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BULLETIN OF THE INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY

Aims and Scope

The Indian Society of Earthquake Technology was founded in November 1962 with the aim of advancing knowledge of the earthquake technology in all aspects.

The Bulletin of the Indian Society of Eathquake Technology, covering all aspects of earthquake technology presents a cross section of technical papers, reviews, comments and discussions in the field of earthquake technology, news of the Society, serving as a medium for recording recent research and development work.

Publication

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WORLD RIFT SYSTEMS, THEIR VOLCANISM AND SEISMICITY

R. S. Tipnis* and L. S. Srivastava†

Abstract

The past seismic activity in the Western Maharashtra as well as the recent earthquake in the Koyna area appears to be confined along the continental divide and the coastal region and apparently related with the origin of the Deccan traps covering this area. A world wide survey of areas showing volcanism associated with seismicity indicate that these form a global pattern of rifts occupying the continents as well as the oceans, and possess striking similarities with the volcanism and seismicity of the Deccan trap region.

Purpose of the Study

The purpose of the study of the rift valleys in the world has been to find out if there are any similarities between rift valleys and their volcanics to the erruptive lavas of Deccan Traps. These Deccan traps in the central and western part of India occupy an area of about 650,000 sq km and are considerd to have their maximum thickness of 3000–3500 meters along the western coast of Maharashtra. Apart from the volcanicity the western region has undergone seismic activity in not only geological epochs but even in historical past. Therefore it was also the object to see whether seismic activity in western region of India also follows a global pattern of rift volcanicity and their seismicity. It is likely that the Deccan trap region of India may have the same origin as other rift valleys and their associated volcanics. Moreover, it would appear that since the Deccan Trap region is essentially a land mass as seen today, it has very little in common with rift phenomenon of oceanic regions. However, since Deccan trap expanse extended beyond to the west, if not major, at least some igneous phenomenon associated with it has been submarine.

Rift Systems

The continuous seismic activity following the mid-oceanic ridges and the close geo-morphorlogical and structural resemblance of the central rifts (Fig. 1) with many of the continental counterparts and their associated volcanism have revealed several interesting features. Their alignments in various parts of the earth indicate that these rifts form a part of a global system of epiorogeny comparable to the well known orogenic girdles represented by mountain belts and island arcs.

In such rift systems the following four rift patterns are very conspicuous.

(i) Rifts situated meridionally between the mid-oceanic ridges and extending over large distances (e.g., Rifts along Mid-Atlantic and Carlsberg Ridges).

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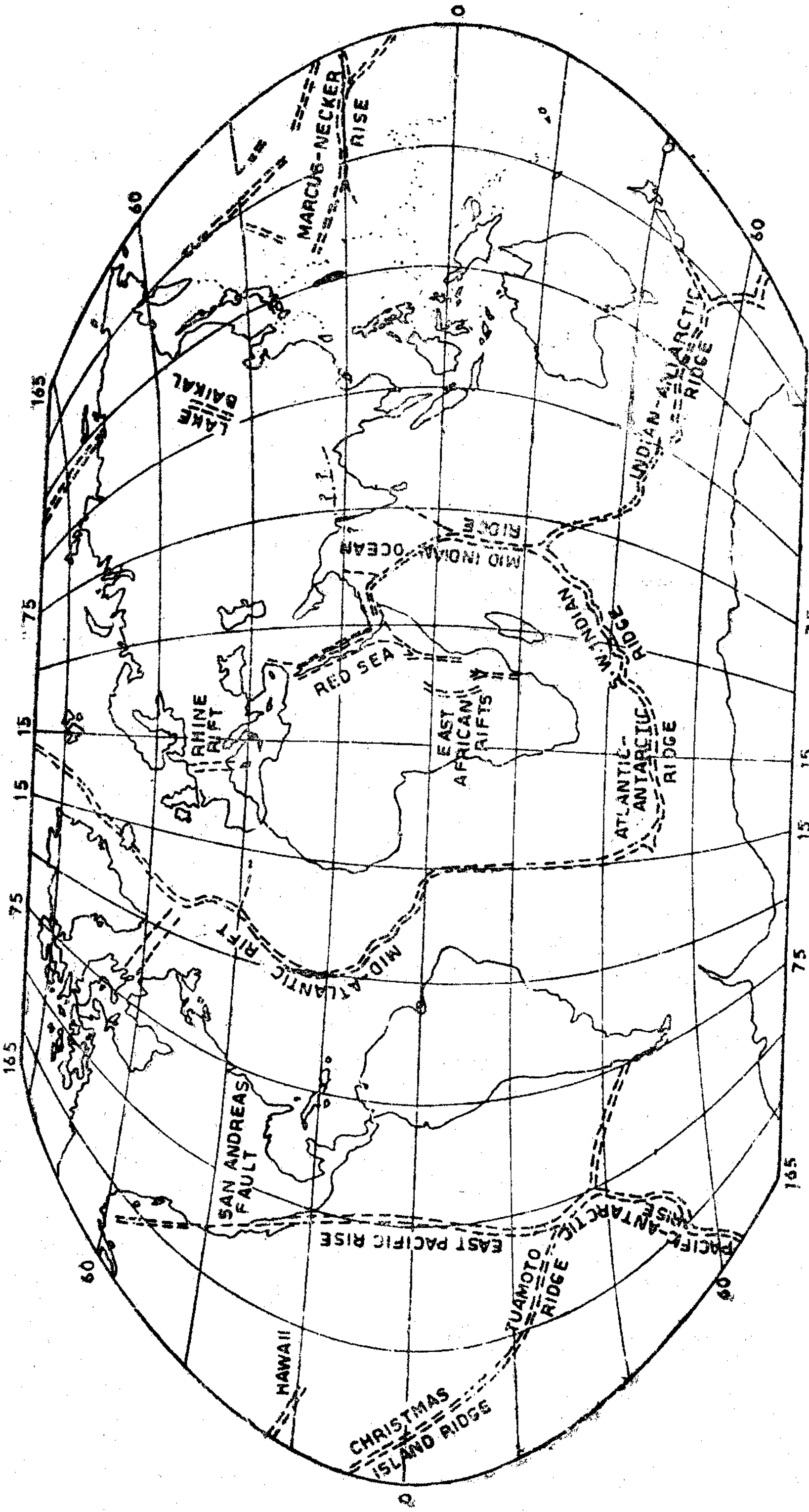

Fig. 1 Map Showing the World Rift System

(ii) Rifts situated meridionally over the continents and often extending for considerable distances (e.g., East African Rifts, Rhine Rhone Rifts etc.).

(iii) Rifts situated along continental margins, inland gulfs and seas (e.g., Red Sea Rift, Gulf of Aden Rift, Gulf of California Rift, etc.).

(iv) Rifts situated within continental shields controlled by the pre-existing orogenic lineaments and with limited extensions (e.g., Baikal Rift, USSR, Satpura Rift, India etc.).

The above do not include the shallow elongated rift like depressions similar to Gondwana rifts of the continents, which show random trends and are of limited extent.

Geological investigations, over various rifts suggests that they originated (Mc Connell 1967) as complex strike slip zones of fractures diverting along or around pre-existing cratons. Their tendency to remain parallel to the major structural trends is perhaps an accomodative effect of the rift to a previously established structural grain. Strike slip movements appear to have attained a greater proportion in the earlier phases, but after the widening of the system, caused by splitting of the axes along cratons, probably deep seated migmatization and granitization sealed the sutures, thereby restricting further strike slip movements (Mc Connell, op. cit.). The renewal of further activity in later geological times along such rifts was essentially vertical, thus following the same structural trends. This explains the close correlation of the later rifts with the pre-existing lineaments.

The earliest known rift structure dates from Jurassic and the youngest are of Tertiary era. Presence of rifts in the pre-Jurassic times can not be ruled out, as it is likely that subsquent movements may have obliterated their traces.

Structural set up of the Rifts

The rifts described above form trains of elongated narrow troughs bordered by parallel normal faults. In the absence of direct observations regarding oceanic rifts, the knowledge of their structure is essentially based on indirect observations and conditions on the adjacent continental masses.

In general the rifts have been effected by transverse fractures and occur as discontinuously off-set units. The width of the main rifts vary from 30–60 kms and in the subsidiary rifts 10–20 kms. Wide rifts, like Red Sea and Gulf of Aden rifts, are also found to occur at many places. The depths of the rifted blocks vary in different patterns.

Many rifts have asymmetrical structure and their deepest segments do not lie at middle, but are situated near to the flanks. Within the rifts, the down faulted block is often observed to have been dissected by numerous faults into small horsts and grabens.

Transverse off-setting of the rift blocks have taken place along east-west strike slip faults in case of both the continental and oceanic rifts. In the case of oceanic rifts, however, these fractures are for more spectacular, and give an appearance of slicing the rifts into seperate elongated crustal blocks. The rift along the Carlsberg Ridge, in the Arabian sea floor of the north western Indian ocean, is off-set by three fracture zones, the Owens fracture zone in the north trending NNE–SSW, the Vema trench trending NE-SW and the Rodriguez fracture zone showing E–W to ESE–WSW trend. Similar off-setting is also noted along the rifts in the Mid–Atlantic and Pacific ridges. In the continental rifts transverse lineaments in the Ethopian rift system (Mohr, 1967), sinistral movements in the Dead sea rift, Lake Albert and Lake Edward rift (Brock, 1965) and transvers movements in other rifts have been observed.

Rifts and Volcanism

In general, both the continental and oceanic rifts are associated with alkaline olivine basalts. The tholeiitic basalt characteristically found within the continental rift valleys also have their counterpart in the oceanic rift zones. There are certain areas having both associations (mixed provinces) of the alkaline olivine basalts as well as the tholeiitic basalts.

On the basis of regional studies carried out in various parts of the world it is noted that though alkaline olivine basalts have close affinities with the oceanic as well as the continental rifts, carbonatites and alkaline igneous erruptive centres are generally confined to the continental rifts. Often in these regions the erruptive rocks are noted to occur in particular geographic patterns. For instance in the East African Rift the oldest Mesozoic lavas occur in the southern part, and as one proceeds northward, younger and younger lavas appear. The older lava are tholeiitic while younger of alkaline olivine type. Menard (1965) has drawn attention to certain characteristics of oceanic rifts and their magmatic associations, and has pointed out the possibility of continental plateau basalts being associated with such rifts in the adjacent ocean floors. The Indian, Atlantic and Pacific ocean floors are associated with rifts which are regions of extrusive activity, and where ever either a branch of the rift or the main rift itself meets or overlaps the continental margin, the surrounding region also become the seat of similar volcanism. Some of the continental basalts considered to be associated with the various oceanic rifts are : Snake River and Columbia River basalt of Western U.S.A. associated with East Pacific Rifts and their landward extensions (Hamilton, 1968); Brito-Arctic Province including part of Greenland islands of Iceland, Forerol, San Mayen, Hebrides, part of Scotland and Ireland, all associated with the northern Mid-Atlantic Rift System; Indian, Arabian and Ethopian flood basalts related to the Carlsberg Rift System; Patagonian basalts related with the Chile Rift; St. Lawrence basalts related with the West Atlantic rift; and Lake Baikal Volcanics related with the possible Circum Eurasian Rift.

No definite correlation exists between the type of volcanics and tectonic framework of the various rifts. Both type of magmas possibily originated from same common source rock of the Upper Mantle (Kuno and Kushiro, 1963; Poldervaart, 1962). At such depths, depending upon the pressure and temperature conditions, melting would occur and magma tapped from base. This hypothesis partially explains the phenomenon, but the actual depth of the magma source and its generation is still a matter of conjecture. But due to the very close correspondence observed between rifts and volcanism it is suggestive that magmatism and rifting could be results of the same major process.

Flexures

The close correlation between volcanism and rift structures is also surported by the presence of flexures. These occur along the rift zones within the basaltic regions, as more or less monoclinal structures. They could be the seats of large sequence of thick plateau basalts having deep seated connections extending upto the Upper Mantle (Gibson, 1966). It is also possible that the extension of flood basalts from the oceanic rifts on the continental margins following the downwarps would have produced these flexures. There is a close association between the flexures and the swarms of dykes which often trend parallel to the flexures. In some cases the flexures are marginal to the continents and also trend parallel to the mid-oceanic ridges (Menard, 1965). Some of the flexures associated with known rift systems and volcanics are : East Greenland Flexure and the associated system of dyke swarms and lava flows; Snake River basalts and the western Snake River Plain graben and warp; Columbia River Basalts and the flexures in the southern and eastern

Columbia plateau ; Red sea volcanics, Red sea graben and Gulf of Aden flexure ; and Deccan Lavas and Panvel Flexure near Bombay.

The Panvel flexure and Deccan lava flows along the west coast of Maharashtra, India, have not been correlated to any rift system. The likely presence of a rift has been postulated by Srivastava and Tipnis (1968) as a cause of the seismicity of the area. Sukeswala (1966) on the presence of acid differentiates and alkaline erruptive rocks has also suggested such as possibility. The presence of Panvel flexure and the associated dykes further supports the possible existence of a rift below the Deccan traps.

Evolution of Rifts

The different characters of the oceanic and continental crust has led some to believe that the rifts in the oceanic regions and the continental masses are a result of different dynamic processes. Continental rifts indicate operation of vertical tectonism, whereas the oceanic rifts are considered to have resulted due to lateral forces.

A rift valley is generally defined as "an elongated sunken area which has descended between two parallel faults". The development of the bounding faults in such situations would be the result of horizontal tensional forces acting in the crust. Such horizontal tension leads to uniaxial stress release forming a normal fault plane through the crust (Anderson in Girdler, 1964). Isostatic readjustment would lead to further crustal deformation and development of faulting. Thus a squence of events in the development of rifts due to tensional stresses would be (1) development of initial tension cracks which may have deep seated connections with the Upper Mantle along which extrusion of volcanic rocks may take place, (2) subsurface and surface movements due to sliding or movements of magma thereby giving rise to the formation of sets of subsequent fractures bounding the central rift and (3) the isostatic adjustments of the subsided crustal blocks inside the rift zones (Girdler, 1964). Althogh this hypothesis and sequence of events can explain the formation of rift structure and the geophysical anomalies associated with it, geomorphological evidence show considerable uplifts, foldings and faultings within the rift sediments, which indicate that compressive forces have a greater role in the development of the rifts.

As an analogy the continental crust can be compared to a plate under compression or subjected to vertical forces. The outer convex layer of this plate will be subjected to tension as the plate is warped up. Growing tension would lead to the formation of fractures along which a rift could develop. Subsequent rise of the walls of such rifts due to continued forces would increase their apparent subsidences. The width of the rifts so produced by such processes would be of similar dimensions as the thickness of the crust (Holmes, 1964) and the interval of fractures may also be more or less at similar spacings. The structure of East African Rift Valley system is an example of such a hypothesis of development of the rifts.

Although floors of some exceptionally deep rifts give indications of subsidence, on closer examination it seems that this may be due to their lagging behind during the general uplift of the adjoining plateaus. The vertical warping manifested by large scale arching is followed by the formation of grabens. If this general warping and uplift is also accompanied by crustal distention (Holmes, op. cit.) to give freedom of movement to rift blocks, the situation would be very favourable for volcanic activity along the resultant fractures. The association of volcanic activity with rifted upwarps supports this hypothesis. It is also likely that such upwarps or uplifts, distentions or the fractures in the crust, and the ascent of gases and lava may be related to localised sources of magmas in the rifts themselves (Holmes, op. cit.). However, it may be noted that both the rifts and the extrusions are the result of readjustments in the crust and the subcrust. It is, therefore, conceivable that full readjustment may also take place with rifting alone, i.e. not followed by volcanic activity. This may be the possible explaination for the lack of volcanism associated with some of the deep rifts (e.g. lake Tanganiyka Rift).

The geomoprhological analysis along rift systems also indicate that the sides are rising since the Jurassic times and that location of the rift is along the more uplifted portions of the continents forming the continental divides. Rise of erosion levels also points to the continental crust being warped in phases since Jurassic times, thereby accentuating the rifts within the crust. Although there is a period of quiescence when planation take place, similar pattern of warping, rifting, magmatism, etc. are repeated again and again in course of time. Initial flexures in such regions of warping would tend to be parallel to the orogenic trends of preexisting flexed or uplifted areas.

Another evidence of uplift is in the exposure of carbonatites from the central parts or cores of the volcanic necks by erosion of the superincumbent material, caused by upheaval of the region. This presence of the rock suits of carbonatites of various ages also indicate different periods of upheaval and associated activity. Upwarping of the crust would produce relief of rock load below the flexure which may initiate an elastic release fracture, and such a fracture zone would be repeatedly affected by succeeding periods of tectonism.

In the case of oceanic rifts the activating forces are in general considered to be horizontal due to the rising convection currents from below, and their subsequent lateral spreading. Spreading of oceanic floor is also considered to be due to such convection currents (Dietz, 1961 and Hess, 1962). The motion in the convection current cell may be vertical along the sides and parallel to the earth's surface along the top horizontal portions. The horizontal drag in opposite directions due to two such cells in the mantle would result in development of tension in the overlying crustal layer gixing three seperate zones of rupture. In the middle axial zone the rock would undergo vertical movements, whereas the two flanks would be dragged outwards. The actual rate of spreading or widening would be proportional to the speed of the horizontal drag of the cell. The width and depth of the gap forming the rift would depend on the dimensions of the affected crustal layers. The tension developed in the flanks may also produce horsts and grabens bordering the rifts

The oceanic rifts are mostly associated with the oceanic ridges. Along such ridges expansion of sea floor, thining of the crust, horsts and grabens and volcanoes transverse or parallel to the rifts are observed (Menard, op. cit.). The continental counterparts of these rift sytsems also show most of these features. This is so, because the central rift or graben of such zones is a complex of parallel or en-echelon major faults with many transverse and even diagonal faults. Transverse fracture zones in both cases are roughly perpendicular to the general trend of the rifts and may have been produced by the same mechanism which gave rise to the rifts. Because of these similarities in the continental and oceanic rift systems it was considered that same processes were responsible for their development, and the continental rift were explained as resulting from rising convection currents (Heezen 1960). But in such cases no evidence of spreading of the crust is observed. Also, the convection current hypothesis cannot explain the discordant trends of Red Sea Rift, Gulf of Aden Rift and other rifts, unless very small convection current cells operating under every individual rift are assumed to have existed. This hardly seems likely. The real cause of rifting thus appears to be related to the fundamental difference between the continental and oceanic crust. The latter is influenced by ocean spreading and formation of new crust, where as the former is a product of nearly vertical faulting associated with wide arching (cymatogeny).

Seismicity of the Rift Systems

From the foregoing discussion it is noted that there is no single decisive theory which could explain the complexities of the Global Rift systems. However, all the rift valleys appear to be closely connected with the Upper Mantle and thus the rift axis represents a deep lineament. The various orogenic tectonic and thermodynamic processes acting at depth along these lineaments would build up strain energy, release of which would be responsible for the seismic activity as observed along the rift systems. Earlier it was considered that the whole of the rift zone, along the oceanic ridges is seismically active. Later studies indicate that this activity is confined mainly between the two crests of the mid oceanic ridges bounding the central rift zone. Similar set up is observed along continental rifts indicating concentration of earthquake activity along the bounding rift faults (Sutton 1965). Wherever the rifts intersect a fault or other tectonic or volcanic lineament an increased seismicity is observed.

In case of the continetal rifts the epicentres are noted as more or less confined to the bounding faults and other faults transverse and parallel to the margins of the rifts, produced by associated warpings and volcanism. Rift valleys without transverse faults and other lineaments in general show lesser seismic activity. Some of the continental rift systems with pronounced seismic activity are L. Baikal rift, St. Lawrence Rift, Rifts in S.E. Australia, Cainozonic Rift zones of western USA, Satpura Rift (India), Rhine Rift (Europe) and East African Rifts.

A comparision of the geology and tectonics of the various rift system with the western part of Maharashtra indicates that this region may also have a similar origin and the Deccan traps may have been poured out through a series of fissures and fractures.

On the basis of the present distribution of the Deccan traps it seems likely that a north-south main rift, associated with a series of transverse subsidiary fractures or rifts, form the main fissures for the out pouring of lavas. Although little is known about the tectonic movements and the earthquake activity in this region, it appears quite probable that prior to the formation of volcanics the region had suffered earth movements responsible for the later warping, rifting and magmatism in the area. This activity in the earlier geological epoch has continued, perhaps intermittently, to historical times. It is reported that nearly 30 earthquakes of significant intensity have occured in this region during a priod of 350 years. It is also very significant to note that this activity in the Western Maharashtra follows the pattern of seismicity as observed in the other important rift systems. The epicentres of the earthquakes are in general aligned in North-South trend possibly following the bounding faults of the North-South oriented rift. The focal mechanism as revealed by detailed seismological study of the Dec. 11, 1967 Koyna Earthquake show that strike slip movement along a fault plane striking N 26° E and dipping at an angle of 66° towards North-West (Tandon and Chaudhary, 1968) was probably responsible for this shock. This perhaps represents a fault diagonal to the main rift along which, in general a higher seismicity is observed. The other epicentres in the region off-set from the general North-South alignment would also be lying along or at the intersection of other faults. Seismicity at the junction of Narbada rift with postulated Maharashtra rift zone is another evidence of seismic activity along transverse junctions.

The seismicity of the rift system has a global pattern comparable to other seismic belts. Along the main rifts there is in general a tendency for dip slip movements for readjustments of stresses, whereas along the transverse fracture zones there is predominence of strike slip movements. The earthquakes resulting from latter process have in general been observed to be higher in magnitude. This may be due to the fact that dip slip movement undergo relaxations due to isostated adjustment prohibiting accumulation of strain energy.

Conclusion

The distribution of the various rift systems on the surface of the earth shows that they form a global system of epiorogeny similar to the Pacific island arcs and Alpine-Himalayan orogenic girdles. The major process in the development of the rift system appears to be the general up-warping of the crustal layers, fractures and rifting due to resulting tensile stresses in the crust or release of pressure below the crust, and magmatism. The rifts are characterised by abundant volananic activity of alkaline and carbonatitic types related to the common source rock comprising the Upper Mantle. Most of the oceanic rifts are found to be situated meridionally along the mid-oceanic ridges and along continental margins, inland gulfs and seas. The continental rifts are also noted to occur meridionally but in many situation the pre-existing orogenic trends appear to control their distribution to considerable extents. This continental rift pattern along with the occurence and distribution of carbonatite, rise of erosional surfaces and the present situation of some of the rifts along contintinental divides, all indicate that continental rift systems have been subjected to repeated uplifts in various geological times. In most of the rifts, associated with volcanic activity occurence of earthquakes is a very common phenomenon.

Along the west coast of Maharashtra it is seen that pattern of fractures and other lineaments are parallel to the alignment of Dharwars immediately south of the Deccan traps. The Satpura rift in Central India is seen to strike parallel to the pre-Cambrian belt of the same name. Both these areas are occupied by volcanics, and show mild seismicity. Such an association of the volcanics with the rifts formed over the ancient orogenic belts prompts the conclusion that the region is not dissimilar to the global rift valley systems reviewed above. That such areas are seats of mild seismicity has also been indicated. It is, therefore, no wonder that the Deccan Traps too have been subjected to seismic activity. Even the patently shield areas which are supposed to be immune to earthquakes have been found to be active wherever combination of rift valleys and volcanics occur. The case of East African, St. Lawrence, Baikal, Rhine and some South Australian rift valleys are in point. To this list should also be added the Satpura Rift and the rift along the Western coast of Maharashtra.

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STUDY OF DAMAGES AND THROWS IN KOYNA EARTHQUAKE*

R. N. Joshi, Chartered Engineer

Synopsis

The paper reports the observations of damages to structures during the Koyna earthquake. Most of the building structures in the area are single storeyed structures built in masonry. They are mostly in ruins. Study of cracks and failures of structures indicates several specific types which have been broadly analysed and presented. Likely seismic acceleration has been worked out in some cases. Direction-wise study of the throws of free objects has been made to draw inferences therefrom regarding the shaking during the earthquake. A scrutiny has been made to find out if the structural failures indicate any similarity by direction and if this one fits in with the throws of free objects. Figures have been given to indicate the observations made. Desirability of upgrading of seismic zones in the Deccan has been discussed. The necessity of educating the public in adopting the basic principles of earthquake resistant structures is emphasised as it is felt that any possibility of recurrance of earthquakes in the Peninsular region should be taken with alertness and caution rather than with panic. It is also necessary to take a nationwide review of materials to be used extensively for earthquake resistant construction.

Introduction

Since the Koyna Dam was brought in commission some tremors have been experienced in the Koyna region. But the tremors were of very shallow focus, say 4 km deep and of magnitudes upto 3.5. It is possible that such minor tremors might be due to some adjustment on account of sagging in the basin due to water load. But the heavy earthquake of 11th December 1967 must be having a deep-seated cause and cannot be attributed to the reservoir, though the tremors in the reservoir area might have triggerred the action at lower depth which was even otherwise in the being.

Heavily Damaged Area

Fig. 1. shows the heavily damaged area based on the observations of the author and a team of accompanying engineers which included Sarvashri M.N. Kulkarni, N.V. Joshi and Professrs. A.G. Patwardhan, Bapat and L.S. Sane. Damages to well-built structures in villeges were observed in particular and were taken as a measure rather than damages to poor type of construction or the rehabilitation activity. Buildings in many villeges which fetched useful information were the school building. the village panchayat office and a couple of houses and shops of leading merchants of the village. Damage to mud house of poor construction certainly aggravated the hardship of the villagers but such damage would not be a proper measure of the shock felt at the place.

Koynanagar experienced very heavy shocks. There was large concentration of buildings in Koynanagar. The township accomodated the offices of the Project in its subsequent stages and housed some 10,000 pessons. The earthquake took a toll of a hundred persons in Koynanagr and dishoused the entire population. Many of the structures were razed to the ground and others were badly damaged.

Surrounding villages which experienced heavy shock and damaged well-built structures include Helwak, Ghatmatha, Nechal, Goshtwadi (Gojegaon), Kamargaon, Nanel,

* Presented at Symposium on Koyna Earthquake of December 11th, 1967, and Related Problems, June 1-2, 1968, Calcutta.

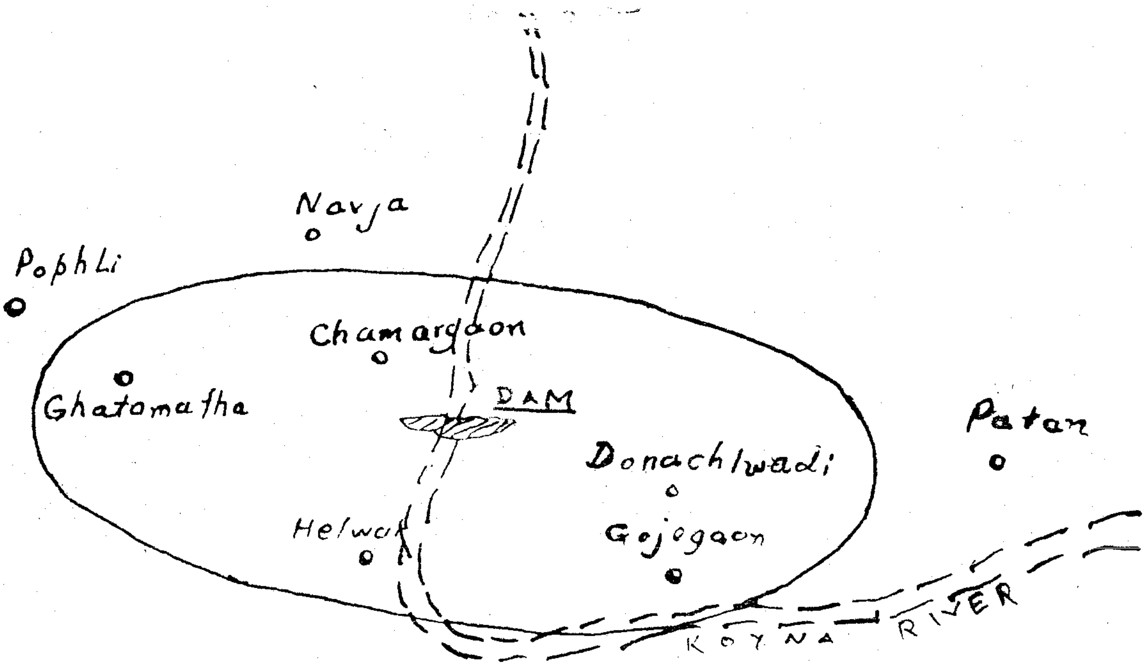


Fig. 1. Meizoseismic Area

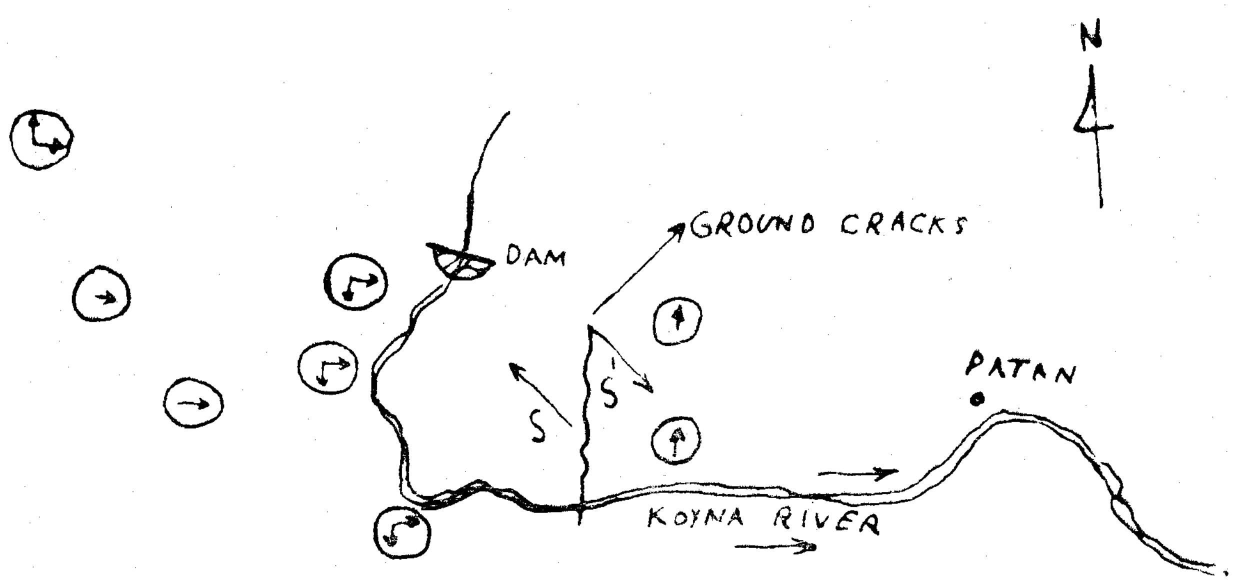


Fig. 2. Direction of Throws, Ground Cracks and Direction of Likely Tectonic Shear

Donachiwadi, Kadoli, Naneri, Lendori, and Morgiri. Navja only 4 miles upstream (north) of the Dam does not exhibit heavy damage to well-built structures. Heavy damage area extended to a distance of 6 to 8 miles east of the Dam and thus saved the Taluka capital of Patan. On the west Pophali, down the ghats, a distance of 6 miles as the crow flies has been considerably damaged though much less than Koynanagar. At Alore two miles further west well built buildings have cracked but the damage is much less.

Thus the zone of heavy damage looks to be an ellipse with major axis 20 km. in length mostly east-west and minor axis 6 km in length mostly north-south direction. Subsequent isosiesmals would be almost circular in shape touching Panchgani and Mahabaleshwar in the north. Nagthana and Karad in the east, and passing througn Sangmeshwar Taluka in the south and Chiplun Taluka on the west. Damages at Panchgani and Mahableshwar may be due to being situated on high peaks and further might have received more publicity than the damages at other less known places.

Ground Disturbance

It appears the ground disturbance is more exhibited along a more or less north south strip shown in Fig. 2 along Nanel and Donachi Wadi. This is confirmed by the observations of a team of Professors of Geology from Poona and Bombay.

Amongst the ground disturbances popularly reported are the heavy landslides and fall of heavy boulders in the area. The region being very hilly and consisting of steep mountain slopes, there is no wonder if large size landslides take place along the slopes. Such landslides might result in obstruction to roads and also involve loss of life and property. But they are not of much significance from the point of view of study of ground disturbance, for with a shock of such intensity landslides along steep slopes are inevitable.

Ground disturbances of significance are cracks across the slopes. Such cracks mostly running north-south are clearly exhibited across the hill slopes and across the Koyna river along the line indicated in Fig. 2. The ground disturbance has resulted in cracks across the road to Patan. There also appears a system of radial cracks on the road in the said region. This is indicated by a sketch in Fig. 3. The exact implication if any of radial pattern of cracks is not understood. Any way the observation in this respect is recorded here. The team of Geologists mentioned above have reported cracks in the ground in a width of 100 ft, the cracks being in a parallel system running N 15°W – S 15°E. The implication of the ground disturbance in relation to the direction is discussed later along with the throws of free objects.

There was no heavy concentration of structures in this area and there were no well built structures worth the mention. It appears this belt was shaken with a heavy intensity. The solitrary school building at Rammala Gojegaon (Gosthawadi) was a well built structure and has been razed to the ground with rubble from walls being thrown a distance of 7 to 8 ft.

Types of Damages to Buildings

Buildings in Koynanagar and Pophali are mostly ground floor structures founded on 'murum'. There are hardly any failures of foundations, this being in contrast with the observations the author had made during Anjar Earthquake (Cutch) of 1956. Buildings are in stone in cement morter for external walls and either stone or brick masonry for internal ones. Roof consists of asbestone cement sheets on timber trusses or masonry gables in case of interior walls and end walls.

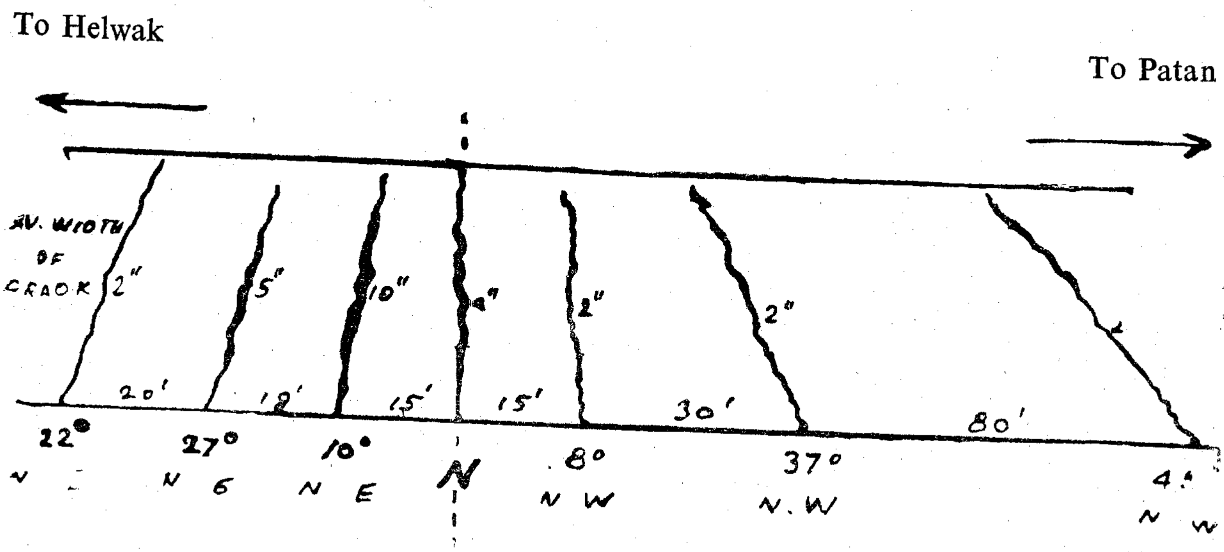


Fig. 3 : Radial Cracks on Guhagar Karad Road at Mile 55/1.

Stone masonry has been more heavily damaged than brick masonary. This may be due to the following reasons : (i) flat bedding afforded by brick masonry, (ii) less attraction of brick masonry to seismic forces because of lightness compared to stone masonry, (iii) stone masonry width of 15 inches adopted in building is often of poor workmanship. Places where timber pieces are embedded in stone masonry, as for instance purlins resting on masonry gables etc., are often weak spots where stones get loosened during shocks. It is likely that after a heavy shock timber frame work whose ends are embedded in masonry start vibrating with its natural period of vibration which is different than the natural period of the masonry wall. This sets up a differential motion between the two resulting in the loosening of stones in the masonry. The damage in such cases is due to the weakness of the joint between the timber piece and the masonry, but for which conjuction of materials of differing vibrational characteristics would have helped each other. A 15 inches wide rubble masonry wall is often weaker at the place where a timber member is embedded because rubble cannot easily be dressed to fitting shapes.

Damages to Trussed Roofs

The following practices in trussed roofs were noticed to produce failures during earthquake in the damages observed at Koynanagar aud Pophali.

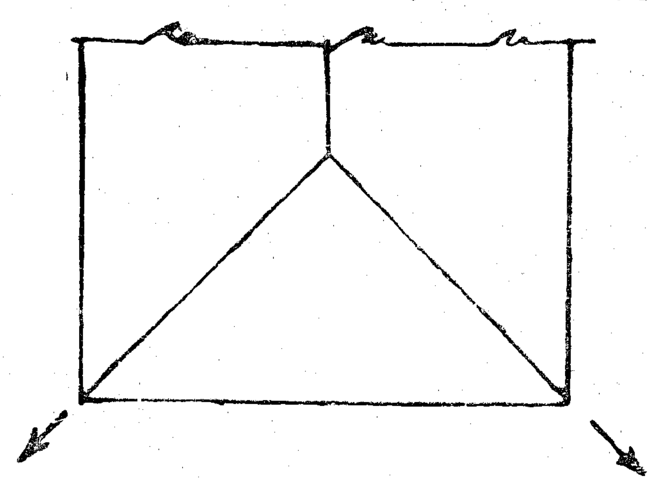


Fig. 4. Hip Rafters

(a) Hip rafters and valley rafters-Hip rafters resting on walls in an inclined position create horizontal thrust in the diagonal direction on the corner joint of two walls. Under seismic loads condition is worsened and hip rafters create a severe thrust on the corners under which the corners are very much prone to being thrown out (Fig. 4). Several cases of such failures were noticed. Valley rafters also create similar conditions.

(b) Collar beam trusses–Collar beam trusses are frequently used in buidings either to save cost or to increase the head room. In the absence of a regulr tie beam the principal rafter exert a push on the supporting walls. The situtation is worsened with horizontal seismic loads on the truss and the roof, with the result that the walls supporting the trusses are thrown out (Fig. 5).

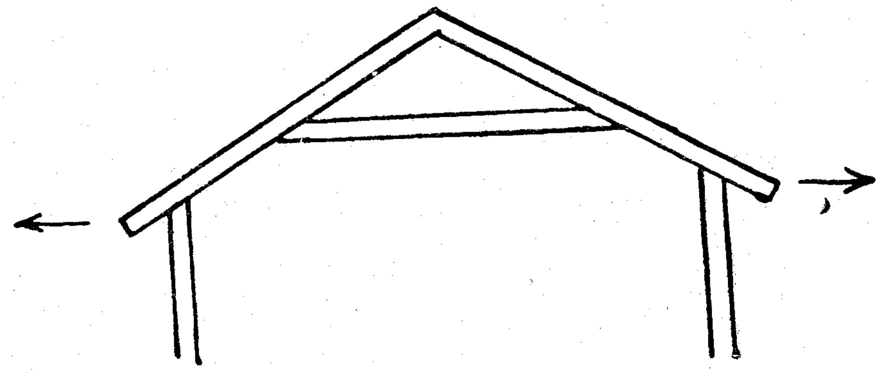


Fig. 5. Coller Beam Truss

(c) Knee braces or knee struts–Sometimes knee struts are provided to support the overhanging portion of the roof, all the more in rainy areas to protect from the weather. The struts are embedded in masonry at the lower end. While acting as a strut it causes a horizontal thrust on the wall. Under seismic loads the condition is worsened causing failure of masonry at the stem of the supports. The vibrations and the push on the roof are transferred to the strut which may either push the masonry at the root of the strut inside or pull it out. (Fig. 6). There are many failures of the latter type at Koynanagar.

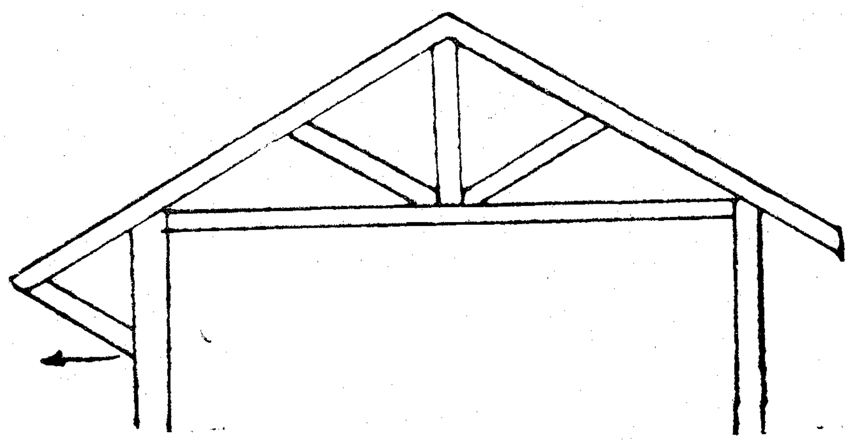


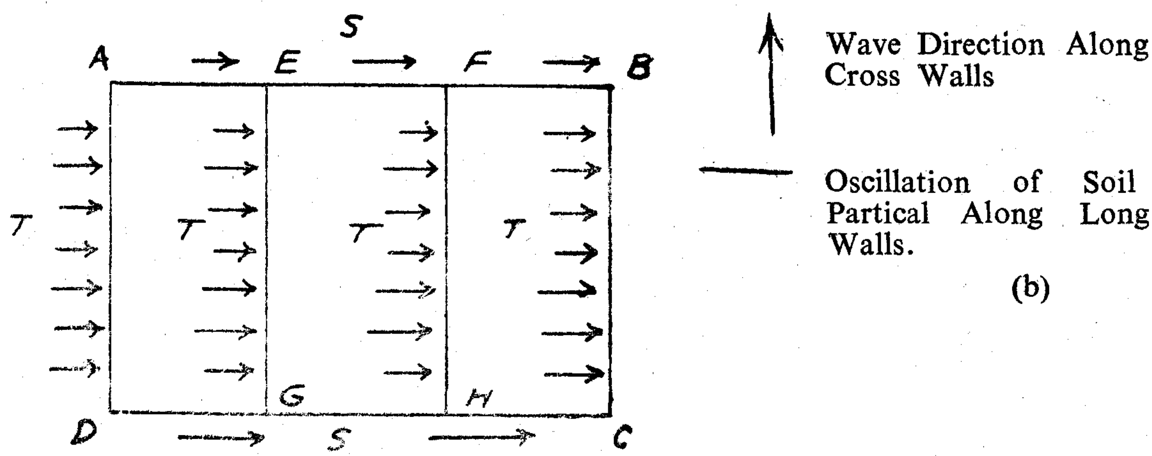
Fig. 6. Knee Brace

(d) Cracks are often observed extending from window corners upto purlins. Window corners are known to be weak spots wherefrom cracks orginate. Places where purlins are embeded in masonry gables are also weak spots as mentioned earlier. This explains the cracks extending from window corners to purlins.

Damages to Walls

Damages to buildings in earthquakes are due to the seismic waves. The actual displacement of a soil particle on the ground is quit complex, but is assumed as combination of different simple harmonic motions. The waves causing structural damage are transverse waves, so that the vibrating movement due to a particular simple harmonic motion is in a direction at right angles to the direction of propogation of the wave.

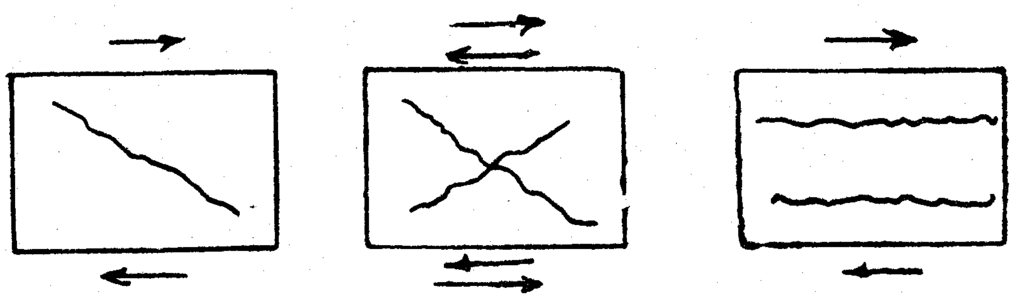
Considering the effect of a wave proceeding in the direction of the cross walls of a room, so that the ground particle describes simple harmonic motion in the direction of the long walls of the room, such wave shall produce shear forces on the long walls and side thrust on cross walls. (Fig. 7)



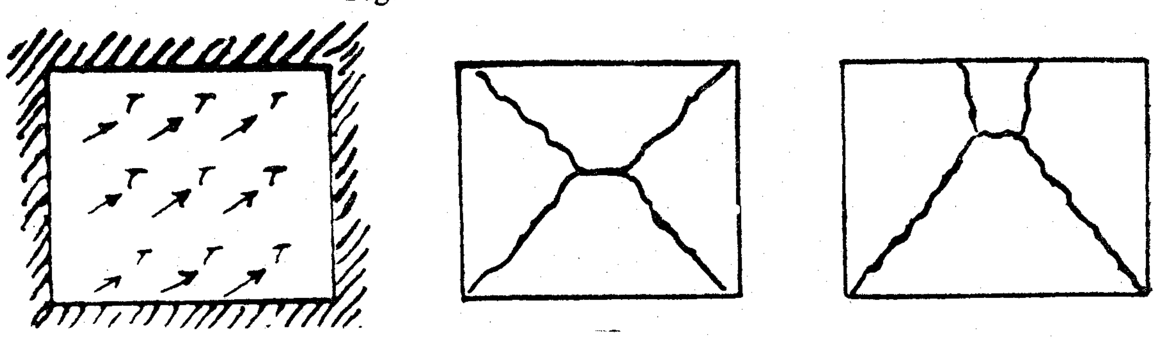
(a)
Fig. 7. Effect of Transverse Oscillation on Walls

The oscillation causes reversible forces and can very well cause damage either in the direction of first oscillation or the reverse direction. But the force of the first oscillation is likely to be very predominent and that of subsequent oscillations rather subdued. All the same there is always a probability of the structure being weakened by the first oscillation and actually failling with the reverse oscillation.

Under the action of the forces as indicated the long walls would come under shear and should produce cracks of the pattern shown in Fig. 8.



(a) (b) (c)
Fig. 8. Crack of Walls in Shear



(a) (b) with adquate support on all sides (c) in case of no top support
Fig. 9. Crack Pattern of Walls with Side Thrust

Under the action of thrust forces T on cross walls shown in Fig. 7, the wall is loaded on its face with a more or less uniform load. With such loading, walls, AB, EG and FE in Fig. 7 will be subjected to a uniform loading, the walls being supported at their edges. The bottom will draw its support from the plinth beam, the top from the slab or roof to the extent such support is available, and on the sides the support will be afforded by long walls acting as butresses (Fig. 9).

With the direction of oscillation along long walls putting them in shear and putting the cross wall in side thrust acting as in Fig. 9A, the system produces a twist in the two corners. The junction of walls because of its partial rigidity transfers the twist to some extent to the long walls. This produces a clockwise rotation in one corner and an anticlockwise rotation in the other. Such rotation is observed at several corners. Sometimes an entire object is seen to have rotated. This is due to the formation of a couple of the seismic force on the object with some point acting as pivot. The direction of rotation merely depends upon which point acts as pivot. At times the motion can be combined one of rotation and translation.

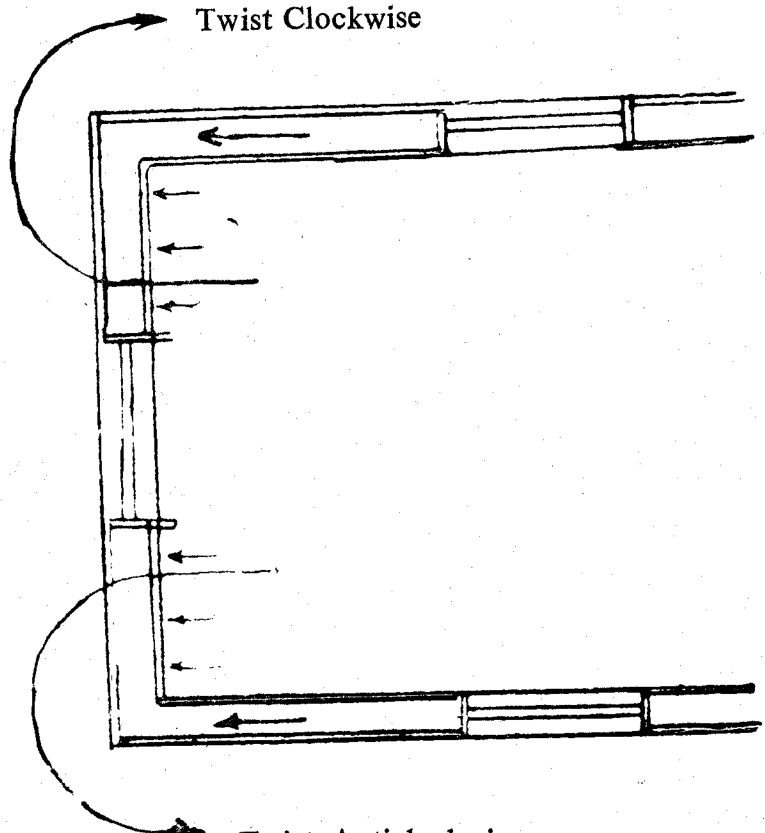
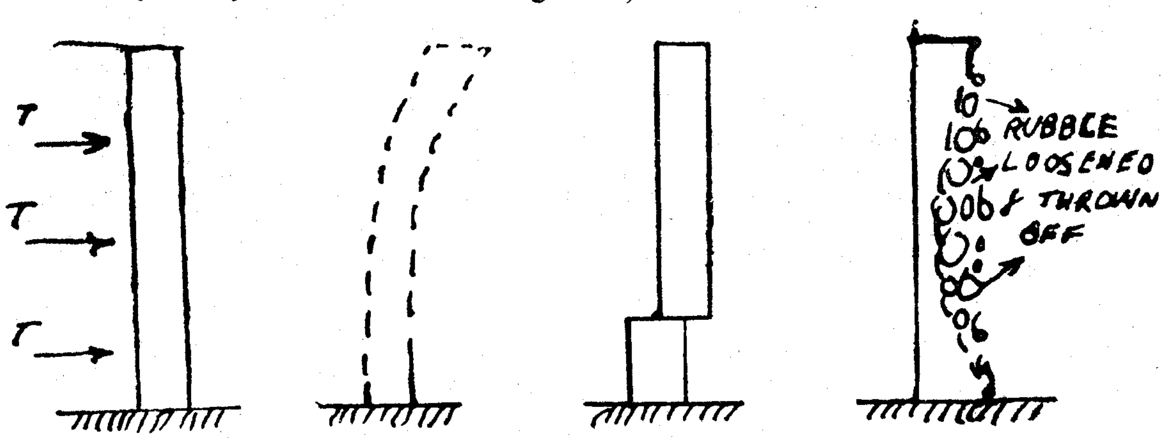


Fig. 9-A. Twisting of Corners

In the case of wall BC in Fig. 7, the side support would be from portions of long walls BF and CH, which come in tension. Under these conditions the cross walls may tear off from the long walls, may bend out in the vertical plane or may slide horizontally at any level as shown in Fig. 10 a, b and c.



(a) (b) (c) (d)

Fig. 10. Effect of Side Thrust on Fartnest External Wall

In the case of Fig. 10 c the upper portion of the wall has slided on the lower portion. There are some failures of this type.

In the case of Fig. 10 d the wall bulges out due to the side thrust T, which produces sagging in the wall with compressive stresses on the inner face of the masonry and tension on the outside face. The outside face may not be able to withstand the tension with the result that the stones would get loosened and fall down. At Koyna there are hundreds of failures of this type where stones on one face have fallen while stones on the other face standing intact. At first sight this is taken by many an onlooker as being due to bad workmanship, poor mortar and non use of bond stones. It appears that even with adequate bond stones, stones on the outside may still get loosened and fall down.

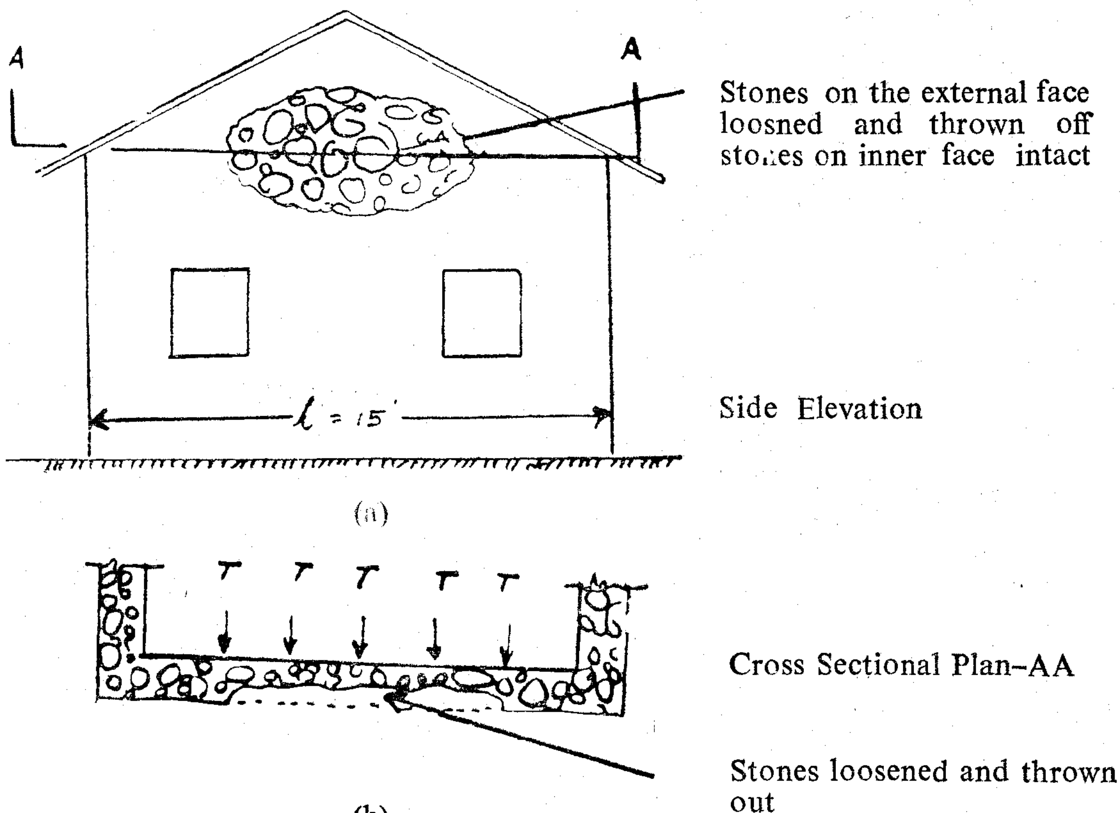


Fig. 11. Effect of Side Thrust on Gable wall

In several cases face stones in masonry gables have been thrown off (Fig. 11). At that level there is hardly any supporting reaction from the top or the bottom. The side thrust T = α. W on the wall at that level produces bending moment with 1 as span. Let the effective seismic coefficient be equal to α, Seismic thrust per sft. $= \dfrac{1.25 \times 120}{2240} \cdot \alpha$ $= .067\,\alpha$ tons/sft.

Span $1 = 15'$

$$\text{B.M. per foot width} = \frac{.067\,\alpha \times 15^2}{8} = 2\,\alpha \text{ ft. tons}$$

Section modulus of the wall per foot widh $= 1/6 \times 1 \times 1.25^2 = 1/4$ ft^3.

Assume the crushing strength of masonry at 10 tons/sft and ultimate tensile strength at 1 ton/sft,

Bending stress produced in masonry $= 2\,a/\frac{1}{4} = 8\,a$ tons/sft.

When bending stress on tension side (i.e. the side with convex flexure) reaches 1 ton/sft., stones would be loosened and thrown off.

For such condition, $8\,a = 1$.

$$\therefore \quad a = \tfrac{1}{8} = 0.125$$

Under the assumptions made the fall of stones on the face of a wall indicates a horizontal seismic acceleration of 0.125 g.

In certain situations face stones on both the faces crumple and a big hole in the masonry is the result. Such ocurance is quite common.

It appears masonry walls fail more by side thrust than by shear as shear stress produced by the seismic force seldom exceeds the ultimate shear strength of masonry.

Damage to Dam Bridge on the Spillway and Control Room

There is not the least doubt that the dam must have shaken during the earthquake because of the very heavy intensity of the shock at dam site. It is very fortunate that the dam has been saved. Some relative movement seems to have taken place in the monoliths resulting in the increase of discharge of water in the galleries. Even the increased discharge is not a source of much worry, as it is not beyond tolerable limits. It appears some horizontal cracks have appeared on the upstream face of the dam at a depth of about 120 ft from top in the deep gorge region. This looks to be the neck section where from the downstream face becomes more steep upwards. As such it is likely to be a critical section. Moreover there may be a construction joint in the dam at that level. As the dam is a gravity dam its stability ordinarily is assured. The only matter of anxiety is the possibility of yet another shock recurring in the locality and then in that case the weakened dam being erquired to face a shock of similar or heavier intensity. From this point of view it looks advisable to strengthen the dam. This can be done by vertical prestressing by anchoring in the foundations on the lines done for Vaitarna Dam. It also looks desirable to provide vertical dowel pins to prevent sliding at the weakened horizontal plane.

The bridge on the spillway has suffered some damage. The concrete in the bearings to a depth of 6″ to 8″ has been crushed so as to expose the anchor bolts. The effect is more apparent at fixed bearings. The concrete of the main longitidunal beams resting on the bearings has also been damaged. This may be due to the impact on the bearing due to the vertical component of the seismic force.

The control room on the top of the dam has been damaged as its wall of cement concrete 12″ thick centrally reinforced has broken along the length and breadth of the room about 5′ from the floor. The crack appears at the construction joint. Even then it shows that the structure must have been severely shaken.

There appear to be some cracks in the body of the dam which look to be under close study. Though such cracks do not endanger the structure ordinarily, their appearance is a matter of some anxiety in view of the uncertainty of recurrence of shocks in the locality. It is gratifying that all efforts are being made to ensure the safety.

Throws of Free Objects

Throws of free objects often gives valuable information about the ground movement during the earthquake. This point was kept in mind during the field observations.

Movement of water storage tank of about 7500 gallon capacity situated on a hill near the A-type quarters fetches some useful information. The tank is 15′ in length 10′ in width and 10′ in height. It was supported on dwarf masonry plinth walls 4 nos. $\times 16' \times 1'-6''$, the height of plinth walls being about $1'-6''$ (Fig. 12).

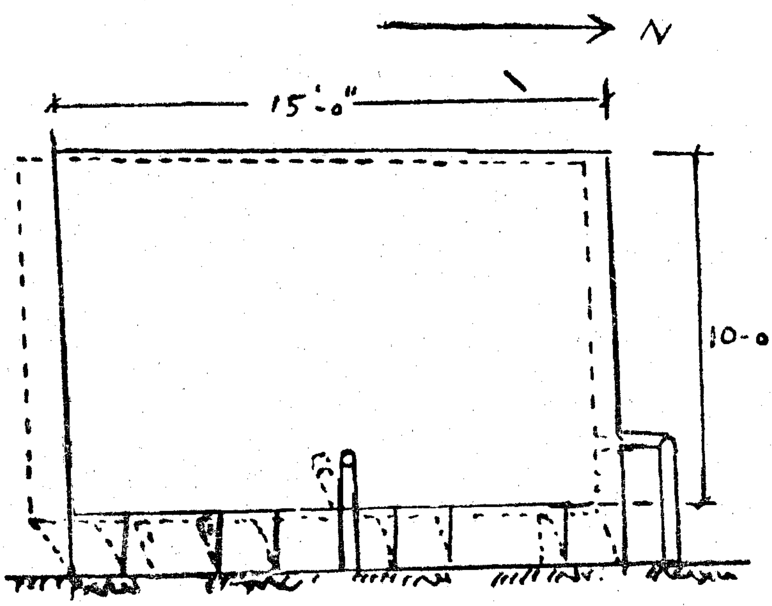


Fig. 12. Throw of Water Storage Tank

The plinth walls have been bent southwards opening up the northern portions and crushing the southern portions. The tank bottom remains attached to the southern wall but has slided on northern walls. A $2\frac{1}{2}''$ dia. G.I. pipe is connected to the tank from the front face, The pipe has been twisted during the movement. In a height of 2′, it has shifted 6″ along the S 40° W long face of the tank and 4″ along E 40° S at right angles to the face of the tank. The resultant shift of the tank is mostly due south.

Another pipe connection to the cross face shows a rotation in plan confirming the shift as above.

An estimate of the horizontal acceleration imparted to the tank during the shock is worked out as under:

Weight of water = 75,000 lb

Weight of water & tank = 80,000 lb

Horizontal acceleration = αg (due South)

Horizontal forces = 80,000 α lb = 35.7 α tons

Leverage of force on the top of masonry = 4′

Moment of horizontal seismic force on 4 masonry walls
$= 35.7\,\alpha \times 4 = 142.8\,\alpha$ ft. tons

Section modulus of 4 walls (width 1′–6″)

$$4 \times 1/6 \times 10' \times 1.5^2 = 15 \text{ ft}^3.$$

Since walls are slender in one direction they would fail in that direction due to the componant of seismic acceleration in that direction α g. cos 40° = .75 α g

Moment of effective horizontal seismic moment on 4 walls in slender direction

$$= 142.8 \times .75\, \alpha \text{ ft. tons.}$$

Bending stresses produced in masonry due to seismic moment

$$= \frac{142.8 \times 75\, \alpha}{15} = 7\, \alpha \text{ tons/sft.}$$

$$\text{Compressive direct stress on masonry} = \frac{35.7 \text{ tons}}{4 \times 10 \times 1.5} = .6 \text{ tons/sft.}$$

Assuming the crushing strength of masonry at 10 tons/sft and ultimate tensile strength of 1 tone/sft.

The net tensile stress in masonry must have reached the value of 1 ton/sft.

$$\therefore \quad 7\,\alpha - .6 = 1$$

$$\therefore \quad 7\,\alpha = 1.6\ ;\ \alpha = 0.23$$

Thus on the presumption made, the least horizontal acceleration imparted to the tank during the shock must have been 0.23 g.

It may further be mentioned that some of the seismic force must have been spent up in twisting the two $2\frac{1}{2}''$ G.I. pipe connections. But this factor would add to the minimum value of seismic acceleration estimated above.

It may further be noted that the actual ground acceleration during the earthquake is much more than the acceleration imparted to an object supported on the ground.

The net movement of the tank = $\sqrt{6^2 + 4^2} = 7.2''$ due south is indicative of the actual movement of soil particles on the surface of the ground. This inference is on the lines of the inference by Oldham regarding the movement of soil particles based on his classic study of tombs at Cherapunji in tne earthquake of 1897, in which case the movement was as much as 12″.

Study of movement of some more water tanks in Koynanagar shows movement in similar direction thought of a lesser extent.

Another study of interest was the southward sliding of a heap of testing concrete cylinders near the testing labotatory at the office building at Koynanagar. The heap consist of 14 layers, one above another resting in the interstitial concave gap between the cylinders. The cylinders were in two rows. The backward row is intact. The cylinders in the front row have slided forward (south) to the extent that the whole heap is on the point of toppling down. Here the concave gap between the lower row of cylinders forms a guide for the movement of the cylinders which can only take place either towards north or towards south. The movement is indicative of one component of the shock.

A definite southwards movement of falling objects was also noticed in Koynanagar as under :

(a) The southernmost stone slab of a tomb at Helwak bridge has been losened and has shifted southwards.

(b) The parapet stone slabs at the end of the railing of Helwak bridge, the central three spans of which have totally collapsed, shows a throw to the south.

An eastwards movement was also noticed in certain cases as under.

(c) A boundry stone buried in ground at Helwak was uprooted falling to the east.

(d) A 20′ long heavy steel rod for operation of gates of the Intake at Navja was hanging in the control room at the tower. It fell during the earthquake by shanking the supporting guides in the east-west direction. Whether the jerk was towards the west or the east cannot however be precisely inferred.

Directionwise Study of Throws and Failures

In Koynanagar throws of free objects shows a predominantly south componant as detailed above. Even as far as structures are concerned the southern ends and gables of large number of F and G type quarters oriented in the the east-west direction in front of Construction Power House have been more heavily damaged than the northern ends and gables. This is in conformity with the type of failure (Fig, 7) of wall BC with the initial direction of oscillation southwards and walls BF and CH not acting as effective buttresses. It must however be said that damages and failures of parts of structures do not yeild a firm evidence regarding direction of initial oscillations, in as much as the motion of ground particles is very complex in nature and that the possibility of a structure weakening during an oscillation and failing subsequently under the influence of another oscillation cannot be ruled out. The observation of damage being predominantly evident on the southern end walls of a large number of similarly placed structures is to be interpreted as broad generalisation. There are also shifts and damages towards east in certain cases.

Observations at Pophali at the foot of Western Ghats indicates damages to end walls and gables facing east in respect of similarly placed D and G type quarters. There also appears a large scale damage towards the north at Pophali. The situation of quarters at Pophali has to be borne in mind. The structures are mostly located on stepped ground sloping northwards joining the east-west valley of Vaitarni River. Pophali again is situated at the foot of Western Ghats at an altitude of 400′ wherefrom the ground slope is considerably reduced. Whether this situation is responsible for the direction of damage needs to be investigated.

The team of geologists from Poona and Bombay who surveyed a lot of damaged area find that the throws and damages is generally towards south in the area from Baja. Nanel and Donchi Wadi extending east (Fig. 2).

All these observations put together suggests that the tectonic shearing might have taken place in the NW–SE direction at about Donachi Wadi, with the seismic wave travelling SW therefrom with an initial oscillation of the ground in NW direction causing throws and failures in the southernly and easternly direction in the zone extending westwards, while the seismic wave might have travelled eastwards from Donachiwadi with the initial oscillation in the SE direction with the resulting throws and failures in the northernly and westernly directions (Fig. 2).

The system of shear might be in the direction NW to the west of tectonic disruption and SE to the east. Such system would induce diagonal tension in the east-west direction. It might also induce cracks in the ground in NS direction. A system of parallel cracks observed by the team of Geologists from Poona and Bombay appears somewhat in conformity with this direction.

Lessons for the Furture

The exact cause for heavy earthquake in the Deccan shield may be a matter of sharp controversy amongst the scientists. Coming to the practical aspect it looks advisable to accept the possibility of earthquake not only in Koyna region but also in Maharashtra as a whole and even the Deccan Peninsula as a fact of life, and upgrade the scismic zoning in the region. There is no reason for panic. Being a seismic zone does not put a bar on industrial or any progress of the region. It may only call for an alertness and watchfulness. People should be advised to bear in mind the basic principles of earthquake resistant design and construction. The structures should be light as far as possible so that they will not attract seismic forces and even if they do their is nothing much to fall to cause damage to life and property. As far as village houses are concerned the philosphy of such houses should be changed. Houses should be earthquake–proof houses rather than burgler-proof houses. As far as city houses are concerned a horizontal seismic force of about 5% is not likely to demand much strength of a structure than the application of wind load of say 10 lb/sft. to 20 lb/sft. Upgrading of zones will not burden the national economy to any considerable extent. If people are advised to adopt framed construction of whatever type either of bullies, timber, R.C.C. steel or prestressed-strength of such structures against any horizontal thrust moments is many times more than that of weight at bearing wall types, whereas increase in cost would be only nominial. Vibrational characteristics of such structures would be amply changed and structures would stand quakes much better. It would be very desirable to review the nation wide resources of suitable building materials which it would be advisable to use to enable earthquake resistant construction on an extensive scale. If that is done Koyna earthquake can prove to be a boon in disguise.

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EARTHQUAKE RESPONSE OF HOMOGENEOUS EARTH DAMS USING FINITE ELEMENT METHOD

S. S. Saini* and A. R. Chandrasekaran**

Synopsis

This paper presents a dynamic analysis of a homogeneous earth dam to actually recorded strong ground motion. Finite element method has been used in the analysis. Two finite element idealizations of the dam have been considered in the analysis and the response compared. The natural frequencies and modes of vibration have been computed by inverse iteration. Mode superposition method has been used to evaluate the dynamic response. The response evaluted are dynamic displacements and dynamic stresses. Static stresses have also been computed as these form a major portion of the stress distribution in earth dams. The effect of the vertical component of ground motion on the dynamic response has also been investigated.

Introduction

Earth dams usually form an important element of multipurpose projects like hydroelectric, irrigation and flood control. The earthquake behaviour of earth dams is an extremely important problem since many important dams are being built in regions of high seismicity at the present time and others will be built in future. Thus, it is essential to obtain some understanding of the response of earth dams to earthquake excitation in an effort to explain their observed behaviour and to arrive at improved methods of design.

Very few studies have been reported on the dynamic analysis of earth dams. An earth dam is a three dimensional continuous system which is highly indeterminate. To attempt the problem, it is necessary to make some simplifying assumptions regarding their behaviour. In all studies reported so far, the true three dimensional nature of the geometry has been ignored. The problem has essentially been analysed by treating the earth dam as a shear structure based on a beam type solution. The beam model is converted into a lumped mass system and analysed[1]. The analysis, in a way, assumes that the stresses do not vary along the width of the cross section of the dam. Also, with this analysis, it is not possible to take into account the effect of variation of material and material properties as is generally encountered in the core of earth dams. Such studies do not furnish adequate information for design purposes. Due to lack of such technical data, the design is essentially based on the experience gained from the past behaviour of dams during earthquakes.

Earth dams are huge structures and their dimensions are such that length to height ratio is large. Further the width of an earth dam is quite significant as compared to height. In such cases, their behaviour will predominantly be two dimensional. Finite element technique[2,3] is more versatile for such analysis and has been used here. This paper describes the application of the finite element technique for earthquake response of homogeneous earth dams. The study has been illustrated with the help of an example of an earth dam 300 feet high. Two finite element idealizations of the earth dam section have been considered and the response compared. The response evaluated are dynamic displacements and dynamic stresses. Static stresses have also been computed as these form a major part of the stress distribution in earth dams.

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To determine the effect of the vertical component of ground motion, the response have been evaluated for horizontal ground motion alone as well as for the combined effect of hoirzontal and vertical components of ground motion. The response to the vertical component of ground motion alone has not been evaluated because such a ground motion will rarely occur. The normal practice in the design of earth dams is to take the seismic coefficient in the vertical direction equal to 50% of that in the horizontal direction. So, accordingly, in one case, the vertical component of ground motion has been taken as 50% of the horizontal component of ground motion and in the other case, the actually recorded vertical component of ground motion has been considered.

This study indicates that the use of a combination of rectangular and triangular elements in the structural idealization is better because for the same order of accuracy, the number of degree of freedom is much less when a combination of rectangular and triangular elements is employed than when triangular elements alone are used. The effect of the vertical component of ground motion on the horizontal dynamic displacements and dynamic stresses is small but the effect on the vertical dynamic displacements is large.

Finite Element Method

The finite element method of analysis is a powerful structural analysis technique. The method is well known for static analysis[3] but its application to vibration problems has been made only in a few cases[4,5,6]. In this method, the continuous system is idealized by introducing finite elements thus converting it into a multiple degree freedom system. In this investigation, it has been assumed that the dam is uniformly loaded along the length so as to produce plane strain behaviour of the cross section. Further it has been assumed that the material of the dam behaves linearly elastic. On the basis of these assumptions, it is possible to calculate the stiffness properties of the dam section which define nodal force deflection relationships and can be represnted as[2].

$$\{R\} = [K] \ \{Z\} \tag{1}$$

Where {R} is the vector of nodal point forces, {Z} is the vector of nodal point displacements and [K] is the stiffness matrix. Here, each nodal point has been assumed to possess two degrees of freedom. Support conditions may be applied by eliminating the rows and columns corresponding to nodal points which impose displacement constraints.

The stresses $\{\sigma\}$ at the nodal points can be computed as[3]

$$\{\sigma\} = [S] \ \{Z\} \tag{2}$$

Where [S] is the stress transformation matrix.

In addition to this, for dynamic analysis, the mass matrix for the idealized model of the dam is needed. This may be defined in various ways including the consistant mass matrix procedure[7]. However, it is convenient if the total mass of an element is assumed to be concentrated equally at its various nodal points as this some what simplifies the analysis as it includes only diagonal terms in the mass matrix. Thus in this investigation, one-third of the mass of each triangular element and one-fourth of the mass of each rectangular element has been assumed to be lumped at nodal point.

Dynamic Analysis

Using Finite elements, the dam structure is reduced to a model with multiple degree of freedom. The equations of motion of such a system can be written using matrix notation as :

$$[M] \{\ddot{Z}\} + [C] \{\dot{Z}\} + [K] \{Z\} = \{R(t)\} \tag{3}$$

Where [M] is the mass matrix, [C] the viscous damping matrix, [K] the stiffness matrix {Z} the vector of nodal point displacements relative to base {R(t)} the load vector caused by earthquake ground motion and dots define differentiation with respect to time.

If it is assumed that the entire base section under the dam is subjected to the same ground motion at any instant of time, then the load vector {R(t)} associated with the ground acceleration caused by an earthquake can be written as

$$\{R(t)\} = -\{P_x\}\ \ddot{X}_g(t) - \{P_y\}\ \ddot{Y}_g(t) \tag{4}$$

where,

$$\{P_x\} = \begin{Bmatrix} M_1 \\ O \\ M_2 \\ O \\ M_3 \\ \vdots \\ \vdots \\ M_N \\ O \end{Bmatrix} \text{ and } \{P_y\} = \begin{Bmatrix} O \\ M_1 \\ O \\ M_2 \\ O \\ \vdots \\ \vdots \\ O \\ M_N \end{Bmatrix} \tag{5}$$

$\ddot{X}_g(t)$ and $\ddot{Y}_g(t)$ represent the horizontal and vertical components of ground motion and N is the number of nodal points used in the structural idealization less support condition Thus the equation of motion (3) can be written as :

$$[M]\ \{\ddot{Z}\} + [C]\ \{\dot{Z}\} + [K]\ \{Z\} = -\{P_x\}\ \ddot{X}_g(t) - \{P_y\}\ \ddot{Y}_g(t) \tag{6}$$

Equation (6) represents a set of second order differential equations which can be solved numerically to compute dynamic respose. However, for the problem considered here, the degree of freedom is large. Also, the contribution of the first few modes of vibration is usually significant in the dynamic response and the contribution of higher modes is small. So, in this investigation the dynamic response was evaluated using mode superposition method. It was necessary first to determine the natural frequencies and mode shapes. These can be determined from the case of free undamped vibration because for small values of damping, damped natural frequency is approximalely equal to undamped natural frequency.

Determination of Natural Frequencies and Mode Shapes:

The equation of motion for free undamped vibrations is

$$[M]\ \{\ddot{X}\} + [K]\ \{X\} = O \tag{7}$$

where {X} is the vector of absolute displacements.

The solution of Eq. 7 takes the form

$$\{X\} = a.\ \ e^{ipt}\ \{\phi\} \tag{8}$$

where a = scalar of dimension L

p = scalar of nimension T^{-1}

t = time

$$\{\phi\} = \text{a nondimensional vector} \quad \begin{Bmatrix} \phi_1 \\ \phi_2 \\ \vdots \\ \vdots \\ \phi_n \end{Bmatrix}$$

and $\quad i = \sqrt{-1}$

Substituting Eq. 8 into Eq. 7

$$-p^2 [M] \{\phi\} + [K] \{\phi\} = 0$$

or $\quad [K] \{\phi\} = p^2 [M] \{\phi\} \quad (9)$

which is a characteristic value problem, the solution of which will yield n distinct values of p^2 where $n = 2N$. For each value of p_r, there will be an associated vector $\{\phi_r\}$.

To obtain the solution of the characteristic value problem represented by Eq. 9, it should first be transformed to the form of standard eigevalue problem represented by $[A] \{Y\} = \lambda \{Y\}$. It may be recalled here that [M] is a diagonal matrix.

Let $[M] = [H]^T [H] \quad (10)$

where [H] is a diagonal matrix and the superscript 'T' indicates the transpose of matrix. Since [H] is diagonal $[H]^T = [H]$

Therefore $[M] = [H] [H] \quad (11)$

and $[H] = [M]^{1/2}$

Substituting Eq. 11 into Eq. 9

$$[K] \{\phi\} = p^2 [H] [H] \{\phi\} \quad (12)$$

Premultiply both sides by $[H]^{-1}$

$$[H]^{-1} [K] \{\phi\} = p^2 [H] \{\phi\} \quad (13)$$

or $\quad [H]^{-1} [K] [H]^{-1} [H] \{\phi\} = p^2 [H] \{\phi\} \quad (14)$

Let $\quad \{Y\} = [H] \{\phi\}$

and $\quad [A] = [H]^{-1} [K] [H]^{-1} \quad (15)$

Then Eq. (14) becomes

$$[A] \{Y\} = p^2 \{Y\} \quad (16)$$

which is of the standard form, $[A] \{Y\} = \lambda \{Y\}$.

Thus the eigenvalues obtained from Eq. (16) will be the true eigenvalues and the true eigenvectors can be obtained as

$$\{\phi\} = [H]^{-1} \{Y\} \quad (17)$$

Method of Inverse Iteration :

This is an iterative process and is based on the fact that if {Y} is an eigenvector corresponding to an eigenvalue λ for the matrix [A], then {Y} and $\mu = \frac{1}{\lambda - b}$ are the corresponding eigenvector and eignvalue for $[A - b\, I]^{-1}$.

This follows by writing the defining equation

$[A] \{Y\} = \lambda \{Y\}$ as

$$[(A) - b\,(I)] \{Y\} = (\lambda - b) \{Y\} \quad (18)$$

Then,

$$[(A) - b\,(I)]^{-1}\,\{Y\} = \frac{1}{\lambda - b}\,\{Y\} \tag{19}$$

Thus if the eigenvalues of [A] are $\lambda_1, \lambda_2, \ldots \ldots \lambda_n$ and λ_r together with $\{Y_r\}$ are desired, then any value of b sufficiently close to λ_r will make $|\lambda_r - b| < |\lambda_s - b|$ if $s \neq r$ and λ_r is not a multiple root. In the present case, [A] is a symmetric matrix and there are no multiple roots.

Therefore $\mu = \dfrac{1}{\lambda_r - b}$ is the dominant eigenvalue of $[(A)-b(I)]^{-1}\{Y\} = \mu\,\{Y\}$. Matrix iteration of Eq. (19) will yield $\{Y_r\}$ and μ. Knowing μ and b, λ_r and hence p_r can be obtained. To start the procedure, a close approximation to the fundamental frequency can be estimated by Rayleigh's method. In actual practice, it is not necessary to carry out the actual inversion process $[(A)-b\,(I)]^{-1}$ and in this study, the iterative scheme has been carried out using a special technique. The frequencies thus obtained are the true frequencies and the actual mode shapes can be obtained as

$$\{\phi_r\} = [H]^{-1}\ \{Y_r\} \tag{20}$$

These mode shapes satisfy the following orthogonality relationship.

$$\left.\begin{aligned} \{\phi_r\}^T\,[M]\ \{\phi_s\} &= 0 \\ \{\phi_r\}^T\,[K]\ \{\phi_s\} &= 0 \end{aligned}\right\} \quad r \neq s \tag{21}$$

where r and s are two different modes of vibration,

Normal Coordinates

The coupled equations of motion (6) can be reduced to a set of uncoupled normal equations using the orthogonality relationship given by Eq. 21. In order to transform the nodal coordinates Z to the normal coordinates ξ, let

$$\{Z\} = [\phi]\ \{\xi\} \tag{22}$$

where $[\phi]$ is a square matrix composed of modal vectors as columns and is given by

$$[\phi] = \begin{bmatrix} \phi_1^{(1)} & \phi_1^{(2)} & \cdots & \phi_1^{(n)} \\ \phi_2^{(1)} & \phi_2^{(2)} & \cdots & \phi_2^{(n)} \\ \vdots & \vdots & & \vdots \\ \phi_n^{(1)} & \phi_n^{(2)} & & \phi_n^{(n)} \end{bmatrix} \text{ and } \{\xi\} = \begin{Bmatrix} \xi_1(t) \\ \xi_2(t) \\ \vdots \\ \xi_n(t) \end{Bmatrix} \tag{23}$$

Substituting Eq. 22 in Eq. 6 and premultiplying throughout by $[\phi]^T$.

$$\begin{aligned} &[\phi]^T\,[M]\,[\phi]\,\{\ddot{\xi}\} + [\phi]^T\,[C]\,[\phi]\,\{\dot{\xi}\} + [\phi]^T\,[K]\,[\phi]\,\{\xi\} \\ &\quad = -\,[\phi]^T\,\{P_x\}\,\ddot{X}_g\,(t) - [\phi]^T\,\{P_y\}\,\ddot{Y}_g\,(t) \end{aligned} \tag{24}$$

Making use of orthogonality relationships, Eq. 21, it may be noted that $[\phi]^T\,[M]\,[\phi]$, and $[\phi]^T\,[K]\,[\phi]$ are diagonal matrices. If the damping is such that the same transformation which diagonalises the mass and stiffness matrices, also diagonalises the damping matrix, that is $[\phi]^T\,[C]\,[\phi]$ is a diagonal matrix, then Eq. 24 could be simplified and solved.

Using the notation

$$\begin{aligned}
\{\phi_r\}^T [M] \{\phi_r\} &= M_r^* \\
\{\phi_r\}^T [K] \{\phi_r\} &= p_r^2 M_r^* \\
\{\phi_r\}^T [C] \{\phi_r\} &= 2 p_r \zeta_r M_r^* \\
\frac{\{\phi_r\}^T \{P_x\}}{M_r^*} &= Q_x^{(r)} \\
\frac{\{\phi_r\}^T \{P_y\}}{M_r^*} &= Q_y^{(r)}
\end{aligned} \tag{25}$$

where ζ_r = Fraction of critical damping in rth mode of vibration.

Equation 24 can be written as a set of nnormal equations

$$\ddot{\xi}_r + 2 p_r \zeta_r \dot{\xi}_r + p_r^2 \xi_r = - Q_x^{(r)} \ddot{X}_g(t) - Q_y^{(r)} \ddot{Y}(t) \qquad r = 1, 2 \ldots\ldots n \tag{26}$$

Thus an n degree of freedom system represented by Eq. 6 has been reduced to n single degree freedom systems represnted by Eq. 26 using a transformation given by Eq. 22. It may be noted that either of the ground acceleration components can be considered separately or they may be combined to give the total effective force as

$$R_r^*(t) = - Q_x^{(r)} \ddot{X}_g(t) - Q_y^{(r)} \ddot{Y}_g(t) \tag{27}$$

The solution of Eq. 26 can be written as

$$\xi_r = \frac{1}{p_{dr}} \int_0^t R_r^*(\tau)\, e^{-p_r \zeta_r (t-\tau)} \operatorname{Sin} p_{dr} (t-\tau)\, d\tau \tag{28}$$

where $p_{dr} = p_r \sqrt{1-\zeta_r^2}$ = damped natural frequency.

The displacemet relative to base of ith mass in the rth mode of vibration can be written as

$$Z_i^{(r)} = \phi_i^{(r)} \frac{1}{p_{dr}} \left[\int_0^t R_r^*(\tau)\, e^{-p_r \zeta_r (t-\tau)} \operatorname{Sin} p_{dr} (t-\tau)\, d\tau \right] \tag{29}$$

or $$Z_i^{(r)} = \phi_i^{(r)} (W)_r^* \tag{30}$$

where $$(W)_r^* = \frac{1}{p_{dr}} \left[\int_0^t R_r^*(\tau)\, e^{-p_r \zeta_r (t-\tau)} \operatorname{Sin} p_{dr} (t-\tau)\, d\tau \right] \tag{31}$$

Hence, the displacements relative to base in the rth mode of vibration can be expressed as

$$\{Z\}^{(r)} = \{\phi_r\} (W)_r^* \tag{32}$$

The stresses at the nodal points in the rth mode of vibration can be computed using Eq. (2) as

$$\begin{aligned}
\{\sigma\}^{(r)} &= [S] \{Z\}^{(r)} \\
&= [S] \{\phi_r\} (W)_r^* \\
\{\sigma\}^{(r)} &= \{D_r\} (W)_r^*
\end{aligned} \tag{33}$$

where $\{D_r\} = [S] \{\phi_r\}$

The total displacement and stresses can be computed by making use of the principle of superposition as :

$$\{Z\} = \sum_{r=1}^{n} \{\phi_r\} (W)_r^* \tag{34}$$

$$\{\sigma\} = \sum_{r=1}^{n} \{D_r\} (W)_r^*$$

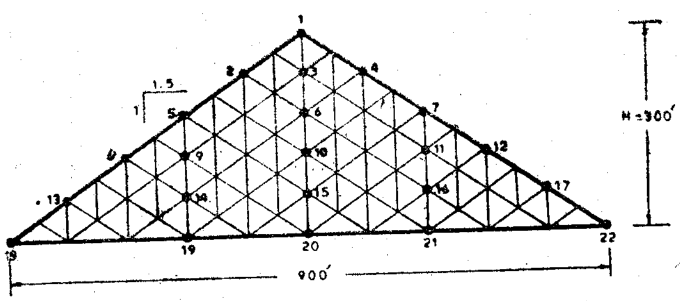
Knowing normal and shear stresses, principal stresses can be computed using well known relationships from Theory of Elasticity.

Specifications of the Example

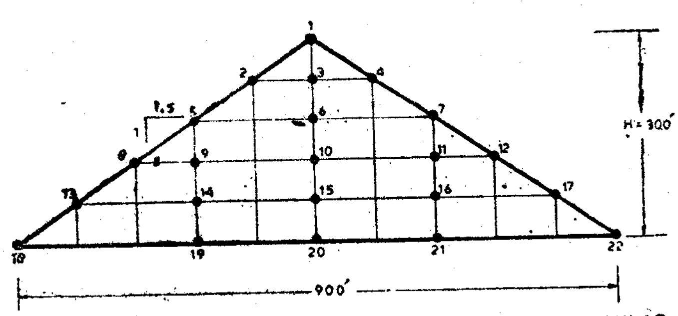
The study has been illustrated with the help of an example. A homogeneous dam of height 300 feet and symmetric triangular crossection with side slopes of 1.5 has been taken. Following data has been taken for the material of the dam section :

Moduls of Elasticity of dam material	$E = 81{,}300$ p.s.i.
Weight density of dam meterial	$\rho = 130$ lbs/cft.
Poisson's ratio of dam material	$\gamma = 0.45$

These values are associated with a shear wave propagation velocity of $V_s = 1000$ ft/sec. and a longitudianal wave propagation velocity of $V_l = 1700$ ft/sec. The section of the dam taken for the example is the same as used in some of the previous investigations[4,5,6]. In the present study, only homogeneous dam has been analysed though the analysis described in this paper is general and can very conveniently take into account the nonhomogeneity of the dam material. In this investigation, two finite element idealizations of the dam section have been considered (Fig. 1) and the response compared. In one case (Case–A), only triangular elements have been used in the structural idealization and in the other case (Case–B), a combination of rectangular and triangular elements has been used. Case A consists of a 110 degree freedom system and Case B consists of a 50 degree freedom system only.



CASE A – IDEALIZATION USING TRIANGULAR ELEMENTS ALONE



CASE B – IDEALIZATION USING COMBINATION OF RECTANGULAR AND TRIANGULAR ELEMENTS

Fig. 1. Finite Element Idealization of the Earth Dam

Discussion of Results

Natural Frequencies

For the two finite element idealizations of the dam section considered, natural frequencies of vibration and mode shapes have been determined by inverse iteration for the first fifteen modes of vibration. The natural frequency of vibration is given by :

$$p = c_p \frac{V_l}{H} \tag{35}$$

where p = undamped natural frequency in rad./sec.

c_p = frequency coefficient

V_l = longitudinal wave propagation velocity in ft/sec.

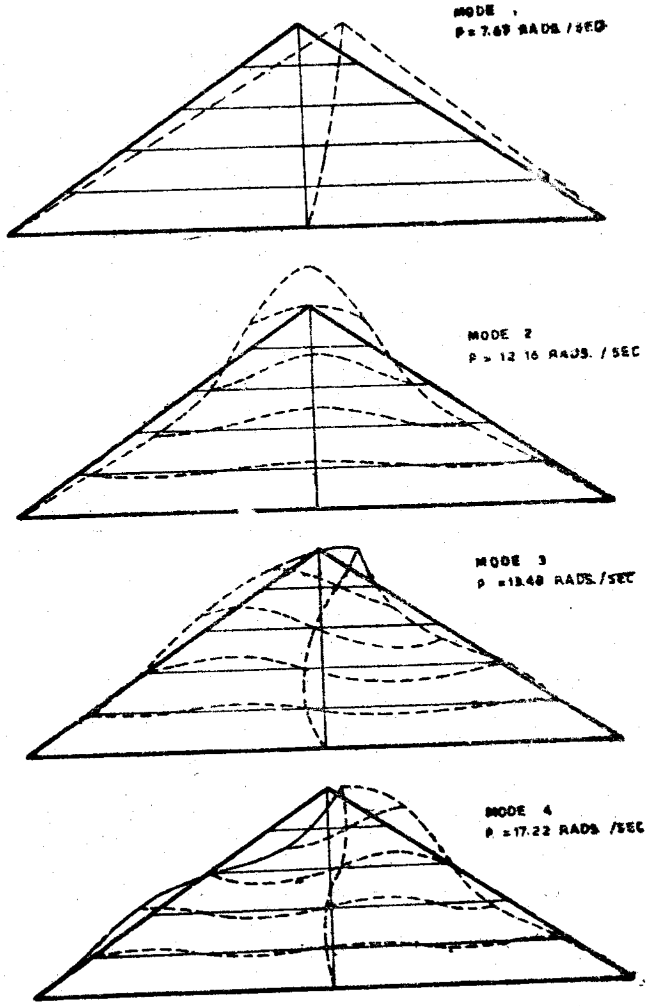
H = height of dam in feet

The value of the frequency coefficients for the two cases have been presented in Table 1. These coefficients can be used for homogeneous dams of any height H having side slopes of 1.5. Considering the case of the dam 300 feet high with the material properties given, $\frac{V_l}{H}$ works out to 5.67. Using this, the values of natural frequencies can be calculated using Eq. 35 and these values are also presented in Table 1. It may be noted from Table 1 that for Case–B, the frequencies in the various modes are somewhat lower than those for Case–A. The amount of difference is quite small in the fundamental mode but gradually increases in higher modes of vibration.

Table 1

Comparison of Frequencies

Freq.	Values of Cp		Values of p	
No.	Case–A	Case–B	Case–A	Case–B
1	1.36	1.35	7.71	7.67
2	2.21	2.14	12.51	12.16
3	2.57	2.38	14.58	13.48
4	3.40	3.04	19.29	17.22
5	3.54	3.20	20.09	18.15
6	4.07	3.58	23.07	20.31
7	4.18	3.59	23.72	20.38
8	4.57	3.96	25.92	22.45
9	4.71	4.04	26.73	22.93
10	5.07	4.13	28.73	23.44
11	5.47	4.29	31.04	24.34
12	5.53	4.50	31.33	25.54
13	5.70	4.63	32.34	26.23
14	5.93	4.68	33.60	26.51
15	6.02	4.84	34.14	27.43



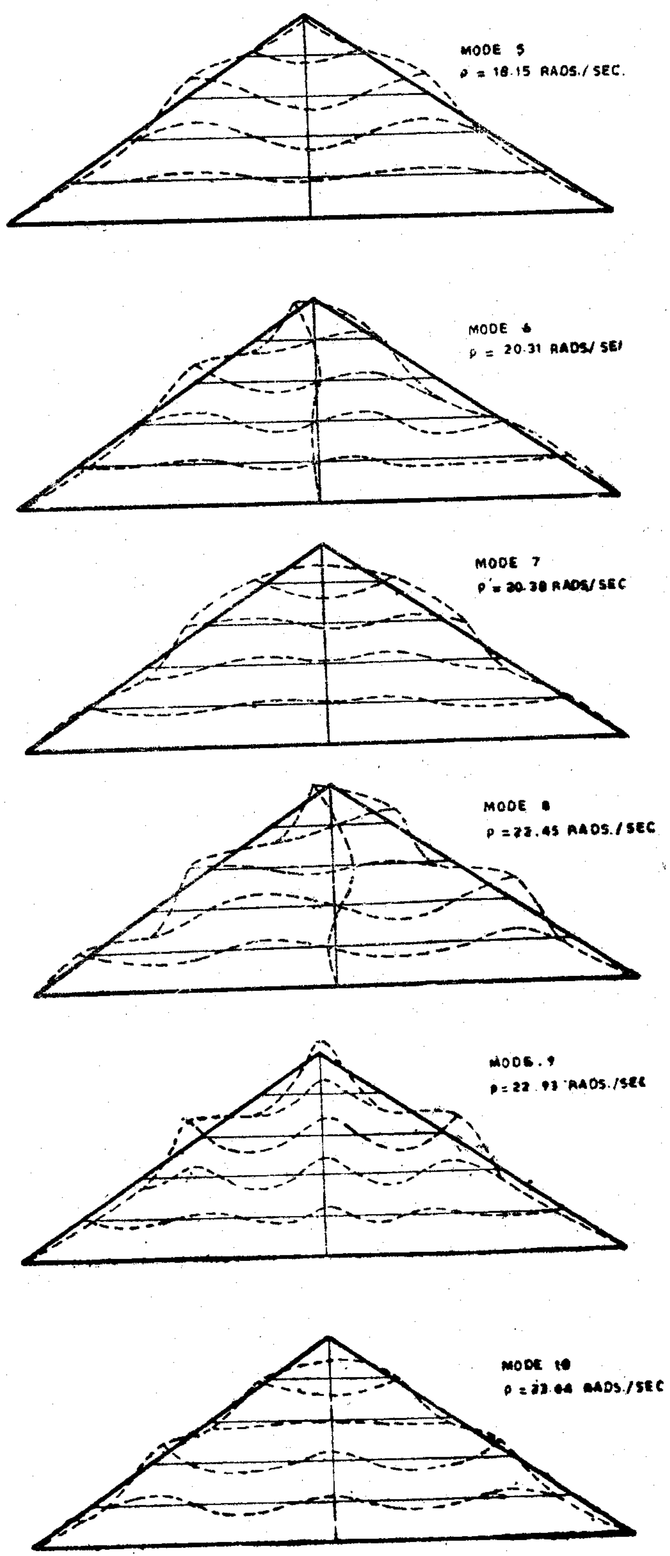


FIG 2(CONTD) - NATURAL FREQUENCIES AND MODES OF VIBRATION OF EARTH DAM (CASE-B)

The first ten modes of vibration as obtained for Case–B are shown in Fig. 2. These mode shapes resemble those for Case–A reported earlier[4,5]. It may be noted from figure 2 that for the symmetrical homogeneous dam considered, there are two basic types of vibration modes, viz. antisymmetric and symmetric which will give rise to antisymmetric and symmetric stress distributions respectively. The antisymmetric modes will be excited by horizontal ground motion only and symmetric modes will de excited by vertical ground motion only. It may be noted here that with the present analysis, the dam can have both horizontal and vertical displacements even under horizontal excitation alone unlike for the case of beam analysis where horizontal excitation will cause horizontal displacements only.

Table 2

Comparison of Mode Participation Factors

Nodal Point No.	Along the Central Axis of Dam				Nodal Point No.	Along the Slope of the Dam			
	First Mode		Second Mode			First Mode		Second Mode	
	Case–A	Case–B	Case–A	Case–B		Case–A	Case–B	Case–A	Case–B
1	1.651	1.580	0.877	0.869	1	1.651	1.580	0.877	0.869
3	1.513	1.481	0.895	0.857	2	1.477	1.466	0.595	0.564
6	1.254	1.258	0.732	0.723	5	1.142	1.151	0.078	0.006
10	0.880	0.882	0.448	0.450	8	0.718	0.725	–0.066	–0.078
15	0.440	0.441	0.158	0.159	13	0.295	0.292	–0.014	0.012

Mode Participation Factors

Mode participation factors for displacement are presented in Table 2 for the first two modes of vibration for the two cases under consideration. The comparison has been made along the central axis of the dam as well as along the slope of the dam. The displacement in any mode of vibration will be given by mode participation factor multiplied by spectral displacement in that mode of vibration. From Fig. 2, it is noted that the first mode of vibration is an antisymmetric mode and will be excited by horizontal ground motion only and the second mode is a symmetric mode and will be excited by vertical ground motion only. In Table 2, are presented, the participation factors in the first mode, for horizontal displacements due to horizontal vibrations and the participation factors in the second mode, for vertical displacements due to vertical vibrations. In the first mode, the participation factors for vertical displacements due horizontal vibrations and in the second mode, participation factors fcr horizontal displacements due to vertical vibrations are not presented as these are small for purposes of comparison. From Table 2, it is noted that the participation factors for the two cases A and B compare quite well though the number of degree of freedom in case B is much less than that in case A.

Static Stresses

Static stresses form a major portion of the stress distribution in earth dams and these have been obtained for the two cases under consideration. Principal stresses have also been computed and are give by :

$$\sigma_{s1} = C_{s_1} \cdot \rho H$$
$$\sigma_{s_2} = C_{s2} \cdot \rho H$$
$$\sigma_{s12} = C_{s12} \cdot \rho H$$

where σ_{s1} = major static principal stress
σ_{s2} = minor static principal stress
σ_{s12} = maximum static shear stress
c_{s1} = coefficient for major static principal stress
c_{12} = coefficient for minor static principal stress
c_{s12} = coefficient for maximum static shear stress.

For the example considered, ρH = 270.83 lbs/sq. in. Using this value, the principal stresses can be evaluated and these are presented in Table 3*, for the two cases. From the Table, it is noted that the stresses in the two cases are close to each other though the number of degree of freedom in Case–B is much less than that in Case–A. Thus the use of a combination of rectangular and triangular elements may be preferred in comparison to the use of tringular elements only as this results in a considerable saving of memory space and computation time. Further, it is noted form Table 3 that the static stresses for the two cases are wholly compressive. The stresses are maximum at the base and decrease towards the top of the dam. As regards the variation of stresses along the width of the dam, the stresses are maximum along the central axis and decrease towards the slopes and are minimum at the slopes.

Dynamic Response

Dynamic response has been evaluated for the two cases A and B. The contribution of the first fifteen modes of vibration has been considered. Damping has been considered to be equal to 20% of critical damping in all the modes of vibration. The digitalized ground motion data of El Centro earthquake of May 18, 1940 has been used. For the horizontal ground motion, NS component has been taken. To study the combined effect of the horizontal and vertical components of ground motion, in one case, the vertical componet has been considerd to be 50% of the horizontal component and in the other case, the actually recorded data of the vertical component has been used. Results have been obtained for the horizontal ground motion alone as well as for the combined effect of horizontal and vertical components of ground motion. The response obtained are dynamic displacements and dynamic stresses.

Dynamic Displacements

Dynamic displacements have been obtained for the two Cases A and B due to horizontal ground motion alone and due to the combined effect of horizontal and vertical components of ground motion and these are presented in Table 4. It may be noted from the table that due to the horizontal ground motion alone, the maximum horizontal displacement occurs at the top and is of the order of 3.0 inches and decreases towards the base. These are maximum near the central axis of the dam and decrease towards the slopes. The maximum vertical displacement is of the order of 0.15 inches and occurs near the top of the dam.

Further it is noted from Table 4 that the combined effect of the horizontal and vertical components of ground motion has little influence on the horizontal displacements in comparison to those obtained for the horizontal ground motion alone but the vertical displacements are increased. The maximum vertical displacement is of the order of 0.60 inches when the vertical component is taken to be 50 per cent of the horizontal component and is of the order of 0.25 inches when the actually reeorded data of the

* Comparison of static and dynamic response has been made at twenty two selected points as marked in Fig. 1. (Case A and B). In this study, compressive stresses have been marked as positive and tensile stresses as negative.

TABLE 3*

Comparison of Static Stresses

Nodal Point No.	Case–A			Case–B		
	σ_{s1}	σ_{s2}	σ_{s12}	σ_{s1}	σ_{s2}	σ_{s12}
1	13.19	14.25	.51	.30	18.06	8.91
2	3.66	33.58	14.98	6.17	29.87	11.86
3	6.50	32.74	13.14	12.76	40.08	13.65
4	3.66	33.58	14.98	6.17	29.87	11.86
5	5 17	32.39	13.62	9.10	30.90	10.91
6	14.92	81.68	33.39	10.37	84.85	37.24
7	5.17	23.39	13.62	9.10	30.90	10.91
8	11.67	42.11	15.22	14.92	40.19	12.65
9	27.76	65.84	19.04	25.92	63.02	18.55
10	46.45	129.19	41.38	41.06	131.84	45.39
11	27.76	65.84	13.04	25.92	63.02	18.55
12	11.67	42.11	15.22	14.92	40.19	12.65
13	13.03	63.16	25.05	13.97	63.56	24.78
14	66.87	125.37	29.25	65.03	125.07	30.01
15	101.78	174.52	36.35	94.01	175.88	40.92
16	66.87	125.37	29.25	65.03	125.07	30.01
17	13.03	63.16	25.05	13.97	63.56	24.78
18	8.77	52.08	21.64	0.00	56.20	28.09
19	96.77	163.61	33.42	93.44	158.38	32.47
20	152.50	201.77	24.65	157.03	195.67	19.31
21	96.77	163.61	33.42	93.44	158.38	32.47
22	8.77	52.08	21.64	0.00	56.20	28.09

* Static Stresses presented are in p.s.i.

TABLE 4*

Compariso of Dynamic Displacements Due to

Nodal Point No.	Horz. Ground Motion				Horz. and Vert. Motion Vert. Being 50% of Horz.				Horz. and Vert. Motion Actually Recorded Vert.			
	Case–A		Case–B		Case–A		Case–B		Case–A		Case–B	
	Z_x	Z_y	Z_x	Z_y	Z_x	Z_y	Z_x	Z_y	Z_x	Z_y	Z_x	Z_y
1	2.96	0.00	2.74	0.00	2.96	0.59	2.74	0.60	2.96	0.19	2.74	0.23
2	2.58	0.15	2.54	0.15	2.57	0.40	2.51	0.42	2.59	0.21	2.55	0.24
3	2.67	0.00	2.57	0.00	2.67	0.59	2.57	0.57	2.67	0.19	2.57	0.22
4	2.58	0.15	2.54	0.15	2.59	0.46	2.57	0.48	2.57	0.21	2.53	0.25
5	1.87	0.13	1.89	0.15	1.98	0.22	2.01	0.27	1.94	0.13	1.95	0.14
6	2.10	0.00	2.13	0.00	2.10	0.47	2.13	0.46	2.10	0.17	2.13	0.18
7	1.87	0.13	1.89	0.15	1.77	0.19	1.83	0.18	1.79	0.16	1.83	0.17
8	1.17	0.05	1.23	0.08	1.32	0.12	1.38	0.11	1.26	0.07	1.28	0.09
9	1.34	0.09	1.36	0.11	1.49	0.19	1.53	0.21	1.44	0.13	1.45	0.14
10	1.46	0.00	1.50	0.00	1.46	0.29	1.50	0.28	1.46	0.10	1.50	0.11
11	1.34	0.09	1.36	0.11	1.29	0.19	1.34	0.20	1.30	0.12	1.36	0.14
12	1.17	0.05	1.23	0.08	1.16	0.11	1.20	0.10	1.16	0.12	1.23	0.07
13	0.50	0.02	0.56	0.02	0.56	0.03	0.57	0.03	0.51	0.03	0.55	0.03
14	0.71	0.06	0.81	0.06	0.81	0.09	0.85	0.09	0.77	0.08	0.81	0.08
15	0.79	0.00	0.85	0.00	0.79	0.12	0.85	0.10	0.79	0.07	0.85	0.04
16	0.71	0.06	0.81	0.06	0.74	0.12	0.83	0.12	0.72	0.07	0.82	0 07
17	0.50	0.02	0.56	0 02	0.47	0.02	0.56	0.03	0.50	0.02	0.57	0.02

* z_x — Horizontal Displacement in inches.
z_y — Vertical Displacement in inches.

vertical component is used. This indicates that the vertical component when taken to be 50 per cent of the horizontal component is more intense than the actually recorded vertical component of ground motion. It may be of interest to mention that based on specrum intensities, for damping equal to 20 per cent of critical, the actually recorded vertical componet is only about 25 per cent as intense as the horizontal component.

For the two cases A and B, the displacements are close to each other.

Dynamic Stresses

Dynamic stresses are the changes in stresses over the static stress condition. These have been evaluated for the two cases A and B. Principal stresses have also been computed and are given by :

$$\begin{aligned} \sigma_{d1} &= c_{d1} \,.\, \rho \,.\, H \\ \sigma_{d2} &= c_{d2} \,.\, \rho \,.\, H \\ \sigma_{d12} &= c_{d12} \,.\, \rho \,.\, H \end{aligned} \tag{37}$$

where σ_{d1} = Major dynamic principal stress

σ_{d2} = Minor dynamic principal stress

σ_{d12} = Maximum dynamic shear stress

c_{d1} = Coefficient for major dynamic principle stress

c_{d2} = Coefficient for minor dynamic principal stress

c_{d12} = Coefficient for maximum dynamic shear stress

For the example considered, ρ H = 270.83 lbs/sq. in. Using this the principal stresses can be evaluated using Eq 37. Maximum values of the principal stresses during the history of ground motion have been obtained and these are presented in Tables 5, 6 and 7.

It may be noted from Table 5 that the maximum principal dynamic stresses do not occur at the top but occur at about 2/5 height from the top of the dam. The maximum principal stress is of the order of 54 p.s.i. These are maximum near the slopes and decrease towards the central axis of the dam. The maximum shear stress is of the order of 31 p.s.i. and occurs at the base of the dam. These are generally maximum near the central axis of the dam and decrease towards the slopes. Due to the combined effect of horizontal and vertical components of ground motion, the maximum principal stress is of the order of 58 p.s.i. and the maximum shear stress is of the order of 32 p.s.i. The general pattern of stress distribution is similar as obtained for horizontal ground motion alone. A comparison of tables 5, 6 and 7 indicates that due to the combined effect of horizontal and vertical components of ground motion, the dynamic stresses are not appreciably altered than those obtained due to horizontal ground motion alone. Further for the two cases A and B, the stresses are close to each other.

Conclusions

On the basis of this study, following conclusions can be drawn :

1. The use of a combination of rectangular elements in the structural idealization is better in comparison to the use of triangular elements alone as for the same order of accuracy, the number of degree of freedom in the former case is much less than in the latter thus resulting in considerable saving of memory space and computation time of the digital computer.

TABLE 5**

Comparison of Dynamic Stresses
Due to Horizontal Ground Motion

Nodal Point No.	Case–A			Case–B		
	σ_{d1}	σ_{d2}	σ_{d12}	σ_{d1}	σ_{d2}	σ_{d12}
1	—14.08	14.08	14.08	—11.05	11.05	11.05
2	—38.35	28.95	23.72	—31.82	24.16	19.80
3	—19.15	19.15	19.15	—17.22	17.22	17.22
4	—28.95	38.35	23.72	—24.16	31.82	19.80
5	—53.90	45.50	28.71	—51.05	41.27	27.79
6	—27.81	27.81	27.81	—27.68	26.68	27.68
7	—45.50	53.90	28.71	—41.27	51.02	27.79
8	—47.23	45.91	23.56	—50.18	46.20	25.32
9	—50.54	46.37	26.92	—49.83	43.31	28.36
10	—29.47	29.47	29.47	—30.82	30.82	30.82
11	—46.37	50.54	26.92	—43.31	49.83	28.36
12	—45.91	47.23	23.56	—46.20	50.18	25.32
13	—35.10	38.57	20.39	—35.86	39.92	22.18
14	—42.57	42.74	26.22	—38.65	40.33	26.79
15	—28.76	28.76	28.76	—28.90	28.90	28.90
16	—42.74	42.57	26.22	—40.33	38.65	26.79
17	—38.57	35.10	20.39	—39.92	35.86	22.18
18	—22.91	26.92	15.79	—24.21	30.90	21.96
19	—37.65	40.68	28.00	—35.37	39.70	31.04
20	—31.12	31.12	31.12	—32.80	32.80	32.80
21	—40.68	37.65	28.00	—39.70	35.37	31.04
22	—26.92	22.91	15.79	—30.90	24.21	21.96

**Dynamic Stresses presented are in p.s.i.

Table 6**

Comparison of Dynamic Stresses Due to Horizontal and Vertical Ground Motion

Vertical Component Being 50% of Horizontal Component

Nodal Point No.	Case-A			Case-B		
	σd_1	σd_2	σd_{12}	σd_1	σd_2	σd_{12}
1	−14.41	15.00	14.16	−12.67	10.86	11.21
2	−36.10	26.08	24.24	−32.72	24.65	21.69
3	−18.17	21.45	19.20	−18.28	18.50	17.28
4	−34.64	41.22	23.32	−24.86	32.47	18.52
5	−49.43	41.71	28.36	−48.40	39.43	27.57
6	−34.02	35.88	27.81	−25.89	31.39	27.68
7	−50.67	58.36	29.11	−44.09	53.95	28.25
8	−47.26	46.85	25.33	−46.53	46.12	24.70
9	−46.56	44.80	28.33	−43.22	41.79	28.46
10	−43.52	42.52	29.66	−36.29	37.97	31.01
11	−52.05	56.17	26.24	−51.62	57.33	28.82
12	−46.56	48.91	22.21	−49.83	54.95	27.33
13	−37.62	43.87	23.26	−40.49	44.63	24.08
14	−40.46	47.86	29.60	−38.38	47.31	30.28
15	−43.79	48.61	29.01	−38.30	43.03	29.47
16	−51.51	53.62	25.62	−45.80	47.53	26.79
17	−37.70	32.80	19.72	−38.16	33.53	21.12
18	−24.70	30.01	17.52	−30.14	31.31	22.40
19	−40.87	50.40	31.61	−38.81	48.97	32.66
20	−41.98	53.14	31.23	−38.57	47.99	32.85
21	−49.29	51.16	29.11	−41.44	44.01	31.50
22	−25.35	21.42	14.87	−32.64	22.72	21.69

**Dynamic Stresses presented are in p.s.i.

Table 7**

Comparison of Dynamic Stresses Due to Horizontal and Vertical Ground Motion

Actually Recorded Vertical Component

Nodal Point No.	Case-A			Case-B		
	σd_1	σd_2	σd_{12}	σd_1	σd_2	σd_{12}
1	−14.62	13.57	14.08	−11.59	10.51	11.05
2	−37.97	30.09	23.78	−32.39	25.43	20.45
3	−19.28	19.20	19.15	−18.23	16.25	17.22
4	−28.17	38.97	23.70	−22.89	31.25	19.15
5	−53.35	44.98	28.79	−50.86	41.08	28.03
6	−27.71	29.01	27.87	−27.52	28.09	27.81
7	−46.53	54.44	28.65	−41.49	51.21	27.57
8	−47.86	46.91	25.02	−50.51	46.18	25.76
9	−51.16	46.99	27.44	−49.51	42.90	28.87
10	−35.32	30.60	29.55	−31.23	31.63	30.87
11	−45.80	50.54	26.41	−44.17	50.73	27.84
12	−44.90	46.64	22.13	−46.20	49.91	25.02
13	−34.40	41.11	21.53	−35.26	41.36	22.18
14	−44.77	43.68	28.57	−41.03	42.33	28.33
15	−44.80	36.89	28.76	−38.08	34.42	29.03
16	−42.22	41.87	25.57	−40.00	38.92	26.89
17	−38.02	35.86	20.39	−39.84	36.43	22.18
18	−22.51	27.52	16.09	−25.05	30.44	21.75
19	−42.36	44.36	30.06	−39.54	42.71	30.79
20	−48.10	40.87	31.12	−38.13	38.05	32.80
21	−42.25	40.35	28.27	−40.79	37.73	31 28
22	−26.92	23.43	15.79	−31.63	23.67	22.18

** Dynamic stresses presented are in p.s.i.

2. Due to the combined effect of horizontal and vertical components of ground motion, the dynamic stresses are not appreciably altered than those obtained due to the horizontal ground motion alone. Also the effect on the horizontal displacments is small though the effect on the vertical displacements is significant and is more pronouced near the central axis of the dam.

Acknowledgements

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SOIL INVESTIGATIONS AND DESIGN OF A FORGING HAMMER FOUNDATION

Shamsher Prakash* and D.C. Gupta**

Synopsis

The paper describes soil investigations at the site of a forging hammer foundation proposed to be installed by M/s Forging Private Limited, Faridabad. The design of the hammer foundation is given along with the working drawings.

Introduction

Soil investigations required for the design of forging hammer foundations at any site are similar to those required for slow speed reciprocating machines but differ considerably as compared to those for conventional foundations subjected to static weight and moment only. For machine foundation design, these investigations usually comprise of determining the dynamic properties of soil at site such as co-efficient of elastic uniform compression of soil. The usual tests carried out to determine the same are the static cyclic plate load test on 30 cm square plate and dynamic vertical and horizontal tests on an R.C.C. Block 1.5 m × 0.75 m × 0.7 m.† The damping of soil being small, is usually neglected in the design of forging hammer foundations. The co-efficient of elastic uniform compression at site, so obtained, helps in estimating the maximum amplitudes of the foundation block and anvil due to the impact of the tup on the forge piece and also the maximum stress on the pad below anvil. The estimated amplitudes and maximum stress on the pad are then compared with the permissible values.

The paper describes the investigation of soil at the site of forging hammer foundation proposed to be installed by M/s Forgings Private Limited, Faridabad. The design of the hammer foundation is given along with working drawings.

Subsoil Conditions at Site

Auger boring was done to depth of 7m below ground level at the centre of the proposed site as shown in Figure 1. Soil samples were obtained at every 1.5 m depth and where there was a change of stratum. Standard Penetration Tests were carried out and N values obtained at various depths for different soil stratum. Figure 2. shows the boring log and N values observed at various depths.

The tests carried out at site to determine the co-efficient of elastic uniform compression are :

(i) Cyclic plate load test,

(ii) Vertical dynamic test, and

(iii) Horizontal dynamic test.

Cyclic Plate Load Test

Cyclic plate load test on 30 cm square plate was carried out at a depth of 2.44 m below ground level to determine the co-efficient of Elastic Uniform Compression, (Cu) at

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† Dimensions proposed in I.S. Methods of Tests for the Determination of In situ Dynamic Properties of soils

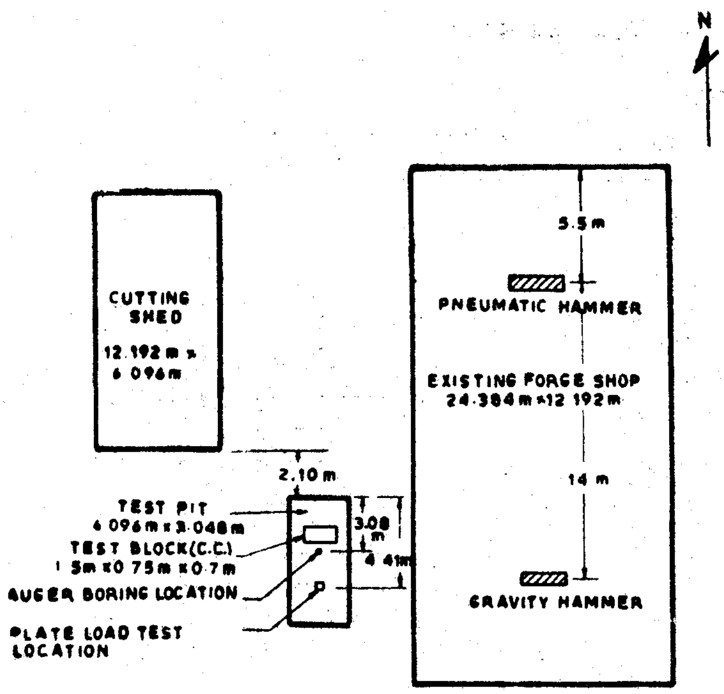


FIG 1 _LOCATION OF FIELD TESTS

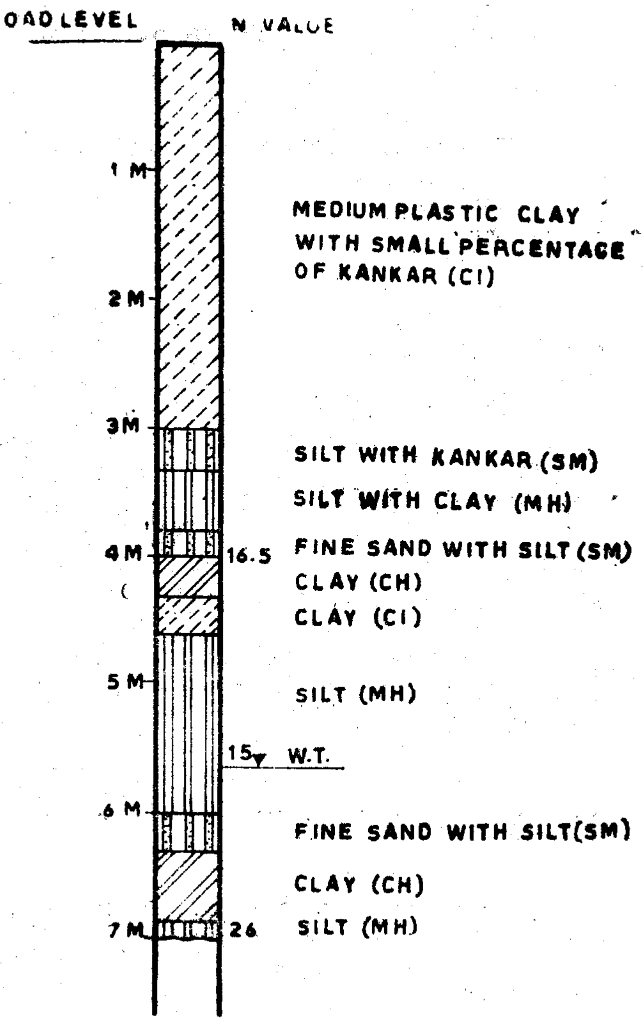


Fig. 2. Poring Log at Site

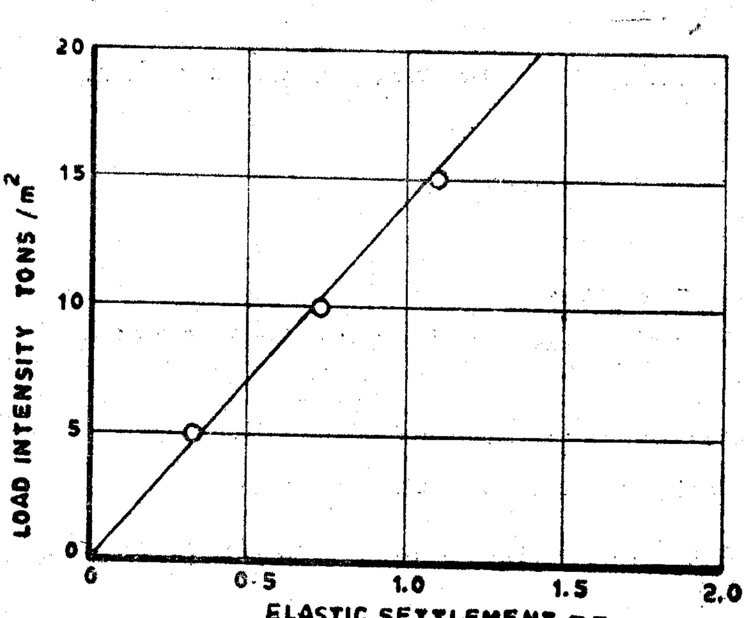


Fig 3. Load Intensity VS Elastic Settlement

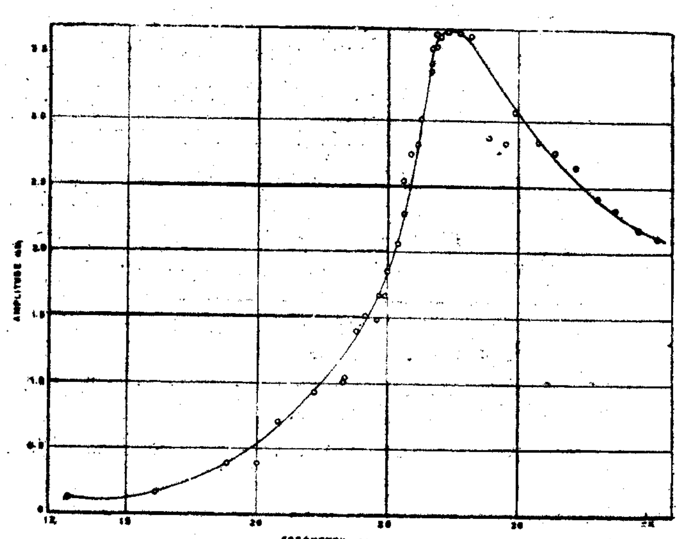


FIG 4 _ AMPLITUDE VS FREQUENCY- VERTICAL VIBRATIONS site. The test was performed at the location shown in Figure 1. Load vs elastic settlement was ploted with the help of the load settlement curve obtained from this test and is shown in Figure 3. The value of Cu, which is the slope of the line in Figure 3 is 15.16×10^3 tons/m³ for the test plate having an area of 0.0929 m².

Deducing for foundation area of 10 m²,

$$u = 15.16 \sqrt{\frac{0.0929}{10}} \times 10^3 = 1.465 \times 10^3 \ t/m^3.$$

Vertical Dynamic Test on Model Block Foundation

Model block foundation of plain cement concrete measuring 1.5 m × 0.75 m × 0.7 m high was constructed with its bottom at 2.44 m depth below the ground level. The test was carried out by mounting a Lazan type oscillator on the top surface of the block. The block was vibrated in the vertical direction. The vibration of the foundation were picked up at the top surface of the block by means of Miller accelerometer. The signal of the accelerometer was amplified by means of a "Brush Universal Amplifier" and recorded on the "Brush Direct Writing Oscillograph".

Figure 4. shows a plot of amplitude of vibration vs frequency of excitation. The natural frequency of the oscillator–foundation–soil system is seen to be 27.3 cps. Assuming soil to be mass-less elastic spring and using expressions developed by Barkan (1962), the value of Cu is computed as follows :

$$Cu = \frac{4\pi^2 f^2_{nz} m}{A}$$

$$= 5040 \ T/m^2 \text{ for } 1.125 \ m^2 \text{ base area} \qquad (1)$$

For 10 m² area,

$$Cu = 5040 \sqrt{\frac{1.125}{10}} = 1.69 \times 10^3 \ T/m^3.$$

Horizontal Dynamic Test

In this test, the concrete block was vibrated in the longitudinal direction. The vibration of the foundation were picked up at three heights of the block. The unbalanced force of the Oscillator at a height of 50 cm above the c.g. of the block caused the footing to translate in the longitudinal direction as well as to have pitching about the transverse axis of the block. The system, thus, constituted a two degress of freedom system.

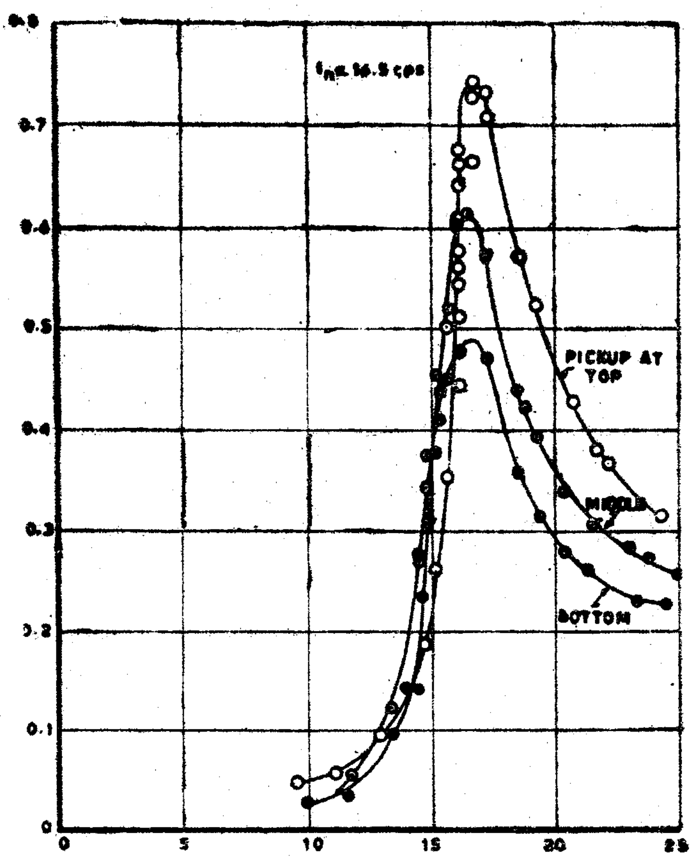
Figure 5. shows the records of amplitude of vibration vs frequency of excitation at top, mid height and bottom of the block. The first natural frequency is observed to be 16.5 cps. Assuming soil to be mass-less elastic spring and using expressions developed by Barkan (1962) the values of C_τ and C_u of soil have been computed as follows :

Computations of C_τ and C_u

Moment of Inertia of the Foundation Block :

The moment of inertia I of the foundation contact area with respect to the axis passing through its centre of gravity perpendicular to the plane of vibration is

$$I = \frac{0.75 \times (1.5)^3}{12} = 0.211 \ m^4$$



Frequncy cps

Fig. 5. Amplitude vs Frequency of Vibrations

Mass M.I. about an axis passing through C.G. of the System :

The moment of inertia of the foundation block with respect to the axis passing through the centre of gravity of the whole system perpendicular to the plane of vibration is

$$M_m = \frac{m}{12}(a_y^2 + a_z^2) = \frac{1.77}{9.81 \times 12}(2.25 + 0.49)$$

$$= 0.0412 \text{ tons} \times \text{m} \times \text{sec}^2$$

where a_y = length of the block = 1.5 m

and a_z = height of the block = 0.7 m

Mass M.I. about an axis passing through the C.G. of the base area :

$$M_{mo} = M_m + m\left(\frac{a_z}{2}\right)^2 = 0.0412 + \frac{1.77}{9.81} \times 0.1225$$

$$= 0.0633 \text{ tons} \times \text{m} \times \text{sec}^2$$

The ratio between the two mass moments of inertia is

$$\gamma = \frac{0.0412}{0.0633} = 0.65$$

Assuming the following relationship between coefficients of Elastic non-Uniform Compression (C_ϕ) and Uniform Shear (C_τ)

$$C_\phi = 3.46\ C_\tau$$

The following expression is obtained for the natural frequencies (Prakash and Gupta, 1967) ;

$$\omega^2_{n1,2} = \frac{1}{2\gamma}\left[\left(\frac{A}{m} + \frac{3.46I}{M_{mo}}\right) \pm \sqrt{\left(\frac{A}{m} + \frac{3.46I}{M_{mo}}\right)^2 - \frac{13.84\gamma AI}{m\ M_{mo}}}\right] \times C_\tau \qquad (2)$$

Substituting the values of A, m, I, M_{mo} and γ for the block under investigation, Equation (2) reduces to,

$$\omega^2_{n1,2} = 4.96\ C_\tau \text{ and } 22.35\ C_\tau\ \text{sec}^{-2}$$

f_{n_1} has been measured for the foundation block as 16.5 cps. C_τ can be computed from the relationship :

$$C_\tau = \frac{\omega_{n_1}{}^2}{4.96}\ \text{t/m}^3$$

$$= \frac{(16.5 \times 2\pi)^2}{4.96} = 2.17 \times 10^3\ \text{t/m}^3.$$

The value of C_u for the soil is obtained by the empirical relation :

$$C_u = 2\ C_\tau$$

$$= 2 \times 2.17 \times 10^3\ \text{t/m}^3 \text{ for } 1.125\ \text{m}^2$$

$$= 4.34 \times 10^3\ \text{t/m}^3$$

and C_u for 10 m^2 foundation area

$$= 4.34\sqrt{\frac{1.125}{10}} \times 10^3\ \text{t/m}^3$$

$$C_u = 1.455 \times 10^3\ \text{t/m}^3$$

A comparison of the values of C_u obtained from cyclic plate load test, vertical dynamic test and Longitudinal dynamic test has been made in Table 1.

The minimum value of C_u equal to 1.455×10^3 t/m³ for an area of 10 m² has been taken for design purpose.

Table 1

Value of C_u From Various Tests

Type of Test	C_u
Cyclic Plate Load Test	1.465×10^3 t/m³
Vertical Dynamic Test	1.69×10^3 t/m³
Longitudinal Dynamic Test	1.455×10^3 t/m³

Design Data

The hammer has the following specifications :

Tup weight without die	:	1150 kg
Maximum tup stroke, h	:	900 mm
Supply steam pressure, p	:	100 psi = 70 t/m^2
Anvil block weight	:	22.5 tons
Total weight of hammer	:	34.3 tons
Bearing area of anvil	:	$2.1 \times 1.3 = 2.73$ m^2

Soil at site consists of silty clay of medium plasticity mixed with Kankar and intermitant layers of fine silty sand of medium relative density. The water table is at a depth of about 5.65 m below ground level. The coefficient of elastic uniform compression of soil, C_u is equal to 1.455×10^3 t/m^3 for an area of 10 m^2 as determined by tests at site.

Data Assumed

For design purposes, the following data was assumed :

1. Material of pad below anvil—Teak heart wood
2. Modulus of Elasticity of pad—5×10^4 t/m^2
3. Thickness of pad below anvil—0.61 m
4. Dimensions of the foundation blok—6.50 m $\times$ 5.70 m $\times$ 1.75 m
5. Dimensions of R. C. C. walls—0.50×1.34 m all around anvil as shown in Figure 6.

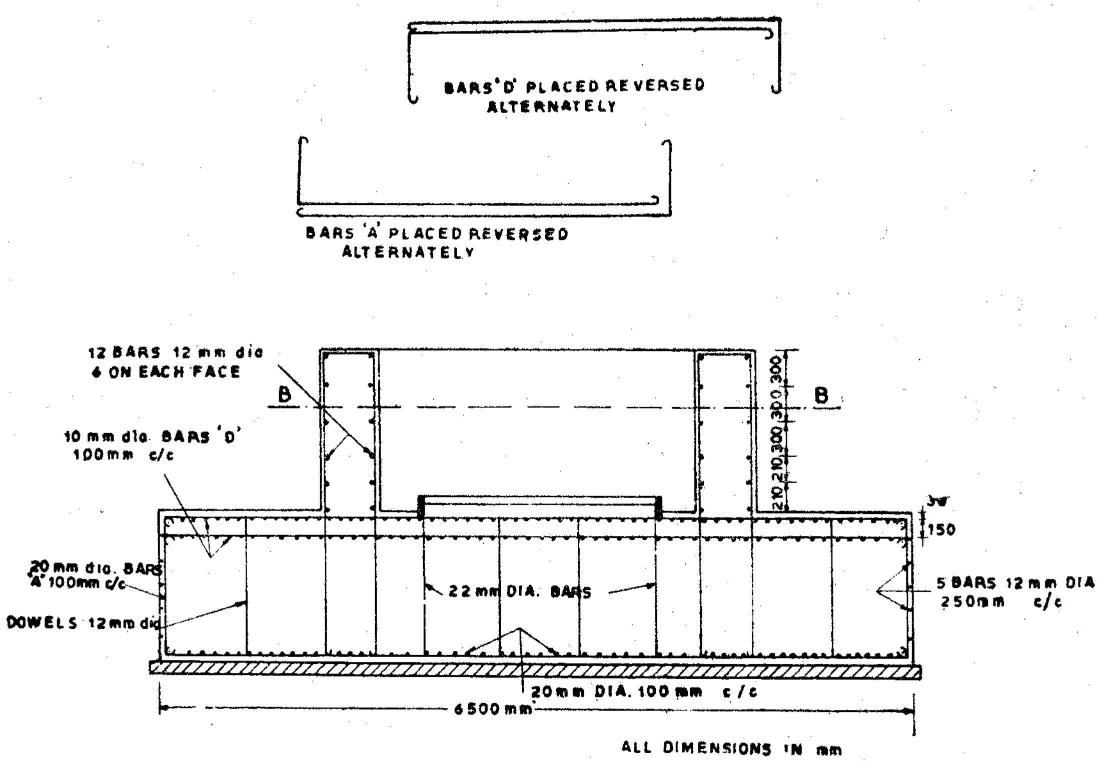


Fig. 6. Section of Hammer Foundation

6. Unit weight of R. C. C. $= 2.4$ t/m^3
7. Unit weight of backfill $= 1.76$ t/m^3
8. Modified coefficient of Elastic Uniform Compression for Impact loading $C_u' = 3. C_u$.
9. Coefficient of restitution, $e = 0.5$
10. Coefficient which takes into account counter-pressure and frictional forces, $\eta = 0.65$

Requirements for Design

The following are the main requirements for satisfactory design of the foundation :

(a) The amplitudes of vibration of the foundation and anvil should be within allowable limits.

(b) The dynamic stress on the soil and pad should be within permissible limits.

According to Barkan (1962) and Indian Standard code af Practice 2974–1966, the following are the permissible limits for amplitudes of vibration :

$A_{foundation}$ = 1.0 to 1.2 mm

A_{anvil} = 1.0 mm for 1 T hammer and 2.0 mm for 2 T hammer

According to Indian Standard code IS : 883–1961 the allowable limit for stress on Teak heart wood loaded perpendicular to grains is as follows : σ allowable $= 400$ t/m^2 in compression.

Computations

Figures 6 and 7 show the assumed dimension of the foundation.

Foundation area in contact with soil $= 6.50 \times 5.70 = 37.05$ m^2

Weight of foundation and backfill :

Volume of the Block $= 6.50 \times 5.70 \times 1.75 = 64.8375$ Cu m

Walls $= 2 \times 3.70 \times 0.50 \times 1.34 = 4.958$ Cu m

$2 \times 1.90 \times 0.50 \times 1.34 = 2.546$ Cu m

72.3415 Cu m

Weight of Concrete $= 72.3415 \times 2.40 = 173.62$ tons.

Volume of the fill $= 2 \times 6.50 \times 1.40 \times 1.34 = 24.40$

$2 \times 2.90 \times 1.40 \times 1.34 = 10.80$

35.29 Cu m

Weight of back-fill $= 35.29 \times 1.76 = 62.20$ tons

Total weight of foundation and backfill $= 235.82$ tons

$$\text{Total mass, m} = \frac{235.82}{9.81} = 24.0 \text{ t sec}^2/\text{m}$$

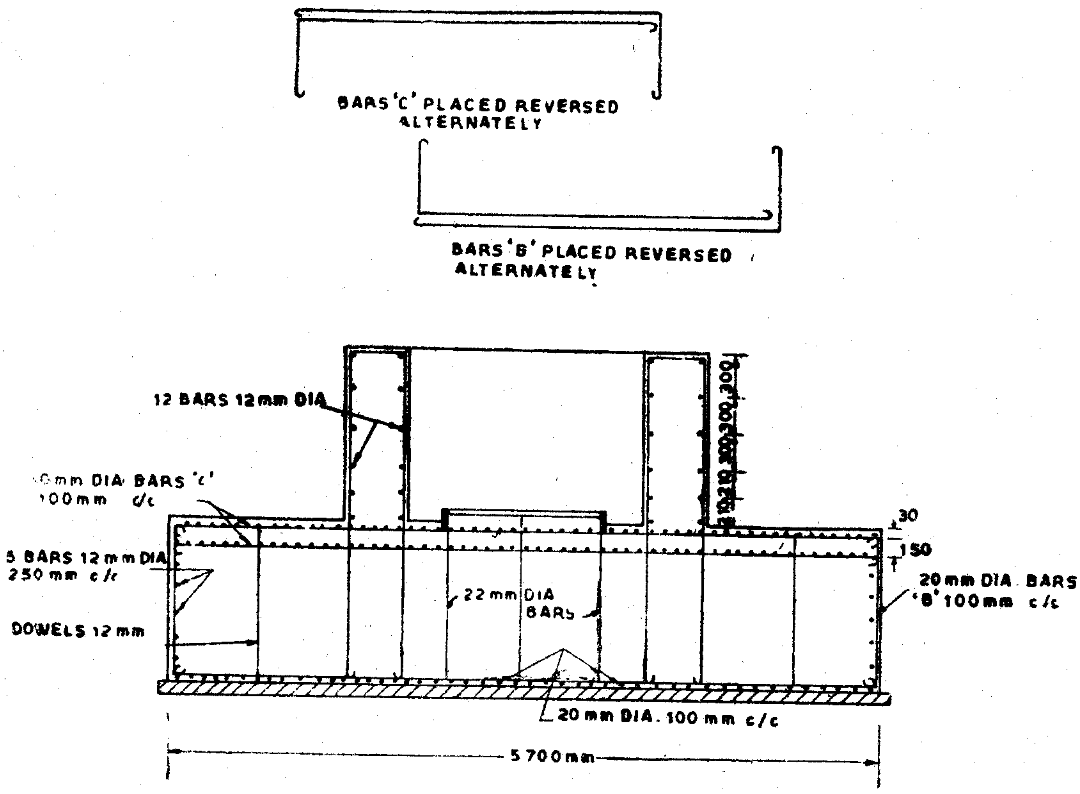


Fig. 7. Side View of Hammer Foundation

Natural Frequencies of Foundation—Hammer System

The modulus of elasticity of the pad, $E_2 = 5 \times 10^4$ t/m²

Thickness of the pad, b = 0.61 m

Coefficient of rigidity of the pad, $k_2 = \frac{E_2 A_2}{t_2} = \frac{5 \times 10^4 \times 2.73}{0.61}$

$= 22.4 \times 10^4$ t/m.

The mass of the anvil and frame, $m_2 = \frac{34.3}{9.81} = 3.5$ t sec²/m

The limiting natural frequency of anvil on pad is

$$\omega^2_{na} = \frac{k_2}{m_2} = \frac{22.4 \times 10^4}{3.5} = 6.4 \times 10^4 \text{ sec}^{-2}$$

now, $Cu = 1.455 \times 10^3$ t/m³ for an area of 10 m²

According to Barkan, if Cu is the coefficient of elastic uniform compression for an area A, then the coefficient of elastic uniform compression Cu_1 for an area A_1 is given by

$$Cu_1 = Cu\sqrt{\frac{A}{A_1}}$$

Cu_1 for an area of 37.05 m^2 is given by

$$Cu_1 = 1.455\times10^3 \sqrt{\frac{10.0}{37.05}} = 0.755 \times 10^3 \ t/m^3$$

$$Cu' = 3.Cu_1 = 3 \times 0.755 \times 10^3 = 2.265 \times 10^3 \ t/m^3$$

Coefficient of rigidity of the soil, $k_1 = Cu' \times A_1 = 2.265 \times 10^3 \times 37.05$
$= 8.4 \times 10^4$ t/m

The limiting natural frequency of the whole system.

$$\omega_l^2 = \frac{k_1}{m_1+m_2} = \frac{8.4 \times 10^4}{3.5 + 24.0} = 0.305\times10^4 \ sec^{-2} \text{ and}$$

$$\mu = \frac{m_2}{m_1} = \frac{3.5}{24.0} = 0.1458$$

The two natural frequencies of the combined system are given by

$$\omega^4_n - (\omega_l^2 + \omega^2_{na})(1 + \mu)\,\omega^2_n + (1 + \mu)\,\omega_l^2\,\omega^2_{na} = 0$$

$$\omega^4_n - (0.305 + 6.4)\,1.1458 \times 10^4\omega^2_n + 1.1458 \times 0.305 \times 6.4 \times 10^8 = 0$$

$$\omega^4_n - 7.69 \times 10^4\omega^2_n + 2.24 \times 10^8 = 0$$

$$\omega^2_n = \tfrac{1}{2}\,[7.69 \pm \sqrt{(7.69)^2 - 4 \times 2.24}] \times 10^4$$

$$= \tfrac{1}{2}\,[7.69 \pm \sqrt{59 - 8.96}] \times 10^4$$

$$= \tfrac{1}{2}\,[7.69 \pm 7.10] \times 10^4$$

If $\omega_{n1} > \omega_{n2}$

$$\omega^2_{n1} = \tfrac{1}{2} \times 14.79 \times 10^4 = 7.395 \times 10^4 \ sec^{-2}$$

$$\omega^2_{n2} = \tfrac{1}{2} \times 0.59 \times 10^4 = 0.295 \times 10^4 \ sec^{-2}$$

$$\omega_{n2} = 54.3 \ sec^{-1}$$

Velocity of Dropping Parts at the beginning of impact

$$V = \eta \sqrt{\frac{2\,g\,(W + A_c\,.\,p)\,h}{W}}$$

where A_c = Area of Piston

$$\therefore \quad V = 0.65 \sqrt{\frac{2\times9.81\times(1.55+0.129\times70\times0.9)}{1.55}} = 7.13 \text{ m/scs}$$

Initial velocity of Anvil Motion

$$Va = \frac{1 + e}{1 + \mu_a}\,V = \frac{1 + 0.5}{1 + \frac{34.2}{1.55}} \times 7.13 = 0.463 \text{ m/sec}$$

Amplitude of Vibration of Foundation

$$A_z = \frac{(\omega^2_{na} - \omega^2_{n2})\times(\omega^2_{na} - \omega^2_{n1})}{\omega^2_{na}\,(\omega^2_{n1} - \omega^2_{n2})\,\omega_{n2}}\,V_a = 1.14 \text{ mm}$$

and that of anvil

$$A_a = \frac{(\omega^2_{na} - \omega^2_{n1})\times V_a}{(\omega^2_{n1} - \omega^2_{n2})\times\omega_{n2}} = 1.195 \text{ mm}$$

These are within permissible limits.

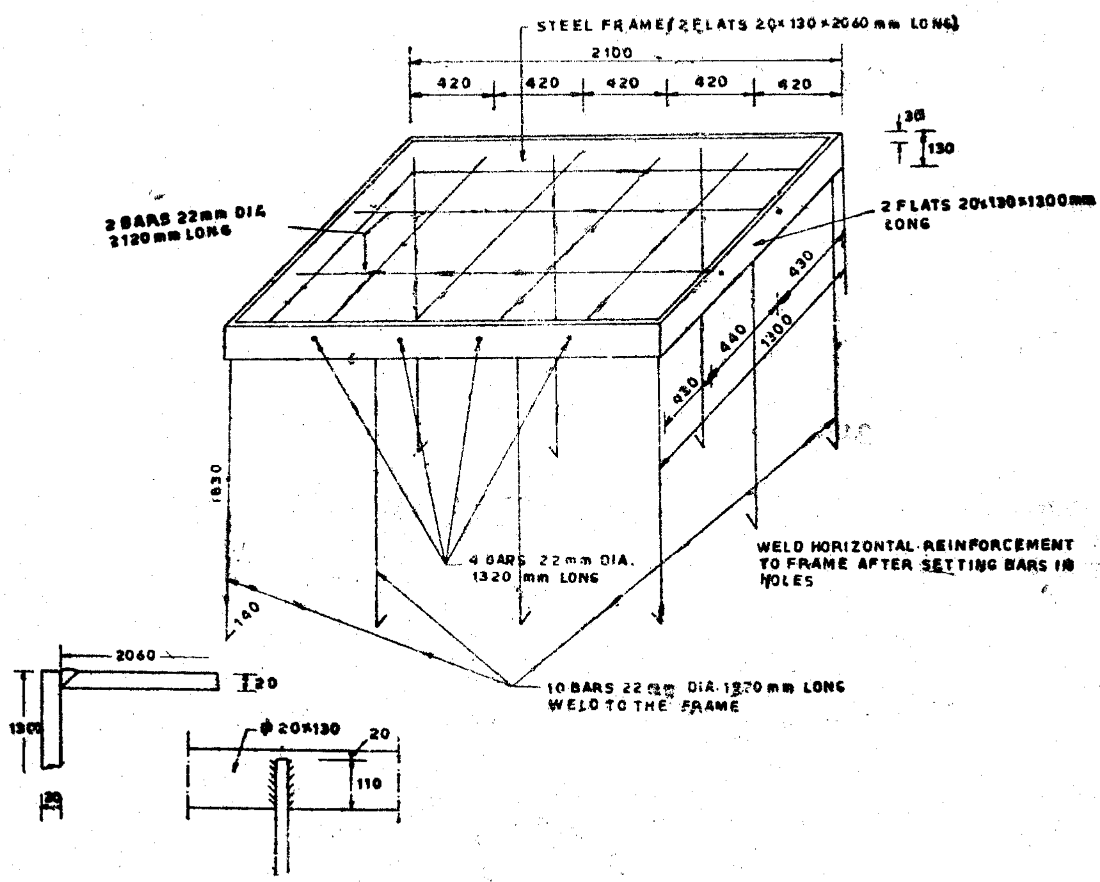


Fig. 8. Details of Reinforcement Under the Anvil

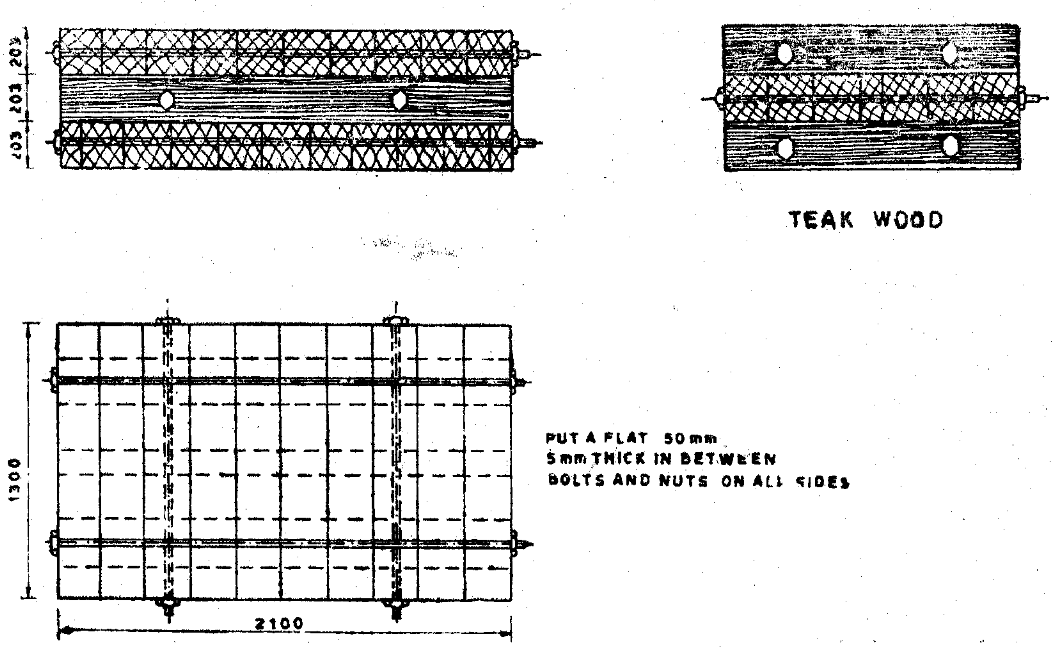


Fig. 9. Details of Pad

Dynamic Stress in pad

$$= \frac{k_2 (A_a - A_z)}{A_2} = \frac{22.4 \times 10^4 \, (1\text{-}14 + 1.195) \, 10^{-3}}{2.73} = 192 \text{ t/m}^2$$

This is also within permissible limits.

The details of various dimensions and reinforcement are shown in Figures 6,7 and 8.

Recommendations for the Material of the Foundation and Pad Under the Anvil

Foundation block under the anvil snould be made of concrete type M 150 with coarse aggregate of hard rocks with a compressive strength of not less than 250 kg/cm^2. Normal portland cement should be used for concrete. The latter should be reinforced.

The pad under the anvil should be made of Teak heart wood. Timber of best quality, having a moisture content below 15 to 18 per cent should be used.

Remarks on Construction Proceshre

It should be ensured that no water penetrates the soil after the excavation of the pit to the required level. The soil should be properly compacted before the lean concrete is laid over it.

While installing the anvil, it should be ensured that the centre of gravities of the anvil and the foundation block coincide with the line of fall of the hammer tup and the impact of forging always remains central.

Pads under the anvil should be made of square timbers 20.3 cm by 20.3 cm in cross section. Timber are to be laid flat in 3 rows one over the other, such that it is loaded perpendicular to grains only. The upper row of timbers is to be laid along the short side of the anvil base. The 3 layers of timber should be glued together. Each row is braced by transverse bolts as shown in Figure 9. To prevent decay resulting from moisture, it is advisable to impregnate timbers with wood preserving solutions.

The mats must be strictly horizontal and smoothly planed. Levels of the excavated pit and the mats should be checked by means of a sensitive water level.

The space between the pad and side walls may be filled with asphalt. In order to prevent the horizontal displacement of the anvil along the pad, four timbers are placed around it near the base, properly wedged.

A trench 3.5 m deep below floor level and minimum of 30 cm wide all around the foundation at a distance of about 1 m from the edges of the foundation base may be filled with saw dust mixed with bitumen or similar shock absorbing material. This is required primarily to cut down the noise level away from the foundation.

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Notation

The following are the notations used in computing the amplitudes of vibration of the anvil and foundation block :

W_0 = Weight of dropping parts
h = Weight of drop
W_2 = Weight of hammer-Anvil
A_2 = Base area of anvil
b = Thickness of pad under anvil
p = Steam pressure
A_c = Cylinder area
C_u = Coeff. of elastic uniform compression
V = Veolcity of dropping parts at the beginning of impact
V_a = Initial velocity of motion of the anvil
W_1 = Weight of foundation and backfill
A = Foundation area in contact with the soil
E_2 = Modulus of elasticity of pad under anvil
k_2 = Coefficient of rigidity of pad
m_2 = Mass of hammer plus anvil
ωna = Limiting frequency of natural vibration of the anvil + hammer
k_1 = Coefficient of rigidity of base under foundation
m_1 = Mass of foundation plus bacafill
ω_l = Limiting natural frequency of the whole system
μ = m_2/m_1
$\omega_{n1,2}$ = Natural frequencies of vibrations of the foundation — hammer system
A_z&A_a= Amplitude of vibration of foundation and anvil respectively

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RECENT EARTHQUAKE SERIES IN KOYNA REGION—A POSSIBLE MECHANISM AT THE SOURCE*

T. K. Dutta**

Abstract

The occurrence of the recent earthquake series, with a peak event of magnitude around 6 on Richter scale, in Koyna area poses a number of problems. It is difficult to relate this occurrence to measurable movement along fault of known activity in recent times. Besides the energy associated with this earthquake also preclude the possibilities of surface structural or load changes as a causative factor.

In this note an attempt has been made to build up a possible source mechanism for this event. On the basis of geological evidence regarding intense volcanic activities in the past a thermodynamically possible process, capable of releasing neccessary energy, is put forward by applying the principle of energy transfer during permissible phase transition in crust and mantle. This hypothesis can explain most of the features of the present series of events.

Introduction

On 11th December, 1967, a severe earthquake affected parts of Deccan Peninsula, so long considered seismically stable. United States Coast and Geodetic Surveys determined following parameters for this event :

Epicentre	...	17.66 N : 73.93 E
Depth of focus	...	33 kms
Origin time	...	04 h. 21 m. 24.3 s. I.S.T.
Magnitude	...	6.0 (U.S.C.G.S.)

About 100 peoples died, over 1300 were injured and major property damages were reported. No satisfactory mechanism to explain the occurrence has yet been evolved. The region affected by this earthquake is characterised by horizontal bedding planes that remained practically undisturbed during long geological epochs. In fact the assumption that Deccan Peninsula is seismically stable is based partly on this fact of observation and partly on a tacit acceptance of the principle of fault movement as the cause of all major earthquakes. As against this, the catalogue of destructive earthquakes (A.D. 7 to 1899) compiled by Milne (1911) mentions as many as three severe earthquakes from Deccan region during the nineteenth century. The present event is the third of its kind during the present century—earlier two being the Coimbatore earthquake of 1900 and the Satpura earthquake of 1937. There are, thus, reasons to believe that the seismic activity of Deccan is such that these events do not result in marked structural changes of the surface geological formations. Important seismic belts of the world are believed to possess typical surface structures associated with fault movements.

Reid (1910) explained the occurrence of the San Fransisco earthquake of 1906 on the basis of release of locked up strain energy in an active fault. Since then movements along active faults were considered as cause of all major seismic events even though in many cases no manifest fault movements were observed. The basis for such association probably was the success of Byerley's correlation of compression and dilatation distribution of first

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movements of earthquakes with fault plane direction. With the increase in the number of seismological stations equipped with high sensitive seismographs a better perspective of first movement analysis has now become possible. In a recent review Stauder (1964) indicated possibilities of widely different quadrantal interpretation for the same earthquake. This is contrary to the unique fault plane solution of the Reid-Byerley hypothesis. Further Evison (1967), from statistical studies on 68 well recorded earthquakes, found that first motion data indicate predominant compression for certain categories of earthquakes, while for certain others they show predominant dilatation. He interpreted this information to indicate that the mechanism at the source of earthquake may not, in all cases, be of the type of movement along faults. As second solution, Evison (1963, 1967), therefore suggested volume changes accompanying polymorphic transition in the crust-mantle as a possible energy source for earthquakes. The transition that he considered permissible, were, however only solid-solid transition. Events at Koyna area points towards a source much more localised than in so called tectonic earthquakes with manifest fault movements. The purpose of the present study is to suggest a possible energy source for this earthquake series.

Nature of Seismie Activity of Deccan Area

From record of past activities, it seems that swarm activities play a dominant role in the seismic behaviour of Deccan and adjoining areas of Saurashtra and Rajasthan. It may be of interest to note that while in other active regions of India, major earthquakes are usually preceeded by a period of relative quiescence the major Koyna earthquake was preceeded by a gradual increase in seismic activity of an otherwise stable region. In fact the main event may be considered as the physical peak of a series of activity. There are many regions of the world where swarm activities are recorded and most of these regions have history of volcanic activity–dormant or active. It would therefore be responsible to consider volcanism as a correlating phenomena for understanding the occurance of such earthquakes.

The character of seismic phenomena of Deccan and adjoining areas may be summarised as follows :

1. The frequency of occurrence of earthquakes in general, and destructive events, in particular, is not very high.
2. There is not much evidence of wide spread tectonic movement of crustal layers in the region as the bedding planes show near horizontality.
3. Swarm type of seismic activity predominates in this region.

Any explanation for the earthquake phenomena in Deccan area should thus be built on these facts of observation.

Volcanism of Deccan Area

Large parts of Deccan and adjoining regions were subjected to intense volcanic activities in the past, eruptive phase of the activity apparently covered a period of over 25 million years and bulk of surface manifestations probably ceased around 50 million year ago. The flow composed of primary basalt, welled out through long cracks or fissures and due to general crustal weakness, the location of activity shifted continously. The resultant series of superimposed lava flows ultimately led to the formation of lava plateaus (Figure 1) of the so called Icelandian type. One of the significant difference between Deccan and Icelandian volcanism is that while in Iceland the flow of lava, after a repose of many million years, revived again in Pleistocene time ; there is as yet no definite indication of such revival in Deccan area.

The relationship between intense eruptive phase of a volcano and the associated swarm type of earthquake activity is now well established. Not much work have, however, been reported on earthquake activity associated with pre and post eruptive stages, but it is reasonable to expect that changes in volcanic manifestations are charccterised by changes in the nature of associated phenomena like earthquakes. In his studies on Japanese earthquakes, Imamura (1939) recognised two distinct type of earthquakes, associated with volcanism and designated a class of earthquake resembling the so called tectonic events as "non-eruptive" earthquakes. Mallet considered these as results of "uncompleted efforts to establish a volcano".

Formation, Growth and Decay of Volcanism

Recent studies on properties of rocks under high pressure and temperature confirms the possibilities of phase transitions of rock materials at great depths. The formation mechanism of basalt magma and their final transfer from mantle to surface has not yet been fully understood. Friggs and Handin (1960) in their studies on the mechanism of deep seated earthquakes ; proposed a shear melting hypothesis according to which the corners of a microflow or some blade like mineral grain can act as locations for superposed tensile stress under the action of distorting forces. The resultant pressure drop may induce a solid to pass into liquid phase. This hypothesis, may be extended to explain the formation of primary magma pool in upper mantle. Once formed, the stability, growth and activity of the pool depend upon thermodynamic conditions of phase stability. Yoder and Tilley (1962) determined such conditions in this melting diagram of basalt compositions as given in Figure 2.

The stability of the formed magma (liquid phase) generally increases with reduction of pressure at constant temperature or increase of temperature under constant pressure. Any increase of temperature is improbable as no heat source is available. But reduction of pressure is possible through adiabatic expansion of magma. Such mechanism leading to fissure formation following solid-liquid transition was recently discussed by Robson, Barr and Luna (1968). The magma pool can thus be sustained, first through the repetition of shear melting forces and then through adiabatic expansion of magma and opening out of fissures. According to Newton (1966) basalt magma is generated by partial fusion of peridotite mantle at depths of 80–250 kms. As the force system sustaining the growth of volcano is not continuously applied, the internal structure of a volcano follows a complicated pattern and may be visualised as shown in Fig. 3.

Decay in volcanic activity on the other hand starts with the weakening of internal forces. The first manifestation of such effect is the gradual reduction of intensity of mass transfer. This is followed by a period when there is a thickening of magma mass due to gradual fall in temperature. Low total heat capacity of materials within vents and fissures and high temperature contrast in the upper parts of the crust first results in sealing off vents and fissures. The resultant isolation of the system quicken the pace of pressure rise within the system and hasten the passage of the magma to a critical stage (Fig. 2). The final stage of the process is reached through the transformation of the magma to a stable solid phase.

From the point of view of energy balance the activity of volcano is thus not confined to eruptive phase only. It starts long before eruptive stage and continues till entire magma returns to the normally stable solid phase.

Earthquake Associated with Volcanism

From the foregoing discussions it will be seen that it is only during the eruptive phase that energy balance is maintained through bulk mass transport. Other factors on

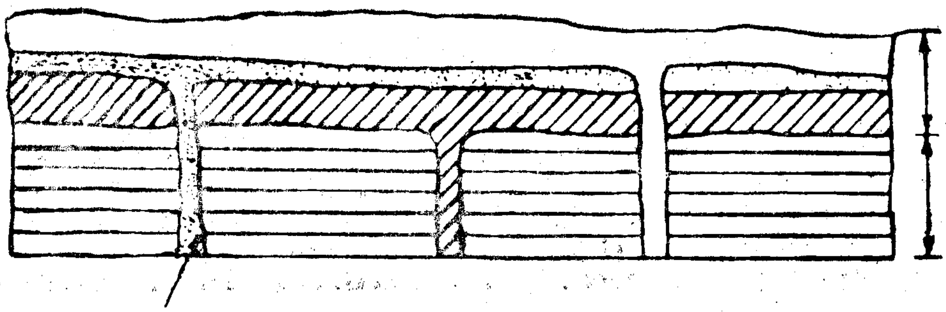

Fig. 1

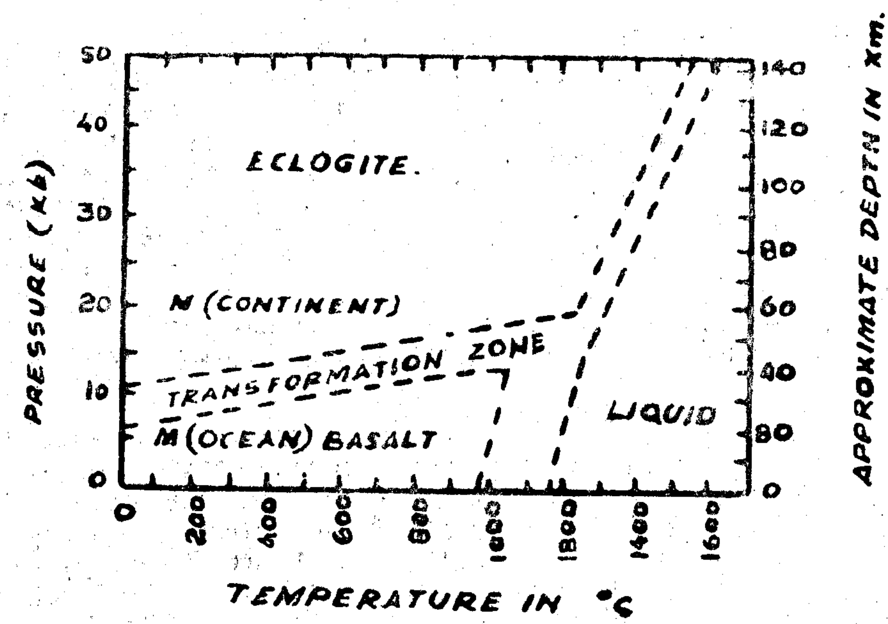

Fig. 2

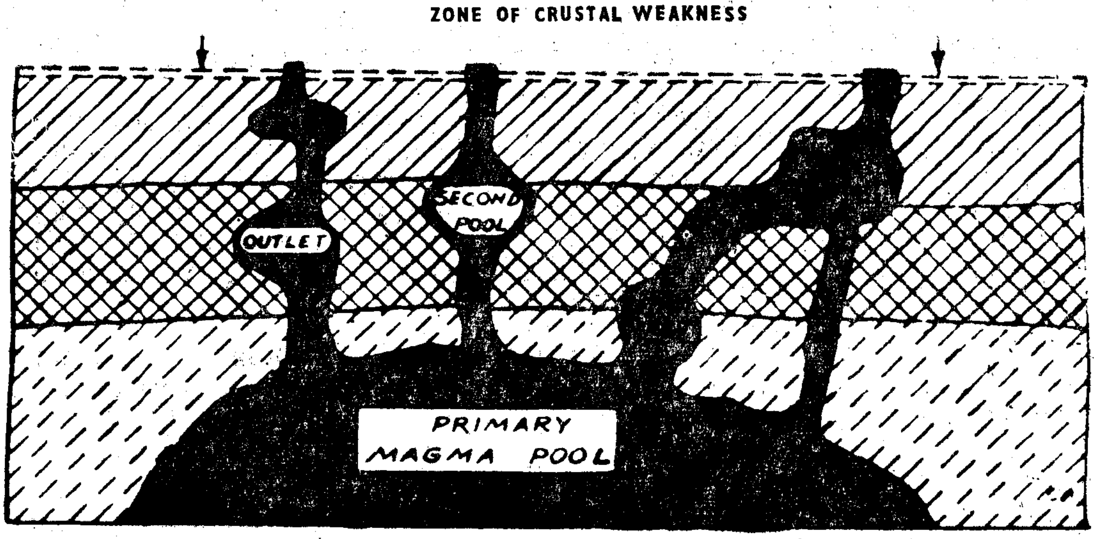

Fig. 3

the debit side, including earthquakes, play insignificant part. Accordingly, eruptive phase is not accompanied by major earthquakes. During pre-eruptive stage, on the other hand, the energy balance is obtained through phase transition and induced volume and structural changes in crust and mantle. The involved process is such that it may cause large scale upheavals and major earthquakes.

The process of decay of volcanism, on the other hand, is characterised by energy dissipation. From consideration of accompanying earthquake phenomena slow processes of heat transfers play insignificant part; but the process of liquid to solid transition, involving sudden release of large energies, is important.

The transition of the heterogenously composed material of the pool will commence slowly at first, in batches, and gradually increase to maximum and finally fall off. This will be because of different critical stages for different batches transformation. The accompanying earthquakes will thus have the character of the so called swarm type of activity.

The total energy release from a spherical magma pool, may be represented by the following simple equation :

$$E = \tfrac{4}{3} \pi r^3 \rho_{T,P} L_{T,P} J \text{ ergs} \tag{1}$$

where r — radius of the sphere.

$\rho_{T,P}$—Sp. gr. of the magma, at the temperature and pressure of the material of pool at the critical stage.

$L_{T,P}$—energy of phase transition of magma materials.

J — Joules heat equivalent.

In reality this is an integral equation of the type.

$$E = \Sigma \Sigma m' L' J \tag{2}$$

where summation are with respet to m′, L′

But as both ρ and L. vary only within narrow limits an average value may be sufficient for the purpose. At present no reliable data for P and L are available but by taking values suggested by Matuzawa (1964), energy of phase transition of a magma pool of 1 km diameter comes to the order of 10^{25} args. This energy is sufficient to account for a swarm in which major earthquakes are included.

Application of the Principle : Koynanagar Earthquake

It is necessary to consider in this connection evidences in support of location of magma chambers within the crust and upper mantle. Gorshkov while studying a number of Japanese earthquake records observed the absence of 'S' phase. He presumed the existence of magma chambers in those paths and from that identified and calculated the depth, size and shape of magma chambers. Hayakawa (1961) recently observed that in India, we have not any active volcano, but we have big areas of Deccan trap, which is the remains of old volcanic activity and also we have a broad distribution of granite and charnockite. From physical properties of such rocks, we can study the nature of magma". There are at present no definite evidences in support of existence of magma chambers in Deccan region but none the less the possibility of such occurrence can not equally be ruled out with existing information on the subject. Thus, on the basis of the principle postulated in earlier chapters, it is possible to arrive at the following general conclusion on the recent earthquake series at Koyna area.

1. The earthquake may be directly traced to volcanic activity in the last phase of decay. The present series originated from the reverse phase transition of magma pool.
2. Frequency of the earthquake in the Deccan region depend upon the frequency of phase transition of pools. Limitation in the number of such pools determine the frequency of earthquake occurrence. The low frequency of earthquake in general and larger events in particular indicates low occurrence of magma pools in critical stage of phase transition.
3. The so called 'tectonic forces' may not have direct bearing on the recent upheavals in this area.
4. Swarm type activity is the normal feature of the thermodynamic process postulated above. This is partly due to the heterogenous composition of the magma and partly due to release of mechanical stresses that develop during sudden phase transition of large volume of magma.

Discussion

The Koynanagar earthquake rudely disturbed the applecart of complacency in regards to earthquake occurrence in Deccan area. The question asked is–what precautions are necessary to overcome future earthquake hazards in this area? This question is asked with the tacit belief that earthquakes at Koynanagar indicate the development of new forces. It, however, appears possible that instead of development of new forces, the earthquake is indicative of degeneration of force system that will give the region increasing stability. However, this does not mean that earthquake will not happen in future. It only means that the frequency of occurrence of such events in future may not rise but may actually fall. The philosophy of earthquake insurance is based upon a correct assessment of the earthquake risks and relative economics of the protective measures. From this point of view, is it necessary to introduce protective measures in a general way? It is felt that the occurrence of Koynanagar earthquake should not itself be considered as evidence regarding increase in activity in the region. As such any programme of introducing additional insurance for structure in the region should be scrutinised properly. Of course, it should simultaneosly be made obligatory that all constructions, whose failure may mean a general hazard to public life and property, should have, in their design substantial earthquake resistance factor. Nuclear Power stations, other Nuclear installations, dam across rivers etc. should only be put under this category. The exact value of such factor can be worked out only after necessary data are obtained.

Acknowledgement

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QUANTITATIVE MEASURES OF SEISMICITY APPLIED TO INDIAN REGIONS

V.K. Gaur* and R.K.S. Chouhan**

Abstract

Three Measures of seismicity have been discussed and their comparative significance analysed. These are (a) The rate of strain generation and relaxation (b) Tectonic flux and (c) Frequency magnitude analysis. Examples of the behaviour of Indian regions is given and relevant results have been tabulated.

Introduction

Seismicity studies have evolved greatly since the time when Oldham (1911) and Montessus de Ballore (1911) first sought to express the seismic susceptibility of Assam by plotting the epicentres of earthquake as circles of varying dimensions to depict their relative sizes. Later, as more data accrued, other parameters such as radii of perceptibility and maximum observed intensities were variously used to denote the seismic activity of a region. However, they all suffered from an undue weightage on personal judgement and local rock conditions. The adoption of the Magnitude scale which related it to the energy released by an earthquake, although subject to certain constraints of experimental requirement, greatly changed the picture and has given earthquake statistics an objective look and a promising future.

Depending upon their completeness, earthquake data pertaining to a given region can be put to three different kinds of analyses—(a) Studies of the strain rebound increments, (b) Studies of the tectonic flux and (c) Frequency–Magnitude analysis.

These studies have several advantages over the older one of merely plottiug the sizes and locations of earthquakes. Apart from their objective character related to the physically sensible quantity representative of an earthquake i.e., its energy, they are relatively less susceptible to errors in epicentral determinations. They also emphasize the remarkable coherence which seismic activities possess both on region as well as on global scales.

Studies of the Strain Rebound Increments

Tectonic earthquakes occur in regions which are undergoing strain in response to certain geological processes. When the consequent stresses exceed the breaking strength of rocks, the potential energy of strain is released by fracturing and subsequent rebound of the rock masses toward equilibrium. The adoption of the earthquake magnitude which has made it possible to determine an objective quantity related to earthqukes i.e., the energy released by them, in turn permits the calculation of a quantity related to the strain rebound or of the equivalent fault slip.

If an elementary volume 'dv' within a fault rock suffers a strain e_{ij}, the resulting strain energy per unit volume can be considered to consist of two parts : one due to symmetrical or hydrostatic stresses and the remaining due to deviatoric stresses. In order to calculate a quantity proportional to strain, the following symbols are used.

1. $T_{ij} = \frac{1}{3} \sum_{k} T_{kk}.\delta_{ij} + P_{ij}$ — components of the stress tensor
2. $\frac{1}{3} \Sigma T_{kk} = - p$ — hydrostatic stress

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3. P_{ij} — components of the deviatoric stress tensor
4. $e_{ij} = \frac{1}{3} \Sigma e_{kk} \delta_{ij} + E_{ij}$ — components of the strain tensor
5. $\Sigma e_{kk} = \theta$ — cubical strain
6. E_{ij} — components of the deviatoric strain tensor
7. μ — coefficient of rigidity
8. K — bulk modulus
9. W — Potential strain energy per unit volume
10. J — Seismic wave energy
11. V — Total volume of rocks involved in the strain
12. γ — Fraction of potential strain energy converted into seismic wave energy. This is taken as constant for the entire volume V.

Where $p = -k\theta$, $P_{ij} = 2\mu E_{ij}$ (Bullen, 1963, p. 32), and the subscripts 'h' and 'd' signify quantities explained above relating to hydrostatic and deviatoric components. We then have the following relations :

$$W_h = \tfrac{1}{2} p\theta = \tfrac{1}{2} K\theta^2 \tag{6}$$

$$\begin{aligned} W_d &= \tfrac{1}{2} P_{ij} . E_{ij} = \mu (E_{ij})^2 \\ &= \mu (e_{ij} - \tfrac{1}{3} \Sigma e_{kk} \delta_{ij}) (e_{ij} - \tfrac{1}{3} \Sigma e_{mm} \delta_{ij}) \\ &= \mu (e^2_{ij} - \tfrac{1}{3} \theta^2) \end{aligned} \tag{7}$$

Also, $$J = \gamma \int_0^V W \, dv \tag{9}$$

If a simplification is introduced by considering $\gamma_h = \gamma_d = \gamma$, and $V_h = V_d = V$ and further if $K = 2\mu$ (Bullen 1963 pp. 233) we get,

$$J = J_h + J_d = \gamma\mu \int_0^V (e^2_{ij} + \tfrac{2}{3} \theta^2) \, dv \tag{10}$$

The above integral can be replaced by a more tractable expression by assuming an average value of strain ϵ which is constant throughout the valume V, so that,

$$J = \tfrac{1}{2} \gamma\mu V\epsilon^2 = C^2 . \epsilon^2 \tag{11}$$

where $\epsilon^2 = (2e^2_{ij} + \tfrac{4}{3} \theta^2)$

or $$C.\epsilon = J^{1/2} \tag{12}$$

where C is a constant dependent upon the volume of the strained rock mass and the elastic coefficients. In general. it is not possible to determine this constant but it may be assumed to be uniform for a given fault system.

Further, if it is assumed that the total strain ϵ is completely released by a slip X_f along a fault then,

$$\epsilon = C_1 . X_f$$

or $$C'.X_f = J^{1/2} \tag{13}$$

where C' is another constant dependent upon the elastic coefficients and the shape and volume of the strained rock.

The square root of the seismic wave energy released by an earthquaqe i.e. $J^{1/2}$ which can be readily calculated from the value of the magnitude is thus a measure of the associated strain release or the equivalent fault slip. A study of the cumulative strain release over a given period thus offers a useful means of comparing the seismic activities of different regions and also of the same region over different periods of time. In practice, this is done by plotting the cumulative values of $J^{1/2}$ corressponding to all the earthquakes considered, against the time of their occurrences which are reckoned in terms of Julian days. Studies of the elastic strain rebound characteristics over varying periods of time have been carried out for a number of regions of the world by Benioff (1949, 1951, 1954, 1955) both for shallow as well as deep focus earthquakes. He also plotted these values corresponding to the shallow and deep focus earthquakes (Benioff, 1951) for the whole world in order to study the total strain build-up in the world. These curves prove the understandable result that whilst in the upper elastic layer, where locking of a fault can only be relieved by subsequent fracture, the strain release occurs in active periods which alternate with relatively quiescent periods, the strain release in deeper regions is almost steady.

Similar study has been made for both shallow and intermediate focus earthquakes for the Indian subscontinent including Baluchistan and Tibet, covering a period of sixty years from 1905 to 1964. The resulting strain rebound characterestics are shown in Figures 1 and 2. The study has been further extended to different seismic regions of the country and will be published elsewhere.

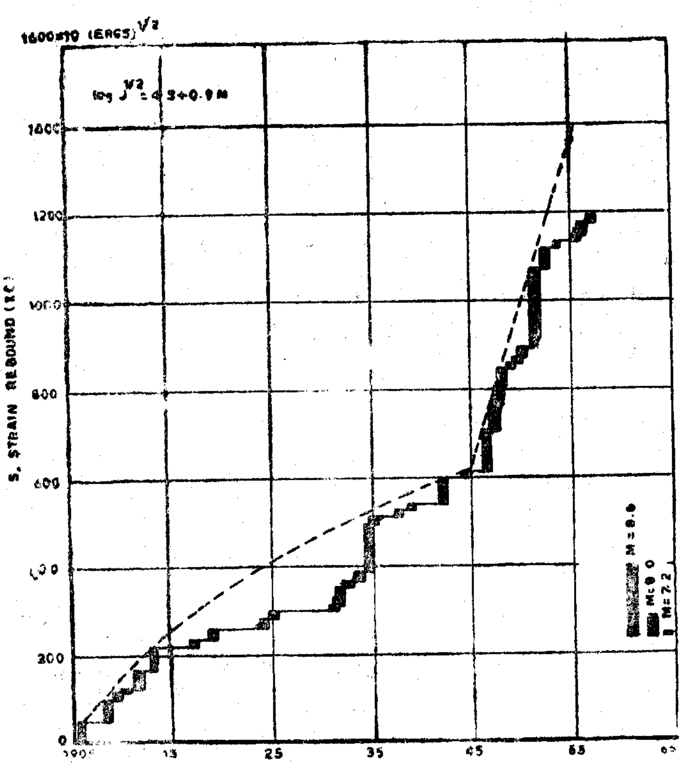

Fig. 1. Strain Accumulation and Release by Indian Shallow Earthquakes

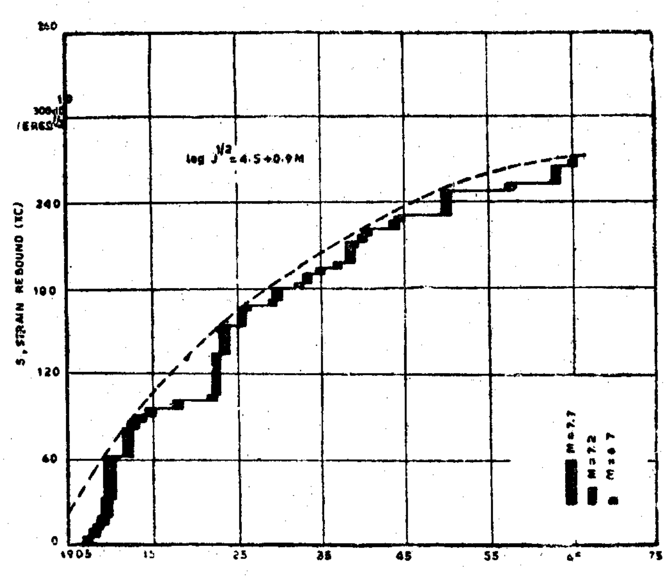

Fig. 2. Strain Accumulation and Release by deep Earthquakes having focal depth greater than 70 kilometers

It is well to recall here the various assumptions which are made whilst drawing the cumulative strain rebound curves and to examine their validity in the light of the results obtained. These assumptions are :

1. That the constant C which relates the strain to the square root of seismic wave energy is uniform throuhout the region. As C is a function of three quantities γ, μ and V as defined by equation (11) let us consider them one by one. γ which in fact represents the conversion efficiency of strain energy into wave energy can almost be taken to be unity as the pattern of energy release in earthquake aftershock sequences suggests that the principal earthquake is produced mainly by almost instantaneous elastic processess. However, variations in the values of the remaining two quantities μ and V from one fault system to another would be expected to destroy the coherence, if any, in the pattern of strain rebound increments. On the contrary, the strain rebound characteristics of various regions and even of the whole world considerd as a single unit (Benioff 1951, 1954) show remarkable consistency, and demostrate that possibly both μ and V or their product remains substantially constant for the fault systems.

2. That all the strain energy stored purely elastically is released by a single principal earthquake.

3. That seismic wave energy is radiated equally in all directions—an assumption which is implicit in the definition of M values which, in turn, form the raw material for the above studies.

Strain Relaxation

Strain rebound characteristics of all but deep focus shocks exhibit spurts of seismic activity separated by relatively quiescent periods. The resulting figure is thus a saw-tooth curve, the upper peaks of which, marking the end of an active period, represent a near exhaustion of the accumulated strain. A line drawn through these points (see straight line in figure 3a) therefore represents the rate of secular strain generation. Considering a mean rate for this in a given region it is then possible to illustrate the relative strain level obtaining in that region at different times, by means of a strain accumulation and relaxation curve. One such curve can be seen in figure 3b. At the begining of the period under study, the strain curve starts from an arbitrary level which represents the store of accumulated strain in that region at that time. It is then made to follow a slope equal to the

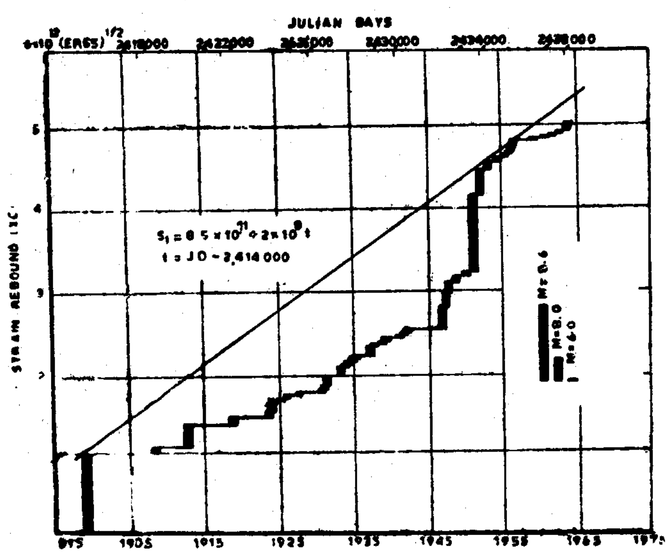

Fig 3a. Elastic strain rebound increments (times constant) of the assam region using shallow earthquakes having M $\leqslant$ 6.0

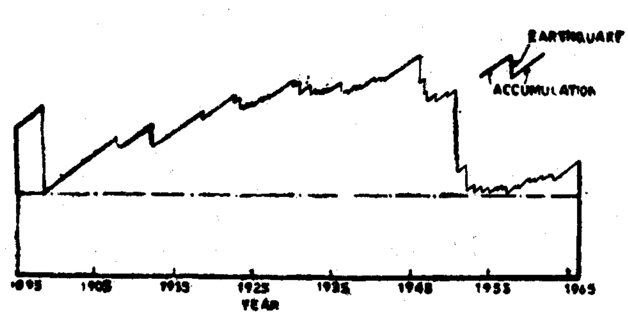

Fig. 3b. Relative strain accumulation and relaxation of the Assam region

mean rate of strain generation, which is derived from the strain rebound increments in a manner described above, until an earthquake representing a discontinuous release of strain takes place. At this point on the time axis, the curve drops vertically by an amount equivalent to the strain released by the earthquake and again follows the slope of strain generation until the occurrence of the next earthquake. Thus we have a yearwise strain accumulation and relaxation curve. Figure 3 referred to above pertains to the Assam region. Similar curves have been obtained for other seismic regions of the country.

A remarkable point illustrated by these curves is that although the strain level in a given region following an earthquake or a series of earthquakes may fluctuate from one active period to another, every region is characterised by a certain minimum level of strain which is perhaps seldom crossed.

This minimum level of strain which may or may not be zero perhaps represents the remanent strain that may persist even after the entire accumulated strain had an opportunity to be released by a large earthquake and therefore suggests itself as a suitable reference from which to estimate the amount of strain available in a region for future earthquakes. Assuming the worst possible situation that all the strain available at a given time in the near future is released by a singile earthquake it is then possible to predict the maximum probable magnitude of that earthquake. Thus the curve for Assam shows that around the year 1950, the region had enough strain to produce an earthquake of magnitude 8.5, which actually did occur.

Estimates of magnitudes of earthquakes liable to occur in the near future in other regions of the country are given in table 1, though it must be noted that these values refer to the maximum probable magnitudes and do not preclude the occurrence of smaller shocks. It is felt that this method would prove valuable for the purpose of general statistical risk prediction.

Table 1

Result of strain release characteristics of various regions giving the maximum probable size (Richter magnitude) of an earthquake that may occur there in the near furture.

Region, Magnitude range Period	Rate of strain generation × 10^7	Maximum probable size of earthquake that may occur in near future
The Andman-Nicobr region M = 5.5 to 8.1, 1915–1964	7.2	7.4
The Assam region M = 6.0 to 8.6, 1897–1964	20.0	8.2
The Bihar-Nepal region, M = 5.5 to 8.3, 1913–1964	8.5	7.3
The Kashmir region M = 5.5 to 7.6, 1924–1964	9.3	7.4
The Kutch region M = 5.0 to 7.0, 1928–1964	0.96	6.5

Tectonic Flux

If instead of plotting the strain released in a given region against time without regard to the spatial distribution of earthquakes, the rate of strain release per unit area is calculated for small units of the region and contoured the resulting map reveals some striking trends of seismic activity in close parallelism with the tectonic features. Such a map is called a tectonic flux map. The quantity plotted is the rate of flux of $J^{1/2}$ or simply the flux $J^{1/2}$ over a given period and is defined after Amand (1956) as follows :

$$F = \frac{1}{AT} \int_A \int_T J^{1/2} \, dA dt \tag{14}$$

where, A — is the area chosen

T — is the duration for wich the sum has been formed, and

$J^{1/2}$ — is the elastic strain times a constant.

A tectonic flux map has the built-in advantage of compromising the number of shocks with their sizes. The visual impressions produced by such maps can indeed be quite revealing and may help unify many a hiatus in an incomplete tectonic map of a region.

In order to prepare a tectonic flux map the region under consideration is first divided into small units of area by parallels of longitudes and latitudes. The strain release times a constant, which is given by the square root of the wave energy, is then calculated from the magnitude values corresponding to all the earthquakes above a certain magnitude that have occured in the region during the selected period. These are distributed amongst the various areal units as follows :

(a) If the epicentre is located within the boundary of the unit, the entire strain is assigned to it.

(b) If the epicentre is located on a boundary of two units, the strain released by the earthquake is divided equally between the two units.

(c) If the epicentre is located at the intersection of two boundaries, a quarter of the strain is assigned to each of the four adjoining units.

Finally for every unit area the sum of all the $J^{1/2}$ values assigned according to the above criteria is calculated, plotted at the centre of the unit and contoured. Before discussing a practical example however, it is well to question the unit of area and time interval chosen for preparing the tectonic flux map of a region. The smallest unit of area is largely limited by the accuracy of epicentral determinations. Thus in many cases, particularly if past earthquake records are used, a unit of area larger than desirable may have to be used, although this may partially compensate for the fact that the strain released by an earthquake covers a finite area, usually elongate, parallel to the fault system and not just a point represented by the epicentre.

The interval of time chosen, on the other hand depends upon the purpose behind the map. Tectonic flux maps covering shorter periods will normally be useful for studying the changing pattern of seismic activity of isolated regions. Longer period on the other hand will obliterate such temporal changes and tend to produce a more unified spatial picture of the seismic activity.

Tectonic flux maps are especially useful in studying the seismic behaviour of geologically complex regions, such as the Himalayas. Accordingly, a map was prepared for the entire Himalayan region including Baluchistan in the west and Tibet in the north. A period of 60 years from the year 1905 to 1964 was chosen in order to delineate most of the active seismic trends in this region which may have widely separated return periods. As the purpose of this study was only to mark the spatial pattern of active weak zones rather than emphasize the absolute values of strain released, values of $J^{1/2}$ were plotted directly without being reduced to unit time, which is constant for all points.

The above map (see figure 4) shows a boundary line dividing a northern zone having $\Sigma J^{1/2}$ greater than 10^9 $(ergs)^{1/2}$ from a southern zone having $\Sigma J^{1/2}$ values less than this. This boundry line closely follows the great boundry fault of the lesser Himalayas which runs from Kashmir to Assam. Of course this parallelism does not exist at the western and eastern extremeties as the tectonic patterns in these regions have been greatly infiuenced by the wedged in blocks of the Peninsular shield. However, it is quite apparent thatt his great boundary fault is more or less active all along its length, which incidently controverts the general feeling amongst geologists that only some parts of it are active. This is a significant result obtained purely from seismic studies. Further, the distribution of tectonic flux north of the boundry correspond to the not so continuous great boundary thrusts of the Himalayas. The maximum value of the flux is found to be in the Sadiya region north east of Assam which was the seat of one of the biggest earthquakes in history having a magnitude of 8.6.

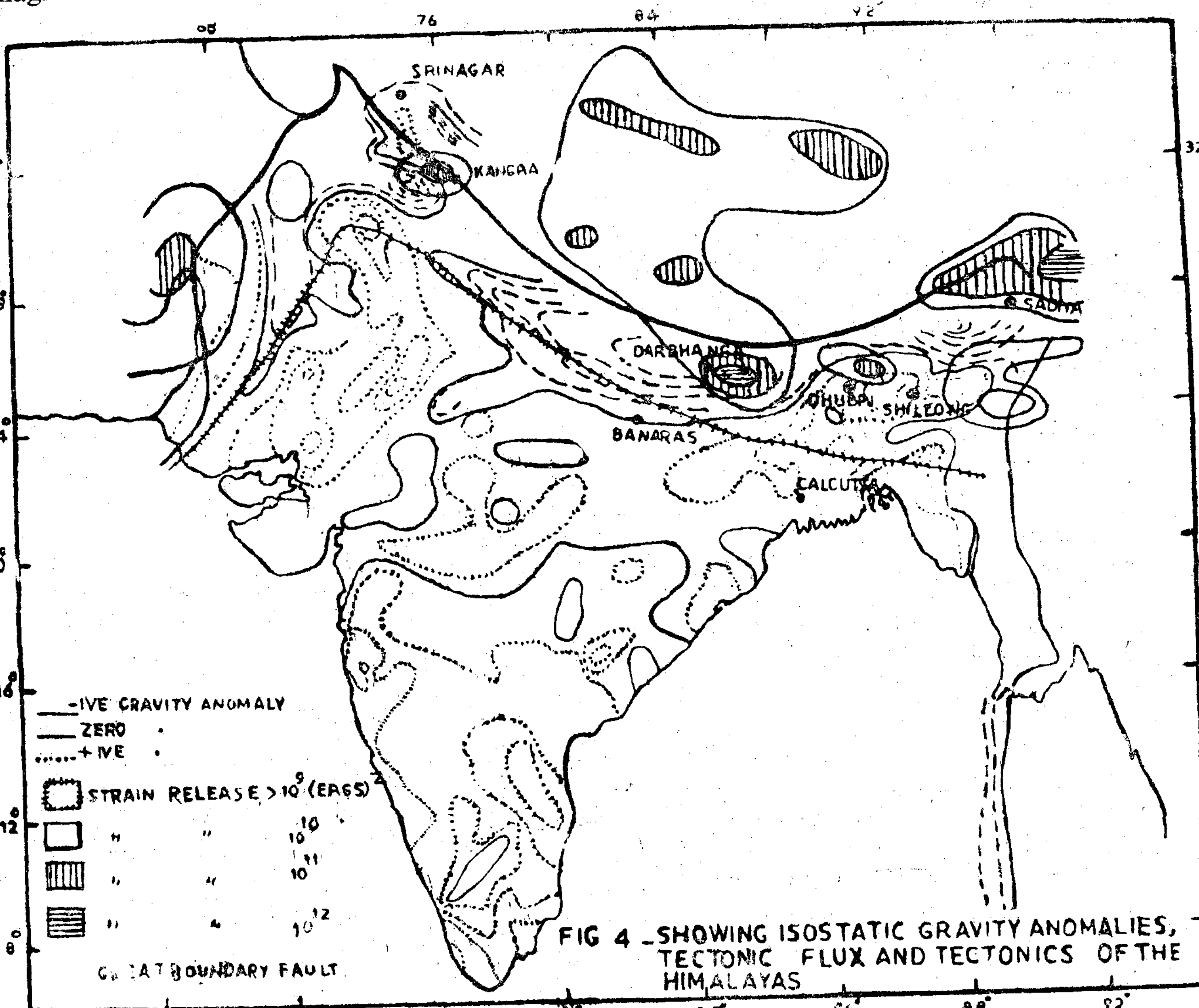

FIG 4 _SHOWING ISOSTATIC GRAVITY ANOMALIES, TECTONIC FLUX AND TECTONICS OF THE HIMALAYAS

The only other region of India for which sufficient data exist to enable the preperation of a tectonic flux map is the Assam region. For this purpose the Assam region was divided into small units of 0.01 square degree by parallels of Latitudes and Longitudes 0.1° apart (Chouhan, Gaur and Mithal 1966). This interval was chosen in view of the errors in epicentral determinations as quoted by the U.S. Coast and Geodetic Survey. All earthquakes of magnitude 5 and above which occurred during the 12 year period between January 1953 and December 1964 were considered for this study. The choice of the beginning of the interval was dictated by the availability of data having the above mentioned accuracy, whereas the length of the period was chosen after trial and error, so as to reveal maximum coherence in the prominent trends of seismic activity.

The tectonic flux for the entire period was calculated for each unit of areas and plotted accordingly. Thereafter, contours of equal tectonic flux were drawn at intervals of 0.5×10^{10} $(\text{ergs})^{1/2}$.

A visual inspection of this map shown in figure 5 reveals some conspicuous features which appear to coincide with known tectonic trends of the Assam region. Thus the dashed line in the EW direction at about 25.2°N which joins two centres of high activity, follows the Dauki transcurrent fault pointed out by Evans (1964). Further, their close coincidence corrobarates the strike slip nature of the fault. Two other trends of seismic activity can be similarly noted on the north and the south east corner of the Shillong Plateau. These are respectiveiy parallel to (a) the Brahamputra valley which flows in a trough and (b) the NE-SW running belt of Schuppen. They, however, do not coincide with them. This offset is indeed to be expected as the thrusts dip into the crust at a low angle.

It may, however be mentioned that in a tectonic flux map such as this covering a limited period, certain tectonic features associated with seismic activity of longer return periods may be entirely missed. Thus figure 5 referred to above is silent about the Sadiya region (28.6°N, 96.6°E) where a big earthquake of magnitude 8.6 occurred on 15th August 1950. This can, however, be seen in figure 6 showing a similar map for the 12 year period between 1940 and 1952. The map of course shows only the values of tectonic flux at discrete points as contouring was not possible owing to widely scattered epicentres. A better course would have been to superimpose these values on to other map and draw new contours, but the large differences between the errors of epicentral determinations made before and after the year 1952 would render this meaningless.

Inspite of what has been said above, which was meant to emphasize the possible defects of tectonic flux maps when constructed indiscriminately, the map obtained for the Assam region (1953–1964) appears to be remarkably objective. In fact an examination of the second map (1940–1952) suggests that a superposition of the tectonic flux values indicated therein on to the previous map would have the effect of leaving most of the trends unaltered apart from introducing another region of significant activity near Sadiya.

Frequency Magnitude Analysis

Both the studies of time variation of the strain release as well as of the tectonic flux of a region basically make use of cumulative values of strain released over a predetermined period without emphasizing the magnitudes of individual activity. This is indeed the purpose of these studies that is to highlight the relative capacity of a region to store strain and to delineate prominent zones of high activity.

However, if instead of plotting the cumulative strain release with time, the cumulative frequency of occurrence of shocks is plotted against tne corresponding magnitudes,

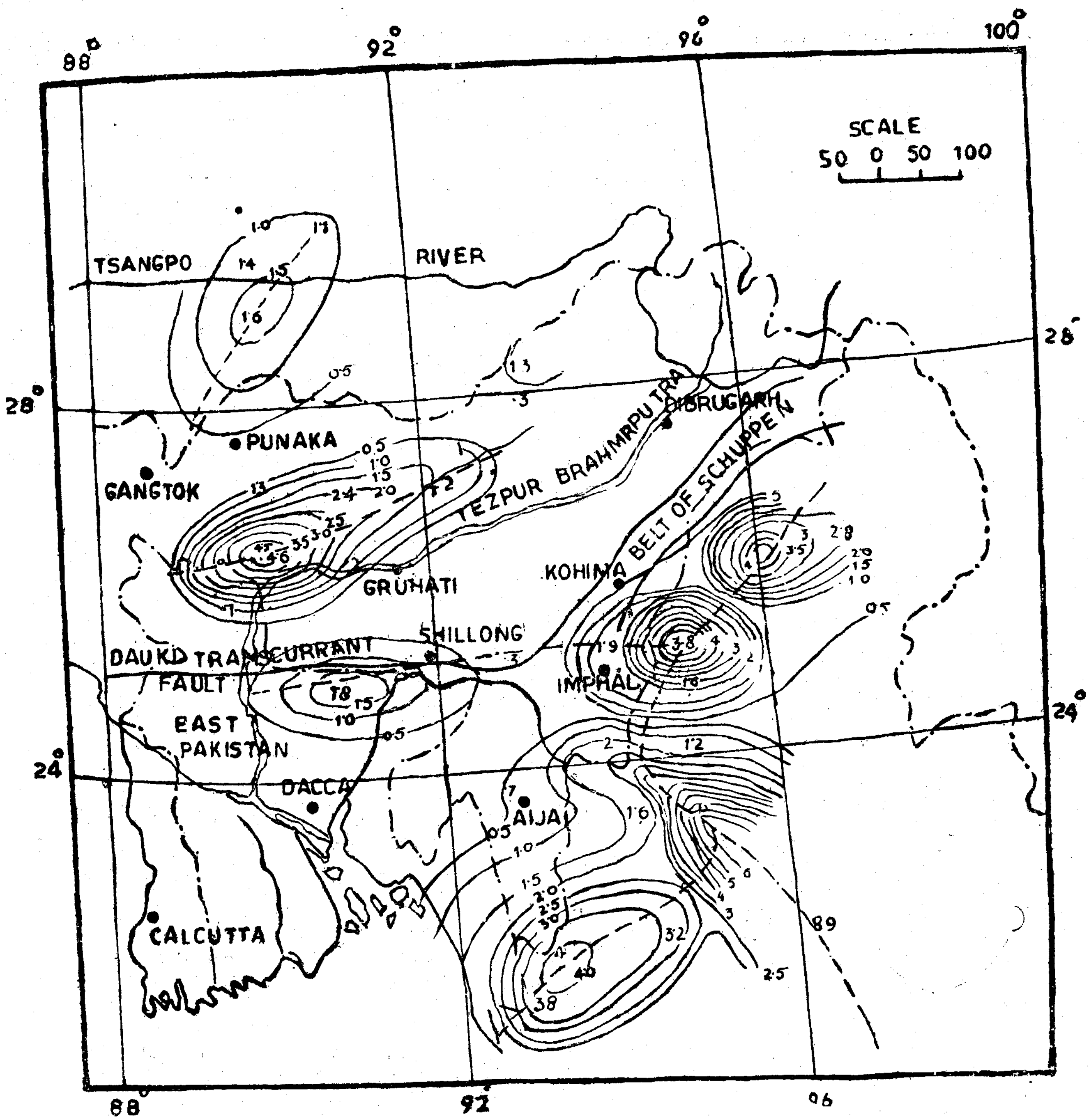

Fig. 5. Strain Energy Release Map of the Assam Shallow Eartquakes in the Unit of 10^{10} $[ERGS]^{1/2}$ Period 1953–64 Using $M \geqslant 5$

some additional information regarding the seismic behaviour of the region can be obtained. The frequency of higher magnitude shocks in any region drops off for obvious reasons and those of lower magnitude shocks become comparatively more numerous. A closer scrutiny however reveals that even with this general behaviour, the variation of frequency with magnitude is characteristic for a given region

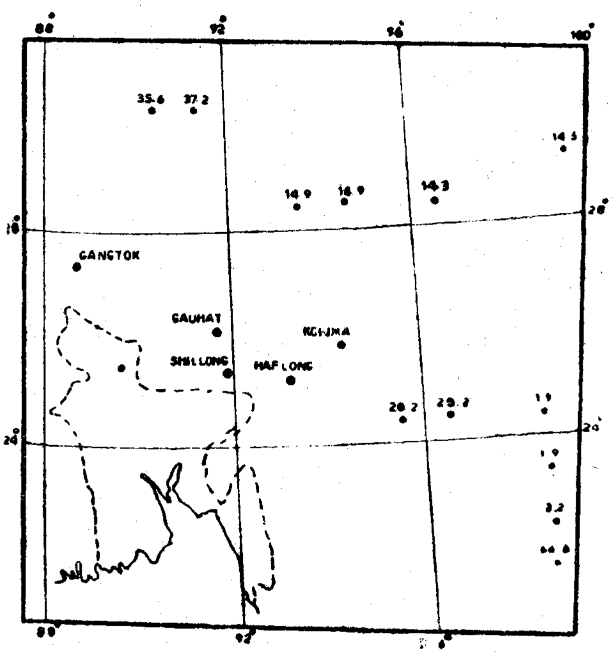

Fig. 6. Strain release map of the Assam shallow earthquake in the unit of 10^{10} $[\mathrm{ERGS}]^{1/2}$ period 1940-1952

In 1939 Ishmito and Lida, observed that small earthquakes occurring in the Kanto district of Japan satisfied the following relationship.

$$NA^{p} = K \qquad (15)$$

where N is the number of earthquake which produced a trace amplitude between A and (A+dA) and p and K are constants. If we now consider the definition of the earthquake magnitude M in terms of a constant trace amplitude A_f produced by an earthquake of fiducial magnitude under standard conditions of observation,

$$M = \log A' - \log A_f \qquad (16)$$

Eliminating A from the above two equations we get,

$$\text{Log } N = a - bM \qquad (17)$$

where a and b are now modified constants. Another way of writing this equation is in terms of the wave energy J

$$\log N = A' + \beta \log J \qquad (18)$$

This last equation has been extensivly used by Russion seismologists for frequency-magnitude sudies but it is felt that no additional advantage can be gained by merely rearranging the terms, which incidentally increases the arithmatical work.

The simpler relation of the type Log N = a — bM was used for the present studies. It has been suggested by many workers in this field that one or the other of the constants a and b may be used as a suitable measure of seismicity though both these suggestions seem to suffer from being effectively a measure of either low or high magnitude activity. For, 'a' represents the frequency of earthquakes whose magnitudes in the limit pass to zero and 'b' of those whose magnitudes attain very high values.

In fact a more meaningful quantity appears to be the ratio (a/b) which signifies a certain reference magnitude, which will be characteristic of the region, corresponding to a unit frequency. The only arbitrariness thus introduced will be on account of the unit of time chosen for frequency determinations.

However, if the frequency is conventionally denoted as the number per year for all studies, this arbitrariness is ironed out whilst comparing the ratio (a/b) for different regions. If, however, a different time unit is chosen say δ years, the new frequency will be defined as the number of events per δ years. Incidentally, such a choice will only have the effect of increasing the value of 'a' by log δ, whilst leaving 'b' unchanged.

In order to bring out the coherence, if any, in the values of the ratio (a/b) for different seismic zones of the country, frequency-magnitude analyses have been made which show that that the relative values of the ratios (a/b) obtained for these regions are atleast qualitatively correct. This work has been further extended to include the statistics of shallow earthquakes in other seismic regions of the world covering earthquakes of magnitudes equal to or greater than 5 as data for lower magnitudes tend to be less reliable.

Figures 7 to 13 show the frequency-magnitude curves pertaining to Indian regions and figures 14 to 20 represent shallow earthquakes of various regions of the world. The resulting values of a and b are compared in tables 2 and 3 respectively.

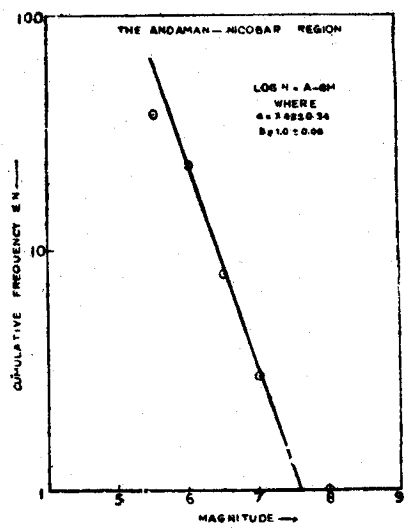

Fig. 7. Magnitude versus fiequency of occurence period 1915–64

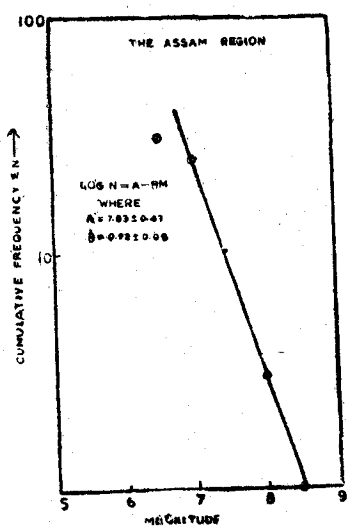

Fig. 8 Magnitude versus frequency of occurence

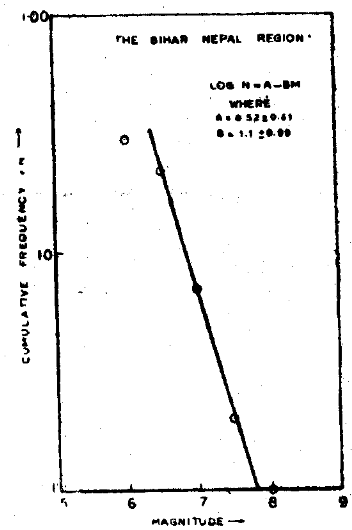

Fig. 9. Magnitu le versus frequency of occurrence period 1913–64

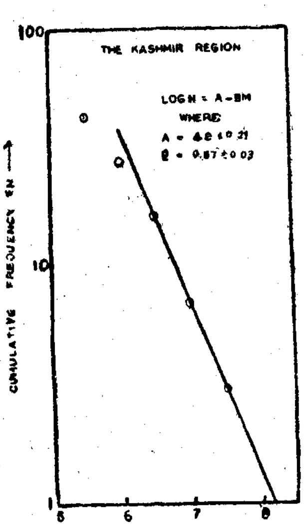

Fig. 10. Magnitude vers'us frequency of occurrence period 1924–64

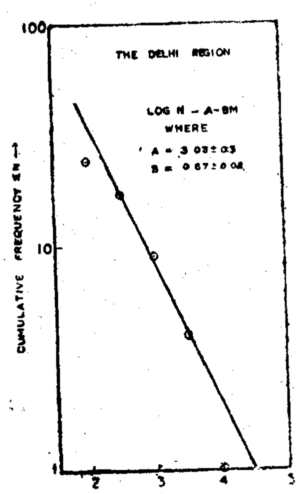

Fig. 11. Magnitude varsus frequency of occurrence period 1963–64

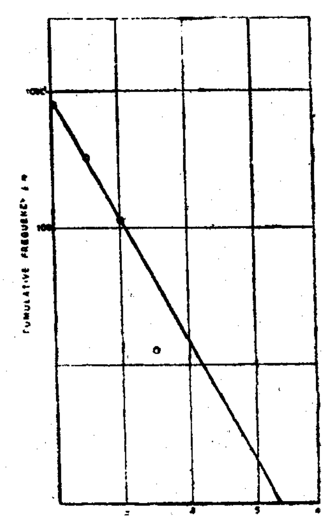

Fig. 12. Magnitude of versus frequency Koyna microtremors period 1963–67

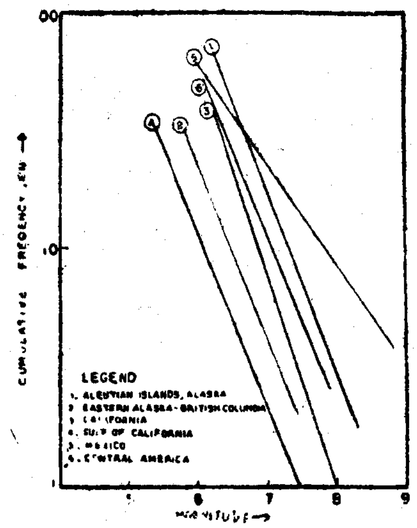

Fig. 13. Magnitude versus frequency of occurrence

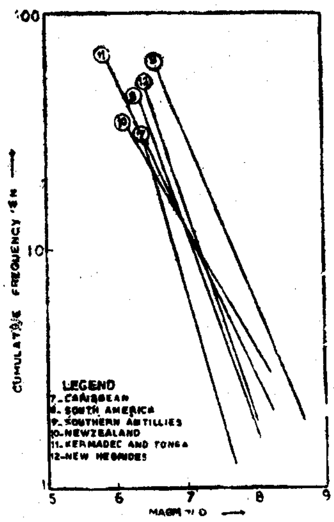

Fig. 14. Magnitude versus frequency of occurrence

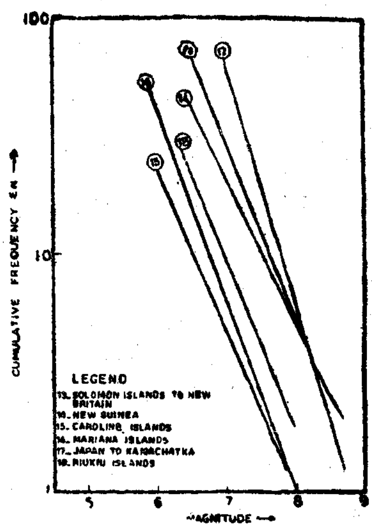

Fig. 15. Magnitude versus frequency of occurrence

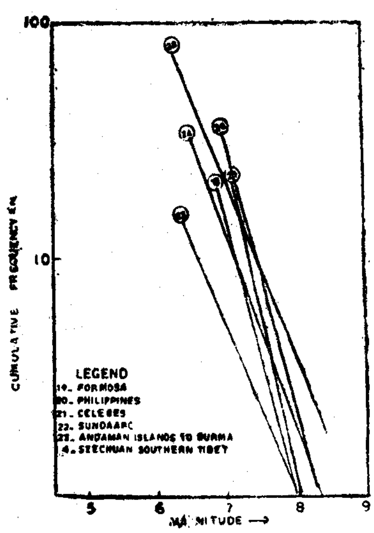

Fig. 16. Magnitude versus frequency of occurrence

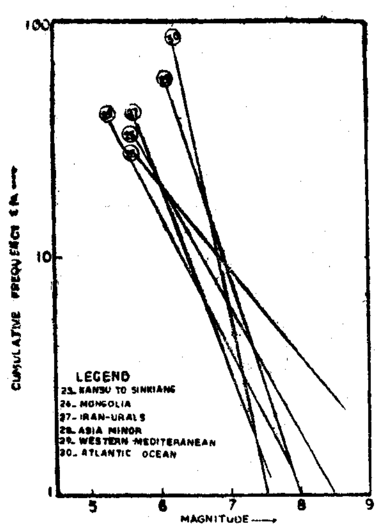

Fig. 17. Magnitude versus frequeney of occurrene

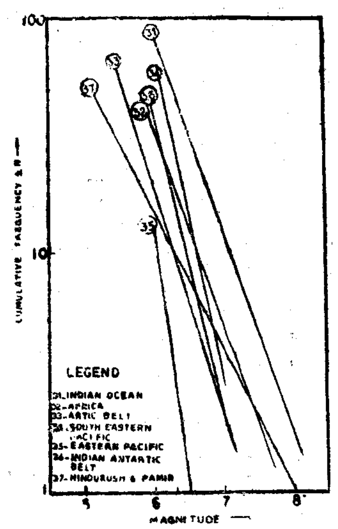

Fig 18 Magnitude versus frequency of occurrence

Table 2

Results of frequeney–magnitude analysis showing the ratio a/b and the return periods for earthquake of given magnitude

Calculated parameters Regio1 period and magnitude range	Values reduced for unit of time = 1 yr		a/b	Return periods for various magnitudes in years				
	a	b		8.5	8.0	7.5	7.0	6.5
The Assam region 1915–1964 M=6 to 8.6	6.13	0.92	6.6	53	22	7	2	
The Bihar Nepal region 1913–1964 M=5.5 to 8.3	6.82	1.1	6.2	...	78	28	8	
The Andman-Nicobar region, 1915–1964 M = 5.5 to 8.1	4.62	0.80	5.8	...	74	26	8	
The Kashmir region 1924–1964 M=5.5 to 7.6	3.2	5.7	5.6	...	...	14	6	
The Kutch region 1928–1964 M=5 to 7	3.38	0.71	4.7	...	...	...	40	
The Koyna region	4.05	0.87	4.6	...	...	...	80	
The Delhi region Jan. 1963 Dec. 1964 M = 2 to 4	2.27	0.67	4.1	...	...	...	100	
Deccan (South India)	...	...	...	...	...	...	...	62±5

Table 3

Showing the values of a/b obtained from the frequency magnitude analyses (world's shallow earthquake)

Region and period	Values of constants reduced correspond to annual frequency		a/b
	a	b	
1	2	3	4
1. Aleution Island Alaska, 1906–1945	5.17	0.79	6.36
2. Eastern Alaska, British Columbia 1908–1955	4.04	0.73	5.54
3. California, 1906–1945	5.16	0.86	5.99
4. Gulf of California, 1907–1945	3.92	0.71	5.49

1	2	3	4
5. Mexico, 1907–1945	2.61	0.41	6.37
6. Central America, 1904–1945	4.42	0.71	6.19
7. Caribbean, 1910–1945	6.98	1.10	6.35
8. South America, 1909–1944	4.84	0.70	6.92
9. Southern Antilles, 1910–1945	6.64	0.87	7.60
10. New Zealand, 1914–1945	3.18	0.51	6.23
11. Karmadec and Tonga, 1910–1943	4.57	0.71	6.40
12. New Hebrides, 1910–1945	5.94	0.90	6.59
13. Solomon Islands to New Britain 1910–1945	4.84	0.75	6.44
14. New Guinea, 1906–1945	4.04	0.64	6.30
15. Caroline Islands, 1909–1943	3.76	0.66	5.58
16. Mariana Islands, 1906–1945	4.87	0.81	5.99
17. Japan to Kamachatka, 1905–1945	7.18	1.00	7.18
18. Riukiu Islands, 1904–1945	4.24	0.70	6.02
19. Formosa, 1914–1943	5.68	0.89	6.36
20. Philippines, 1907–1943	6.72	0.98	6.85
21. Celebes, 1910–1945	8.29	1.42	5.80
22. Sunda arc, 1909–1943	4.95	0.75	6.48
23. Andaman Islands to Burma 1912–1940	4.36	0.73	5.97
24. Szechuan, Southern Tibet 1905–1945	5.22	0.82	6.37
25. Kansu to Sinkiang, 1920–1945	3.00	0.51	5.88
26. Mongolia, 1905–1944	2.64	0.35	6.55
27. Iran–Urals, 1911–1945	3.92	0.79	4.98
28. Asia Minor. 1909–1944	5.76	0.91	6.34
29. Western Mediteranean, 1908–1945	3.12	0.58	5.34
30. Atlantic Ocean, 1922–1944	6.99	1.10	6.36
31. Indian Ocean, 1904–1944	5.64	0.88	6.43
32. Africa, 1906–1945	5.33	0.91	5.86
33. Arctic Belt, 1908–1945	5.72	1.00	5.70
34. Southeastern Pacific, 1912–1944	8.91	1.42	6.24
35. Eastern Pacific, 1926–1944	7.96	1.42	5.57
36. Indian Antarctic belt, 1921–1942	5.06	0.83	6-07
37. Hindukush and Pamir, 1907–1944	3.12	5.90	5.27

The curves are plotted on a semi-logarithmic paper in order to handle long periods of time on a convenient scale. It may also be mentioned here that whilst drawing these curves, the unit of time was chosen to equal the entire period for which data were used. This was done in order to be able to plot whole numbers rather than fractions. The values of a and b have, however, been reduced for all these cases to conform to the annual frequency in a manner described above.

The data used in the above studies have been abstracted from the Catalogue given by Gutenberg and Richter in their book Seismicity of the Earth and from the records of the U.S.C.G.S. and of the Indian observatories. Complete details are given in the first five appendices of the Ph. D. thesis of R.K.S. Chouhan submitted at the University of Roorkee in August 1968.

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FORCED HARMONIC VIBRATION OF SANDWHICH PLATES

K. K. Pujara*

Summary

Static analysis of Sandwich plates by Eric Reissner [1] has been modified by including inertia terms in order to study the amplitude of forced harmonic vibrations of sandwich axisymmetric circular plates. Solution is found for the simply supported case and adapted for a viscoelastic core.

Notations

(2)	Reference No,
h	core layer thichness
ρ, ρ_c	the mass densities of the face and core respectively
t	each face layer thickness
q	axisymmetric load intensity
E_f, G_f, γ	Elastic modulii of isotropic face layer material
E_c, G_c	Elastic modulii of core layer material
w	deflection of the sandwich plate
W	amplitude of vibration
D	bending stiffness factor $= \dfrac{t\,(h+t)^2 E_f}{2\,(1-\gamma^2)}$
r	radial coordinate of any point in the plane of the flat plate
R	radius of the circular plate

$$\nabla^2 = \frac{d^2}{dr} + \frac{1}{r}\frac{d}{dr}$$

Assumptions

1. t/h is small compared with unity
2. $E_f\, t/E_c\, h$ is large compared with unity
3. Each face layer is a membrane without bending stiffness
4. Stresses in the core, which are parallel to the faces are negligible compared with transverse shear

Analysis

Eric Reissner[1] gives the following equation for the static analysis of a sandwich plate

$$D\,\nabla^2\,\nabla^2\,w = q - \frac{D\,\nabla^2 q}{(h+t)\,G_c} \tag{1}$$

It is abvious that to modify the equation to describe the dynamical problem, it is necessary only to add the inertia terms to the equation. The deflection then becomes a function of both radius and time.

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If a load intensity $q = q_o \sin pt$ is assumed, the above equation gets modified to the following, to describe the vibration $w(r,t) = W(r) \sin pt$.

$$D \nabla^2 \nabla^2 W - Ap^2 W = q_o - \frac{D \nabla^2 q_o}{(h+t) G_c} - \frac{D \nabla^2 A p^2 W}{(h+t) G_c} \tag{2}$$

where $A = 2 \rho t + \rho_c h$

the time factor sin pt gets cancelled throughout in equation (2) confining attention to a circular axisymmetric plate, simply supported edges lead to the following boundry conditions :

$$\text{at } r = R, \quad W = 0$$

$$\text{and } \frac{d^2 W}{dr^2} + \frac{\gamma}{r} \frac{dW}{dr} = 0 \tag{3}$$

$$\text{at } r = 0, \frac{dW}{dr} = 0 \quad \text{(because of axisymmetry)}$$

W. Nowacki[2] modifies the edge moment condition (Equation 3) to

$$\frac{d^2 W}{dr^2} + \frac{1}{r} \frac{d W}{dr} = 0$$

This modification has been observed to have negligible effect on the results[4]. This approximation is taken.

Now introducing Hankel transforms.

$$W^* = \int_0^R r J_o(a_1 r) W \, dr$$

$$q_o^* = \int_0^R r J_o(a_1 r) q_o \, dr$$

$$\int_0^R r \nabla^2 W J_o(a_1 r) = \left[r \frac{dW}{dr} J_o(a_1 r) - a_1 r W J_o'(a_1 r) \right]_0^R - a_1^2 \int_0^R r W J_o(a_1 r) \, dr$$

The expression in brackets vanishes for upper limit provided $J_o(a_1 R) = 0$ and for the lower limit it always is zero.

Thus if parameter a_1 satisfies the equation $J_o(a_1 R) = 0$

$$\int_0^R r \nabla^2 W J_o(a_1 r) = - a_1^2 W^*$$

$$\int_0^R r \nabla^2 \nabla^2 W J_o(a_1 r) = a_1^4 W^*$$

$$\text{and} \quad \int_0^R r J_o(a_1 r) \nabla^2 q_o = - a_1^2 q_o^*$$

Substituting the above Hankel transforms in Equation (2)

$$D\,\alpha_1^4\,W^* - Ap^2\,W^* = q_o^* + \frac{D\,\alpha_1^2\,q_o^*}{(h+t)\,G_c} + \frac{DAp^2\,\alpha_1^2\,W^*}{(h+t)\,G_c}$$

or $$W^*\left(D\,\alpha_1^4 - \frac{D\,A\,p^2\,\alpha_1^2}{(h+t)\,G_c} - A\,p^2\right) = q_o^*\left[1 + \frac{D\,\alpha_1^2}{(h+t)\,G_c}\right]$$

or $$W^* = \frac{q_o^*\left[1 + \frac{D\,\alpha_1^2}{(h+t)\,G_c}\right]}{D\,\alpha_1^4 - \frac{D\,A\,p^2\,\alpha_1^2}{G_c\,(h+t)} - Ap^2}$$

By inverse transformation

$$W = \frac{2}{R^2}\sum_{i=1}^{i=\infty} \frac{q_o^*\left[1 + \frac{D\,\alpha_1^2}{(h+t)\,G_c}\right]}{\left(D\,\alpha_1^4 - \frac{D\,A\,p^2}{(h+t)\,G_c} - Ap^2\right)}\;\frac{J_o(\alpha_1\,r)}{[J_1(\alpha_1\,R)]^2}$$

$$= \frac{2}{R^2}\sum_{i=1}^{i=\infty} \frac{\left[\int_0^R r\,J_o\,(\alpha_1\,r)\,q_o\right]\left[1 + \frac{D\,\alpha_1^2}{(h+t)\,G_c}\right]}{\left(D\,\alpha_1^4 - \frac{D\,A\,p^2}{(h+t)\,G_c} - A\,p^2\right)}\;\frac{J_o\,(\alpha_1\,r)}{[J_1\,(\alpha_1\,R)]^2} \qquad (4)$$

where α_1 satisfies the equation

$$J_o\,(\alpha_1\,R) = 0$$

By eliminating the terms containing G_c, the equation (2) yields the expected form of solution for the homogeneous plate[2].

The solution is a series solution when each term is a constant multiplied by zero-th order Bessel function and as

$$\nabla^2\,J_o\,(\alpha_1\,r) = -\,J_o\,(\alpha_1\,r)$$

at $r = R$

$$\nabla^2\,J_o\,(\alpha_1\,r) = -\,J_o\,(\alpha_1\,R) = 0$$

This is the apprximate expression for the edge moment condition (Equation 3).

So the edge moment condition is satisfied.

Similarly it can be easily seen that the solution satisfies the other boundary conditions also.

To adapt equation (4) to a viscoelastic core, G_c can be made a complex modulus[3].

Let $G_c = G_{c1}\,(1 + j\,\beta)$

The solution for the viscoelastic core would modify to

$$W = \frac{2}{R^2}\sum_{i=1}^{i=\infty} \frac{[G_{c1}\,(1+j\,\beta)\,(h+t) + D\,\alpha_1^2]\int_0^R r\,J_o\,(\alpha_1\,r)\,q_o}{G_{c1}\,(1+j\,\beta)\,(h+t)\,[D\alpha_1^4 - A\,p^2] - D\,A\,p^2\,\alpha_1^2}\cdot\frac{J_o\,(\alpha_1\,r)}{[J_1\,(\alpha_1\,R)]^2}$$

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KOYNA EARTHQUAKE INVESTIGATIONS*

A.S. Arya, A R. Chandrasekaran and L.S. Srivastava**

The area around Koynanagar, district Satara, Maharashtra State, witnessed a very severe easthquake on the early hours of December 11, 1967 and caused considerable loss of life and damage to structures. Due to lack of data, sporadic minor earthquake activity and the sheild character of the Peninsula, the region was generally considered inactive and stable, and the occurrence of this major earthquake has caused great concern and panic in the area. This report gives a summary of the investigations being carried out at the School of Research and Training in Earthquake Engineering, Roorkee.

The School was established in 1960 as a grant-in-aid unit of the Council of Scientific and Industrial Research at the University of Roorkee. One of the projects of this School is to collect data on damage and nondamage to various types of buildings during major earthquakes and behaviour of the ground and strong ground motion. These surveys pinpoint the good and weak and vulnerable features of the various structures and construction practices in the region, and help in evolving suitable modifications in the design and construction of structures to make them earthquake resistant with minimum additional expenditure. With this in view a team of scientific staff visited the site immediately after the occurrence of this earthquake to study the behaviour of various structures in the region and related problems.

The field survey conducted by the School (Chandrasekaran, Srivastava and Arya, 1968) show that the traditional construction in the area was non-seismic and had little resistance against lateral forces. In the villages the basic frame work was of timber which could offer lateral resistance and failure was mostly confined to the cladding walls. In the modern construction random-rubble masonry was used which has very poor lateral resistance and these structures suffered extensive damage. The severe damage is limited in a small area of nearly elliptical shape, about 7 miles wide and 13 miles long and the maximum observed intensity in the region was VIII+ on the MM scale (Fig. 1). Investigations are being carried out to determine the effect of the earthquake upto distances of 150 miles and evaluate the forces to which the various structures were subjected to during the earthquake. This will lead to a better and more correct assessment of the seismic coefficient for the design of structures in the region.

A study of damage and non-damage indicate that it is possible to have earthquake resistant construction with locally available materials. The great devastation done by the earthquake necessitated immediate rehabilitation and reconstruction in the area, and as a first step towards this, the school suggested an earthquake resistant design with timber or pipe supports for rebuilding the houses utilising the plinths, roof trusses and sheets of damaged buildings. Detailed recommendations for construction of small buildings (Arya, 1968) were also given for the reconstruction of the houses in the area.

Data of true ground motion (displacement, velocity, acceleration) due to the earthquake at a site is required to determine forces acting on the structure for its earthquake resistant design. Special strong motion instruments are required to record the character of these movements. There are two accelerogaphs located in the Koyna dam, and the accelerograph situated in the right abutment in Monolith 'I a' functioned at the time of the earthquake. The accelergram recorded is the strongest ever recorded from the point

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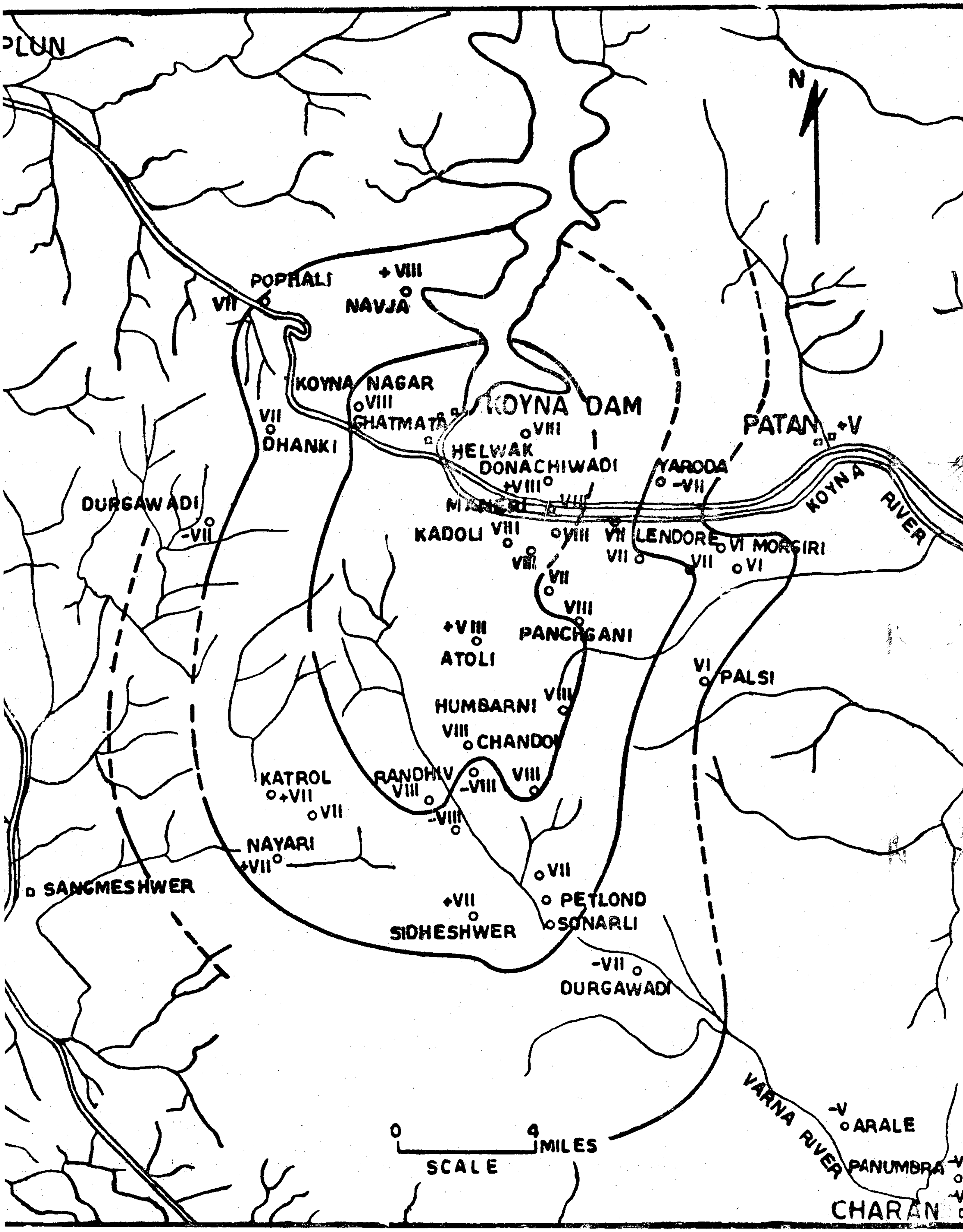

Fig. 1. Isoseismal Map of the Koyna Earthquake of December 11, 1967, (M.M. Intensity Scale).

of view of maximum base acceleration. Response spectra curves have been drawn for the various components of this accelerogram (Chandrasekaran and Saini, 1968). Fourier spectra have also been determined which give an indication of the amplitudes associated with various frequency components. Comparing spectral intensities of this earthquake with some of the earlier records it is observed that Koyna accelerogram is not the most intense, even though its peak ground acceleration was the maximum.

In addition to the two accelerograph located in the body of the dam, the School has suggested to the project authorities to instal two accelerographs in the tallest monolith of the dam, one in the foundation gallery and other at the top. This would give the ground acceleration and the response of the monolith respectively. The School has designed and manufactured accelerographs and will be supplying the accelerographs to the project authorities for installation.

Because of the relatively high cost and complexity of the accelerograph, and the necessity for obtaining more information on the effect of local geological conditions and greater instrumental coverage, a low cost simplified strong motion earthquake recorder has been developed by the School, which has been termed as Structural Response Recorder. The recorder does not measure ground motion, but rather records the maximum response of a mechanical system idealising dynamic characteristics of a structure which effect its over all behaviour. A structural response recorder simulating Koyna Dam has been installed in the region. Similar recorders simulating other structures in the region are being manufactured and will be installed shortly.

The Koyna dam and the appurtenent structures have with-stood the shock admirably well, though a little distress was noted in some parts. Theoretical investigations to determine response of various monoliths of the Koyna dam have been carrid out assuming the monolith to behave as a one-dimensional structure (Chandrasekaran, Arya and Saini, 1968). This study indicated the likely forces to which the highest monolith of the non-overflow section was subjected to during the shock, and the stresses produced at various sections of the monolith have been worked out. Model studies on a horizontal shake table were carried out to determine the dynamic behaviour of the monoliths of the Koyna dam. Models were made of plaster of paris and sand mixture. These were subjected to free vibrations, steady state sinusoidal vibrations and impulsive shock type (initial velocity) vibrations (Arya, Chandrasekaran, Mathur, Gosain, Thakkar and Khurana, 1968). Hydro dynamic pressures were also evaluated. Based on the therotical and experimental study the stresses acting on the dam monoliths due to static and dynamic loads have been evaluated and the required strengthening at various points to safe-guard the structure against furture shocks have been indicated. Further model experiments are under progress. The various monoliths are also being anlysed as two dimensional structures. Investigations are also in progress concerning strengthening measures to be adopted for future shocks.

As stated earlier the region was considered seismically inactive and stable. A study of past historical documents and records reveals occurrence of many earthquakes in the region. Geologic and seismo-tectonic studies of the Peninsula show evidences of recent movements in the region (Srivastava and Tipnis, 1968), and detailed geological and tectonic mapping of the area has been undertaken by the School. The geological and tectonic set up indicates that the region has a relatively higher level of seismicity and requires modification in the existing seismic zoning of the country. The seismic zones, as hither-to adopted in the country, has more emphasis on the seismic history of the country. The occurrence of this earthquake indicates that greater emphasis is needed on the tectonic features vulnerable to movements in the various zones. Based on these considerations revision of seismic zoning map has been proposed.

It has been observed that in the active seismic belts with gradual accumulation of strain the ground undergoes deformations. These deformations are an indication of the tectonic and seismic activity of the region. In the long range the measurment of these deformations may help in predicting future earthquake activity. The school has designed and manufactured a portable water tube tiltmeter (Agrawal, 1968) which would measure changes in ground level correct to a few microns. These water tube tiltmeters are being installed in the area, and would lead to a better assessment of the causes of the earthquakes in the region.

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SEISMOLOGICAL NOTES

(India Meteorological Department, New Delhi)

Earthquakes in and near about India During January – March, 1968

Date	Origin time (G.M.T.) h.	m.	s.	Epicentre Lat. (°N)	Long. (°E)	Region	Approx depth (Kms.)	Magnitude	Remarks
1	2	3	4	5	6	7	8	9	10
Jan. 5	06	02	40.5 (NDI)	30.2	80.2	U.P.-Nepal Border	—	5.3 (NDI)	—
Jan. 6	15	13	28.7 (CGS)	16.4	92.1	Bay of Bengal	33	5.1 (CGS)	—
Jan. 12	04	17	43.1 (CGS)	13.4	93.1	Andaman-Island	33	5.5 (CGS)	—
Jan. 12	04	37	26 (NDI)	Koyna Region		S. Maharashtra	—	—	Aftershock.
Jan. 18	19	57	12.9 (CGS)	24.3	93.2	Burma-India Border	100	4.7 (CGS)	—
Jan. 22	10	35	36.6 (CGS)	38.2	75.6	S. Sinkiang Province, China	108	5.3 (CGS)	—
Jan. 23	03	22	46.2 (CGS)	26.0	95.5	Burma-India Border	103	5.0 (CGS)	Felt at Shillong.
Jan. 25	20	15	17 (NDI)	Koyna Region		S. Maharashtra	—	—	Aftershock.
Jan. 29	05	00	10.0 (CGS)	36.3	70.4	Hindukush	225	5.5 (CGS)	Felt at Peshwar, Rawalpindi & Lahore.
Jan. 30	08	17	32.3 (CGS)	36.4	70.4	Hindukush	205	5.2 (CGS)	—
Jan. 31	11	45	15 (NDI)	29½	92	E. Tibet	—	—	—
Jan. 31	11	45	16.9 (CGS)	29.9	92.1	Tibet	18	5.2 (CGS)	—
Feb. 7	08	09	13 (NDI)	Koyna region		S. Maharashtra	—	—	Felt upto Poona.
Feb. 7	12	23	03.2 (CGS)	36.2	70.7	Hindukush	155	4.9 (CGS)	—
Feb. 9	14	39	46.8 (CGS)	29.8	68.7	W. Pakistan	33	4.6 (CGS)	—
Feb. 10	17	03	03.8 (CGS)	34.1	78.5	Kashmir-Tibet Border	37	5.2 (CGS)	—
Feb. 11	02	25	01.2 (CGS)	34.2	78.4	Kashmir-Tibet Border	33	4.8 (CGS)	—

1		2	3	4	5	6	7	8	9	10
Feb.	11	20	38	29.4 (CGS)	34.2	78.6	Kashmir-Tibet Border	44	5.1 (CGS)	—
		20	38	28 (NDI)	33.7	79.2	Kashmir	—	4.8 (NDI)	—
Feb.	11	23	18	16.0 (CGS)	34.3	78.2	Kashmir-H.P. Border	33	4.5 (CGS)	—
Feb.	12	22	17	23.0 (CGS)	22.9	95.4	Burma	23	4.7 (CGS)	—
Feb.	16	05	37	54.2 (CGS)	33.7	95.1	Tshinghai Province, China	33	4.8 (CGS)	—
Feb.	21	23	32	36.9 (CGS)	38.1	86.9	S. Sinkiang Province, China	28	4.7 (CGS)	—
Feb.	23	05	41	05.7 (CGS)	2.4	98.6	N. Sumatra	39	4.7 (CGS)	—
Feb.	24	12	53	48.8 (CGS)	8.7	94.0	Nicobar Island	58	4.7 (CGS)	—
Feb.	25	12	43	49.5 (CGS)	4.0	95.8	N. Sumatra	33	5.0 (CGS)	—
Feb.	28	09	54	56.1 (CGS)	30.3	67.6	W. Pakistan	25	4.8 (CGS)	Felt at Quetta.
Mar.	3	09	31	20.2 (CGS)	34.7	72.3	W. Pakistan	33	5.2 (CGS)	—
Mar.	4	21	36	35 (NDI)	Koyna region		S. Maharashtra	—	—	Felt upto Poona
Mar.	7	11	44	04.5 (NDI)	25 Kms W. of Delhi		Near Delhi	—	2.8 (NDI)	Felt at Delhi
Mar.	8	23	08	21 (CGS)	8.7	94.1	Nicobar-Island	33	4.6 (CGS)	—
Mar.	9	00	45	50 (NDI)	7.5	94	-do-	—	—	—
		00	46	00.9 (CGS)	8.7	94.0	-do-	33	5.0 (CGS)	—
Mar.	10	01	44	2.5 (NDI)	22 Kms W. of Delhi		Near Delhi	—	2.6 (NDI)	Felt in Delhi
Mar.	21	02	45	56 (CGS)	37.8	72.5	Tadzik SSR	131	4.8 (CGS)	—
Mar.	25	10	25	01 (NDI)	40 Kms NW of Delhi		Near Delhi	—	—	Felt at Sonepat.
Mar.	29	19	00	35	36.6	70.4	Hindukush	209	4.7 (CGS)	—

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3. Three copies of the manuscripts must be submitted.
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Example :—

Aggarwal, S L. (1964) "Static and Dynamic Behaviour of a Vertical Pile Subjected to Lateral Loads", Master of Engineering Thesis, University of Roorkee, Roorkee, 1964.

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