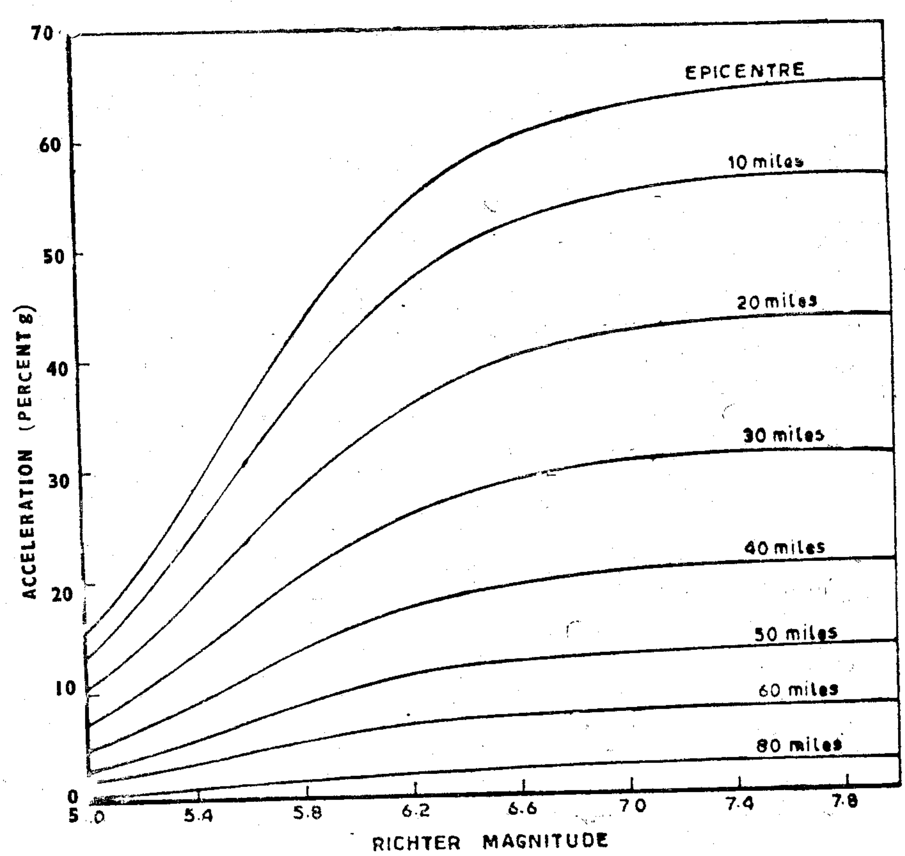

Fig. 1(b). Magnitude-Distance-Acceleration Curves (Depth of Focus=15 miles)

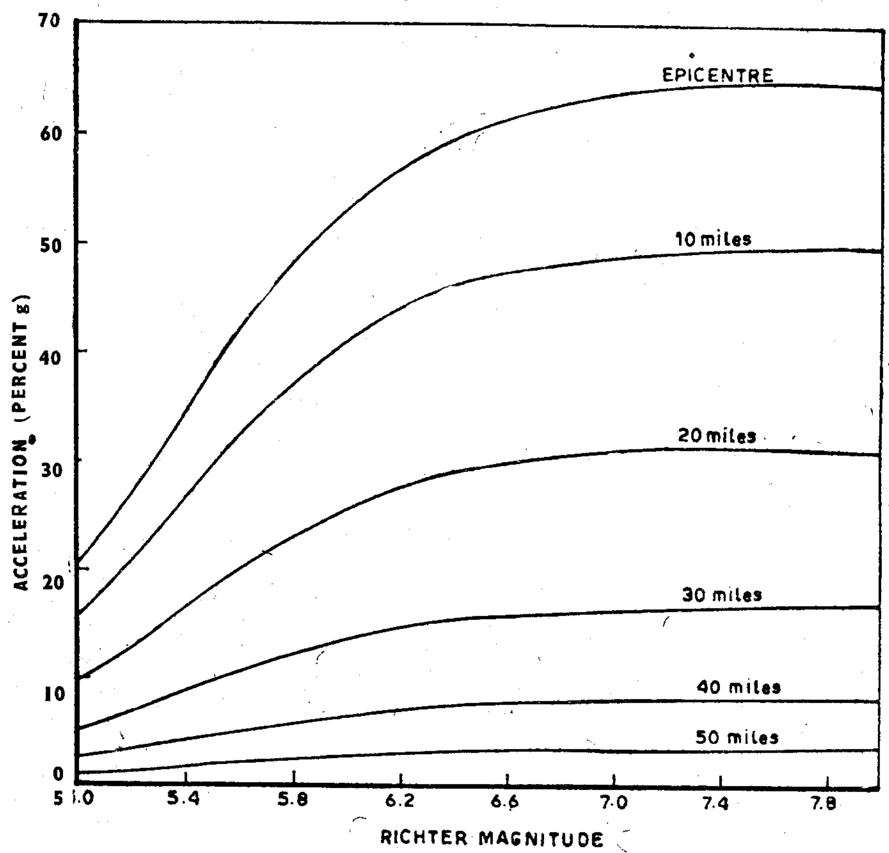

Fig. 1(c) Magnitude Distance Acceleration Curves (Depth of Focus = 10 miles)

Conclusions

For regions where firm soil conditions exist, eqn. 6 can be used to estimate maximum ground accelerations. Parkfield or Koyna accelerogram pattern could be used for sites close to epicentres, while El Centro and Ferendale patterns are recommended for epicentral distances of about 30 and 50 miles. Beyond these distances, "acceleration" will tend to to be less dominant a factor for design compared with "amplitudes" of vibration in comparatively long-period structures.

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IMPACT DAMPERS

A.R. Chandrasekaran*, Y.P. Gupta and L.R. Gupta*****

Synopsis

The application of impact dampers for systems subjected to earthquake type excitations has been investigated. The parent system was idealised as a linear single degree of freedom system. Impact is produced by the collision of a single particle mass against the wall of a container rigidly mounted on the parent system. This study indicates that forces on parent system could be reduced by such devices.

Introduction

The use of impact dampers has so far been studied for systems subjected to steady state excitations(1,2,3)†. An impact damper primarily reorganises the vibrating pattern of a physical system. If the impact is so adjusted that the amplitude of vibration decreases, then the force on parent system is reduced.

An impact damper is a device which reduces the vibration amplitude of a system through the mechanism of momentum transfer by collision and conversion of mechanical energy into heat. It essentially consists of a mass particle within a container, which is fixed to the parent system, such that the particle has specified freedom to move relative to the container. The energy of the mass particle is dissipated in impact.

Here, a study has been made of the application of single mass impact dampers to linear single degree of freedom systems subjected to earthquake excitations. The equations of motion have been given and results obtained for various values of parameters involved in the problem. The efficiency of the impact damper has been worked out for various cases and among them, the maximum reduction was of the order of forty percent.

Equations of Motion

The parent system has been idealised as a linear single degree of freedom system having a mass M, viscous damping constant C and spring constant K. The single particle has a mass m which moves in a frictionless container and has a clearance d in which it is free to oscillate. A single translational component of the ground motion is only considered (Fig. 1).

Between impacts, the equation of motion of the parent system is given by,

$$M\ddot{X}_1 + C(\dot{X}_1 - \dot{y}) + K(X_1 - y) = 0 \qquad (1)$$

where X_1 is the absolute displacement of mass M and y is the ground displacement.

Dividing equation 1 by M and subtracting $\ddot{y}$ from both sides,

* Professor of Structural Dynamics, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee.

** Lecturer in Structural Dynamics, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee

*** Lecturer in Civil Engineering, Regional Engineering College, Srinagar.

† Refers to serial number of references listed at the end.

$$\ddot{Z}_1 = -(\ddot{y} + 2\,p\,\zeta\,\dot{Z}_1 + p^2 Z_1) \tag{2}$$

where $Z_1 = X_1 - y$, the displacement of the mass M, relative to the base.

$p = \sqrt{K/M}$, the undamped circular natural frequancy of the parent system, also, equal to $2\pi/T$, where T is the undamped natural period of the system, also, equal to $\sqrt{g/\delta_{st}}$ where δ_{st} is the static deflection of the parent system and g is the acceleration due to gravity.

$\zeta = \dfrac{C}{2\sqrt{KM}}$, percentage of critical damping of the parent system.

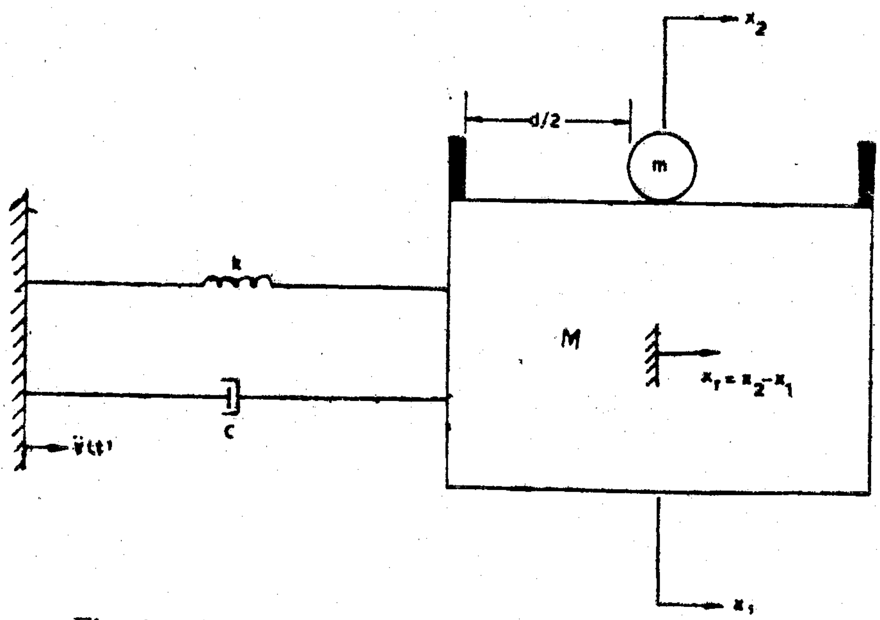

Fig. 1. A mathematical model of a damped elastic system with an impact damper

If X_2 is the absolute displacement of the particle m, then, between impacts, its equation of motion is given by $m\ddot{X}_2 = 0$ (as there is no friction and no spring force). Since m is not zero,

$$\ddot{X}_2 = 0 \tag{3}$$

If X_r is the relative displacement between the masses m and M then, $X_r = X_2 - X_1$

and $$\ddot{X}_r = \ddot{X}_2 - \ddot{X}_1 = -\ddot{X}_1 \ (\text{as } \ddot{X}_2 = 0) \tag{4}$$

From equations 1 and 4,

$$\ddot{X}_r = 2\,p\,\zeta\,\dot{Z}_1 + p^2 Z_1 \tag{5}$$

Since the duration of the impact is very small compared to the period of vibration of the system, it has been assumed that during impact no change in displacement takes place and only velocity changes. If subscripts $-$ and $+$ indicate respectively quantities preceding and following an occurrence then,

at $t = t_{1-}$; $X_r = d/2$, $Z_1 = Z_{11-}$, $\dot{Z}_1 = \dot{Z}_{11-}$

and,

at $t = t_{1+}$; $X_r = d/2$, $Z_1 = Z_{11-}$, $\dot{Z}_1 = \dot{Z}_{11+}$ (6)

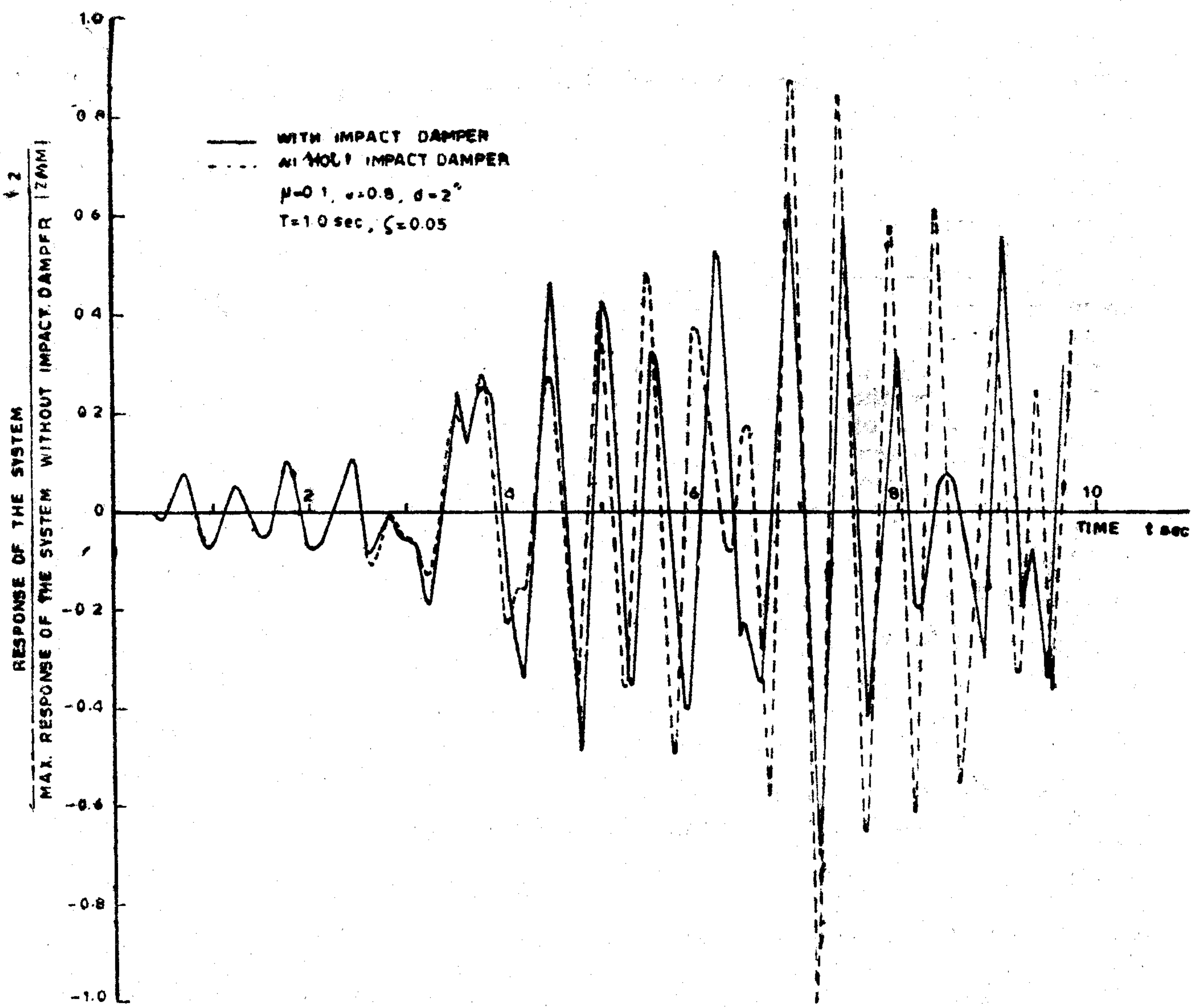

Fig. 2. Influence of impact damper, Taft earthquake.

During impact, momentum equation has to be satisfied. That is,

$$M\,\dot{X}_{1-} + m\,\dot{X}_{2-} = M\,\dot{X}_{1+} + m\,\dot{X}_{2+} \tag{7}$$

Subtracting $(M + m)\,\dot{y}$ from both sides of equation 7,

$$M\,(\dot{X}_{1-} - \dot{y}) + m\,(\dot{X}_{2-} - \dot{y}) = M\,(\dot{X}_{1+} - \dot{y}) + m\,(\dot{X}_{2+} - \dot{y}) \tag{8}$$

or $$M\,\dot{Z}_{1-} + m\,\dot{Z}_{2-} = M\,\dot{Z}_{1+} + m\,\dot{Z}_{2+}$$

From the definition of co-efficient of restitution, e,

$$\dot{X}_{r+} = -\,e\,\dot{X}_{r-} \tag{9}$$

Since $X_r = X_2 - X_1$ and $Z_1 = X_1 - y$, equation 9 can be represented as

$$\dot{Z}_{2+} - \dot{Z}_{1+} = -e\,(\dot{Z}_{2-} - \dot{Z}_{1-}) \tag{10}$$

From equations 8 and 10,

$$\dot{Z}_{1+} = \dot{Z}_{1-} + \frac{\mu\,(1 + e)}{1 + \mu}\,\dot{X}_{r-} \tag{11}$$

where μ = mass ratio, m/M.

Summarising, equations 2 and 5 govern the motion of the masses between two successive impacts. These equations could be solved, using numerical techniques, to obtain velocities $\dot{Z}_1$ and $\dot{X}_r$ and displacements Z_1 and X_r at any instant of time. At the time of impact, defined by $X_r = d/2$, displacements, Z_1 and X_r do not change but velocities, which become discontinuous functions, do change and are given by equations 9 and 11.

Variables

The following variables are involved in the problem :—(i) Undamped natural period, T, and percentage of critical damping, ζ, of the parent system, (ii) ratio of mass of particle to that of parent system, μ, (iii) co-efficient of restitution, e, (iv) clear distance, d, of the container in which the particle is free to oscillate smoothly and (v) ground motion, $\ddot{y}$.

T has two values, namely 0.5 and 1.0 second and ζ was taken as 0.05. μ has values ranging between 0.1 and 0.3 and e between 0.0 to 1.0. d/δ_{st} had values between 0.30 and 2.00. Two ground motion data, namely, (i) NS component of El Centro, May 18, 1940, and (ii) S 21 W component of Taft, July 21, 1952, were utilised.

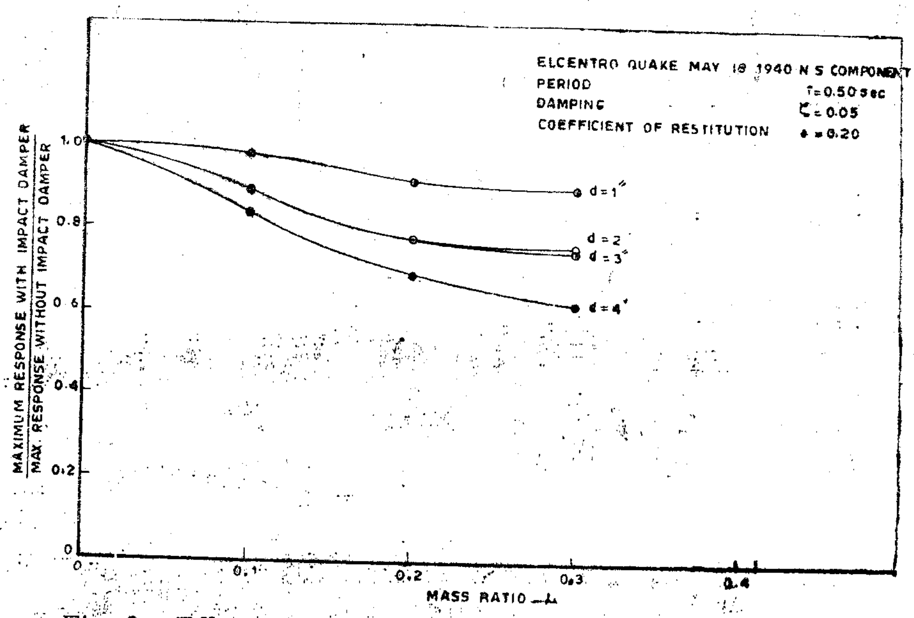

Fig. 3. Effect of mass ratio on the response of the system

The lateral shear force on the parent system is directly proportional to its displacement Z_1, relative to the base. The ratio of displacement response of the parent system with impact damper to that without it has been worked out for all the cases. If Z_M denotes maximum displacement with impact damper and Z_{MM} without it, then the above ratio is equal to Z_M / Z_{MM} and the efficiency of the impact damper is given by $1 - Z_M / Z_{MM}$.

It is known that for a linear system, response decreases with increase in damping. Therefore, the influence of the impact damper can also be represented as an increase in the damping of the system. The results are also expressed as an increase in percentage of critical damping over that already inherent in the system (namely, above 0.05).

Results

Tables 1, 2, 3 and 4 give the efficiency of impact damper as well as equivalent increase in damping factor for the various cases that have been solved. The influence of the various parameters are discussed below.

(a) *Time wise Response*

Figure 2 shows a typical time-wise response of the parent system with and without impact damper for a particular combination of variables. It is seen that the impact damper reduces the amplitude of vibration and is particularly effective at large amplitudes.

(b) *Effect of Mass Ratio*

Figures 3 and 4 show the influence of mass ratio on the response of the system. It is seen that the efficiency of damper increases with increase in mass ratio.

(c) *Effect of Co-efficient of Restitution*

The variation of response of primary system with coefficient of restitution is given in figures 5 and 6. No definite pattern of variation is perceptible from these results.

(d) *Effect of Clearance*

From the tables, it could be seen that for the various cases tried, the efficiency of damper generally increases with clearance.

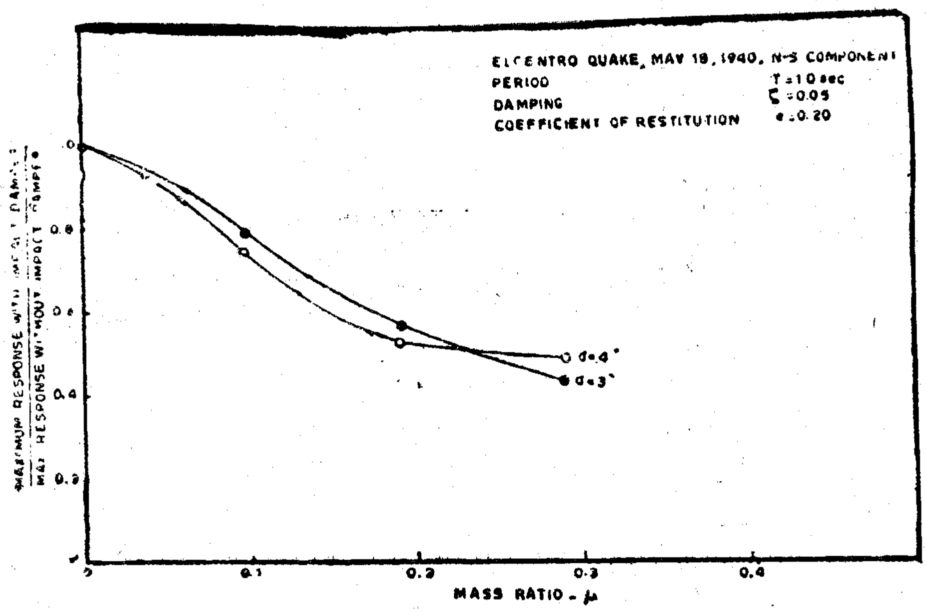

Fig. 4. Effect of mass ratio on the response of the system

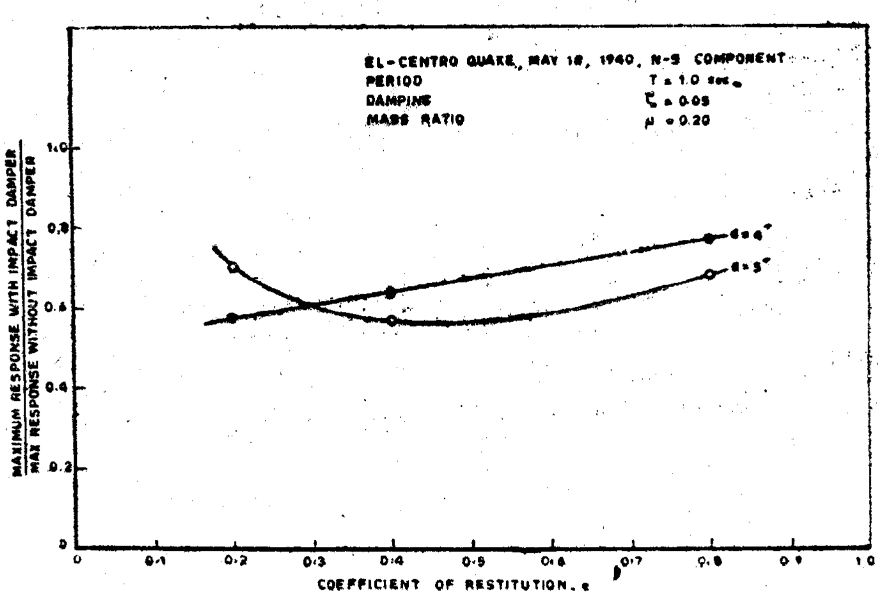

Fig. 5. Effect of coefficient of restitution on the response of the system

(e) *Effect of Period*

Only two periods have been tried. It is seen that impact damper is more efficient for longer period systems.

Conclusion

The analysis indicates that the impact damper could be used to reduce the response of systems subjected to earthquake type excitations. Among the various cases tried, the maximum reduction in response was of the order of forty percent. Probably, a multiple particle impact damper with variable clearances may give greater reduction in response.

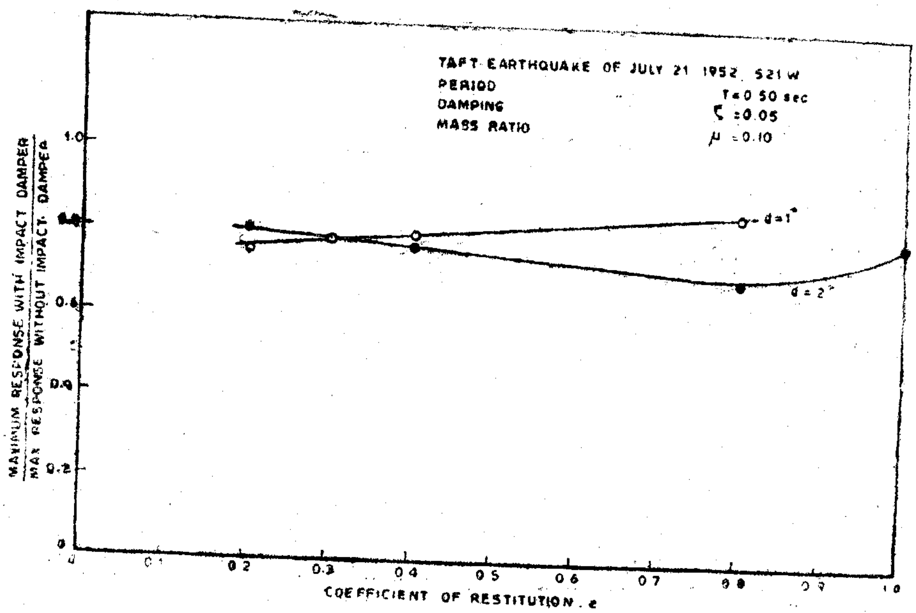

Fig. 6. Effect of coefficient of restitution on the response of the system

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TABLE 1

Maximum Displacement Response of the System to El Centro, Earthquake, May 18, 1940, N–S Component.

Period of primary system $T = 0.5$ sec.

Damping of primary system $\zeta = 0.05$

Maximum displacement response of primary system alone $Z_{MM} = 2.345$ inch

Static deflection $\delta_{st} = 2.450$ inch

Case No.	Mass Ratio μ	Coeff. of Restitution e	Clearance d (inch)	d/δ_{st}	Max. Disp. Z_M (inch)	Efficiency $1-\frac{Z_M}{Z_{MM}}$	Equivalent increase in Damping
1	0.10	0.2	1.0	0.408	2.464	2.70	0.665
2	0.10	0.3	1.0	0.408	2.478	2.20	0.543
3	0.10	0.4	1.0	0.408	2.482	2.00	0.493
4	0.10	0.2	2.0	0.816	2.248	11.25	2.775
5	0.10	0.3	2.0	0.816	2.245	11.34	2.800
6	0.10	0.4	2.0	0.816	2.232	11.86	2.920
7	0.10	0.2	3.0	1.225	2.238	11.62	2.870
8	0.10	0.3	3.0	1.225	2.244	11.41	2.810
9	0.10	0.4	3.0	1.225	2.254	11.00	2.710
10	0.10	0.2	4.0	1.634	2.110	16.70	4.120
11	0.10	0.3	4.0	1.634	2.170	14.30	3.520
12	0.10	0.4	4.0	1.634	2.225	12.13	3.000
13	0 20	0.2	1.0	0.408	2.317	8.52	2.100
14	0.20	0.3	1.0	0.408	2.292	9.50	2 340
15	0.20	0.4	1.0	0.408	2.335	7.82	1.930
16	0.20	0.2	2.0	0.816	1.955	22.81	5.750
17	0.20	0.3	2.0	0.816	1.969	22.27	5.610
18	0.20	0.4	2.0	0.816	1.956	22.78	5.765
19	0.20	0.2	3.0	1.225	1.966	22.38	5.625
20	0.20	0.3	3.0	1.225	1.989	21 47	5.361
21	0.20	0.4	3.0	1.225	2.005	20.82	5.160
22	0.20	0.2	4.0	1.634	1.746	31.06	8.320
23	0.20	0.3	4.0	1.634	1.775	29.90	7.960
24	0.20	0.4	4.0	1.634	1.828	27.80	7.310

Table 2

Maximum Displacement Response of the System to El Centro, Earthquake May 18, 1940, N–S Component.

Period of primary system, $T = 1.0$ sec.

Damping of primary system, $\zeta = 0.05$

Maximum displacement response of primary system alone, $Z_{MM} = 4.591$ inch

Static Deflection, $\delta_{st} = 9.775$ inch

Case No.	Mass Ratio μ	Coeff. of Restitution e	Clearance d (inch)	d/δ_{st}	Maxm. Disp. Z_M (inch)	Efficiency $1-\frac{Z_M}{Z_{MM}}$	Equivalent increase in damping
1	0.1	0.2	3.0	0.306	3.699	19.41	2.900
2	0.1	0.4	3.0	0.306	3.920	14.60	2.185
3	0.1	0.2	4.0	0.410	3.484	24.10	3.600
4	0.1	0.4	4.0	0.410	3.724	18.88	2.825
5	0.1	0.8	4.0	0.410	4.252	7.37	1.103
6	0.2	0.2	3.0	0.306	2.852	37.87	6.367
7	0.2	0.4	3.0	0.306	3.189	30.52	4.570
8	0.2	0.2	4.0	0.410	2.657	42.12	7.970
9	0.2	0.4	4.0	0.410	2.901	36.80	6.040
10	0.2	0.8	4.0	0.410	3.526	23.18	3.470
11	0.2	0.2	5.0	0.512	3.234	29.55	4.430
12	0.2	0.4	5.0	0.512	2.588	43.62	8.120
13	0.2	0.8	5.0	0.512	3.104	32.39	4.850

Table 3

Maximum Displacement Response of the System to Taft Earthquake, July 21, 1952 S21 W, Component.

Period of primary system, $T = 0.5$ sec.

Damping of primary system, $\zeta = 0.05$

Maximum displacement response of primary system alone, $Z_{MM} = 0.909$ inch

Static Deflection, $\delta_{st} = 2.450$ inch

Case No.	Mass Ratio μ	Coeff. of Restitution e	Clearance d (inch)	d/δ_{st}	Maxm. Disp. Z_M (inch)	Efficiency $1-\frac{Z_M}{Z_{MM}}$	Equivalent increase in damping
1	0.1	0.2	1.0	0.408	0.728	20.00	3.820
2	0.1	0.3	1.0	0.408	0.705	22.50	4.295
3	0.1	0.4	1.0	0.408	0.716	21.30	4.070
4	0.1	0.8	1.0	0.408	0.775	14.72	2.810
5	0.1	0.2	2.0	0.816	0.679	25.25	4.870
6	0.1	0.3	2.0	0.816	0.704	22.64	4.310
7	0.1	0.4	2.0	0.816	0.693	23.80	4.540
8	0.1	0.8	2.0	0.816	0.630	30.69	6.402
9	0.1	1.0	2.0	0.816	0.721	20.66	3.940
10	0.1	0.0	5.0	2.040	0.904	0.54	0.103
11	0.1	1.0	5.0	2.040	0.902	0.76	0.150
12	0.2	0.2	1.0	0.408	0.664	26.96	5.237
13	0.2	0.3	1.0	0.408	0.672	26.08	4.980
14	0.2	0.2	2.0	0.816	0.595	34.52	7.600
15	0.2	0.2	3.0	1.225	0.883	2.92	0.557
16	0.2	0.2	5:0	2.040	0.893	1.71	0.326

TABLE 4

Maximum Displacement Response of the System to Taft Earthquake, July 21, 1952, S21 W, Component.

Period of primary system	T	$= 1.0$ sec.
Damping of primary system	ζ	$= 0.05$
Maximum displacement response of primary system alone	Z_{MM}	$= 1.778$ inch
Static deflection	δ_{st}	$= 9.775$ inch

Case No.	Mass Ratio μ	Coeff. of Restitution e	Clearance d (inch)	d/δ_{st}	Max. Disp. Z_M (inch)	Efficiency $1-\frac{Z_M}{Z_{MM}}$	Equivalent increase in Damping
1	0.1	0.0	3.0	0.306	1.488	16.31	3.980
2	0.1	0.0	5.0	0.512	1.631	8.27	2.020
3	0.1	0.1	5.0	0.512	1.614	9.22	2.250
4	0.1	0.2	5.0	0.512	1.573	11.53	2.810
5	0.1	0.4	5.0	0.512	1.505	15.34	3.740
6	0.1	1.0	5.0	0.512	1.451	18.40	4.440
7	0.2	0.2	4.0	0.410	1.315	26.04	7.310
8	0.2	0.4	4.0	0.410	1.282	27.93	8.100
9	0.2	0.8	4.0	0.410	1.415	20.46	5.000
10	0.2	0.0	5.0	0.512	1.558	12.36	3.013
11	0.2	0.1	5.0	0.512	1.497	15.83	3.860
12	0.2	0.2	5.0	0.512	1.445	18.75	4.575
13	0.2	0.4	5.0	0.512	1.434	19.38	4.720
14	0.2	0.8	5.0	0.512	1.411	20.65	5.060
15	0.2	1.0	5.0	0.512	1.400	21.28	5.325

Bulletin of the Indian Society of Earthquake Technology. Vol. 6, No. 1, pp 17–22 March, 1969

ROORKEE SEISMOSCOPE

P. N. Agrawal*

Abstract

A simplified earthquake recorder, termed as the Roorkee Seismoscope has been developed for earthquake engineering requirement of determining response for idealised dynamic models of structures in those regions where expected acceleration response spectral values are lower than 30% of gravity. The design features of the Roorkee Seismoscope have been discussed in relation to the existing Structural Response Recorder. For smaller accelerations, the new seismoscope would yield more useful records.

Introduction

Specially designed simplified recorders are now finding increased applications in strong motion recording programmes for earthquake engineering requirement of determining spectral response values for idealised dynamic models of structures. The reason for their increased applications are (i) low cost for providing satisfactory instrument coverage and (ii) simple and easy interpretation of records in terms of results, directly usable for design purposes. Most of the records produced by such recorders during earthquakes in India (Agrawal 1969) since the initiation of this programme have been of very small sizes for the reasons that (i) the instrument tilt sensitivity in its present design is low, specially so for the shorter period recorders which are of greater interest in the earthquake engineering applications, (ii) the earthquakes recorded have been of moderate size and (iii) generally the recorders have been located at some distance from the epicentre. These earthquake records have therefore been utilised after making photographic enlargements, resulting in increased error in recorded amplitude measurements. Also, as pointed out earlier by Hudson and Cloud (1967), at such low amplitude the damping due to stylus friction increases. Thus, the contribution due to both the stylus friction and eddy currents caused by permanent magnetic field to give 5% and 10% of critical damping (equivalent) are of the same order at such low amplitudes of motion. This results in large uncertainties in determination of damping values. In the new design of the recorder reported here the stylus friction remains practically same at all amplitudes. Thus it reduces the uncertainty which existed in the previous design and can be usefully employed for obtaining small response values when installed along with the earlier recorder

Description of the Roorkee Seismoscope

Figure—1 shows the Roorkee Seismoscope. This is refered as Structural Response Recorder Model—3 in *Indian Strong Motion Earthquake Recording Programme*. It has a heavy circular base of 26.5 cm diameter. The total height of the instrument with its conical cover is 27 cm (not seen in Figure—1). The cover is fixed to the base by three knurled head screws. A handle is provided at the top of the cover for removing it. The total weight of the recorder is about 3.5 kg. A two legged support is fixed to the recorder base. A gimbal suspension, which can be raised or lowered and fixed to recoder support, holds the pendulum at its upper end retaining it in the vertical position and allowing it to have free motion in any azimuth. By using jewel points in the gimbal suspension the pendulum can be made to have practically undamped motion whereas by using suitable size electroplated steel ball in the gimbal suspension it is possible to obtain equivalent viscous damping in the range 2% to 15%. The variation of damping with amplitude has been studied and it is found to be more at

* Scientist, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee., U.P., India.

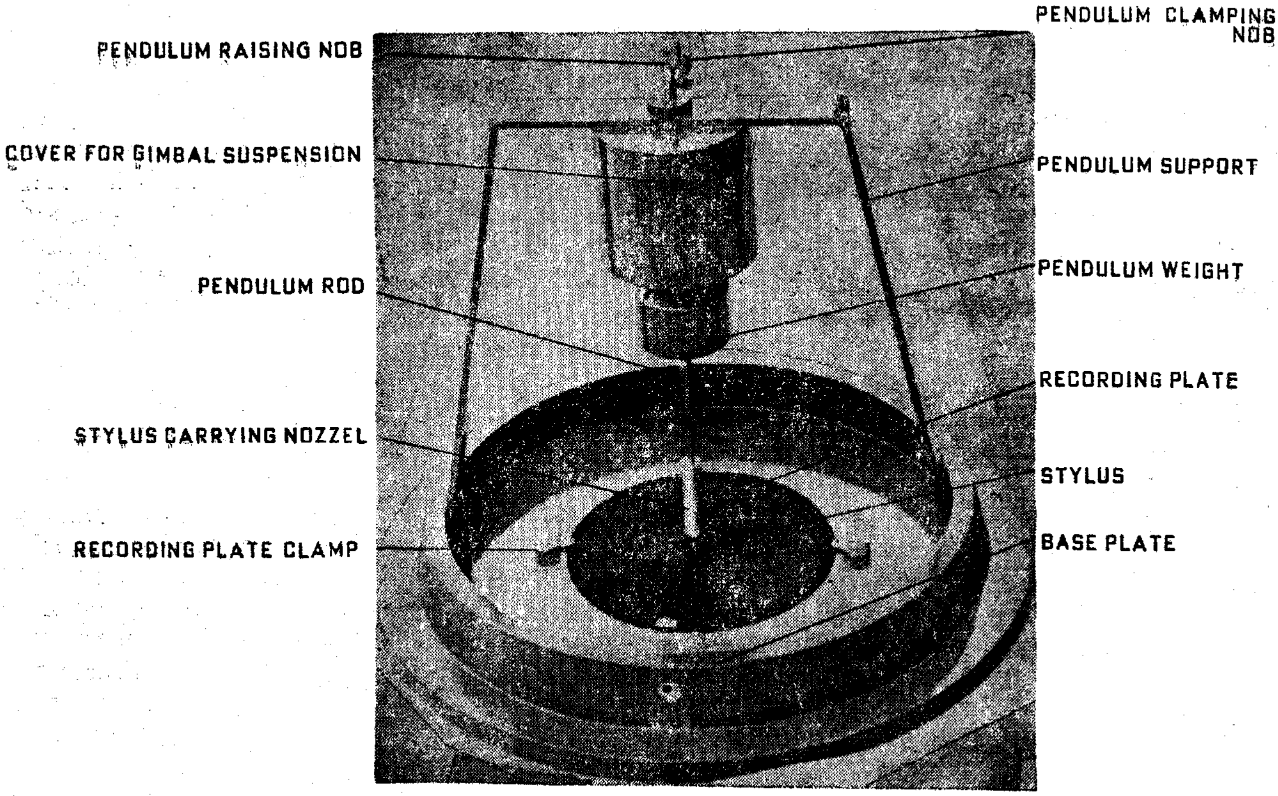

Fig. 1. Roorkee seismoscope (cover removed)

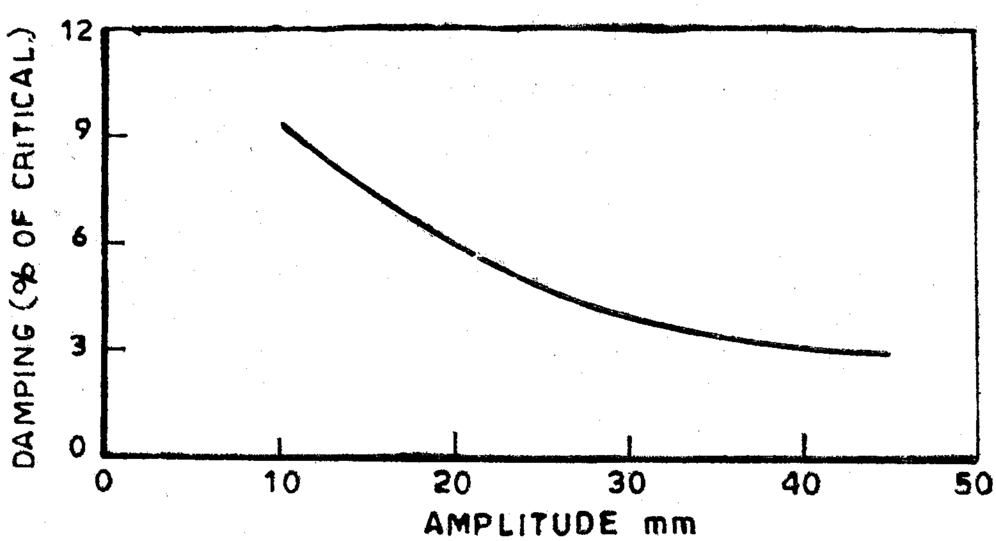

Fig. 2. Variation of damping with amplitude in Roorkee seismoscope

small amplitudes as shown in Figure—2. This variation can be accounted for by calculating damping at the amplitude of recording. At an amplitude of 10 mm which roughly corresponds to small size record of 1 mm on earlier recorder of 0.4 sec period, the damping is easily determinable and does not introduce uncertainties in the results. A protective cover is provided for the gimbal suspension. The pendulum is in the form of a rod carrying a heavy mass as shown in Figure—3. The position of the mass can be raised and lowered for period adjustment. The period is adjustable from 0.4 sec to 0.75 sec retaining the same length of the pendulum in equilibrium condition. The lower end of the pendulum carries a stylus of self adjusting length and scratches a record on the flat circular smoked glass of about 11 cm diameter held in the centre of the base plate. The stylus pressure required for suitable recording is only 1/4 gm. and has a negligible variation with amplitude of recording as the angular deflection for maximum trace amplitude is not large. The entire pendulum assembly could be raised by about 2 cm, bringing the stylus away from the recording plate. This is to avoid scratching of the recording plate while placing it for recording or at the time of removing the record.

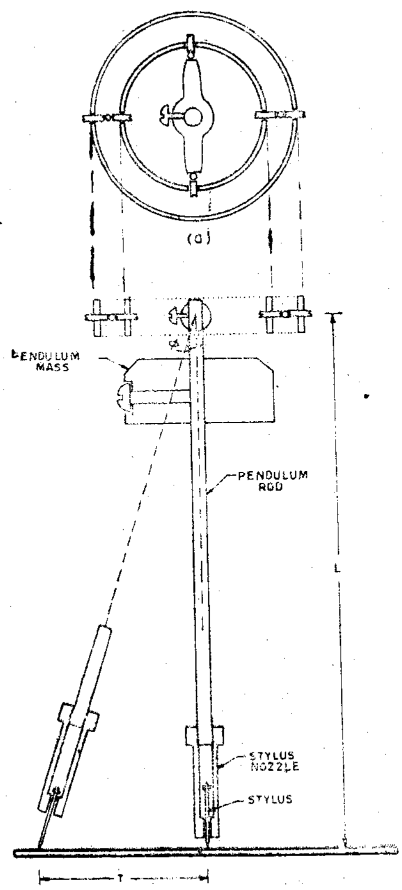

Fig. 3. Sytematic diagram of pendulum, its suspension and stylus.

To avoid water or moisture entering inside the recorder, which may spoil the recording plate, a circular hedging has been provided to the base plate which fits in to the cover. By providing a rubber seal to this hedging and a valve to the cover, the instrument could easily be evacuated aiding satisfactory maintenance of the recorder over long periods.

Dynamic Consideration

The instrument consists of a pendulum with Coulomb damping. Theoretically, the following would be the equation of motion for the system when subjected to an earthquake motion.

$$\ddot{\phi} + n^2\phi + h(\tau) = f(\tau) \qquad (1)$$

where ϕ is the angle of deflection of the pendulum from its position of rest.

n is the frequency.

$h(\tau)$ is the Meander function representing the Coulomb friction A.

$f(\tau)$ is an earthquake function.

The solution of the equation is :—

$$\phi = \phi_0 \operatorname{Cos} nt + \phi_0 \frac{\operatorname{Sin} nt}{n} + \frac{A}{n^2}(1-\operatorname{Cos} nt) + \frac{2A}{n^2}\sum_{k=0}^{n}\left[(-1)^k\left\{1-\operatorname{Cos} n\ (t-kT)\right\}\right] + \int_0^t \frac{1}{n}\operatorname{Sin} n\,(t-\tau)\, f(\tau)\, d\tau \qquad \ldots (2)$$

The first two terms correspond to the homogenous case of free vibrations, the next two to the friction and the last due to the earthquake motion.

Thus, rigorously speaking the present system can not be compared to the single degree of freedom linear viscous damped oscillator used generally to define the response spectrum and if the results of this recorder have to be utilised, a new definition of response spectrum on the basis of equation (2) would be necessary. But the assumption that the system could be considered as mechanical model of single degree of freedom linear viscous damped oscillator for this specific application finds support from following two considerations: (i) The records are to be utilised only to get the maximum deflection and not to obtain the pendulum movement with time, and (ii) the order of damping is very low.

Now let us examine the deviations of the earlier Structural Response Recorder from an ideal single degree of freedom linear viscous damped oscillator. At low amplitudes of motion if small damping was to be introduced then a substantial fraction of damping was due to Coulomb friction. On the other hand at intermediate and large amplitudes non-linear effects due to suspension would become significant. Also, as indicated by Cloud and Hudson (1961) at large amplitudes the damping torque associated with the vertical movement of pendulum would also become effective. Thus, the departures from the single degree of freedom linear viscous damped oscillator and the previous recorder are roughly of the same order as the new system. On the basis of these considerations the response of the Roorkee Seismoscope and the previous recorder were compared by shake table test. On one of the available shake tables two recorders—one of each type calibrated to same period and equivalent damping at average amplitude of records obtained in each case were

Table 1—Shake table seismoscope comparison test results.

	ROORKEE SEISMOSCOPE			PREVIOUS RECORDER		
A	Period 0.40 sec. Damping 5% of critical L* = 15 cm			Period 0.40 sec. Damping 5% of critical, Tilt sensitivity 1.733 cm/radian		
	Double Amplitude (max.) mm	ϕ degrees	Acceleration % of g	Double Amplitude (max.) mm	ϕ degrees	Acceleration % of g
1	93.5	17.28	30.15	10.8	17.88	31.2
2	33.0	6.28	10.95	4.0	6.61	11.5
B	Period 0.75 sec. Damping 5% of critical L = 15 cm			Period 0.75 sec. Damping 5% of critical, Tilt sensitivity 3.623 cm/radian		
	Double Amplitude (max.) mm	ϕ degrees	Acceleration % of g	Double Amplitude (max.) mm	ϕ degrees	Acceleration % of g
1	57.5	10.85	18.9	14.2	11.22	19.5
2	88	16.35	28.5	21.8	17.24	30.0

*Explained under tilt sensitivity.

fixed. A random jerk of suitable intensity was given by hammer impact. The maximum trace amplitudes obtained from both recorders for the same table motion were measured and acceleration response spectral values calculated (Table 1). This was done for 0.40 sec. and 0.75 sec period recorders with 5% of critical damping (equivalent). It is observed that the results from the two types of recorders agree. The utility of the earlier recorder is already established by simultaneous recording of an actual earthquake with strong motion accelerographs. Thus, the new seismoscope giving results equivalent to the earlier recorder is expected to prove satisfactory even in the field use and the records can be utilised as the records of the previous recorder.

Tilt Sensitivity

The most significant advantage of the new seismoscope lies in the higher tilt sensitivity which is independent of the period of the recorder. This is so because the pendulum length is not altered while making period adjustments and only the position of the mass is changed. This would imply that Roorkee Seismoscope would yield large records compared to the previous recorder for the same earthquake motion. However. since the recording is done on a flat glass plate, the sensitivity does not remain constant at all the amplitudes. The values of ϕ can be calculated using following formula (see Figure 3).

$$\phi = \tan^{-1}\left[\frac{T}{L}\right] \qquad (3)$$

where T is the maximum trace amplitude (single) recorded and L is the height of the point of pendulum suspension above the recording plate and is 15 cm in the recorder reported. The calibration for tilt sensitivity in the seismoscope would consist of only measuring the L. The seismoscope described here has reduced range to accommodate a record for acceleration response spectral values upto 30% of g only.

Other Features

Some of the other features of the Roorkee Seismoscope are as follows :

(i) The calibration procedure for the new recorder is much simpler.

(i i) The system is very sturdy compared to the previous design. A strong jerk to to earlier recorder could result in a kink to the suspension thus changing its calibration but can not be so in this seismoscope.

(iii) The instrument can be carried to field with less skill, much ease and installed in less time.

(iv) The possibility of torsional vibrations which caused difficulty due to instability in larger period recorders does not exist is this case.

(v) With considerably smaller stylus pressure which is reasonably uniform upto the amplitudes of interest satisfactory records are obtained. The variation in the stylus pressure introduced greater uncertainty in determining the damping at low amplitudes in earlier recorder.

(vi) The position of the zero reference is easy to determine and single amplitudes can be measured conveniently.

Conclusions

1. This development was done with a view to obtain seismoscope records during aftershocks in Koyna region. Six such recorders have been installed there and are expected to yield useful records.

2. At locations where expected accelerations are within 30% of g the installation of the Roorkee Seismoscope would yield more useful records.

3. If seismoscope installation is being planned in regions where epicentral tracts are not well recognised i.e. statistically more recorders are likely to be located at some distance from the epicentre in that case the Roorkee Seismoscope would serve better purpose.

4. This development in no way diminish the utility of the earlier recorder for recording higher acceleration and can be installed in addition to it.

Acknowledgement

The author is thankful to Dr. Jai Krishna, Director, Earthquake School, Dr. A. S. Arya, Professor, Earthquake School and Prof. V. K. Gaur of Dept. of Geology and Geophysics for their interest and encouragement in the development. Prof. A. R. Chandrasekaran and Shri L. S. Srivastava have very kindly gone through the manuscript and offered many useful suggestions.

The paper is being published with the kind permission of the Director, Earthquake School, University of Roorkee, Roorkee.

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Bulletin of the Indian Society of Earthquake Technology. Vol. 6, No. 1, March, 1969

DISCUSSION

"Lateral Pressure in Bins Due to Earthquake Type Horizontal Loads". By A.R. Chandrasekaran and P.C. Jain, Bulletin of the Indian Society of Earthquake Technology, Vol. 4, No. 3, pp. 41-50, Sept. 1967.

G.C. Nayak M. ISET[1], The authors, it seems, have tried to make a break through in the determination of lateral pressure in bins due to horizontal Loads.

However, there are many limitations. The theory has been derived only for single circular bin whereas in practice continous bins, having various shapes in plan are constructed. The coefficients of friction μ and μ' may not remain same under dynamic conditions. In general it may depend on so many factors like foundation, soil conditions and relative stiffnesses and so on.

By prescribing arbitrarily any static coefficients without any experimental evidence seems to be highly objectionable.

From the design point of view it may be quite extravagant to spend more on additional safety against earthquake type forces for bins containing coal, wheat and cement. Considering the ultimate strength of the structure this type of additional insurance seems to be quite unnecessary.

Finally, the authors have adopted Airy's theory for computing pressures. It appears to be quite inadequate to assume Airy's theory. As pointed out by Jenike and Johanson[2] the condition of flow is more critical. They also give an extensive bibliography on bin loads.

Reply by Authors. The authors are very thankful to Mr. G.C. Nayak for his comments on the paper.

The paper is an attempt to get an approximate idea of the effect of earthquake forces on bins. An equivalent horizontal static force has been considered to take into account the earthquake effects. A modification to Airy's theory has been made on similar lines, as Mononobe had modified Coulomb's wedge theory of earth pressure for considering the earthquake effects on retaining walls. Airy's theory being more popular for static pressure in bins, has been adopted for computing pressures. The paper as such was published in September 1967 while the reference pointed out by Mr. G.C. Nayak is quite recent (April 1968)

The authors agree that the theory has got several limitations, but it could be used to get first estimate of earthquake effects on bins. Since at the time of earthquake the behaviour is not exactly known, some values for μ and μ' have to be assumed to arrive at approximate results.

A different technique has also been pointed out by the authors for considering the dynamic effects of the fill[3].

While considering the ultimate strength of the bins also, we have to consider the maximum possible ground acceleration due to earthquake forces.

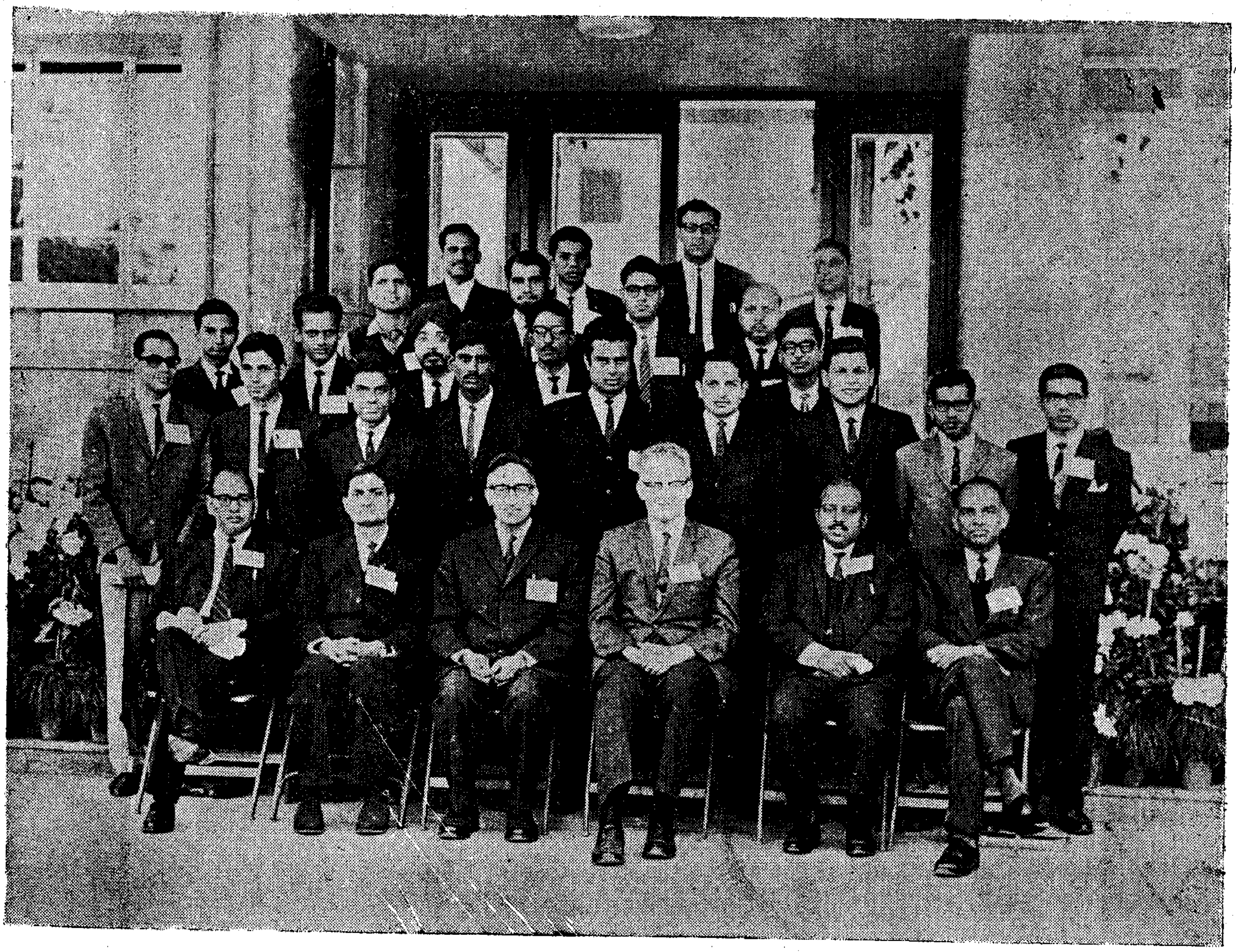
1. Reader in Civil Engg., Univerisity of Roorkee, Roorkee, Now on Study Leave and Working at Deptt. of Civil Engg. University College Swansea (Glam) U.K.
2. A.W. Jenike and J R. Johanson. "Bin Loads" Journal of the Structural Division, Proc. ASCE, April 1968, Paper No. 5916, pp. 1011-1041.
3. A.R. Chandrasekaran and P.C. Jain, "Effective live load of storage materials under dynamic conditions". Indian Concrete Journal, Sept, 1968.

Bulletin of the Indian Society of Earthquake Technology. Vol. 6, No. 1, March, 1969

INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY

Second General Meeting

Dec. 14, 1968



Left to Right Sitting : B.C. Mathur, Shamsher Prakash, Secretary, Jai Krishna, Vice President, G.V. Berg, L.S. Srivastava, Editor, B.N. Gupta.

Left to Right Standing 1st Row : P.N. Agrawal, Y.P. Gupta, V.H. Joshi, B.M. Basavanna, V. Chandrasekaran, A.S.R. Rao, S.K. Sen, C.R. Katakwar, K.K. Sarkar.

II Row : M.K. Gupta, P. Nanda Kumaran, S.S. Saini, K.K. Khurana, S.P.C. Goel.

III Row : S.K. Thakkar, M. Qamaruddin, V. Johri, R.K. Agarwal.

IV Row : D.S. Deshpande, P.K. Mitra, N.K. Gosain, S.K. Mittal,

Bulletin of the Indian Society of Earthquake Technology. Vol. 6, No. 1, pp 23-26 March, 1969

THE SECOND ANNUAL GENERAL MEETING OF THE INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY, ROORKEE

The Business Session of the Second Annual General Meeting of the Indian Society of Earthquake Technology was held in C.B.R.I. Auditorium, Roorkee, on 14th December, 1968 at 10.00 A.M. Dr. Jai Krishna, Vice President of the Society, presided over the meeting. Dr. A. N. Tandon, the President, could not attend the Annual General Meeting because of his other engagements.

An Inaugural Session was held in the afternoon at 1-30 P.M., jointly with the Institution of Engineers (India), Roorkee Sub-Centre, and the Indian National Society of Soil Mechanics and Foundation Engineering, Roorkee Chapter, at which Shri Jagdish Prashad, General Manager, Bharat Heavy Electricals Ltd., Hardwar, was the Chief Guest, and Dr. Jai Krishna, Vice-President of the Society, delivered his Presidential Address on behalf of the Society. The Inaugural Session was followed by a Technical Session from 3-30 to 5-30 P.M. under the Chairmanship of Professor Jagdish Narain, Head of the Water Resources Development Training Centre, University of Roorkee, Roorkee, and the following three papers were presented by the members of the Society.

Earthquakes in India and Growth of Earthquake Engineering Studies—*L.S. Srivastava*

Instrumentation to Record Earthquakes and their Application in Study of Structures—*B.C. Mathur and P.N. Agrawal*

Dynamic Loading of Soils and Foundations—*Shamsher Prakash*

MINUTES OF BUSINESS SESSION OF THE SECOND ANNUAL GENERAL MEETING OF THE SOCIETY, HELD ON 14TH DECEMBER, 1968 AT ROORKEE.

1. To confirm the minutes of First Annual General Meeting held on 5th November 1966 held in Roorkee.

The minutes of the First Annual General Meeting of the Society were circulated by post. No amendments had been received. As such the minutes were approved.

2. To consider and adopt the Secretary's Report for the period 6-11-1966 to 14-12-1968.

The Secretary's report giving main activities of the Society was read before the house and accepted.

3. To ratify the decision of the Executive Committee regarding payment of T.A. to Office bearers of the Society.

The Executive Committee in its 5th meeting held on 25th March 1968 resolved that regular T.A. may be paid to office bearers of the Society when going on official duty of the Society, according to what they are entitled in their organisations.

The decision taken by the Executive Committee was ratified by the General Body.

4. To decide the venue and date of the Third Annual General Meeting of the Society.

Since its inception, the Society has held the following General Meetings.

1st Meeting. Roorkee 1966 – on the occasion of the Third Symposium on Earthquake Engineering.

2nd Meeting. Roorkee 1968 – in joint session with the Annual General Meeting of the Institution of Engineers (India), Roorkee Sub-Centre and the Indian National Society of Soil Mechanics and Foundation Engineering, Roorkee Chapter.

Dr. Berg, suggested that he would explore possibilities of the group at I.I.T. Kanpur to extend invitation to the Society to hold the 3rd Annual General Meeting at Kanpur sometime in Nov.–Dec., 1969. The final decision will be taken after receipt of invitation from other places also.

5. The appointment of the Auditor for the year 1968–69. On the recommendations of the Secretary it was decided that M/s. Goswamy and Co., Chartered Accountants, Saharanpur be appointed Auditors for the Society for financial year 1968–69.

6. Dr. Jai Krishna suggested that local groups may be formed at Kanpur, Delhi and Bombay under the guidance of Dr. Berg, Dr. Mallick and Sri R.N. Joshi respectively.

The Secretary was requested to persue the matter and report on the above at the next Annual General Meeting.

7. Dr. Jai Krishna suggested that the Society may move Railway Board for concession to members for attending the Annual General Meeting of the Society. It was decided that this matter may be persued further by the Executive Committee of the Society.

8. The Executive Committee had decided in its 7th meeting that Government of India's permission be sought to invite International Association of Earthquake Engineering to hold their 5th World Conference in India in 1973. The implication of inviting world body to India was explained by the Secretary to the House. An expense of about 2.5 lakhs of rupees was expected. The General Body was in favour of inviting the world body to hold this Conference in India and also authorised the Executive Committee to take up the matter further with Government of India.

The meeting ended with a vote of thanks to the chair.

SECRETARY'S REPORT ON THE WORK OF THE SOCIETY FOR THE PERIOD 6-11-1966 to 14-12-1968.

Meetings

Five meetings of the Executive Committee of the Society were held at Roorkee during the period under report on 5–6–67, 25–3–68, 10–8–68, 20–11–68, and 14–12–68.

Fourth World Conference on Earthquake Engineering

The IVth WCEE is scheduled to be held in Santiago, Chile from Jan. 13–18, 1969. The Society constituted an Indian Delegation, to represent India at the World Conference. The Reserve Bank of India were requested to release foreign exchange to the delegation and the Government of India was requested to offer financial and other assistance to the delegates. The proposed delegation consisted of the following :

Dr. Jai Krishna, Dr. A.N. Tandon, Sri P.M. Mane, Dr. Shamsher Prakash Dr. Y.C. Dass, Sri R.L. Nene, Sri R.H. Mahimtura.

The subjects selected for the main sessions are as under :

Jan. 13, 1969 Observations in Recent Earthquakes.

Jan. 14, 1969 (a) Seismicity and Simulated Earthquakes. (b) Vibration Tests on Structures. (c) Ground Motion and Instruments. (d) Behaviour of Structural Elements.

Jan. 16, 1969 (a) Elastic Response of Structures. (b) Large Buildings and Structures Details. (c) Inelastic Response and Design of Structures.

Jan. 17, 1969 (a) Soils and Soils Structures. (b) Design Criteria and Research. (c) Foundations and Soil Structures Interaction. (d) Small Buildings, Insurance, Damage, Repair.

Jan. 18, 1969 Special Papers.

The papers were invited directly by the Organising Committee of the World Conference. To the best of our information, seven papers have been accepted from the Members of our Society.

The Society has initiated action for formal affiliation of the Society with the International Association of Eartquake Engineering.

Symposium on Koyna Earthquake of Dec. 11, 1967 and Related Problems

A Symposium on Koyna Earthquake was held in Calcutta on June 1, 2, 1968. This symposium was co-sponsored by ISET, Indian Society of Engg. Geology, Institution of Engineers (India) West Bengal Centre, and Mining Geological and Metallurgical Institute of India, Calcutta. The Society was represented by Dr. A.N. Tandon, Sri L.S Srivastava, Sri R.N. Joshi and several other members.

Publications

The following publications of the Society were issued during the period under report :

Bulletins : Vol. III No. 1 (Dec. 1966), Vol. IV No. 1, 2, 3, 4. (1967), Vol. V No. 1, 2, 3, 4. (1968)

The frequency of the Bulletin was increased from 2 issues per year to 4 issues per year with effect from issue of March 1967.

List of Members : A list of members has been published alongwith the Bulletin of December 1968 (Vol. V, No. 4). The total memberships is as under :

Institution Members	Life Members	Individual Members
21	21	214

All the members have been assigned a number for easy reference.

Establishment of Exchange Relations

The Society has established exchange relations with the New Zealand Society of Earthquake Engineering for exchange of Society Publications.

Election of Members of Executive Committee

As per the Constitution of the Society, the working of the Society is governed by the Executive Committee elected for the period of one year. The term of the present Executive Committee will expire on 31-3-1969.

The following is the composition of the Executive Committee of the Society :

1.	Dr. A.N. Tandon	President
2.	Dr. Jai Krishna	Vice-President
3.	Dr. Shamsher Prakash	Secretary and Treasurer
4.	Sri L.S. Srivastava	Editor
5.	Dr. A.S. Arya	Member
6.	Dr. A.R. Chandrasekaran	Member
7.	Sri V S. Krishnaswamy	Member
8.	Dr. Jagdish Narain	Member
9.	Dr. R.S Mithal	Member
10.	Sri R.N. Joshi	Member
11.	Sri A.P. Bagchi, Sahu Cement Service, New Delhi	Member
12.	Chief Design Engineer, Bhakra and Beas Designs Organisation, New Delhi.	Member

Funds

The main source of income of the Society is the Membership subscription from Institution Members, Life Members and Individual Members. The income is just sufficient to print only four issues of the Bulletin in its present form. Attempts are being made to obtain grant-in-aid from the Government of India.

Accounts

The accounts of the Society for the year 1967-68 have been audited by M/s B.G. Goswamy and Co., Chartered Accountants, Saharanpur. The statement of audited accounts as prepared and submitted by the auditors has been printed in December 1968 issues of the Bulletin.

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PHILOSOPHY OF ASEISMIC DESIGN*

Jai Krishna**

The practice of designing engineering structures in seismic zones has consisted of assuming a seismic coefficient almost arbitrarily and using it as a proportion of the vertical loads of the structure to be applied as a seismic force horizontally at the centre of gravity of the mass of the structure. In each country the seismic zones are divided on the basis of frequency of occurrence of earthquakes and the seismic coefficients are adopted zonewise. These coefficients have in some countries, some experience at their back in as much as many structures designed on their basis have done well during strong earthquakes. At the same time some other structures have failed to do so. This method has, therefore, proved fairly satisfactory and is in use throughout the world for average structures. For important structures like tall buildings, dams, bridges, tall industrial containers and hydraulic structures etc , it is now increasingly realized that a more rational approach is essential. The traditional approach suffers from the obvious defect that it is independent of the type of structure and soil conditions of the site where the structure is going to be built. In some codes of practice the soil condition is taken into account by increasing the seismic coefficient in the case of poorer soils, but in most countries this is not done. It may be stated here that whatever provisions exist in this respect are really not adequate for soils having a safe bearing capacity less than about 1 ton/sft In this country very vast areas of alluvial plains are thus not covered by these provisions.

A close study of damage during earthquakes in the last 20 years has indicated that the dynamic properties of structure play a great part in determining whether it is going to be damaged or not, besides, of course, the size and position of the earthquake itself. It has been observed that near the epicentral areas, tall structures having long natural periods of vibration have suffered much less than those having short periods of vibration even though they were otherwise stronger, (for example Skopje earthquake of 1963 in Yugoslavia). Similarly, at certain distance away from the epicentre where long period waves only reach the structures, tall structures have been damaged due to large deflections although the stresses induced by the earthquake forces were small (e.g. Alaska earthquake of 1964). Other features which have resulted in damage are unequal settlement (Fukui earthquake of 1948), liquefaction of soils in the foundation (e.g. Niigata earthquake of 1964) and poor construction. A study of the examples of poor construction, particularly reinforced concrete structures, has shown that excess of steel crowded up at joints and inadequate quantity of cement in concrete have resulted in failures of otherwise very well designed sructures.

It would thus be seen that the most important factors in aseismic design namely, (i) the dynamic properties of the structure (ii) foundation behaviour and (iii) the actual earthquake motion expected at site are not taken into account in the traditional method. A structure is designed for a static 'force' which is assumed to be equivalent to a dynamic motion of random nature.

Such a practice is quite understandable in small structures or some isolated structures where a detailed examination is not feasible, but for major structures it is possible now to examine the problem somewhat more rationally.

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** Professor and Director, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee.

In the last 30 years a large number of strong motion accelerograms have become available for several conditions, namely, epicentral areas such as Koynanagar earthquake, Parkfield earthquake etc., for point 30 to 50 miles away from the epicentre on rocky soils such as El Centro earthquake and Ferndale earthquake, and on soft soils such as Seattle and Taft earthquakes etc.

It will, of course, be an ideal situation if the strong motion records for the site itself were available since that will give the most reliable shape of ground motion, but in the absence of such a record it should be possible to choose one of the accelerograms recorded elsewhere for similar soil conditions. The accelerogram may then be used to test the stability of the structure through dynamic analysis under an actual ground motion. The seismic coefficient may be used as an equivalent force for preliminary design purposes. This would give a far more realistic picture of the structure under an actual ground motion.

Another aspect of rational design is to consider the overall capacity of a structure to resist dynamic forces by taking into account the total energy absorbing capacity rather than the stress level at some special points. A structure possesses a certain capacity by virtue of the properties of materials in it, its form and the maximum limit of deformation imposed by functional requirements and permissible damage. This concept results in considerable economy in design or at least a clearer understanding of how a structure is expected to behave during an earthquake.

The two ideas explained above are in increasing use in scientifically advanced countries in aseismic design practice and I hope they will be used increasingly in this country as well.

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EARTHQUAKES IN INDIA AND GROWTH OF EARTHQUAKE ENGINEERING STUDIES

L. S. Srivastava*

Abstract

Earthquakes occur very frequently in various parts of India and considerable loss of life and property has occurred in the past during major earthquakes. This paper describes in brief the earthquake occurrence in India and highlights the significance and growth of earthquake engineering studies in the country during the last ten years. Brief descriptions of some of the important earthquakes during the present and last century has also been given.

Introduction

Since the evolution of man, he has battled against the forces of nature and one of the battle fronts has been the field of natural disasters due to earthquakes. History and legend record these events in succeeding generations and their unforeseen character has frequently been classed as 'acts of god', but scientists and engineers are beginning to find ways to protect man from its fury, even if as yet no means are available for forecasting or preventing them. It is, therefore, very essential to mobilise the various technical skills on a properly organised basis, so that the devastation and the appalling loss of human life and property in prevented or, at least, reduced.

With the rapid industrialisation and spread of urban civilisation throughout the world, the toll taken by earthquakes has, during the past hundred years, been steadily increasing. Systematic records of the damage and loss of life caused by earthquakes are only available for less than hundred years. These records show that over 350,000 people were killed in the second quarter of this century between 1926 and 1950, and damage to buildings and public works totalled near to Rs. 7,500 crores ($ 10,000 mln.) – a yearly average of 14,000 deaths and Rs. 300 crores ($ 400 mln.) damage. The worst years were 1927 (76,615 deaths), 1932 (76,202 deaths) and 1939 (61,082 deaths). In the first quarter of the century records are less complete, but five earthquakes alone are known to have claimed a total of 500,000 lives—more than all earthquake of the second quarter combined. Fatalities since 1950 have ranged from as few as about 100 in 1959 to 20,257 in 1960. During 1967 twenty disastrous earthquakes in various parts of the world killed 949 persons and did great damage.

After every large earthquake, various organisations and individuals that are concerned with earthquakes are flooded with question from the public and often the administrator: Why do we have earthquakes? What can be done about all this? But these are the questions that should be asked not after an earthquake, but before. Much progress has been made in the study of causes and probable effects of earthquakes, and every Indian can save himself a great deal of the worry and grief and perhaps thousands of rupees—by understanding the fact about earthquakes and learning to live with them. The most practical attitude would be to admit that India is a seismic country and earthquakes would continue to occur for countless centuries. Though there is no way of eliminating the causes of earthquakes, there is much that can be done about effects to eliminate or reduce damage; and we should always be ready for the day when the shaking under our feet suddenly becomes the greatest earthquake of a decade.

* Reader in Applied Geology, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee (U.P.) India.

Earthquakes in India

India is one of the most active earthquake country and holds a significant position in the world of earthquakes. The country faced five "monster" earthquakes during the nineteeth and this century : 1819 in Cutch, 1897 in Assam, 1905 in Kangra, 1934 in Bihar and then again in 1950 in Assam. Most of the earthquakes which occur at short intervals are confined to the northern mountainous region and the plains, but past earthquake data has shown that there are no regions in India where the earthquake problem can be completely negleted. The following gives a brief account of some of the important earthquakes which occurred in various parts of India and significant damage was observed.

September 1, 1803, Garhwal Earthquake : This earthquake has also been referred as Muthra earthquake where the earthquake is reported to be very violent and lasted several seconds with many 'kucha' buildings thrown down. Extensive fissures in fields through which water rose with considerable violence were noted. Many buildings were ruined in Kumaon. It was very violent in upper Ganges Valley, Sirmoor and Garhwal. 200 to 300 persons died at Barabal. Badrinath also suffered severely. Several villages were destroyed. The upper portions of Qutub Minar in Delhi were destroyed.

June 16, 1819, Cutch Earthquake : This earthquake was the second largest earthquake in India in the nineteenth century, next to 1897 Assam earthquake. This shock was felt through out India and did considerable damage in very large area. Maximum damage occurred near Cutch. Bhooj was reduced to ruins and 2000 people perished. Extensive damage to buildings occurred in whole of the Kathiawar, in the north upto Jaisalmer and in the east beyond Ahmedabad. The damage region was quite comparable to ~~that~~ of 1897 Assam earthquake and 1934 Bihar earthquake. At Ahmedabad great damage to mosques and other buildings occurred and 500 people assembled for a wedding feast perished in the ruins. The western parts of the town of Sindri and adjoining parts were inundated and submerged by tremendous rush from the ocean due to approximately 14 feet sinking of the ground. While 5 miles to the north of this tract a low ridge of about 15 miles width extending for fifty miles in east-west was formed. The natives called this "Allah Band" or Mound of God. In the runn of Cutch numerous jets of blackish muddy water were thrown out from fissures and cones of sand 6 to 8 feet high.

June 6, 1828, Kashmir Earthquake : During this earthquake which probably had its epicentre about 10 miles east of Srinagar about 1000 people were killed and 1200 houses collapsed. Fissures in the ground, and water fountains were observed. The earthquake was followed by numerous after shocks for two month at the rate of 100 to 200 per day.

August 26, 1833, Bihar-Nepal Earthquake : This earthquake was felt at great distances. About 125 houses were destroyed at Kathmandu and similar fate overtook other affected areas also.

April 1, 1843, Deccan Earthquake : This earthquake which was felt with violent intensity at great distances, and damaged many houses throwing down walls and roofs, had its epicentre near Bellary. The various other places affected included Sholapur, Maktal, Singrurgarh, Kurnool and Belgaum.

January 10, 1869, Cachar Earthquake : This earthquake was felt over an area of 250,000 square miles, with its epicentre on the north-east side of the Shillong plateau. Earth fissures and sand craters were very abunant.

May 30, 1885, Kashmir Earthquake : This was another major shock which affected the Kashmir valley, with its epicentre close to the 1828 Kashmir earthquake. Great damage occurred to the buildings and ground was badly cracked and fissured. About 3000 persons lost their lives in this earthquake. This is a large number considering that the valley is thinly populated.

June 12, 1897, Great Assam Earthquake : This earthquake was probably the largest earthquake which has occurred in the historic times and was felt over an estimated area of 1,750,000 square miles. Over 1600 persons were killed. Great damage was observed in an area of radius 300 miles. Stone buildings in Shillong, Gopalpur, Gauhati, Nowgong and Sylhet were almost entirely destroyed. Pebbles were seen bouncing on the ground "like peas on a drum head", posts shot out of their holes and boulders lifted out of the ground without cutting the edges of their former seats, all indicating that the maximum accleration exceeded that of gravity. This high accleration was consistent with the observed wide spread surface distortion and shattering of granitic rocks of the Shillong plateau. Displacements observed in surface faultings were of the order of 35 feet. Calcutta was also seriously affected.

February 8, 1900, Coimbatore Earthquake : This earthquake was felt in the whole of South India, with severe intensity in Mysore, Madras, Bangalore, Calicut and Negapatam. The epicentre was near Coimbatore where significant damage is reported to have occurred.

April 4, 1905, Kangra Earthquake, Near 31°N: 79°E, M=8.6+ : This is the earliest large Indian earthquake for which a well document instrumental magnitude (8.6+) could be assigned. This earthquake caused great disaster and about 20,000 people lost their life. Kangra, Dharamshala and neighbouring places were completely ruined. Another area, 100 miles away, including Dehradun and the foot hills indicated high intensity, lower than that at Kangra but not approached elsewhere. This could have resulted due to two seperate earthquakes in this tectonic belt.

July 8, 1918, Srimangal Assam Earthquake, 24.5°N: 91.0°E, M=7.6 : This earthquake was felt over an area of 800,000 square miles and the epicentre was 8½ miles south of Srimangal on an alluvial tract. Many tea gardens were ruined.

July 3, 1930, Dhubri Earthquake, 25.8°N: 90.2°E, M=7.5 : The earthquake was felt over an area of 350,000 square miles. Dhubri suffered considerable damage. Maximum intensity felt at Gauhati, Rangpur and Berhampur was M.M. IX.

January 15, 1934, Bihar-Nepal Earthquake, 26.5°N: 86.5°E, M=8¼ : This was one of the most severe earthquakes in Indian history, and was felt over an area of 1,900,000 square miles. About 10,000 persons were killed and property worth crores of rupees was destroyed. The extent of damage and other effects places this earthquake only a little below that of 1897. M.M. intensity X was assigned to a belt about 80 miles long by 20 miles wide and to two spots almost 100 miles distance from the main belt, at Monghyr to the south and in the Nepal Valley to the north. The three isolated regions of maximum damage may be related to three independent events occurring near about the same time along a N-S trending tectonic lineament. The isoseismal IX, about 190 miles long and of irregular width exceeding 40 miles in places, was demarcated as the slump belt in which tilting and slumping of building due to liquefaction of alluvium and subsidence of roads, cause-ways and railway embankments were marked. Within isoseismal X most of the houses were razed to the ground.

March 14, 1938, Satpura Earthquake, 21.6°N : 75.0°E, M = 6¼ : This earthquake was felt upto distances greater than 600 miles and did damage at Bhusawal, Khandwa, Godhara, Baroda and Nasik. The maximum MM intensity in the epicentral tract at Amalner was VII.

August 15, 1950, Assam Earthquake, 28.7°N : 96.6°E, M=8.6 : This was a disastrous earthquake which affected Assam and bordering regions of Tibet and China. Strictly this was not an Indian earthquake, the epicentre being near Rima outside Indian borders. This shock was more damaging in Assam in terms of damage than the 1897 earthquake. To the effects of shaking were added those of floods ; the rivers rose high after the earthquake, bringing down sand, mud, trees and all kinds of debris. This was largely due to enormous slides. Alterations of relief were brought about by many rock falls in the Mishmi hills and destruction of forest areas. In the Abor hills 70 villages were destroyed with 156 casualties due to landslides. Dykes blocked the tributaries of Brahamputra; that in the Dibang Valley broke without causing damage, but that at Sibansari opened after 8 days and the wave 22 feet high submerged several villages and killed 532 persons. Estimated area in India, Pakistan and Burma over which the earthquake was felt was 650,000 sq. miles. Seiches due to this earthquake were observed as far away as Norway and England. The main shock was followed by a train of after shock some of which reached destructive magnitude near epicentre.

July 21, 1956, Anjar Earthquake, 23.34°N : 70.02°E, M=7 : This earthquake was felt at large distances in Western India and did considerable damage to the village houses. Very few brick masonry structures escaped damage and were badly cracked and partially collapsed. But houses built of bulky walls in randum ruble stone masonry with mud mortar or of mud collapsed completely. The toll of life lay in the faulty construction of these houses.

October 10, 1956, Khurja Earthquake, 28.15°N ; 77.67°E, M=6.7 : This earthquake was felt in a large area. 23 persons were killed in Bulandshahar and some injuried in Delhi.

December 28, 1958, Kapkote Earthquake, 30.01°N : 79.94°E, M=6¼ : The earthquake caused ground ruptures with gaping cracks and few landslides in an area of 100 sq. miles around Kapkote. Partial collapse of walls and cracking of buildings had taken place at several localities including Hatsila, Pharsila, Bhainsoyi and Sulmati.

August 27, 1960, Delhi Earthquake, 28.2°N : 77.4°E, M=6 : Minor property damage at New Delhi. About 50 persons were injured. It was felt also at Jaipur and Kanpur.

September 2, 1963, Badgam (Kashmir) Earthquake, 33.9°N : 74.7°E, M = 5.5 : Though of small magnitude, this shock did considerable damage in the Badgam Tehsil of Srinagar district. About 65 persons were killed and 400 injured. More than 2000 houses mostly of mud construction collapsed, while about 5000 houses suffered partial damage. Timber structure and good masonry houses did not suffer any damage.

June 6, 1966, Hindukush Earthquake, 36°N : 69°E, M=6.1 : Though the epicentre of this earthquake was at Hindukush, it did significant damage in the Kashmir Valley, and was felt upto great distances and even at Roorkee. In Srinagar the Museum building was damaged and 100 houses developed deep cracks. The ceiling of the central hall of the Museum collapsed and almost all the walls of the building had suffered cracks. Show cases containing rare manuscripts were damaged when the ceiling came down. The Civil Secretariate building at Srinagar also suffered minor cracks. The chimney of the High Court building fell down. Most of the telephone connections went out of commission.

June 27, 1966, Darchula (West Nepal) Earthquake, 29.5°N : 81.0°E, M=6.3 : This earthquake did great damage in West Nepal, where about 300 houses were destroyed or damaged, leaving nearly 10,000 people homeless. About 80 people were killed and many injured. Significant damage also occurred in Indian side bordering Nepal. This earthquake shook the whole of Uttar Pradesh and was felt upto Amritsar, Agra, Bahraich and other places. Some buildings in Delhi, Ambala and Nainital developed cracks. In Ambala city the roof of a double storey building, housing the main post office, collapsed. In Moradabad several old houses collapsed.

August 15, 1966, Moradabad Earthquake, 28.0°N : 79.0°E, M=5.3 : This earthquake was felt extensively in Uttar Pradesh and adjoining areas of Delhi, Haryana and Punjab. In Moradabad thirty persons were injured, fifteen of them seriously in house collapses. A few houses, some well built, were damaged. Minor damage also occurred at Barielly and Meerut. 14 persons were killed due to collapse of dilapidated houses in Delhi.

December 16, 1966, West Nepal Earthquake, 29.5°N : 81.°E, M=5.7 : This earthquake was felt extensively in Uttar Pradesh and adjoining areas of neighbouring states. Several houses collapsed at Dharchula due to the earthquake. The earthquake caused cracks in some buildings at Pilibhit and Moradabad.

February 20, 1967, Anantnag Earthquake, 33.5°N ; 75.5°E, M=5–5.7 : This earthquake was felt upto Lahore, Pakistan and other places. During this earthquake about 50 houses of poor construction collapsed and another fifty were damagad in the Anantnag district. One person was killed and another was seriously injured. The maximum M.M. intensity in the region did not exceed VII.

September 13, 1967, Koyna Earthquake, 17.4°N : 73.7°E, M=.58 : This earthquake, which was followed and preceeded by a large number of smaller shocks, was felt over a wide area upto Bombay. It caused some damage in Koynanagar and neighbourhood.

December 11, 1967, Koyna Earthquake, 17.37°N : 73.74°E, M=6.5 : This is considered one of the greatest earthquake of the Peninsular shield. The earthquake caused considerable damage in Koynanagar and neighbouring areas. About 180 people were killed and 2300 injured. This earthquake was felt over a large area of radius about 400 miles It was felt very strongly at Poona and Bombay. The maximum intensity near the epicentre was VIII + in the M.M. scale. There was 100 percent damage to radium rubble stone masonry in Koynanagar and 25 percent were completely demolished. Total loss is estimated to be Rs. 30 lakhs. Production loss in Bombay region due to closure of power was estimated to exceed Rs. 2 crores. About 5000 people were left homeless. Koyna dam resisted this shock well though cracks and other distress feature were developed at several parts of the dam.

Correlating the geology and tectonics with the known epicentres of past earthquakes in India, a definite pattern of earthquake activity is observed, which indicates that all these earthquakes are in general of tectonic origin. In the mountainous Himalayan belt abuting the Indo-Gangetic plains, the major earthquakes are considered to be associated with the movements along faults, thrust and nappes in this region of youthful mountains. In the alluvial tracts these are considered to be related to the movements along fractures, faults and rifts in the basement underlying the sedimentary cover, and in the Deccan Peninsular shield with the various rifts and fracture zones probably produced by the cymatogenic uplift of this crustal block. Some of these faults and thrusts have been well demarcated by detailed geological mapping by various organisations, while the likely presence of others is indicated by indirect inferences based on geophysical surveys as well as the alignment of the epicentres and the pattern of earthquake occurrence. The causative forces which produce these movements are not fully known and various views have been expressed regarding them. In the Himalayan region isostatic imbalance appears to be the main cause and large earthquakes occur very frequently. In the Peninsular shield, whose extensions also form the basement of the alluvial plains, cymatogenic warping of the crust due to subcrustal currents coupled with isostatic readjustment appears to be the main activating force. Although minor earthquake tremors are frequently felt in various parts of Peninsular India and Indo-Gangetic Plains, a study of known earthquakes during the last few centuries. indicates that there were comparatively large periods of quiteness with shorter periods of activity as compared to the very frequent earthquake occurrence in the Himalayan belt. Thus it appears that the smaller magnitude of isostatic imbalance and the tectonically stable character of the Peninsular India inhibits rapid build up of strain which would be responsible for large frequent earthquakes. However, this does not preclude the occurrence of a large earthquake after considerable lapse of time, though in many cases in past having occurred after several decades or having damaged inaccessible or inhabited or small areas may have escaped to arise attention and recorded in historical documents. This lack of data on occurrence of earthquakes led many to believe this region seismically inactive and stable. December 11, 1967 Koyna earthquake has shattered this common concept and has highlighted the necessity of evaluation of the seismic status of this large segment of the country.

Seismic Zoning of India

Based on the data collected during the past earthquakes, various attempts have been made to prepare earthquake zoning maps of the country. In these maps the country was divided into three to four zones indicating probable occurrence of the earthquakes (frequent, occassional, few) or probable accelerations (10 to 30 percent gravity, less than 10 percent gravity, etc.) or likely intensity of damage (heavy, moderate, slight etc.) or factor of safety to be adopted in the design of structures, etc. The construction of these maps utilised data on past earthquakes and geologic and tectonic features of the country.

In order to give a unified picture of the zoning map, the Indian Standards Institution formed a panel of experts to prepare a seismic zoning map of the country and published its first map in 1962 (IS : 1893–1962). This map had seven zones and it appears was prepared considering that a rational approach to the problem would be to arrive at a zoning map which show the maximum intensity (M.M. intensity scale) of earthquakes likely to occur at each point based on data of the known earthquakes, assuming all other conditions as being average, and to modify such an average idealised isoseismal map in the light of tectonics, geology, soil conditions and the maximum intensities as recorded from damage surveys. Zones with M.M. intensity V, VI, VII, VIII, IX and X (and above) were designated as zones I, II, III, IV, V, and VI, and region with M.M. intensity less than V was designated as zone 0. This 'zero zone' was not a zone of 'zero earthquakes', but the designation was given to suggest that no earthquake problems of any significance may occur in this region. However, as only preliminary tectonic map of the country was available this zoining map probably had greater emphasis on available historical earthquake records and the resulting zones mostly wraped around the epicentres of the five 'monster' earthquakes, of the country. Design seismic coefficeint for each of these zones, which depend on the type of structures, materials of construction, soil condition, soil-structure inter-action and duration of shaking, in addition to the intensity of the earthquake and ground accelerations, were also recommended.

With additional knowledge of geology and tectonics and earthquake epicentres in subsequent years it was felt that this map required modifications. A revised seismic zoning map was published in 1966 (IS : 1893–1896), which incorporated data on all past earthquakes of magnitude 5 and more from the earliest times to December 1965. In this map more tectonic evidences were used for interpolation and extrapolation of seismic data, and the expected intensity along marginal depressions and mobile belts bordering the Peninsular shield and along known active faults in various regions was changed, to take into consideration that as earthquakes have occurred with higher intensity in some parts of a tectonic zone, earthquake of comparable intensity may arise along the whole length of the same zone or in another zone with similar structure and history.

Soon after the publication of the revised zoning map, an earthquake of magnitude 6.5 occurred in Koyna valley on December 11, 1967, in a region which was considered to be of comparatively low seismic status, due to lack of data regarding existance of active tectonic lineaments in the region. This event has illustrated that in the modification of seismic zones that may be undertaken in the future, geologic and tectonic features should be given greater recognition. Attempts should be made to identify and demaracate the various active tectonic belts along which an earthquake can occur in future. This would require detailed geologic and tectonic mapping of the various regions and evaluation of complete history of the tectonic evolution of the regions through the various geological ages with particular reference to the recent movements of the crust. Also at present many small tremors, originating from various tectonic zones, along which a devastating earthquake may occur in future, are missed as the net work of the seismological station in India is not close enough. Thus, many zones which may be active have not been recognised. It is, therefore, essential that a closer network of seismograph station is built in the country. It is hoped that with more information a better picture of the seismic zones of the country would be evolved.

Field survey of earthquake disasters has shown that in many cases the damage is due to the failure of the ground. Liquefaction and settlement of soils, landslides and lateral spreading, and fissures, cracks and lurching of the ground have led to the sinking, tilting and collapse of the structures. It is therefore essential that for the planning and development of urban centres and site selection of industrial projects, detailed geomorphological, geohydrological and soil survey be carried out and local seismic zoning maps be prepared. Regions likely to undergo liquefaction and other damage can be left as open spaces. Unfortunately no systematic studies in this connection have been initiated in the country. Many of our towns which are being allowed to grow on land, without assessing their vulnerability to damage during future earthquakes, may be responsible for great loss of life and damage to property.

Growth of Earthquake Engineering Studies in India

During earthquakes man had been the victim, and thus his natural instincts forced him to evolve better means of protection against its hazards. In regions of frequent earthquakes, various types of earthquake resistant construction was adopted, based on the experience during various disasters, but in many regions modifications to safeguard against the rigours of the climatic conditions, non-availability of the suitable construction material and lack of adequate 'know how' in design and construction methods made them unsafe.

Systematic studies of earthquake engineering problems in India started at the beginning of this decade, though a number of organisations engaged in construction and maintenance of buildings had drawn out various design specifications and construction practices based on their experience from past damage. The University of Roorkee played a very significant role in the initiation of earthquake engineering studies in India by organising the First Symposium on Earthquake Engineering in February, 1959 which highlighted the necessity and importance of these studies and enumerated the various problem which needed investigations. The School of Research and Training in Earthquake Engineering was established at Roorkee in 1960, and research and training on various aspects of earthquake engineering is being carried out at this School. The University of Roorkee, has organised two more symposia on earthquake engineering during 1962 and 1966, at which data on the investigations carried out in the country was presented. The work being carried out at the Earthquake School at the University of Roorkee in this field of enquiry, has also lead other organisations and institutions in the country to initiate similar studies. In this brief period, since 1959, research and investigations in the country have helped in creating some confidence and capacity in the country to tackle problems concerning behaviour of structures during earthquakes.

Some of the significant development in the growth of earthquake engineering studies are described as follows :

Standard Recommendations for Earthquake Resistant Design of Structures : No unified code of practice for earthquake resistant design of structure existed in the country. The first Indian standard recommendations were brought out in 1962 (IS : 1893–1962). These have been revised in 1966 incorporating recent advances made in the design practices and methods.

Code of Practice for Earthquake Resistant Construction : This code was brought out in 1967 (IS : 4326–1967) and recommends the construction methods for various type of buildings in various seismic zones of the country. In this special emphasis has been given to utilise local available construction materials, so that the cost of construction is not appreciably altered. Methods of reinforcing brick and stone masonry and timber houses which are the most prevailent type of construction in the country have been dealt in detail in this code.

Period and Damping Measurements : The modern earthquake design practices use the oscillation period and damping of structures as design parameters. Extensive field and laboratory studies have been carried out in the country to measure the periods of vibrations and damping characteristics of various types of structures.

Structural Response : The design seismic coefficients vary with the dynamic characteristics of the structures as indicated by response spectra of recorded strong ground motion. No strong ground motion records were available for any earthquakes in India and studies were being careied out from records obtained in USA and other countries. The first strong ground motion accelerograph records were obtained during the 1967 Koyna earthquakes. Response spectrum curves have been drawn for the various components recorded on the accelerograph for December 11, 1967, Koyna earthquake, and in future would enable us to calculate structural response on the basis of recorded ground motion incorporating the local effects of geology and soil conditions. Strong ground motions is recorded on accelerographs, and we have very few accelerographs installed in India. This is mostly due to their non-availability in the country. These are now being manufactured in Roorkee and it is essential that necessary steps be undertaken to establish a network of accelerograph stations in seismic regions. Because of the relatively high cost and complexity of the accelerograph, a low cost simplified strong motion earthquake recorder (structural response recorder) has been developed at Roorkee which, though does not measure ground motion, records the maximum response of a mechanical system idealising the dynamic characteristics of the structures. These structural response recorders give the overall effects of the geological and soil conditions of the site and serve very useful purpose in the greater instrumental coverage for strong ground motion studies in the country. More than forty structural response recorder stations have been established in various parts of India.

Model Studies : Laboratory investigations of models for evaluating the behaviour of the prototype structures is an essential part of earthquake engineering studies. Facilities now exist for carrying out these model studies, as well as, the study of the dynamic behaviour of the actual full size structures. Shake tables, acceleration, velocity and displacement pick ups, pore water pressure pick ups, earth pressure cells and vibration generators for exciting various models and prototype structures have been designed and fabricated in the country.

Dissemination of Result of Research Investigations : Dessemination of results of investigations is an essential requirement for the growth of the subject. The publications of the proceedings of the three symposia in earthquake engineering held at Roorkee, and Bulletins of the Indian Society of Earthquake Technology, have helped greately in the growth of earthquake engineering studies in the country. Publications in other allied technical journals and periodicals have also helped in bringing these problems to the attention of the various persons and organisations interested in this field. These provide an avenue for the presentation, discussions and exchange of views and results of the research investigations being carried out by engineers, geologists, seismologists and others with the aim of advancing knowledge of earthquake technology in all its aspects.

Conclusion

Considerable progress in research in earthquake engineering has been made in the country. This has resulted due to coordinated efforts of the engineers and scientists of the various organisations. There is no place in India where the earthquake engineering problem can be completely neglected, and therefore it is essential that better and more economical eathquake-resistant design and construction methods for different types of buildings, dams, power houses, factories and other structures and foundations be evolved.

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SYMPOSIUM ON KOYNA EARTHQUAKE OF DECEMBER 11, 1967 AND RELATED PROBLEMS

Calcutta, June 1 and 2, 1968

The Proceedings of **the Symposium on Koyna Earthquake of December 11, 1967 and Related Problems** held at Calcutta on June 1 & 2, 1968 are being published by M/s. Indian Construction News, Construction Garden, Budge Budge Road, P.O. Maheshtala, 24 Parganas, West Bengal. The Publishers have agreed to supply the *Proceedings* at a discount of 25% to the members of the Indian Society of Earthquake Technology and other sponsoring bodies of the Symposium. Each author will get 25 reprints of his paper. If the paper is in joint authorship the 25 reprints will have to be distributed amongst the various authors. Those who wish to have additional reprints are requested to approach the publishers, who will indicate them the price of the same. The publishers would like to get an idea of additional requirements of reprints as early as possible. All members of the Society who wish to purchase the above *Proceedings* are requested to place their orders with the publishers or the Secretary, Mining Metallurgical and Geological Institute of India, 29 Chowringhee, Calcutta-16.

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INSTRUMENTS TO RECORD EARTHQUAKES AND THEIR APPLICATION IN STUDY OF STRUCTURES

Brijesh Chandra* and P.N. Agrawal*

Abstract

Ground motion data obtained from strong motion earthquake recording instruments makes possible more efficient design of engineering structures in seismic areas. The strong motion instruments available in the country have been briefly described. Their usefulness in the study of structural response has been explained. The need for extending strong motion earthquake recording programme in the country has thus been emphasised.

Introduction

Earthquake resistant design of structures stands as a big problem for several countries frequently visited by earthquakes. For an efficient design, one has to correctly estimate forces that are caused on a structure during an earthquake. Before earthquake recording instruments came in use, the earthquake occurrence could only be described in qualitative terms, basing observations on the extent of damage caused. These observations could only give a rough idea regarding the forces that might have been caused on engineering structures. It was around the middle of nineteenth century that Robert Mallet first recognized the need for having instruments for earthquake recording. Palmieri of Italy was first to develop an instrument for detecting feeble earthquake vibrations around the same time. It was only towards the close of nineteenth century John Miline of Japan developed the first effective earthquake recording instrument. Thereafter the developments in instrumental seismology have been rater rapid. The earthquake recording instruments maintained in seismological observatories yield useful scientific information regarding the earthquake and the ground properties. In general these do not give complete information during earthquake in the area where it has been of any consequence to engineering structures, because they are more sensitive and invariably go out of scale during strong ground motions. This difficulty has been overcome by the development of the strong motion instruments designed to yield ground motion records specially useful for interpretation in terms of data required for design purposes[1]. In the following paragraphs, these are briefly discussed. Their usefulness in studying structural response is also explained.

The Seismograph

A seismograph records many useful details of ground movements following an earthquake. It consists of a sensitive device called a seismometer and a recorder. Generally the recorders employed are the photographic type and yield continuous records. These records as a routine are utilized for determining the origin time, epicentre, depth of focus, magnitude and regional travel time etc. for the earthquake. If such data is available for a sufficient period, then it would help estimating the following :

(a) The magnitude of the largest shock that has occured in the area.

(b) Earthquake frequency-magnitude relationships.

* School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, U.P., India.

(c) Predominent periods of ground vibration.
(d) Predominent period of earthquake waves.

The above data would help designers to make an estimate of the largest size of earthquake expected within the lifetime of proposed structures in the area apart from providing other useful guide lines.

However, such instruments go out of scale and get damaged during strong motion earthquakes. Therefore, in order to obtain records of strong ground motions, specialised instrumentation has to be provided.

The Accelerograph

The strong motion accelerograph differs from a conventional seismograph used for tele-seismic purposes in the following respects :

(a) Is is less sensitive so as to stay in operation during violent motions.

(b) It has to be located at sites where important structures are to be constructed. The other seismographs are generally housed in observatories in remote areas.

(c) Recording speed of an accelerograph must be much higher compared to the other seismographs. This would enable clear interpretation of records.

(d) Since only once in several years such instruments would generally yield an earthquake record, their continuous maintenance with high recording speed would be expensive and wasteful. A self starting device during an earthquake solves this problem.

(e) Since this instrument has to start suddenly for earthquake recording, use of electronic components has to be avoided.

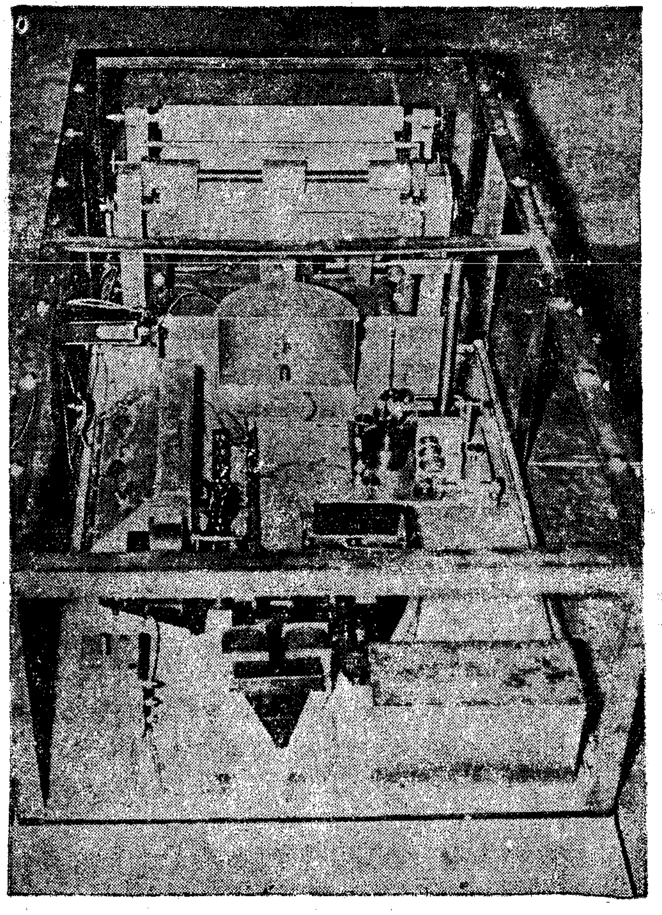

Fig. 1. Strong Motion Accelerograph

Fig. 1 shows the strong motion accelerograph made at the School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee. It contains three torsion seismometers, each having natural period of vibration as 1/20 seconds. Two of these respond to horizontal components of ground acceleration in mutually perpendicular directions and the third to the vertical component. The seismometers are critically damped by magnetic eddy currents produced

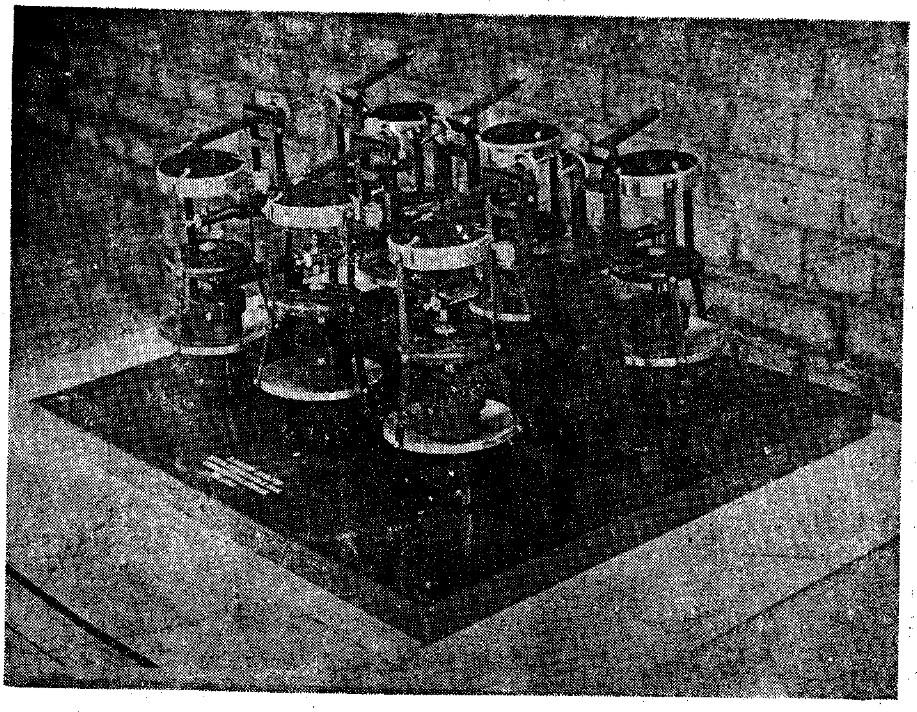

Fig. 2. Structural Response Recorder Set (MD–2)

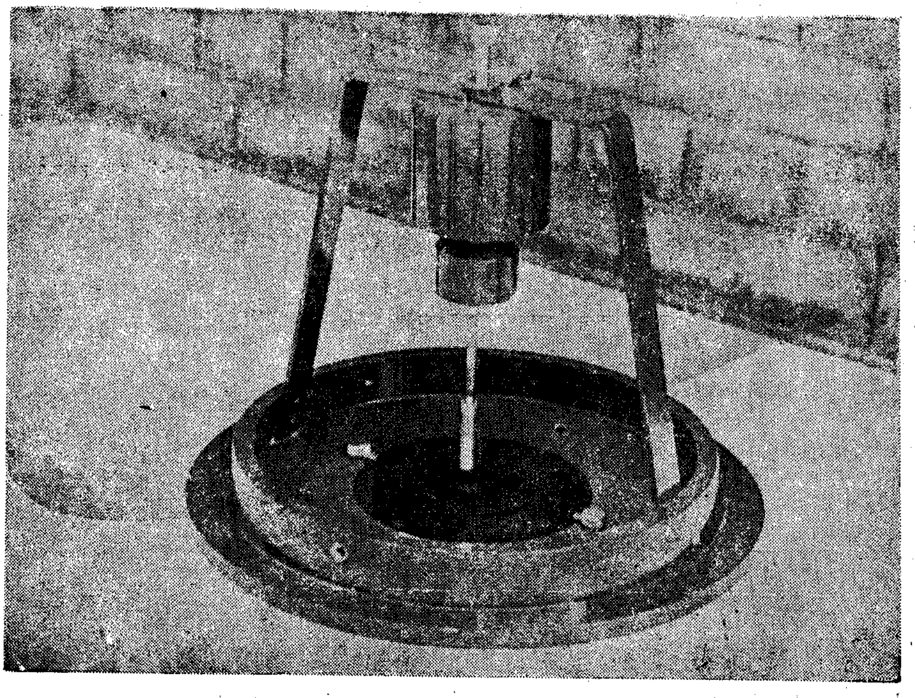

Fig. 3. Roorkee Seismoscope

in a vane attached to the pendulum. Electromagnets are employed for the purpose. A mirror is attached to each seismometer pendulum which reflects a light beam from a suitably placed light source to a recording drum through a lens, enabling the recording on a photographic paper. The recording speed is 1.5 cm/sec. The instrument gets started by a pendulum starter, when the ground acceleration has exceeded 5% of g. This instrument gives ground acceleration records (accelerogram) during strong earthquakes. Analysis of these records would yield useful information regarding the forces caused on structures during earthquakes[2]. These accelerographs are however very expensive. Also the records have to be processed through a digital computer for obtaining response of structures. This makes the channel even more expensive. Therefore, simplified instruments called structural response recorders or seismoscopes are now finding increased application in strong motion earthquake recording programmes for obtaining response of structures directly.

The Structural Response Recorder

These simplified earthquake recorders[3] measure directly the maximum dynamic response of structures having the same vibration characteristics, namely damping and natural period, as the instrument. Fig. 2 shows the Structural Response Recorder set manufactured at School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, incorporating six pendulums representing the crucial range of response spectrum. These have already been installed at more than forty selected sites in seismic areas of the country. The interpretaion of the records[4] obtained on these instruments is relatively simple. The instrument has a self recording device for recording the shock on a smoked glass. Recently another simplified and more sensitive recorder—the 'Roorkee Seismoscope'[5] has been developed. This would yield a bigger trace on the smoked plate for smaller earthquakes. The new recorder will be more effective for recording spectral accelerations upto 30% g. This is shown in the figure 3.

Study of Structures

The acceleration records obtained through the strong motion accelerograph enable calculation of the forces experienced by structures situated close to this instrument[6]. Referring to fig. 4, the following equation governs the behaviour of the structure shown therein—

$$M\ddot{Z} + C\dot{Z} + KZ = -m\ddot{y}\,(t) \qquad (1)$$

where M, C and K represent the mass, damping and stiffness characteristics of the structure and Z is the relative displacement of mass (M) with respect to ground. $\ddot{y}(t)$ is the ground acceleration recorded as function of time. The solution of eqn. 1 is obtained by using standard numerical techiques. For small values of damping, the solution could be expressed as,

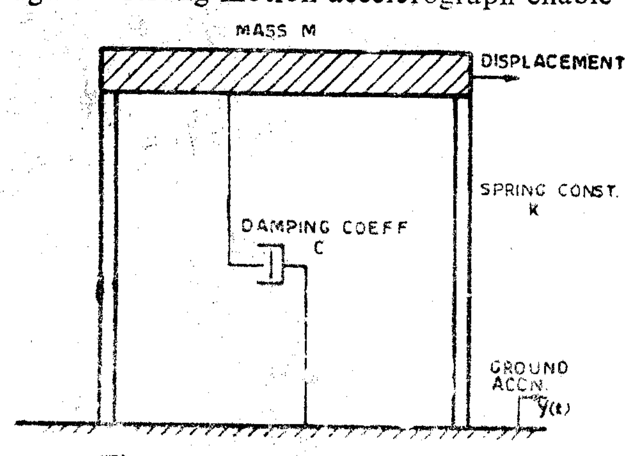

Fig. 4. The Idealised Structure

$$Z_{max} = \frac{1}{p}\,Sv \text{ where } p = \sqrt{K/M}, \text{ and } n \text{ is defined as } C/2M.$$

The maximum force experienced by structure is given by

$$F_{max} = K.\ Z_{max} \tag{2}$$

Expressing F_{max} as fraction of mg, the so called seismic coefficient α works out as

$$\alpha = \frac{K}{Mg} \cdot \frac{Sv}{p} \tag{3}$$

Calling the period of structure as T, and putting $K/M = p^2$, we have

$$\alpha = \frac{2\pi}{T} \cdot \frac{Sv}{g} \tag{4}$$

The maximum trace obtained on the smoked glass of the Structural Response Recorder is proportional to the seismic coefficient, 'α' mentioned in eqn. 4, Instruments having different periods and damping would yield response spectra[7] of a wide range of structures, during actual earthquakes. This information will be of utmost value in estimating forces on structures in the region where such instruments are installed.

Strong Motion Measurement Programme

For studying earthquake occurrences and recording scientific data for computing of structural response in seismic zones, a network of strong motion instruments to obtain ground motion records would form a basic requirement. For this, a number of seismographs, accelerographs and structural response recorders must be installed at selected location throughout the active seismic zones of the country. Whereas a set of structural response recorders is expected to yield response spectra of the earthquake at the site directly, an accelerograph must be installed to record the basic data which could be used for estimating response of any particular system with any restoring force and damping characteristics. The seismograph installed at various such stations would produce useful information regarding the characteristics of earthquake waves and the ground properties. Installation of such instruments in requisite number may appear to be a costly preposition, but the expenditure involved will be in general a very small fraction of the expected expenditure on various construction projects in those areas. Also more efficient design of structures on the basis of this data would result in an over all economy. Thus the strong motion earthquake recording programme must be intensified in the active seismic zones considering the invaluable scientific data that would be obtained for design of structures.

Conclusion

For design of engineering structures in seismic zones, the basic data regarding earthquake motion is obtained by stsong motion instruments like the accelerograph and the structural response recorders. The strong motion earthquake recording programme must, therefore, be intensified to get this basic data for future. In the absence of strong motion data at any particular site, data obtained in other location with similar ground properties could be used as guide-lines.

Acknowledgements

The paper is being published with the kind permission of the Director, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee.

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DYNAMIC LOADING OF SOILS AND FOUNDATIONS

Shamsher Prakash*

Synopsis

In this paper, the problems of dynamic loading of soils and foundations have been briefly described. These include stress-strain characteristics of soil, stability problems and problems associated with machine foundations and compaction of soils and application of blasting techniques for civil engineering purposes. Areas where fruitful research is needed have also been indicated. A comprehensive bibliography has also been appended.

Introduction

Dynamic loads may be caused due to (a) earthquakes, (b) bomb-blasts, either surface or air, (c) machines, (d) fast moving traffic and (e) construction operations e.g. pile driving. The nature of loads may be repetitive, and such loads pose a variety of problems to a foundation engineer in the design of earth dams, retaining walls and machine foundations. The phenomenon of dynamic loading can be usefully employed for constructive purposes e.g. pile driving for compaction of loose soils and for large scale excavations. In this paper, different facets of these problems have been described in brief along with suggested problems for research. A comprehensive list of bibliography has been appended for further reference of the reader.

Nature of Loads

The loads occuring in nature may be classified into the following categories:—

(a) Slow repetitive

(b) Fast repetitive

(c) Transient

  (i) Single impulse

  (ii) Multiple-impulse

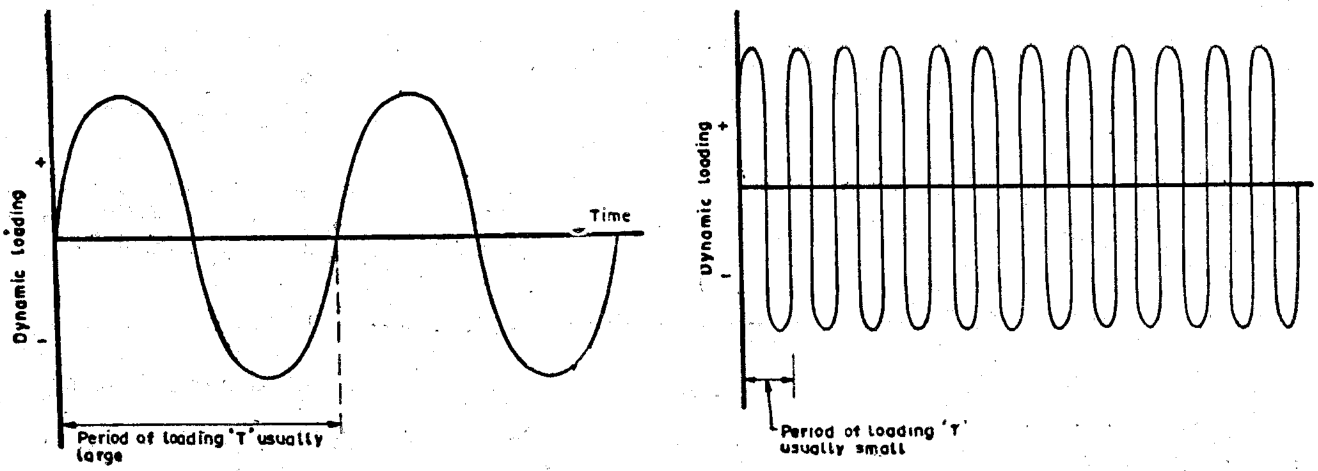
Figures 1 and 2 show schematically the diagrams of dynamic loods versus time. These are very simplified presentation of actual loading conditions. Actual loads in the field may often differ from the simplified loading diagrams presented above in the following respect:—

(a) The loading in nature is not truly periodic. Also, the magnitude of load in subsequent loading cycles is not the same as in the first cycle. This is especially so during an earthquake, Figure 3. Figure 4 shows trace of vertical acceleration of ground due to pile driving operations.

(b) Purely dynamic loads do not occur in nature. It is always a combination of static plus dynamic load. Static loads are caused by the dead weight of the structure and the sub-structure.

* Professor of Soil Dynamics, University of Roorkee, Roorkee, U.P., India.

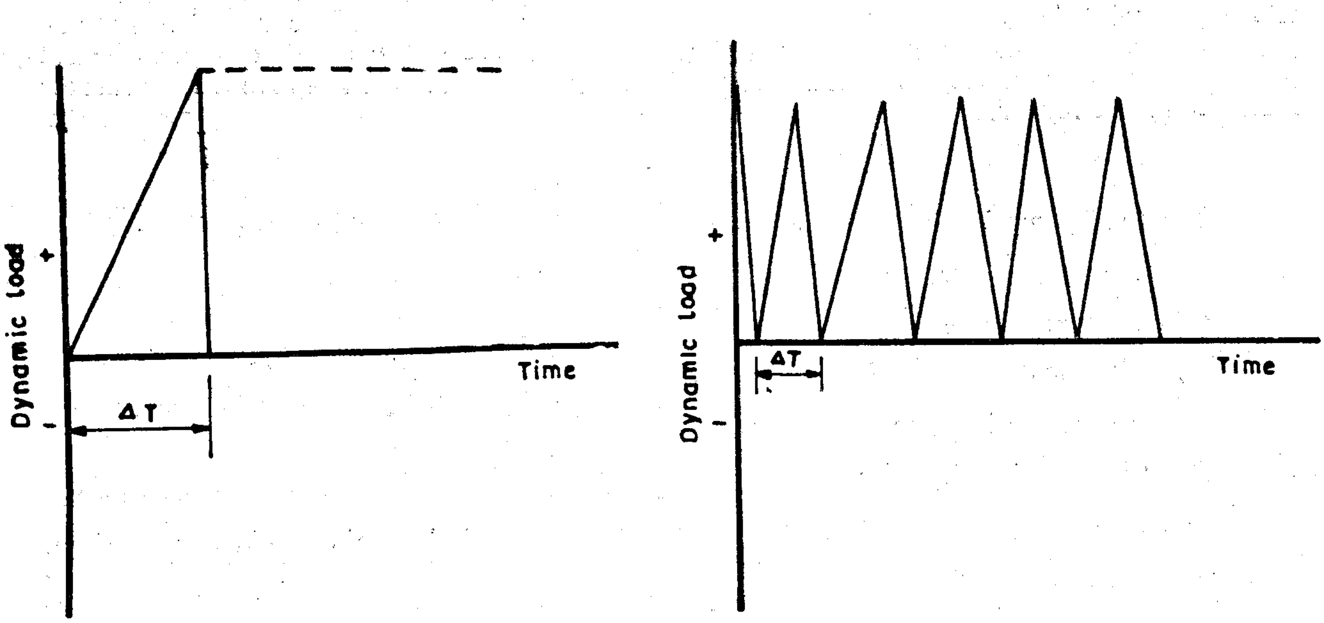
** Abridged from paper presented to Second Annual General Meeting of the Indian Society of Earthquake Technology, held at Roorkee on 14th December, 1968.



a–Slow repetitive loading

b–Fast repetitive loading

Fig. 1—Steady state dynamic loading



a–Single impulse transient load

b–Multi impulse transient load

Fig. 2—Transient loading

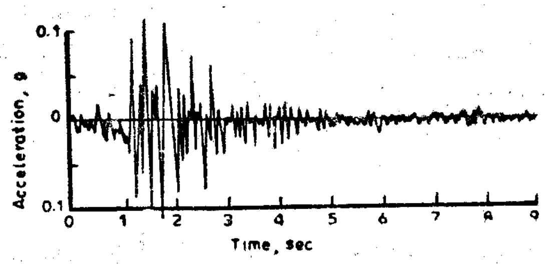


Fig. 3—Typical earthquake record Elcentro, May 18, 1940 N–S component

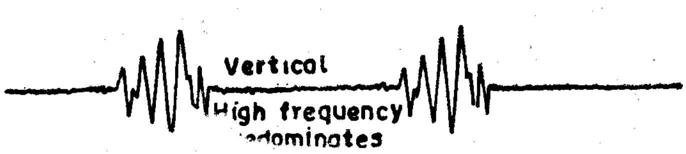


Fig. 4—Trace of vertical acceleration of ground due to pile dreving

If the frequency with which the load is applied is close to the natural frequency of the system, the effect of both dynamic nature and resonance is felt. However, if the frequency of loading is far removed from the natural frequency of the system, the effect of only rate of loading is perceptible.

It is not quite easy to determine precisely the natural frequency of systems involving structures made up of soils e.g. embankments, fills and foundations. Therefore, many a time an arbitrary criterion is employed.

Problems of Dynamic Loading of Soils

In the following sections different aspects of the various problems of dynamic loading of soils are briefely discussed.

1. Stress Deformation and Strength Characteristics of Soils.

Soils differ from other engineering materials like concrete and steel in many respects. One of the factors of great importance is that the soils are formed by processes of nature over which no human agency has any control. As a result of this, the soils are hetero-geneous in nature and determination of soil properties is a necessary job in connection with any foundation project. The dynamic loading introduces an additional variable. In any case, an effort has to be made to simulate the loading conditions of the field in the laboratory.

All the soil tests reported in the published literature may be classified into the following categories :—

(a) Transient loading tests.

(i) Single impulse transient load (Casagrande and Shannon 1948).

(ii) Transient pulses repeated at regular short intervals a number of times (Seed 1960, Seed and Chan 1966).

A combination of static and dynamic loads simulates the field conditions to a better degree.

(b) Vibratory loading tests.

(i) Samples subjected to vibratory loading of known characteristics i. e. stress level, frequency and amplitude of vibrations and then sheared as in routine tests.

(ii) Soils samples being under vibratory load during tests. This loading condition simulates field loading conditions to a better degree.

Various aspects of this problems are as follows :—

(a) Special equipment needs be designed to simulate field loading conditions. The recording of load and deformation and pore pressures is done on automatic recording devices.

(b) Effect of increased rate of loading on shear parameters c and ϕ, and shear modulus and strain at failure.

(c) Effect of pore pressure on strength in saturted soils.

2. Stability Problems

These include stability of earthen embankments, retaining walls and other retaining structures and stability of foundations. In earth retaining structures, the problem is to compute active and passive earth pressure. Mononobe-Okabe (1929) modified Coulomb's theory and determind an expression for dynamic earth pressures behined retaining walls. Kapila (1966) suggested modified Culumann's construction. The main difficulty is in ascertaining the point of application of the dynamic earth pressures. Experimental investigations carried out in Japan do indicate that the dynamic earth pressure acts at an elevation higher than one-third above the base of the retaining wall assumed in Mononobe-Okabe theory. Analytical and experimental investigations on flexible model retaining walls performed at Roorkee also indicate that point of application of the dynamic increment lies at a higher elevation than one third height from the base. Indian Standard Code (1893–1966)* arbitrarily recommends that dynamic increment of earth pressure be applied at two thirds the height of the retaining wall instead of that at one-third height.

Also, no information is at all available on the change of static "earth pressure at rest" if the basements are subjected to an excitation by an earthquake. These are fruitful areas for future research.

In the stability of earthen embankments, several problems are there. A design engineer is interested in determining the response of the dam to anticipated ground motion (Krishna 1962). He also wants to have tools to perform stability analysis, accounting for changes in properties of the soils due to dynamic nature of loading. (Seed 1960 Prakash, 1966). Information on field behaviour is extremely scanty. Therefore many a time, investigators resort to model tests. (Clough and Pirtz (1958), Seed and Clough 1963, Krishna and Prakash (1965, 1966) Krishna, Prakash and Thakker (1969). However there are uncertainties involved in extrapolating the model test results to prototypes. Perfecting of model testing technique is very fruitful area for future research. Analytical research on stress distribution and deformation pattern within the body of dam and in foundation has been attempted by various investigators by finite element analysis. Fundamental assumption of all such analysis reported to-date (1969) is that the soil is a linearly elastic material. Clough and Chopra (1966), Chopra (1967), Lian Finn and Khanna (1966), Saini and Sekaran (1968 a, b).

Stability of dam foundations is equally important. If the foundation material happens to be medium or loose sand, it may be subjected to complete or partial liquefaction during an earthquake. The problem of liquefaction has been studied in detail by Maslov (1957), Florin and Ivanor (1961), Prakash and Mathur (1965), Krishna, Prakash, Mathur and Gupta (1967) and Prakash and Gupta (1967, 1968), Seed (1968). The investigations fall under two categories : one, tests on soil models on the vibration table and the other, tests on triaxial specimens. In both the cases there are uncertainties in infering the laboratory results to field. Blast test were carried out both at Obra dam site in Uttar Pradesh and Tenughat dam site in Bihar in order to ascertain the possibility of the respective sands to liquefy during an earthquake. Laboratory tests on both the sands were also performed. Correlations of field and laboratory data are promising. (Krishna and Prakash 1967, Prakash and Gupta 1968 b).

Stability of footings during and earthquake and bomb blasts is a problem which has not been paid adequate attention. A footing may be subjected to excessive differential settlement and thus cause damage to the superstructure. In Niigata earthquake of 1964,

* Revised.

the buildings sometimes tilted as much as by 60° with the vertical (Prakash, Gupta and Chandersakaran 1968). This resulted from excessive differential settlements. No precise methods are available to compute total settelements of footings under dynamic loads. Differential settlements on account of even known total settlements is any body's guess. The problem does not appear to be prone to mathematical treatment without over simplifying assumptions because of heterogeneous nature of soils. However, carefully conducted model studies for a specific field problem may offer an alternative and workable solution. The scale of the model is likely to play an important role.

Bearing capacity failure of the soil is not likely to occur primarily because the initial factor of safety in well designed footing is quite high.

(d) Machine Foundations

Design of Machine fondation is a very complex job. The design enigeer is first faced with the problem of slecting a method of analysis out of a host of them available, based on theory of elasticity, soil spring concept and several empirical ones. The other problem is to determine suitable soil constants to be used in the design. The soil constants can be evaluated by (i) laboratory tests, (ii) field static tests and (iii) field dynamic tests. Field dynamic test including resonance test on block is recommended since it gives more realistic values. Such a test is usually performed by mounting a mechanical oscillator either on test block or a test plate*. The force of excitation in such a block varies with the frequency. It may therefore be desirable either to incorporate a device in the oscillator so that the force of excitation remains constant with variation in frequency or different oscillators may be used, which would satisfy the above requirement.

In resonance tests, the natural frequency of the soil foundation system depends upon the force of excitation. The ratio of unbalanced force to static weights in an actual machine is usually not more than 10 percent. It is therefore desirable that in a field test this criterion may be adhered to.

In many cases, it may be difficult to excite a block to resonance. Natural frequency in such cases can be determined by studying the free vibrations of the system. The natural frequency so determined is usually higher than the one determined in forced vibration tests. A correlations betweeh the two natural frequencies has been attempted by Puri (1969) based on typical model tests on blocks. Further verification of this correlationship will go a long way in simplifying the interpretation of field tests.

3. Peaceful Uses of Blasts

In earth dam construction, compaction of soils is a must. If the foundations are of loose or medium density and comprise of sands, vibratory technique of compaction may be more effective. Desired densities mey be achieved either by vibroflotation, blasting and pile driving. Vibroflotation is a patented process, while the other two methods can be adapted for use freely. Sufficient quantitative data on planning a field job by blasting technique is available (Prugh 1963, Lyman 1962, Hall 1962), while no such information is available on the amount of compaction achieved by pile driving. Some studies on this aspect of the problem are desirable.

Large excavation jobs can be handled efficiently by mechanical equipment. American engineers are postulating the use of nuclear blasts for excavation of a sea-level Isthmian

* Draf Indian Standard Methods of Test for the Determination of in-situ Dynamic Properties of soils DOC BDC 43 (1476) Jan. 1969.

Canal to connect the Atlantic with the Pacific. (Magnusan 1964, Vortman 1964). This will offer a route parallel to the existing Panama Canal. It is estimated that the technique of nuclear blasting would prove to be cheaper for especially large jobs. It is believed that with cheap manual labour available in India, it may not be necessary to envisage the use of these techniques in the immediate furure.

4. Vibratory Drilling

Piles are driven by conventional methods in this country so far. Considerable effort is spent in the actual operation of driving the pile with the help of pile hammer. Russian engineers have developed vibratory techique of driving piles and caissons. The effort involved in driving a pile by a vibratory hammer is reduced to almost nil. Russians have perfected pile drivers to drive caissons of as much as 5m outer diameter (Levkin 1960). In U.K. and USA also, vibratory drivers are gaining grounds. Once the technique is perfected, several other uses can be made e.g. in soil exploration.

Prakash and Singh (1967) report that for pulling out a 10 cm of casing about 18 m long on the campus of Roorkee University, it took them about a couple days. With a vibratory rig, this could be accomplished in a few minutes. Since large construction activity is ahead for us, development of vibratory hammers will prove to be a very desirable effort.

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DISCUSSION

Discussion on "Soil Investigations and Design of Forging Hammer Foundation" by Shamsher Prakash and D.C. Gupta, Bulletin, Vol. V, No. 1 and 2.

D. Raghu[1], The authors have to be congratulated for the nice work and excellent presentation of paper. However, it is felt that clarification may be made on the following points.

1. The number of strokes of hammer per minute is not mentioned. This is very important, particularly to check whether resonance will occur or not. If the frequency of the hammer strokes and any one of the natural frequencies of the system coincide, resonance will result.

2. It is mentioned in the article that the dynamic tests were made at a depth of 2.44 metres from ground level. It may kindly be mentioned whether the above tests were performed at the level corresponding to the bottom of the foundation.

3. The field data obtained by dynamic tests may be compared with those obtained in design calculations.

4. It is not known whether in the design of the foundations, the synchronism between the two hammers was taken into account or not. If during operation, the frequencies of these two machines synchronise, resonance will occur.

Reply by Authors, The authors are grateful to Sri Raghu for his interest in our paper.

(i) The frequency of operation of the hammer was 120 strokes per minute.

(ii) The dynamic tests had been performed at the depth of foundation proposed by the manufacturer.

(iii) We are planning a study of the performance of the hammer. The results will be reported when they become available.

(iv) The new hammer has been installed in a new shop and not in the existing forge shop. Hence the question of synchronism of the machines does not arise.

1. Department of Civil Engineering, B.I.T.S., Pilani (Rajasthan).

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SEISMOLOGICAL NOTES

(India Meteorological Department, New Delhi)

Earthquakes in and near about India during April—September, 1968

Date	Origin time (G.M.T.) h.	m.	s.	Epicentre Lat. (°N)	Long. (°E)	Region	Appox. depth (Kms.)	Magnitude	Remarks
1	2	3	4	5	6	7	8	9	10
April 4	01	44	26.4 (CGS)	24.6	66.0	West Pakistan	33	5.0 (CGS) 5.6 (NDI)	—
April 9	01	14	52.7 (CGS)	35.5	73.3	Northwestern Kashmir	14	4.4 (CGS)	—
April 12	10	33	58.3 (CGS)	36.7	69.1	Hindukush region	67	—	—
April 13	23	31	31.0 (CGS)	24.6	94.8	Burma-India border region	123	4.7 (CGS)	—
April 17	09	50	39.1 (CGS)	36.3	71.4	Afghanistan-USSR border	94	4.8 (CGS) 4.7 (NDI)	—
April 17	13	11	26.2 (CGS)	36.4	71.5	Afghanistan-USSR border	113	5.2 (CGS) 5.1 (NDI)	—
April 23	06	45	11.5 (CGS)	36.3	71.2	Afghanistan-USSR border	114	5.2 (CGS) 5.2 (NDI)	—
	06	45	11 (NDI)	36.0	71.0	-do-	—	5.2 (NDI)	
May 2	00	26	02.9 (CGS)	26.2	92.2	Eastern India	53	4.8 (CGS)	Felt at Shillong Dibrugarh, Jorhat, Dhubri etc.
May 6	00	26	00.5 (NDI)	25.0	91.7	Assam	—	—	—
May 6	20	49	45.5 (CGS)	36.4	70.8	Hindukush region	230	5.0 (CGS)	—
May 8	22	45	08.3 (CGS)	37.1	71.9	Afghanistan USSR border	160	5.1 (CGS)	—
May 14	02	41	16.1 (CGS)	36.1	70.9	Hindukush region	128	4.7 (CGS)	—
May 21	03	59	11.5 (CGS)	38.9	65.2	Southeastern Uzbek SSR	130	5.4 (CGS)	—

1		2	3	4	5	6	7	8	9	10
May	27	18	35	57.0 (CGS)	29.7	80.4	Nepal India border	27.0	5.1 (CGS)	—
		18	36	02 (NDI)	28.8	81.0	-do-	—	4.3 (NDI)	—
May	31	03	01	35.7 (CGS)	29.9	80.0	Nepal India border	33	5.1 (CGS)	—
		03	01	42 (NDI)	30	79.5	-do-	—	4.2 (NDI)	—
Jun	4	05	10	52 (CGS)	35.7	82.1	Tibet	33	4.8 (CGS)	—
Jun	5	00	09	41 (CGS)	36.1	66.2	Hindukush region	33	4.8 (CGS)	—
Jun	9	04	13	08 (CGS)	6.4	95.2	Nicobar Island region	33	4.2 (CGS)	—
Jun	10	17	36	30 (CGS)	39.0	75.1	S. Sinkiang Province, China	51	4.8 (CGS)	—
Jun	12	04	24	25 (NDI)	26.0	91.1	Assam	—	5.5 (NDI)	Felt at Shillong
Jun	13	15	37	43 (CGS)	24.7	66.4	West Pakistan	33	4.3 (CGS)	—
Jun	13	22	38	01 (CGS)	36.6	71.5	Afganistan USSR border	213	4.8 (CGS)	—
Jun	14	04	02	22 (CGS)	31.2	70.2	West Pakistan	25	4.9 (CGS)	—
Jun	17	06	25	55.3 (CGS)	37.4	72.3	Tadzik SSR	195	4.8 (CGS)	—
July	1	03	11	10.0 (CGS)	30.3	94.5	Tibet	28	4.3	—
July	3	19	46	53.7 (CGS)	34.7	75.1	Eastern Kashmir	113	4.5	—
July	4	06	45	58.0 (CGS)	30.3	94.9	Tibet	33	4.7	—
July	8	13	14	29.9 (CGS)	38.0	67.6	Southeastern Uzbek SSR	28	6.3	Felt widely. (outside India)
July	13	06	05	54.2 (CGS)	30.3	94.6	Tibet	33	5.0	—
July	14	18	12	41.0 (CGS)	30.3	94.8	Tibet	22	4.9	—
July	15	01	25	36.0 (CGS)	36.3	68.4	Hindukush region	35	4.1	—
July	15	05	09	05.9 (CGS)	30.3	95.0	Tibet	22	4.8	—

1		2	3	4	5	6	7	8	9	10
July	16	11	13	40.0 (CGS)	36.0	71.2	Hindukush region	5	—	—
July	16	22	23	07.0 (CGS)	30.3	94.8	Tibet	40	4.8	—
July	18	17	20	29.0 (CGS)	8.9	93.9	Nicobar Island region	33	4.8	—
July	18	17	43	24.0 (CGS)	8.8	93.8	Nicobar Island region	33	4.3	—
July	19	04	56	27.2 (CGS)	8.7	93.6	–do–	33	5.3	—
		04	56	20 (NDI)	9	95	Nicobar Island	—	5.9 (NDI)	—
July	19	06	07	22.0 (CGS)	8.9	93.8	Nicobar Island region	33	4.8	—
July	19	16	42	15.9 (CGS)	8.7	93.7	–do–	8	5.1	—
July	19	18	48	59.0 (CGS)	30.2	94.9	Tibet	33	4.9	—
July	20	08	22	08.6 (CGS)	39.4	73.8	Tadzik Sinkiang border	22	4.8	—
July	23	20	51	47.9 (CGS)	30.3	94.9	Tibet	30	4.9	—
July	25	03	34	13.0 (CGS)	30.2	94.8	–do–	33	4.8	—
July	26	12	44	03.0 (CGS)	29.4	95.0	India China border region	33	4.9	—
July	26	20	48	03.0 (CGS)	32.1	70.1	W. Pakistan	35	4.8	—
Aug.	2	13	30	23.3 (CGS)	27.5	60.9	Eastern Iran	62	5.7	—
Aug.	3	14	01	46.5 (CGS)	25.8	62.8	W. Pakistan	41	4.7	—
Aug.	5	02	41	11.9 (CGS)	35.7	70.2	Hindukush region	9	3.6	—
Aug.	9	02	24	53.2 (CGS)	25.2	94.4	Burma India border region	33	4.7	—
Aug.	18	14	18	59.5 (CGS)	26.4	90.6	Eastern India	31	5.2	—
		14	19	10 (NDI)	27	89.5	Near Kalimpong, N. Bengal	—	5.7 (NDI)	—
Aug.	23	12	01	16.6 (CGS)	30.3	94.9	Tibet	33	4.8	—

1		2	3	4	5	6	7	8	9	10
Aug.	24	14	26 (CGS)	07.4	30.0	95.1	Tibet	56	4.6	—
Aug.	25	17	55 (CGS)	05.3	30.4	94.8	-do-	19	4.8	—
Aug.	26	18	23 (CCS)	41.3	36.4	70.7	Hindukush region	203	5.0	—
Aug.	29	19	51 (CGS)	24.6	30.2	95.1	Tibet	33	5.0	—
Aug.	31	02	53 (NDI)	40	17.3	74.0	Koyna region Maharashtra	12	5.7	Felt upto Bombay.
Sept.	1	05	59 (CGS)	26.6	30.3	94.8	Tibet	20	5.0	—
Sept.	3	17	45 (CGS)	54.1	30.2	94.8	Tibet	5.3	4.9	—
Sept.	4	01	40 (CGS)	04.0	33.5	97.5	Tsinghai Province, China	33	4.8	—
Sept.	5	05	32 (CGS)	01.1	14.7	96.8	Andaman Island region	33	4.8	—
Sept.	10	05	04 (CGS)	58.3	15.2	93.2	Bay of Bengal	33	4.2	—
Sept.	10	17	18 (CGS)	08.9	36.3	70.8	Hindukush region	18	4.7	—
Sept.	11	03	07 (CGS)	32.0	30.3	94.9	Tibet	38	4.3	—
Sept.	12	15	36 (CGS)	48.8	39.8	77.8	S. Sinkiang Pro., China	8	4.9	—
Sept.	15	14	16 (CGS)	55.8	37.2	72.7	Tadzhik SSR	33	5.2	—
Sept.	16	17	02 (CGS)	40.2	28.6	95.7	India China border region	60	4.7	—
Sept.	18	07	37 (CGS)	21.8	37.2	71.9	Afghanistan USSR border	123	5.0	—
Sept.	25	—	—	—	—	—	—		—	Tremor felt by some people at Trivandrum at 01h 28 m GMT.