Volume III, No. 1 January, 1966

BULLETIN

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OF
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ROORKEE, U.P. (INDIA)

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INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY

Roorkee (U.P.) India

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1966

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REQUEST

The First Bulletin (Vol I. No. 1) published in January 1964 has been sold out. Querries for its availability are still pouring in especially from abroad. The Society will feel grateful to the members who can spare their copy gratis for the use of others. The Society will record with appreciation this gesture of goodwill by the members.

Secretary

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INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY

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Vol. III | January 1966 | No. 1

CONTENTS

EDITORIAL

Dear Member,

We feel great pleasure in presenting the fifth Bulletin (Vol III No. 1) as per schedule. We are happy to inform you that it is proposed to increase the frequency of publication of this Bulletin from two per year to three per year. This year's publications will be in January and May. The third issue may be replaced by the proceedings of the Third Symposium to be held at Roorkee in November 1966 on a concessional rate.

A section on reviews of papers published in other journals on problems connected with Earthquake Technology was introduced in the Fourth Bulletin (Vol. II No. 2). This section has now been enlarged. The cooperation of all the members who contributed reviews is greatly appreciated.

A column on "News of Members" had also been introduced. Through this we shall know the changes in placement, status of members and distinctions achieved and this will assist us in maintaining better contacts. Members are requested to send relevent information for publication in subsequent issues.

Election of office bearers of the Society was held by postal ballot in December, 1965 and the following have been elected for the year 1966:—

(a)	Dr. Jai Krishna	—	President
(b)	Sri V.S. Krishnaswamy	—	Vice-President
(c)	Dr. A.S. Arya	—	Secretary
(d)	Sri S.L. Agarwal	—	Editor
(e)	Sri B.C. Mathur	—	Treasurer
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(i)	Dr. H.C. Visvesvaraya	—	do
(j)	Dr. Hari Narain	—	do
(k)	Indian Institute of Technology, New Delhi	—	Institution Member
(l)	Regional Research Laboratory, Jorhat, Assam	—	do

The Annual General Meeting of the Society will be held in November 1966 at Roorkee along with the Third Symposium on Earthquake Engineering. More information regarding this meeting will be available at a later date. Members are reminded that the last date for the receipt of papers to the III Symposium is may 31, 1966.

The papers published in this Bulletin are open for discussion upto 15th Sept. 1966. Contributions are invited on the following topics:—

1. Analysis of Structural Response and Design of Structures for Earthquake Forces.
2. Design of Dams and other Appurtenant Works in Seismic Zones.
3. Soil and Foundation Behaviour during Vibrations.
4. Seismicity, Wave Propagation and Ground Motion.
5. Instruments for Earthquake Engineering and Seismological Studies.
6. Geological studies of Tectonic Features influencing Occurrence of Earthquakes.
7. Recent Strong Earthquakes and Resulting Damage.
8. Housing in Seismic Zones.

THIRD SYMPOSIUM ON EARTHQUAKE ENGINEERING AT THE UNIVERSITY OF ROORKEE ROORKEE (INDIA)

November 4, 5 and 6, 1966

The University of Roorkee cordially invites engineers and scientists interested in the field of Earthquake Engineering and Seismology to participate in the Third Symposium on Earthquake Engineering in November 1966.

The Sessions are proposed to be divided broadly into the following themes:

I. **Analysis of Structural Response and Design of Structures for Earthquake Forces.**
II. **Design of Dams and other Appurtenant Works in Seismic Zones.**
III. **Soil and Foundation Behaviour during Vibrations.**
IV. **Seismicity, Wave Propagation and Ground Motion.**
V. **Instruments for Earthquake Engineering and Seismological Studies.**
VI. **Geological Studies of Tectonic Features influencing Occurrence of Earthquakes.**
VII. **Recent Strong Earthquakes and Resulting Damage.**
VIII. **Housing in Seismic Zones.**

It is planned to print all accepted papers prior to the opening of the first session of the symposium for distribution to all registered participants. Those interested are requested kindly to send their papers as early as possible.

All correspondence may please be addressed to **Dr. Jai Krishna, Professor and Director, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee U.P. (INDIA).**

The University of Roorkee will be grateful if the receivers of this circular will assist in bringing it to the notice of any person who is likely to be interested in the Symposium.

Note:—1. The paper should be limited to not more than 6000 words.
2. Last date for full papers to reach Roorkee is, May 31, 1966.
3. Early advice of papers would greatly assist the organisers in drawing up the final programme.

A NOTE ON VIBRATIONAL CHARACTERISTICS OF GROUND AT CHATRA, NEPAL

P. N. Agrawal*

SYNOPSIS

Seismograms of Chatra Observatory, Nepal for a period of three years were scrutinised. Mostly the periods of ground vibration were found to occur in three ranges, namely : 0·27 sec. to 0·3 sec., 0·32 sec. to 0·4 sec. and 0·5 sec. to 0·7 sec. This period response lead to the conclusion that the ground at Chatra consists of several layers. The depth of bed rock was calculated utilizing the quarter wave length law of K. Tazime and was found to agree with the results of geophysical investigations in that area.

INTRODUCTION

It has been realised for long that a ground which is good or bad from the static point of view, may not be so under vibrational conditions. Obviously the investigations of vibrational characteristics of ground are important for the design of earthquake resistant structures. This can be achieved by :—

(a) Interpreting seismograms suitably along with the theoretical studies of seismic waves.

(b) Seismic prospecting.

(c) Shaking ground by vibrating machines.

(d) Statistical studies of earthquake damage.

The results given in this paper are based on the study of seismograms of Chatra Observatory, Nepal for the period May, 1959 to April, 1962. The records available for this period were from the following seismographs :

1.	Milne Shaw No. 67297	$To = 12.0$ sec., $V_s. = 250$,	$\varepsilon = 20:1$
2.	Wood Anderson No. 5	$To = 1.0$ sec., $V_s. = 1000$,	$\varepsilon = 20:1$
3.	Benioff Short Period (Z)	$To = 0.72$ sec., $T_g. = 0.4$ sec. $\varepsilon_s = 2.3:1$	$\varepsilon_g. =$ Critical.

These seismograms were scrutinised for local shocks, microtremors and short period microseisms. The periods of P and S waves in the local shock records may be thought to be associated with the periods of ground vibration (Gutenberg 1957). In recent years microtremors have been utilized for such purposes (Kanai and Tanaka 1961). However, short period micro-

* Scientist, Regional Research Laboratory, Jorhat, Assam (India).

seisms have been found most advantageous in such studies (Akamatu 1961). Though these consist of random waves generated from various sources, both natural and artificial; but their periods are found to correspond with the periods of ground vibration. The periods of all the above movements on these seismograms were determined. These can be taken as the periods of ground vibration and can throw some light on the vibrational characteristics of ground at Chatra.

LOCAL SHOCKS

Milne Shaw seismograms contained very few local shocks (Fig. 1) and for these also the period of P and S group of waves could not be measured. Because the paper speed was only 16 mm per minute and the wave periods were less than 1 second. For most of the local shocks recorded by Wood Anderson Seismograph, the period of P and S group of waves could not be picked up as the P movements were small where-as after the onset of S the records were invisible due to the rapid movement of light spot. However, ten shocks (Fig. 2) were selected for the purpose. Benioff seismograph had recorded a good number of local shocks very well (Fig. 3). Thirty suitable shocks well scattered over the period were examined and P and S wave periods were determined.

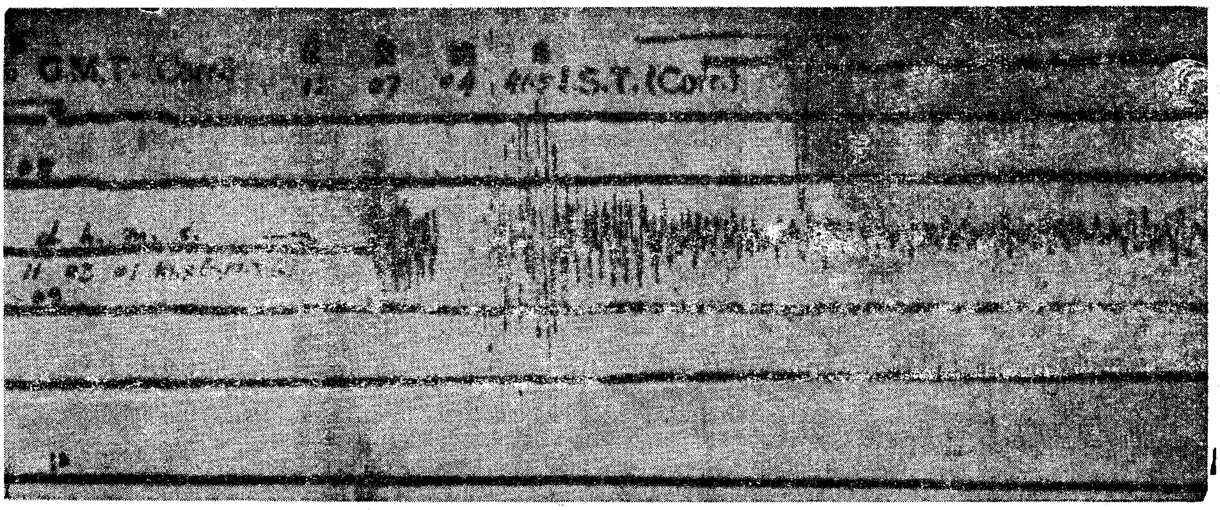

Fig. 1. Section of Chatra Milne Shaw Seismogram, 11–12 Jan. 1962

SHORT PERIOD MICROSEISMS AND MICROTREMORS

Benioff seismograph had very prominently recorded short period microseisms. Occasionally microtremors were also found to be well recorded. A scrutiny of these seismograms revealed that the microseismic activity had a marked daily variation during the period September–October to April–May (Fig. 4) whereas during the period April–May to September–October the daily variation of the same was very small and gradual (Fig. 5). The seismograms for the above two periods were very distinctly different in appearance. From year to year it

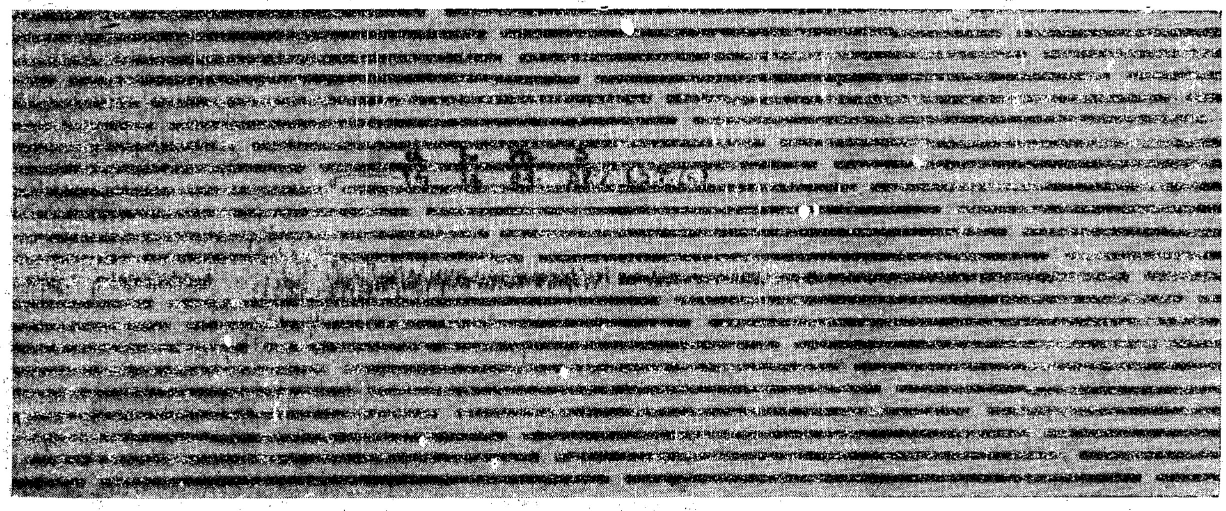

Fig. 2. Section of Chtra W. Anderson Seismogram, 16-17 Dec. 1961

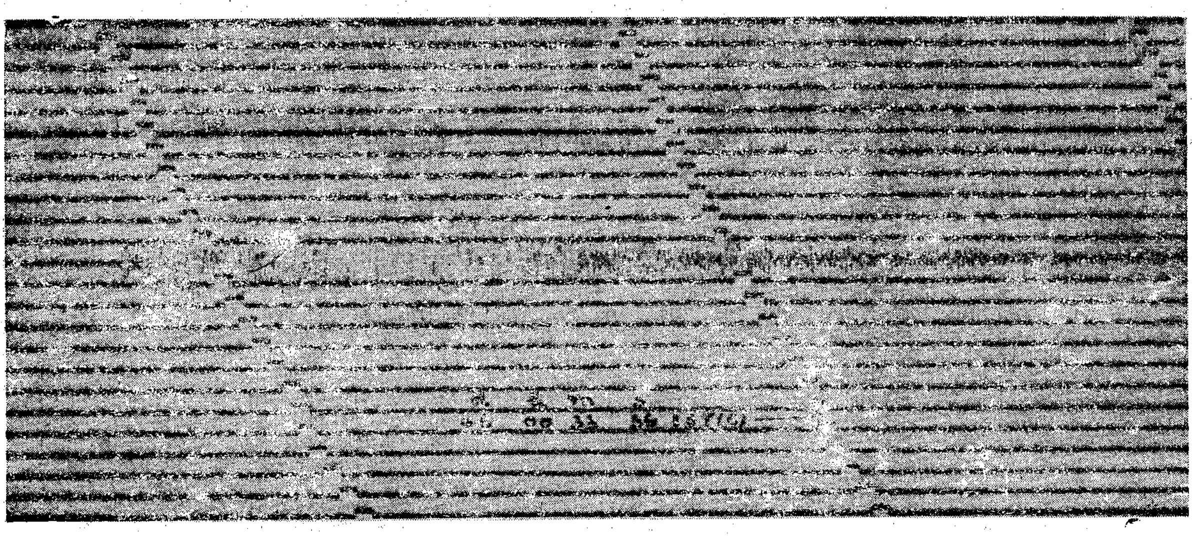

Fig 3. Section of Chatra Vertical, Short Period Benioff Record 05–06 May 1961

was found that the change from one kind of appearance of the record to the other was abrupt and not gradual. One typical day for each of the above periods has been taken and the maximum amplitude of the dominant period microseisms averaged over two minutes around every hour have been plotted so as to represent the daily variation in microseismic activity. Graph corresponding to first period (Fig. 6a) shows that nights were quiet whereas the days were disturbed, only one exception found to this was where the microseismic activity was more even during late evening hours. Corresponding synoptic conditions were examined. The microseisms present were of short period. But during the other period (Fig. 6b) this feature could not be clearly seen due to regular presence of long period microseisms. The nature of daily variations as represented by these two graphs could be very clearly seen on the daily charts

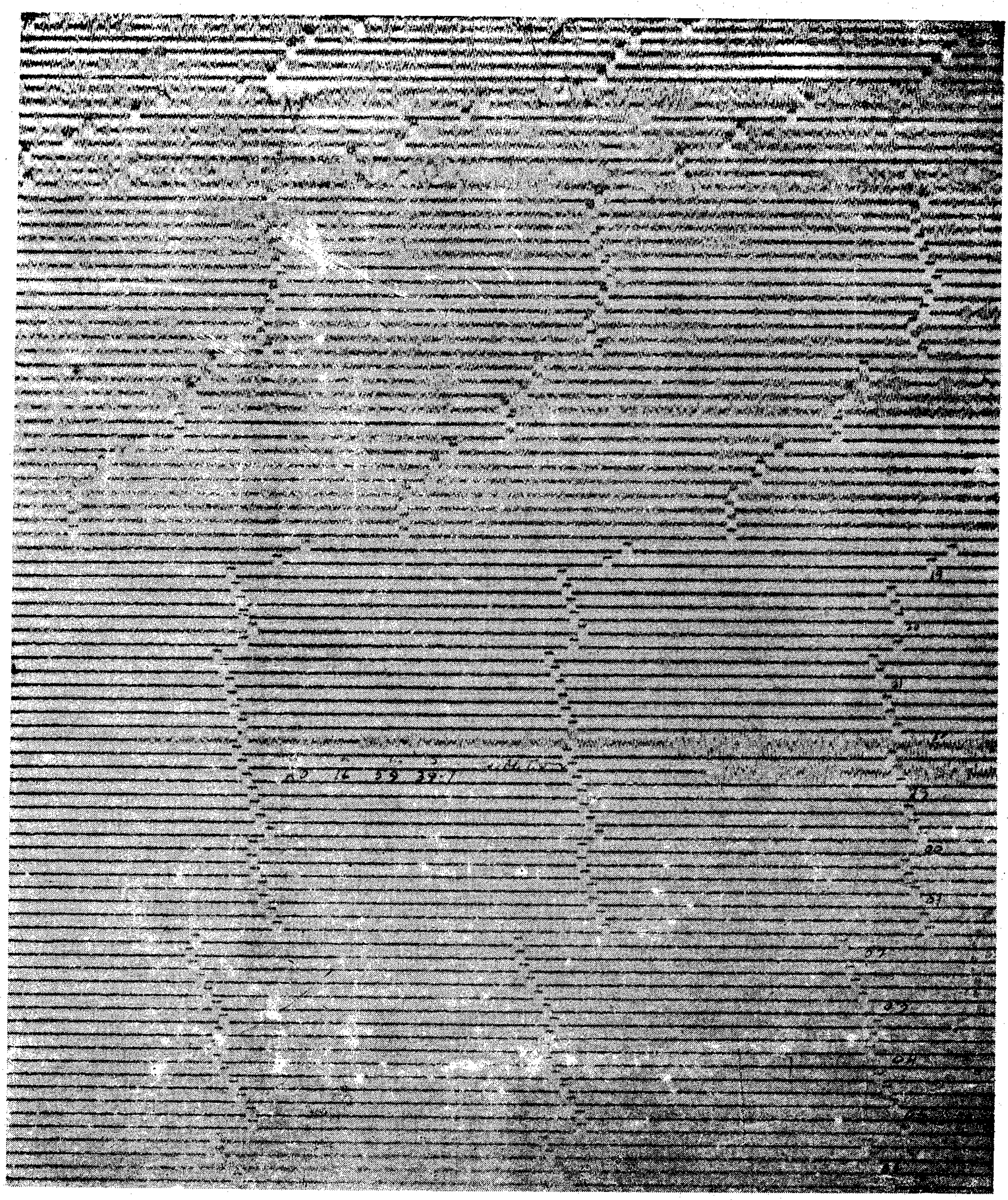

Fig. 4. Section Chatra Vertical, Short Period Benioff Record, 23–24 Nov. 1960

at a glance. Dominant wave periods prevailing in these microseisms of different kind and also microtremors were measured.

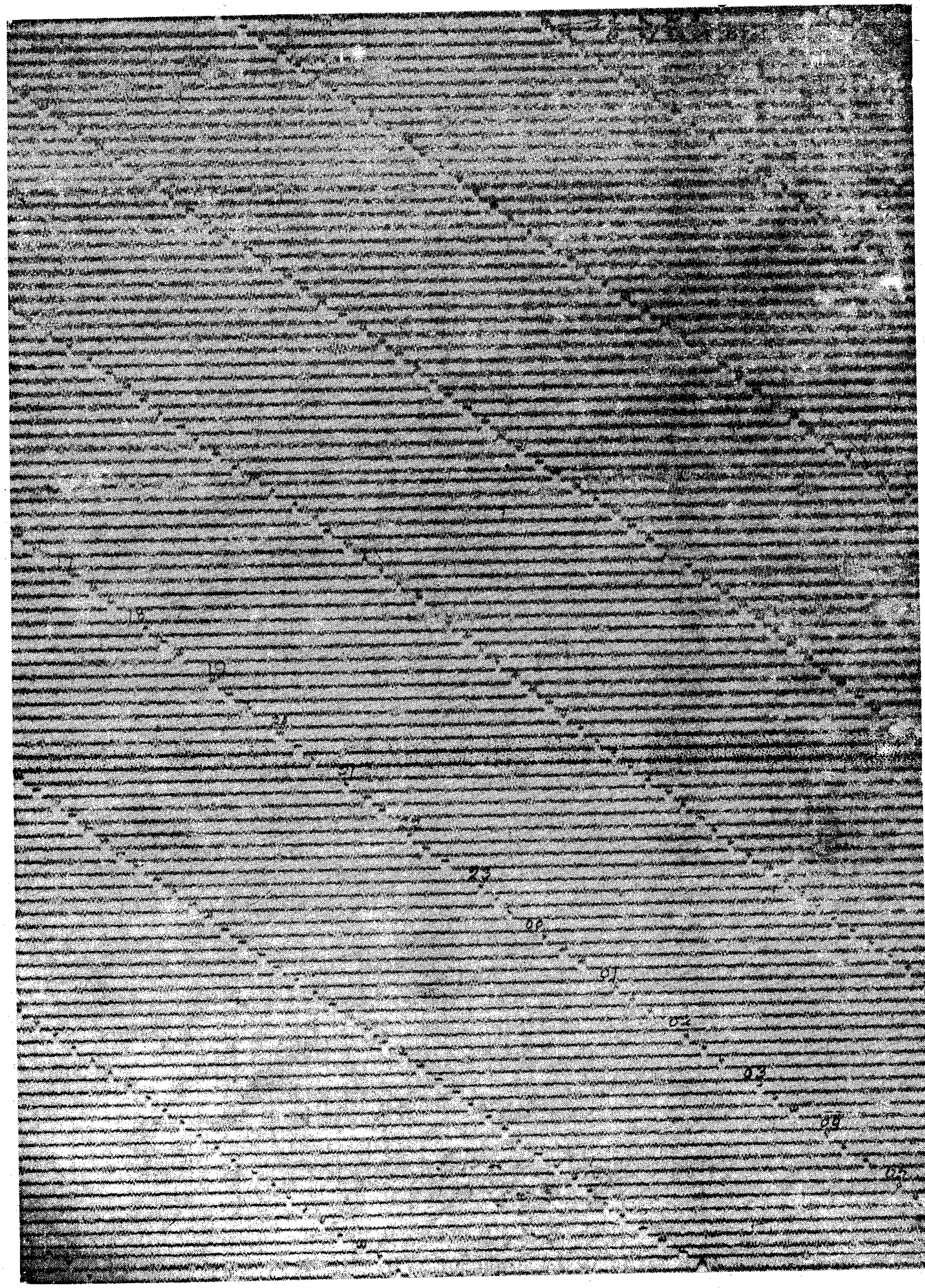

Fig. 5. Section of Chatra Vertical, Short Period Benioff Record, 30–31 July 1960

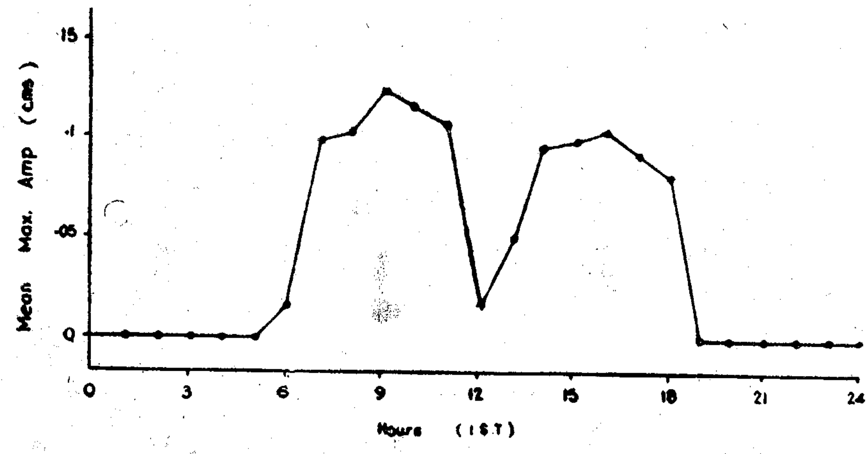

FIG - 6 (a) DATE 3rd OF DEC 1960

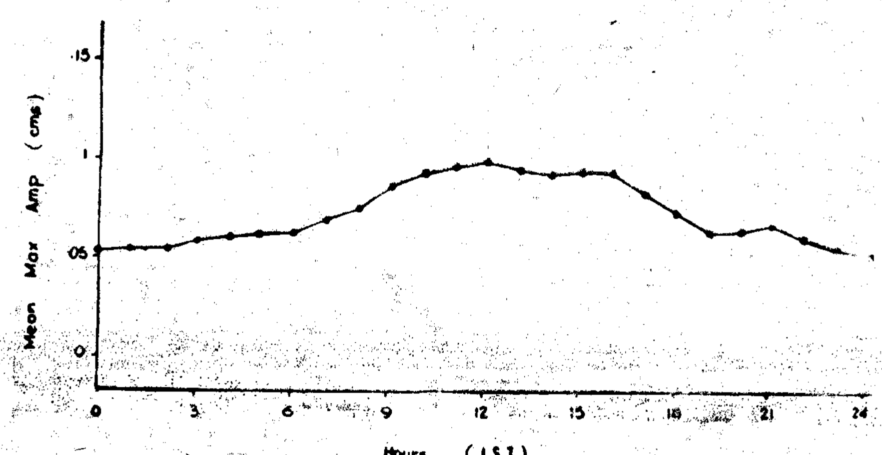

FIG - 6 (b) DATE 10th OF AUG. 1961

FINDINGS AND DISCUSSIONS

The wave periods of P and S movements of local shocks were found to lie between 0.5 to 0.7sec in general. The dominant periods of microseisms and microtremors appeard to be occurring in the ranges: 0.27 sec. to 0.30 sec., 0.32 sec. to 0.40 sec. and 0.50 sec. to 0.70 sec. The shorter periods mostly corresponded to the microsesims of day time. The appearance of these microseisms during the working hours of day time suggests that these might have been due to artificial disturbance of workings in the Kosi Project causing a thin surficial layer to vibrate. The sudden decrease in microseismic activity during mid-day may correspond to lunch break in the work. The occurrence of microseisms with greater periods during April-May to September - October might be associated with the increased flow of water in the Kosi river or some other natural cause like weather. Because. it has already been realized that the microseisms due to natural causes are of longer period than can be induced by artificial causes. Though the short period microseismic activity also appears to exist but it has been almost not seen due to the dominence of long period microseisms. The increased microseismic activity in late evening hours on a particular day during September - October to April - May, unlike the general trend might have been due to the thunderstorm activity over that region on that day as found by the examination of synoptic conditions.

Ordinarily, there are two kinds of spectral response of ground possible—one having a predominant period peak and the other having an irregular shape with number of flattened maxima. Theoretical study of Kanai (1957) indicates : in case of the predominant vibration appearing at the free surface, the surface amplitude seems to be concerned with the rigidity ratio of two layers; and values of rigidity themselves of the two layers have no connection with the predominant period amplitude. Also that as the number of layers increases the number of requisite conditions of the predominant vibration increases. In other words, the case of the predominant vibration of ground becomes rarer. The spectral response of amplitude of the surface layer is very irregular and maximum value of peak is not so large as in the case of single layer since the elastic wave reflections at the various boundaries interfere with one another. Interpretation of Chatra seismograms with this background of knowledge would indicate that the ground at Chatra seems to consist of a series of layers. The middle range of periods seems to be associated with the vibration of the total thickness of weathered layer. The shorter periods may correspond to the vibration of a thin layer of the overburden. According to Tazime (1959), $T = \frac{4\,H}{V_p}$ where T is the corresponding period, H is the thickness of the overburden and V_p is the longitudinal velocity in weathered layer. Taking mean (more common) T as 0.35 sec. for the middle range and the longitudinal velocity as 1270 ft./sec (Banerjee), the thickness of overburden comes out to be $H = \frac{V_p\,T}{4} = \frac{1270 \times 0.35}{4}$ ft. = 110 ft. approx., which is in fair agreement with the findings of Banerjee by geophysical investigations near Chatra.

CONCLUSIONS

1. The vibrational response of ground at Chatra as studied with the help of local shocks, microtremors and short period microseisms points out that it consists of several layers.

2. The predominent period of ground vibration was absent. The more common ground vibration periods ranges from 0.27 to 0.30 sec., 0.32 sec. to 0.40 sec. and 0.50 sec. to 0.70 sec.

3. The thickness of the overburden as calculated from the middle range of ground period is 110 ft. approximately and is in fair agreement with that determined by geophysical

investigations at Chatra. Hence it may be concluded that period 0.32 sec. to 0.40 sec. corresponds to the vibration of overburden.

ACKNOWLEDGEMENT

I wish to express my thanks to Dr. Jaikrishna, Director, School of Research and Training in Earthquake Engineering, University of Roorkee, as he had suggested the problem and gave encouragements during the preparation of this paper. I am also thankful to Dr. V.K. Gaur, Reader, Department of Geology and Geophysics, University of Roorkee, for his going through the manuscript.

Thanks are also due to Shri B.P. Saha, Meteorologist Incharge, Central Seismological Observatory, Shillong who extended all the facilities for the scrutiny of seismograms at Shillong.

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LESSONS FROM NIIGATA EARTHQUAKE

C.S. Jain*

SYNOPSIS

This paper enumerates some of the lessons we may learn from the damage done by an earthquake which struck the Japanese city of Niigata on 16-6-1964. The earthquake damage was characterised by its close relationship with the ground conditions, Tsunami and outbreak of fire. A study of damage pattern to the civil engineering structures reveals instructive information which can be helpful in the future planning, design and construction of earthquake resistant structures in similar areas. Some shortcomings in design and construction of the existing building structures have been pointed out and remedies suggested. The writer visited Niigata in Dec. 1964 and collected some material for this article.

INTRODUCTION

On June 16, 1964 at 1.02 p.m. Niigata a city of about 290,000 persons and Japan's largest petroleum and natural gas producing centre, situated on the Japans ea side 150 km from Tokyo was struck by an earthquake. It has a magnitude of 7.5 on the richter scale, focus 40 km below the earth surface; epicentre 70 km from Niigata and maximum acceleration of 0·16g as recorded at Niigata. This earthquake lasting only for 20 seconds claimed 15 lives, destroyed a large number of buildings and other structures, and caused damage to property in the region of 800 million dollars. The energy released by this earthquake is estimated to be equivalent to 500 bombs of the type which had destroyed Hiroshima

Niigata city is located at the edge of a low lying coastal plain on both sides of Shinano River, and the damaged zone mainly consists of loose uniform sand of fine and medium size, and in the wake of the earthquake portions of the city areas subsided varying in settlement upto a few feet. In this area the ground water-table is also high rising to as much as one meter below the ground surface in some locations. The vibratory response of the structures to the earthquake compacted the sand expelling out water, and setting the structure along with the subsidence of the ground. Similarly the vibrations of the ground resulted in a large number of sand blows. (See Photo 1)

CONSTRUCTION ON SANDY STRATA

The transient state of the phenomena termed Liquefaction of sand reduced the bearing

* Assistant Director, Central Water and Power Commission, New Delhi.

Pho o No 1 "Sand Blows" at Niigata

capacity of the soil due to increase in soil pore pressure, and consequently a number of structures sank into the ground. A group of gasoline pumps had sunk into the liquified sand under their own weight so that only the inclined tops remained visible. Thus it is seen that loose sandy soil is very susceptible to shock vibrations as compared to harder strata in the same area.

To mitigate the damage on such soils due considerations should be given for stabilisation of loose soil before starting the construction work.

FOUNDATION CONSIDERATIONS

At Niigata it has been observed that the buildings having basements comparatively suffered less damage than those without basements. This is illustrated by the tilt in some of the buildings in a group of four storeyed reinforced concrete apartments one km west of the prefectural office, where two apartments having basements suffered little tilt. Buildings having pile foundations behaved better than those without piles. For example four storeyed main railway station of Niigata was undamaged as it was founded on piers. Again, structures with both the basements and piles sustained almost no earthquake damage. The two prefectural offices consisting of four storeyed and six storeyed, and having basements as well as pile foundations did not suffer any damage even though the areas around these buildings showed settlement and distortion. Also buildings having mat foundations suffered less damage than those having individual or strip foundation.

Therefore in the planning and design of earthquake resistant structures provision for basement and pile foundation should be given adequate consideration. Photo II shows a overturned apartment exposing the bottom stones of the foundation and indicating the inadequate depth of foundation.

BRACINGS AND SYMMETRICAL LAYOUT

At Niigata it was observed that the interior of many affected buildings was distorted and walls collapsed though the roofs did not suffer any damage. Hence it is important that the main structural parts of a building such as walls, cornices, parapits, columns etc be tied to the interior frame-work and to the foundation from the lowest floor to the top floor, giving the power or resistance to the buildings as a whole.

The geometric shape of the building is also imp rtant. Structures having rectangular cross section, vibrating in a unit without interface from other structural parts withstand the shock vibration best. In the case of a building consisting of two or more irregularly shaped portions, the earthquake response of the building is different in different parts and consequently may tend to knock against each other. This is supported by the fact that at Niigata the damage to many buildings was limited to entrance canopies and connecting links.

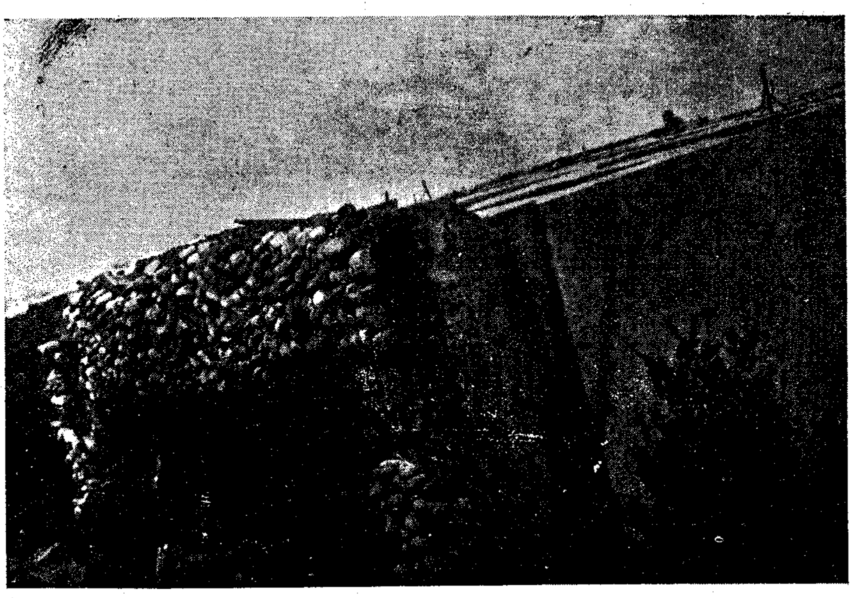

Photo No. 2 Overturned Apartment at Niigata Showing the Foundation Stones

CONSTRUCTION MATERIALS

It is not merely the apparent strength of a structure but rather its true strength in relation to its weight which controls its earthquake resistance. This may perhaps be the reason for comparatively lesser damage to lighter wooden old shrines. At Niigata in most cases it was observed that the heavier of two adjacent buildings had settled most, and the lighter buildings had inclined toward the heavier ones. Thus the buildings constructed with lighter materials are better. Mathematically the period T of vibration of a structure of weight w is given by the relation:

$$T = 2\pi \sqrt{w/gk}$$

This shows that the weight of a building affects the natural period of vibration directly.

Also earthquake resistant structure should be constructed of ductile materials that are strong and have energy absorbing qualities which can withstand dynamic forces without fracture, since stiff and brittle members cannot withstand the vibrations of a severe earthquake. At Niigata this has been demonstrated in some buildings which did not seem to suffer complete collapse as the comparatively ductile steel constituting the side walls had simply bent out. A properly designed structure built of ductile material can absorb a large amount of energy in plastic deformation and hence it is not necessary that a structure must behave elastically in the maximum designed earthquake.

The importance of providing ductility in a structure is twofold. Firstly that when the material is strained in the plastic range due to the earthquake shock the strain energy absorption is much more than that in the plastic range for the same increase of strain and consequently is a source of extra natural damping. Secondly, the failure of the structure is not sudden thus giving sufficient warning before ocmplete collapse thereby mitigating the possible loss of life and property due to an earthquake.

TSUNAMI DAMAGE

A small tidal wave hit Japan sea coastal area less than thirty minutes after the earthquake which caused backflowing of the river Shinano and flooding of the low-lying regions, causing severe damage to life and property. A stronger embankment on both sides of the river could have mitigated the loss. In some cases the onrush of the Tsunami is preceeded by a clearly visible withdrawal of seawater for a short period which may be taken as a warning by the people to rush to the higher grounds.

DESTRUCTION BY FIRE

In the wake of the earthquake fire broke out when a tank of Showa Oil Co. containing 40,000 kilo-litre crude oil blew up. Soon it spread out to more than 80 tanks containing crude and refined oil and damaged more than two hundred buildings. The fire raged for about thirty hours before it could be brought under control by dropping chemical foam bombs from the air. The cause of the fire is not definite, but the fire disaster highlighted the effect of a big earthquake on oil tanks. In this connection three points need considerations. Since the oil refinery and petrochemical plants are completely automatic in Japan it needs investigation whether the automation devices can set off the necessary emergency controls as efficiently as a trained staff before installing such systems in an area vulnerable to earthquakes. Again at Niigata near the oil tanks there were other tanks containing tetra-chloride ethyl and methonal which give poisonous gases and explosive hydrogen gas. Hence the second debatable aspect is the wisdom of locating such tanks in a factory area. Thirdly the non-availability of necessary chemicals and other fire fighting equipment and the inadequate capacity of the available protection devices to deal with the initial outbreak of fire focuses the importance of adequate and quick fire protection equipment near such places.

CONCLUSION

In conclusion it may be pointed out that the intensity of ground motion at Niigata was much less in comparison to the severity of damage to the structures. This accounts for the fact that the damage was not due to the vibrational effects of structures but due to the deformation and decrease in the bearing power of the soil. Though the structures were made sufficiently strong, proper consideration was not given to the foundation requirements, in the soil which could then liquefy due to an earthquake. The damage at Niigata brings to light the dangers inherrent in constructing Engineering structures on fills or reclaimed land. The lessons of this catastrope can well be kept in mind when plans for the construction in similar regions are drawn.

REFERENCE

Falconer, (1965), "Immediate Report on Inspection mission to Niigata City", Niigata earth Earthquake, I.I.S.S.E, Feb. 1965.

Paper No. 33

STATIC AND DYNAMIC BEHAVIOUR OF A LATERALLY LOADED PILE

by

Shamsher Prakash* and S.L. Agarwal**

SYNOPSIS

The behaviour of a vertical pile subjected to static, steady State dynamic loading both at the ground level and above and under transient loading at ground level was studied on aluminium model piles embedded in sand. The steady state dynamic load was applied through vibration table which could be excited with known amplitude and frequency of vibration. The transient load was applied by another shaking table excited at a constant frequency. The load was measured by a proving frame on which electric resistance strain gauges had been mounted. The effect of both transient and steady state loading on the displacement has been studied and compared with the static case.

INTRODUCTION

Large lateral loads act on piles when these are used as support for retaining walls, bridge abutments, piers, fenders, dolphins and anchors for bulk-heads and water-front and off-shore structures. Dynamic loads may be caused because of wave action, impact of ships, earthquakes and machines.

Figure 1 A illustrates the external loads applied to the pile at ground level, while in Figure 1 B the loads are applied above the ground level. P represents the vertical load, while Mg and Q_{hg} represent the moment and horizontal shear respectively at the ground surface. Ms represents the moment produced by rotational restraint supplied by the structure and Q_{hs} represents the horizontal shear force when applied above the ground level. Figure 2 (a) illustrates the deflected shape of the embedded portion of the pile. The pile has a modulus of elasticity E, moment of inertia I and width B. An approximate soil reaction diagram has also been shown. Figure 2 (b) shows deflections of a pile which are due to rigid body rotation and its flexural bending.

An investigation of piles subjected to lateral loads involves the study of interaction between the pile and the foundation soil. Most of the analysis of laterally loaded piles involve

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

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

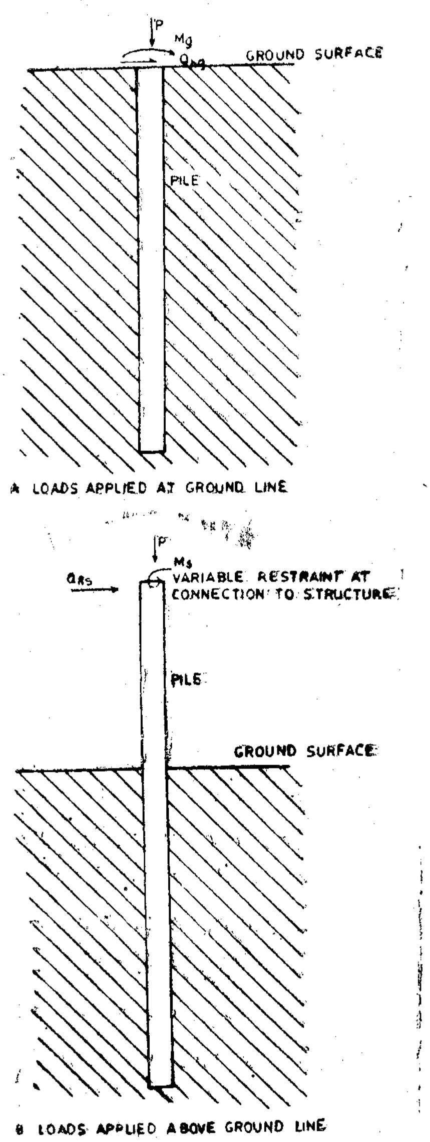

Figure 1 External Loads Applied to Pile

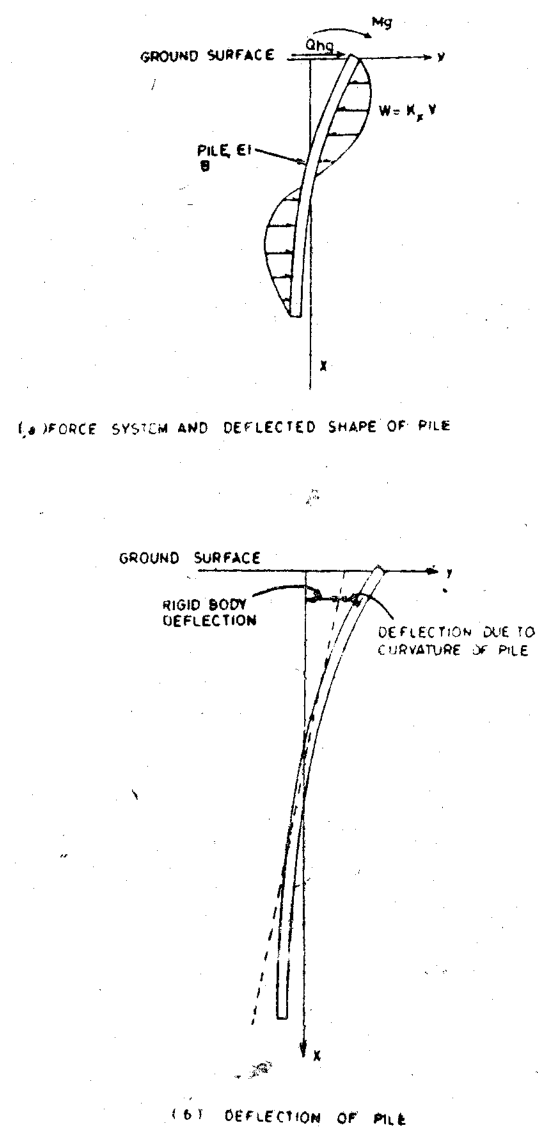

Figure 2 System of Forces and Deflected Shape of Pile

the concept of modulus of subgrade reaction based on Winkler's (1867) assumption, which states that soil strata may be approximated by a series of infinitely closely spaced independent and elastic springs. Spring stiffness k, khown as coefficient of subgrade reaction is defined as

$$k = \frac{w}{y} \tag{1}$$

where,

w = soil reaction at any point and

y = deflection at that point

Soil type and size of loaded area mainly influence k. Terzaghi (1955) suggested constant k for piles in over consolidated clays and k linearly varying with depth for sands,

i.e. $k_x = n_h . x$ (2)

where,

k_x = value of k at depth x and

n_h = constant of horizontal subgrade reaction.

The problem of laterally loaded piles under static conditions has been studied to a considerable extent by Reese and Matlock (1956), Davisson (1960), Broms (1964 b) and Aggarwal and Soneja (1965) for k varying linearly with depth and Broms (1964 & 1965) for k–constant with depth. Davisson considered effect of vertical loads also. Prakash (1962) investigated the behaviour of pile groups subjected to static lateral loads in sands. Davisson and Gill (1963) solved this problem for constant k for two layered medium. Prakash and Subramanyam (1964 a, b) studied the behaviour of battered piles under lateral loads.

According to these analyses, the factors governing the displacement of a pile are :—

(i) Type of soil,

(ii) Soil properties, e.g. density, moisture content etc.

(iii) Variation of soil modulus,

(iv) Pile properties, e.g. pile material, shape, width, length, flexural stiffness etc.

(v) Shear and moments etc acting on the pile.

(vi) Pile end conditions, free or restrained.

(vii) Method of soil and pile placement e.g. driving, jetting in the field.

(viii) Relative stiffness factor T, defined as

$$T = (EI/n_h)^{1/5} \tag{3}$$

for k linearly varying with depth. A non-dimensional coefficient,

$$Z_{max} = \frac{L_s}{T} \tag{4}$$

where, L_s = Length of embedded portion of the pile,

determines the different solutions, which have been presented in the form of non-dimensional curves (Reese and Matlock 1956, Davisson 1960). A pile is considered rigid if $Z_{max} \leqslant 2$ and infinitely long if $Z_{max} \geqslant 5$.

The dynamic lateral loads may be caused because of earthquake, wave action and in machine foundations. Experimental data on actual piles is difficult to obtain because of cost considerations. Model studies have been reported by Gaul (1958), Hayashi and Miyajima (1962) and a few others. A critical review of their work has been presented elsewhere

(Prakash and Agarwal 1965). The information is far from being complete and there is considerable scope for fruitful research in this direction.

In this paper, investigation carried out on a vertical pile under static, steady state dynamic and transient loads has been reported. The lateral load was applied both at and above the ground level on a free head pile. The effect of vertical load will be included in a subsequent investigation.

The tests were performed on a model aluminium pile 15 mm in outside diameter and 2.5 mm wall thickness. The length of the pile (60 cm) was so chosen that it corresponds to a usual pile length in the field i.e. $Z_{max} \geqslant 5$. The pile was placed in a steel tank of 60 cms $\times$ 30 cms $\times$ 61 cms high and sand placed around the pile. The sand was compacted to produce homogeneous soil density. Steady state dynamic load was applied through a vibration table which could be excited with known frequency and amplitude of vibration and transient load was applied through a free vibrating table having constant frequency of oscillation. The load was measured by means of proving frame on which electric resistance strain gauges were mounted.

A comparison has been made between the behaviour of pile under dynamic and static case.

EXPERIMENTAL SET UP

General

The variables in the study of single vertical pile subjected to lateral loads have already been enumerated. For dynamic study, the frequency and amplitude of loading will also govern the behaviour. In this study, a single vertical aluminium pile was embedded in medium sand and static and dynamic loading had been applied both at and above the ground surface.

Soil and Pile Properties

The sand used was medium Ranipur sand with a relative density of 0.584 during the tests (Prakash and Agarwal 1965).

Aluminium pipe section was adopted since strains and deflections at the same stress level are large as compared with those of steel. Other properties of the section were as follows :

(a) Section No. 9442 (ALIND)

(b) Outside Diameter 15 mm

(c) Inside Diameter 10 mm

(d) Stiffness EI 13.7×10^4 kg/cm²

4.77×10^4 psi

Tank

The size of the tank should be large enough so that there is no interference of the walls on the behaviour of pile. At the same time it should be small enough so that volume of sand to be handled each time is not too much.

Preliminary tests under dynamic loading showed that the zones of influence may extend to a maximum of 30 times the pile width in the direction and 15 times its width perpendicular to the load. These spacings are eight to twelve and three to five times the corresponding values for static loading (Prakash 1962). A tank having dimensions of 60 cm × 30 cm × 61cm high was chosen to accommodate even small groups piles:

Loading Device

(a) Static loading was applied by means of a string passing over a ball bearing pulley.

(b) Steady state dynamic loading was applied through a horizontal vibration table† which could be excited with amplitudes of 0.8 mm, 1.7 mm, 2.80 mm, 3.73 mm, and 4.65 mm and at frequency of 0-1000 rpm. The motion of the table does not follow a true sinusoidal path.

(c) Transient loading was applied through a wooden table 170 cm × 120 cm × 9cm thick, mounted on four m.s. rollers 6 cm in diameter and attached to a fixed support by means of a spring. The natural frequency of the table could be varied by using different springs, while the amplitude of motion could be varied by striking the table with hammer having variable fall. In order to attain an amplitude of about 5 mm, the appropriate spring was selected. The natural frequency of the table was close to 2 cps.

Dynamic Load Measurement

The dynamic load was measured by means of a proving frame on which electric resistance strain gauges had been mounted. It was connected to the shaking table on one side and the pile on the other, by connecting rods, adaptors and clamps. The frame was designed based on the criterion laid by Casagrande and Shannon (1948). It was made of spring steel with four Rohit's (India) Electric resistance strain gauges mounted on three of the inner sides (Figure 3). The frame was calibrated both under tension and compression, with strain (ε), deflection (y_{pf}) of the proving frame from its initial position and number of chart lines (y_{pc}) recorded on a Brush two channel oscillograph. Calibration with strain was done to check the linearity of the frame and the other calibration (Figure 4) is required for interpretation of load and displacement of pile (See next section).

The natural frequency of the frame was found to be close to 175 cps. The maximum

† For details, refer to Annual Report 1960-61, School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, U.P. (INDIA).

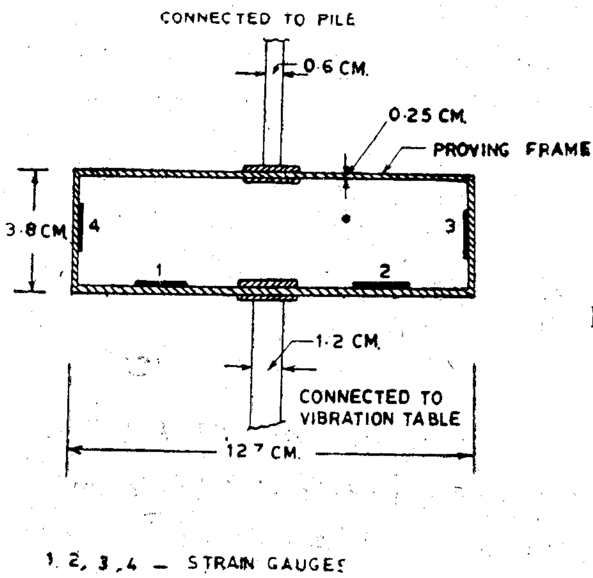

Figure 3 Proving Frame

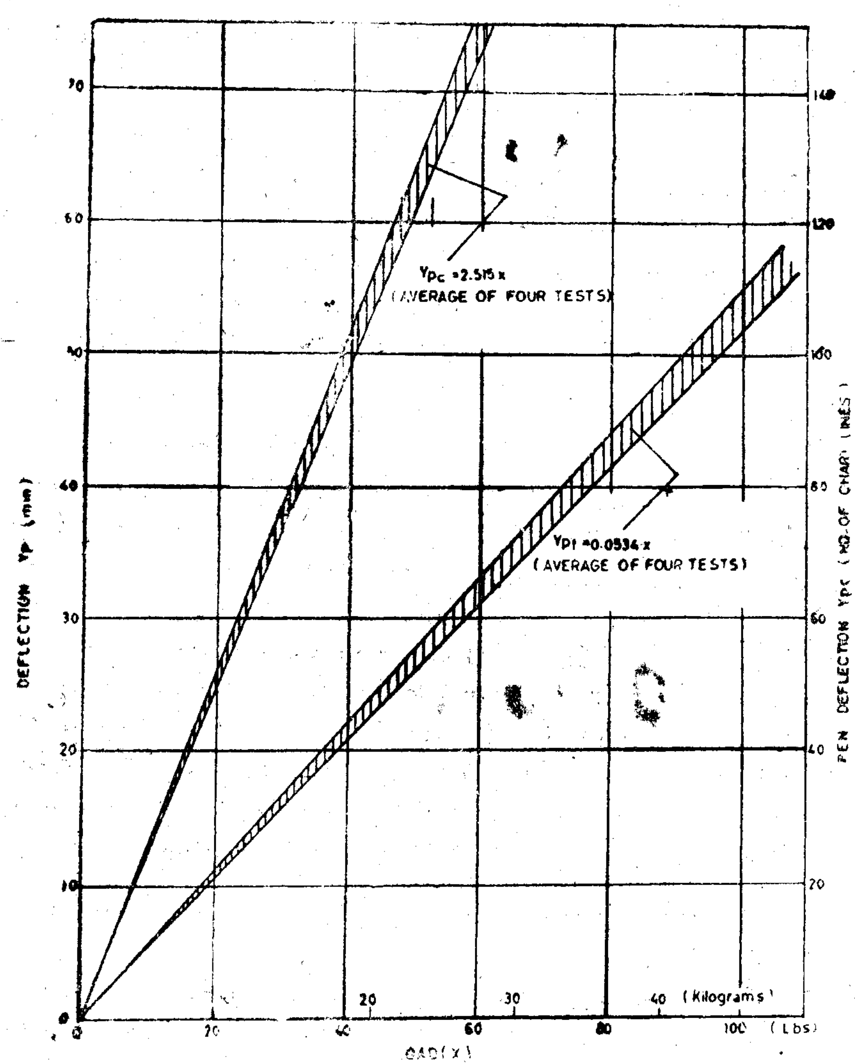

Figure 4
Calibration of Proving Frame

frequency of loading was 10 cps. Therefore, the natural frequency of the proving frame was about 17.5 times that of load application against 20 recommended.

Measurement of Pile Displacement.

(a) Static Test—

The displacement of the pile was measured by a dial gauge abutting against a flat connected to the pile at the ground level.

(b) Steady State Dynamic Test—

Displacement of the pile was obtained by subtracting the deflection of the proving frame (y_{pf}) from the displacement of the vibration table (y_t), Figure 5, when the load was applied at G.L. The displacement of the table motion was set at a known value. The deflection of the proving frame was determined with the help of Figure 4. In case the load was applied above the ground level, a relationship between displacement of pile (y_l) at load level and displacement of pile (y_g) at ground level was established (See Test Procedure). Figure 6 shows the assembly diagram of tank and pile.

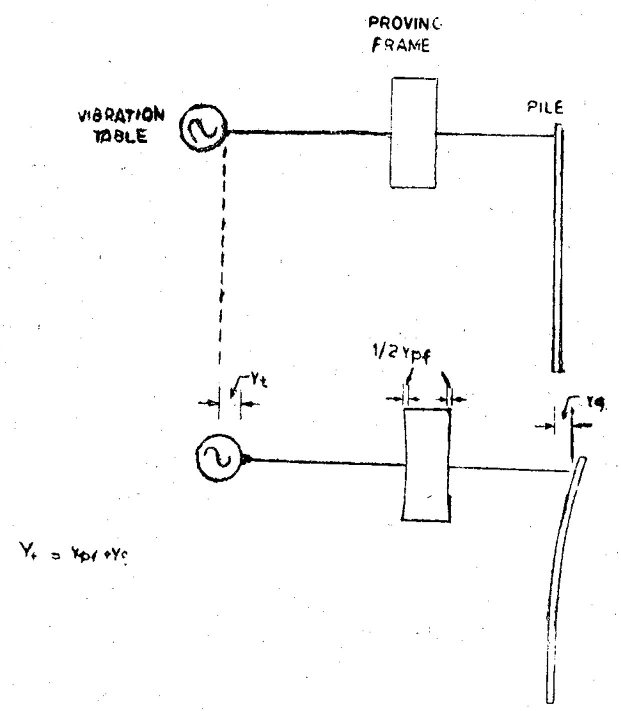

Figure 5 Method of Determining Pile Displacement (y_g)

(c) Transient Test—

The displacement of the vibration table was measured by a CEC vibration pick-up and read on a vibration meter. The natural frequency of the table was 2 cps. As the vibration meter was calibrated from 5 to 1000 cps, the displacement pick-up was calibrated on a small shaking table at frequency of 2 cps (Agarwal 1964).

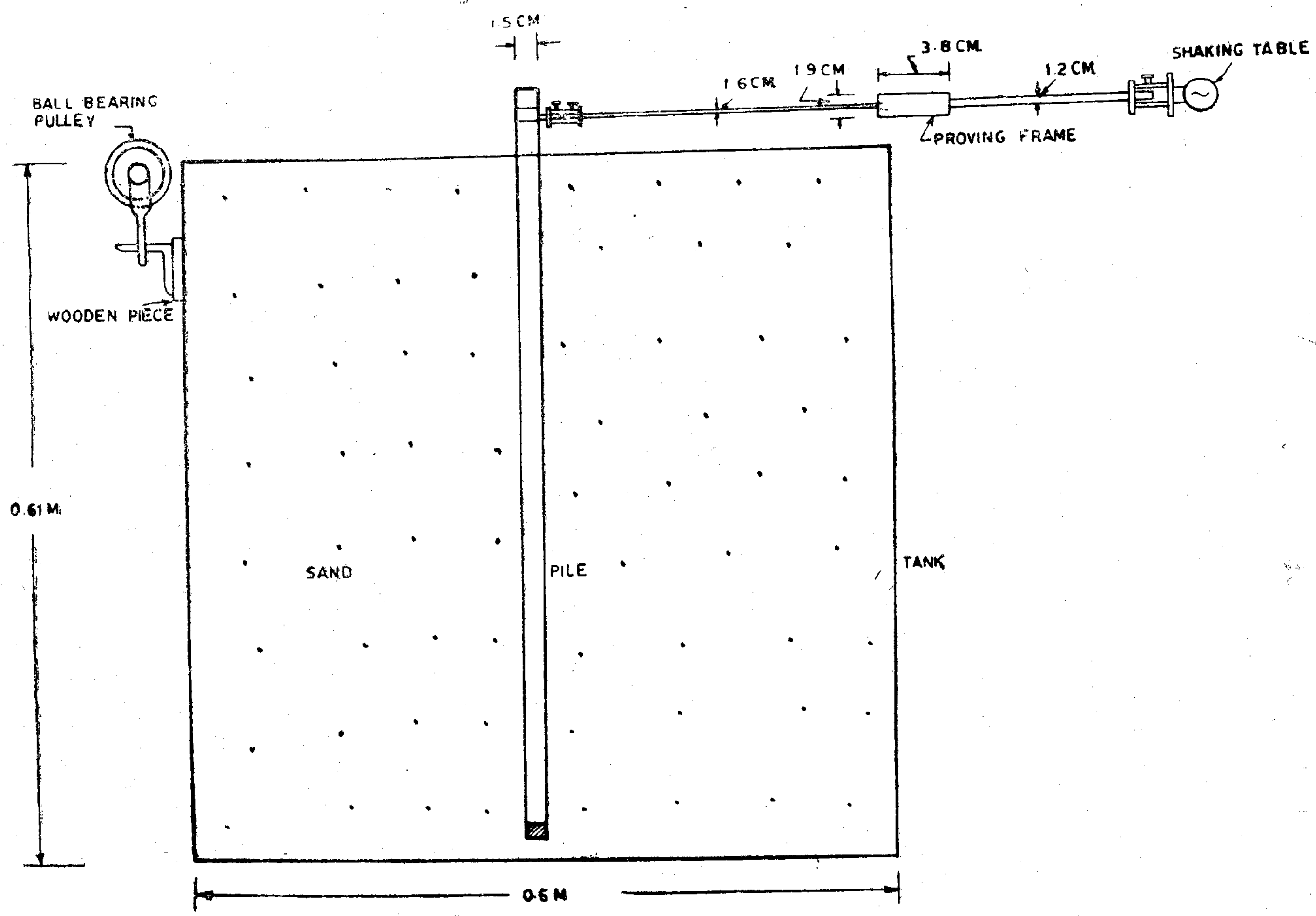

Figure 6 Pile Tank Assembly with Necessary Connections

TEST PROCEDURE

Placement of Sand and Pile

The Sand was placed in 10 cm thick layers and each layer was compacted 300 times with the standard Proctor's hammer. The pile was held in position when the sand was being placed.

Test Procedure—

(a) Static Loading—

Before dynamically loading, the pile was first tested under static loading. Two cases are of interest.

(i) Load applied at Ground Level—The displacement was measured by a dial gauge. Each load increment was allowed to stand for five minutes before deflection was recorded. The procedure was continued upto 40 lbs load.

(ii) Load applied above Ground Level—Displacement was measured both at ground level and at load level, by two dial gauges.

In order to establish a relationship between displacement at load level (y_l) and displacement at ground level (y_g), a plot was made between y_l and y_g obtained during all the static tests of each series. A straight line was drawn which established the relationship between y_l and y_g. Similar relationships were established for all Test Series 2 to 5 which are as follows :—

Load Position	y_l/y_g
G.L.	1.00
2.0 cm above G.L.	1.15
3.0 cm above G.L.	1.20
4.0 cm above G.L.	1.35
5.0 cm above G.L.	1.54

The ratio of y_l/y_g was used to determine y_g from y_l, values obtained directly in dynamic tests.

(b) Steady State Dynamic Loading—

The pile was connected to the steady state vibrating table as shown in Figure 7. The table amplitude of 0 8 mm (1.60 mm peak to peak) was adjusted. The lead wires of the strain gauges No. 2 and 4 were connected to the universal amplifier, which was in turn connected to the Brush oscillograph. The table was then excited with frequency increasing from 0–10 cps, and record of load obtained at almost equal increments of frequency. A record at the same frequency was obtained while decreasing the frequency from 10–0 cps.

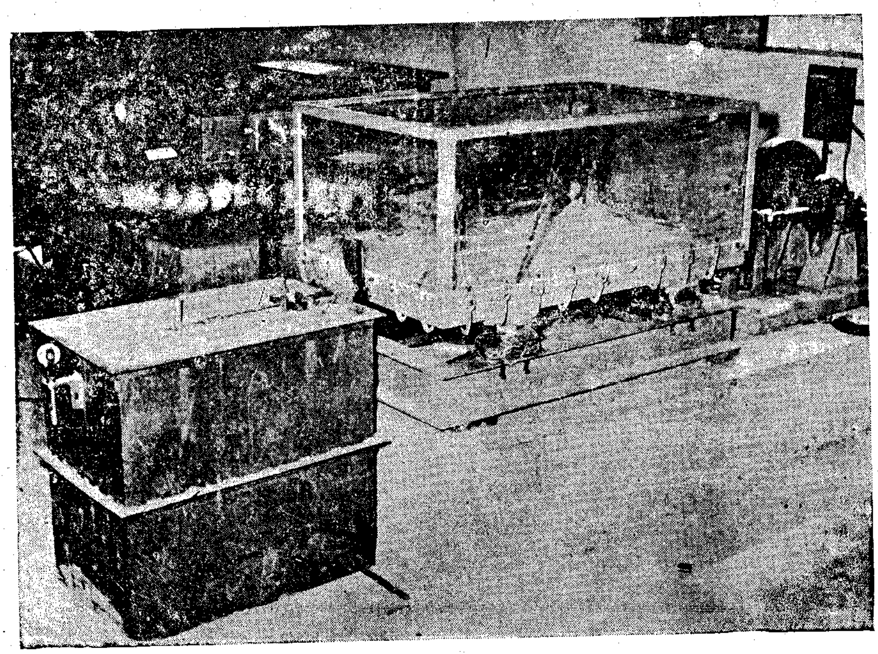

Figure 7 Pile Connected to Steady State Vibration Table

The pile was then tested under static loading. After this test, the table amplitude was set at 1.77 mm and dynamic test performed. Thus one test series consisted of the following tests :—

(i) Static test.

(ii) Dynamic test with table amplitude 0.8 mm.

(iii) Static test.

(iv) Dynamic test with table amplitude 1.77 mm.

(v) Static test.

(vi) Dynamic test with table amplitude 2.8 mm.

(vii) Static test.

(viii) Dynamic test with table amplitude 3.73 mm.

(ix) Static test.

(x) Dynamic test with table amplitude 4.65 mm.

(xi) Static test.

Every static test took about 1 hour and dynamic test about 10 minutes. When the load was applied above the ground level, the same procedure was followed.

(c) Transient Loading—

The pile was connected to the transient vibration table, Figure 8. The displacement pick-up and strain gauges of proving frame were connected to suitable recording devices. The table was first struck gently with a hammer and displacements and loads recorded. Subsequently, it was struck with increasing forces and the corresponding deflections and loads

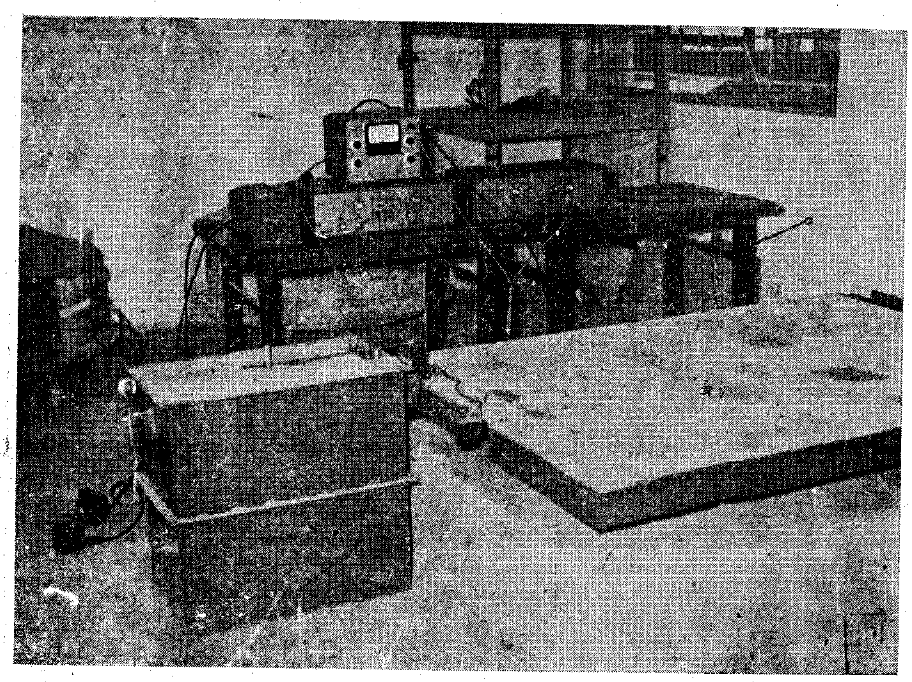

Figure 8 Pile Connected to Transient Vibration Table

recorded. The maximum load applied was about 45 lbs. After the transient test, a static test was also performed. Thus one series of test under transient load consisted of the following :—

(a) Static test in the beginning.
(b) Test under transient load.
(c) Static test at the end.

Tests Performed—

Tests performed included :—

(a) Test Series No. 1 Load applied at the ground level.
Test Series No. 2 Load applied at 2.0 cm above the ground level.
Test Series No. 3 Load applied at 3.0 cm above the ground level.
Test Series No. 4 Load applied at 4.0 cm above the ground level.
Test Series No. 5 Load applied at 5.0 cm above the ground level.

Test Series No. 1 and 5 were repeated to check the reproducibility of the data.

(b) Transient Loading—

Test Series No. 6—Load applied at the ground level.

Depression Around the Pile

During Test Series No. 1 a depression in the soil in the immediate vicinity of the pile was observed at table amplitude of 1.77 mm. Similar depressions were observed at higher table amplitudes, their dimensions being as follows :—

	Size of Depression	
Table amplitude (y_t) mm	Diameter cm	Depth cm
1.77	4.0	3.0
2.80	5.0	3.5
3.73	6.0	4.0
4.65	7.0	4.5

Soil Column Vibrating with the Pile

In Test Series No 1 to 5, a soil column could be seen vibrating with the pile. The dimensions of this column (Figure 9) elliptical in section were 39 0 cm. in the direction of loading and 21.0 cm. perpendicular to it in Test Series No. 5 when the table amplitude was 4 65 mm.

Natural Frequency of Soil-Pile System

The pile while embedded in sand was disconnected from the vibrating table and a CEC vibration (displacement) pick up was connected rigidly to the pile. To determine the natural frequency of the system, the pile was struck gently with a rod and the record of the pick-up signal indicated a frequency of about 50 cps.

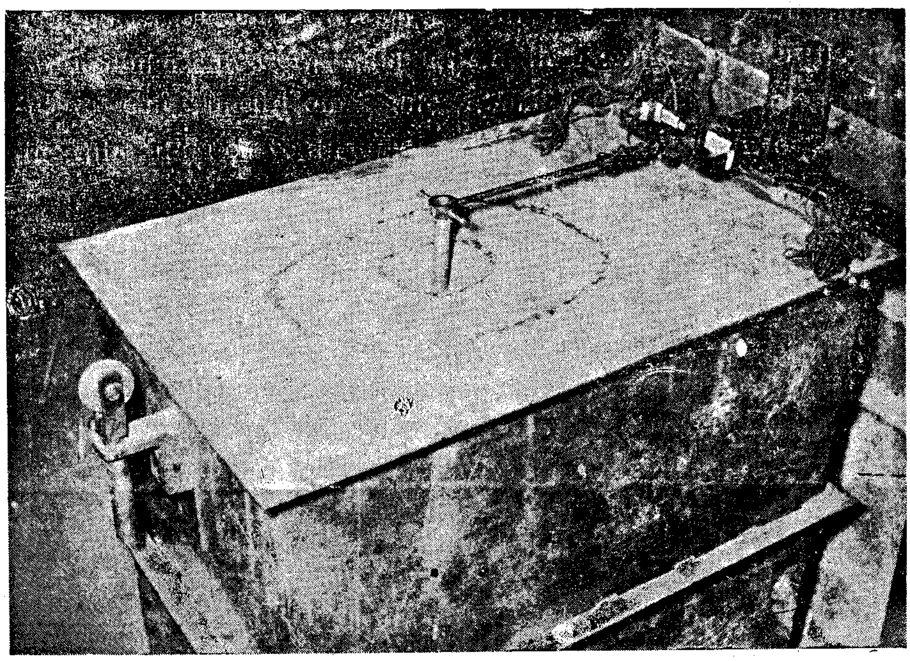

Figure 9 Zone of Influence for Pile Under Dymamic Lateral Loads

RESULTS OBTAINED

Static Tests—

Figure 10 illustrates Load-Displacement at ground level (y_g) curves for all the six static tests of Test series No. 1, when the load was applied at ground level. Figure 11 is a similar diagram for test series No. 5, (Load applied at 5.0 cms above the ground level). Figures for Tests No. 2, 3 and 4 have not been included for want of space.

Steady State Dynamic Tests

(a) Load applied at G. L.

Figure 12 is a plot of frequency and load at constant table amplitude of 0.8 mm, 1.77 mm, 2.80 mm, 3.73 mm and 4.65 mm respectively. The full lines and points indicate that the frequency is increasing while the dotted lines stand for decreasing frequency. Figure 13 contains Load Displacement (y_g) curves with frequency of oscillation of 2, 4, 6, 8 and 10 cps along with similar curves for static tests in the beginning and at the end. At any frequency, the load was determined from Figure 12 and deformation of the proving frame was determined from the calibration curve, Figure 4. The difference of the two gave displacement at G.L. i.e.

$$y_g = y_t - y_{pf} \tag{5}$$

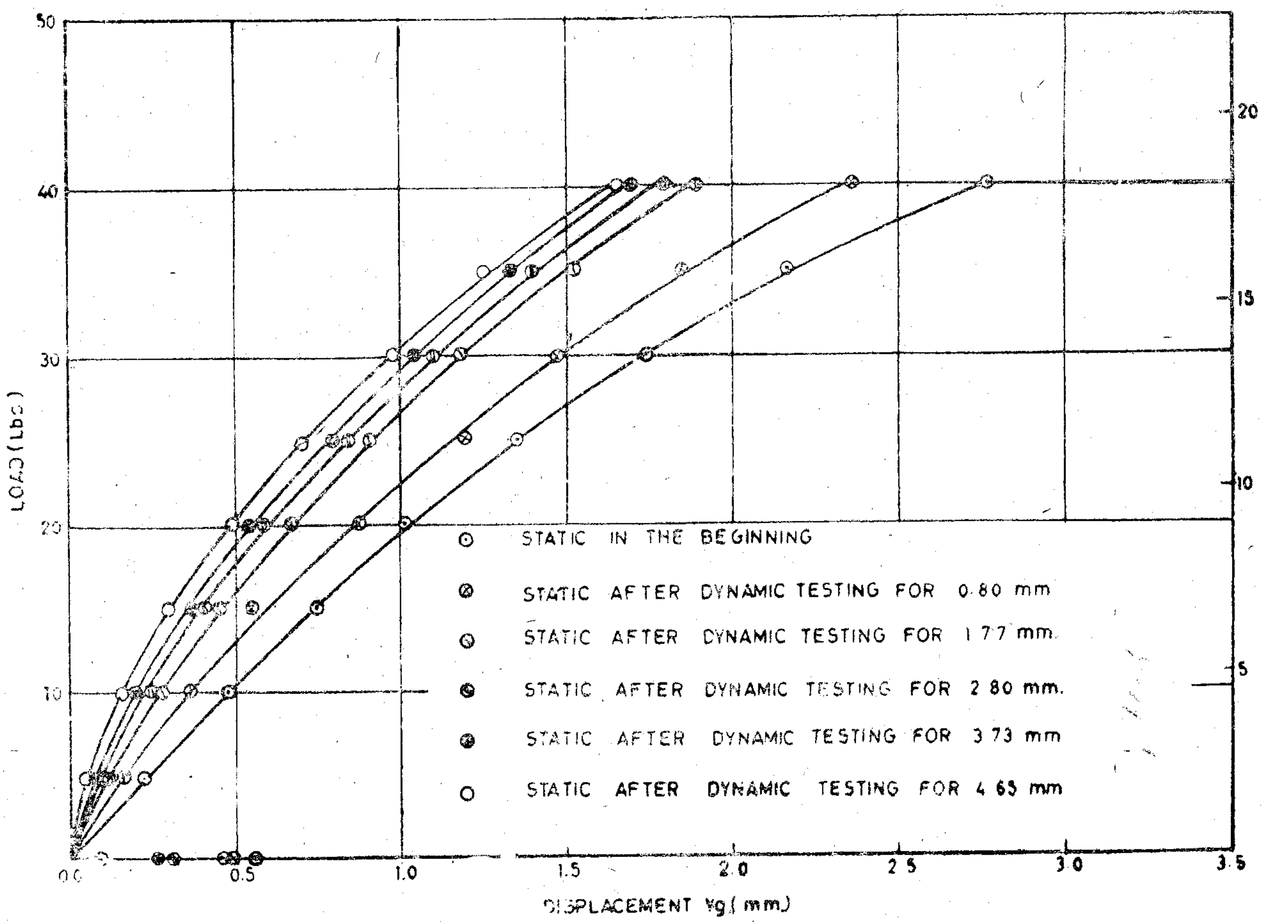

Figure 10 Load v/s Displacement (y_g) Under Static Loads with Load at G.L.

(b) Load Applied above G. L.

The data for Test series No. 2, 3, 4 and 5 were analysed in a similar manner except for the pile displacement at the ground level which was obtained as follows :—

Displacement (y_l) at load level during dynamic loading was obtained by subtracting the deflection of the proving frame (y_{pf}) from the table displacement (y_t). Pile displacement at ground level (y_g) was then obtained by dividing y_l by the corresponding ratio of y_l/y_g, which had already been established from static tests. Load displacement for Test Series No. 5 have been plotted in Figure 14. Similar curves were obtained for Test Series No. 2, 3 and 4.

Transient Loading—

Figure 15 illustrates the specimen of transient load record as obtained on the oscillograph for the 2nd, 10th and 11th blows respectively. Peak to peak amplitude of the first cycle has been analysed. The corresponding displacement was obtained from measurements of the table motion and compression of the spring at any particular measured load. The frequency of oscillation was governed by the spring used and it equalled 2 cps. The load displacement curve for transient loading is shown in Figure 16. Curves for static tests in the beginning and at the end of the transient test have also been shown.

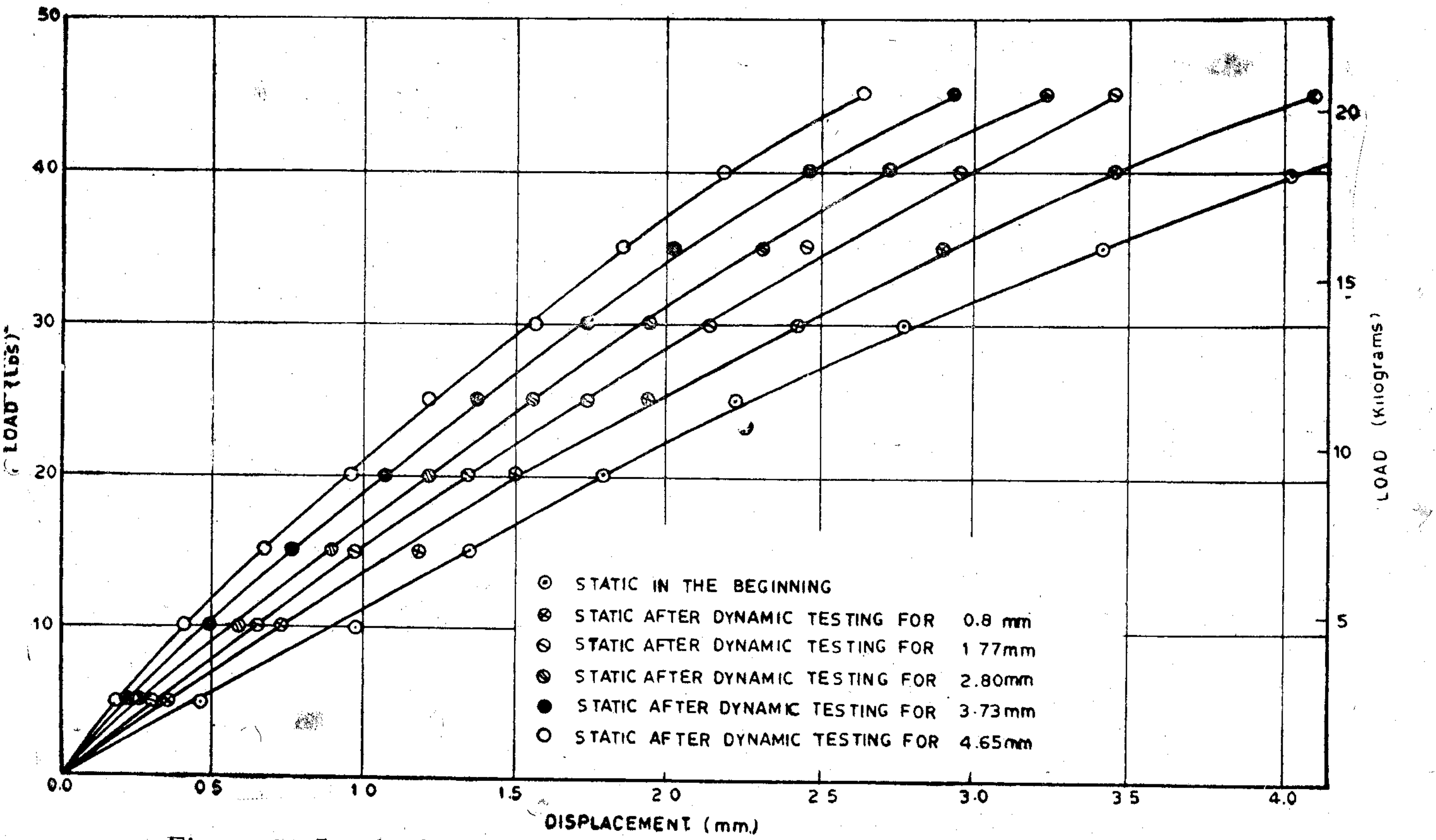

Figure 11 Load Displacement Curve Under Static Loads (With Load 5 cm Above G.L.)

Estimation of n_h

Reese and Matlock (1956) presented non-dimensional solutions for deflection of a vertical flexible pile embedded in sand according to which

$$y_g = A_y \cdot \frac{Q_{hg}\, T^3}{EI} + B_y \cdot \frac{M_g \cdot T^2}{EI} \tag{6}$$

where,

y_g = deflection at ground level.

Q_{hg} = Horizontal shear at ground level.

M_g = Moment at ground level.

T = Relative stiffness factor $= 5\ (EI/n_h)^{1/5}$

EI = Flexural stiffness of pile.

A_y = Non-dimensional deflection coefficient corresponding to shear and

B_y = Non-dimensional deflection coefficient corresponding to moment.

Values of n_h can be computed by solving the above equation for T. For a load of 20 lbs applied at the ground level in a static test and in Test Series No. 1, 2, 5 and 6, the following values were obtained:—

TABLE—1

SOIL MODULUS VALUES

Frequency	Load at G.L.	Load at 2.0 cms above G.L.	Load at 5.0 cm above G.L.	Remarks
	n_h. (lbs/in^3)			
0	103	76	68	Static Test
2	190	145	132	Steady State Test
4	215	170	145	Steady State Test
6	226	203	162	Steady State Test
8	262	240	170	Steady State Test
10	287	279	191	Steady State Test
2	250	—	—	Transient Test

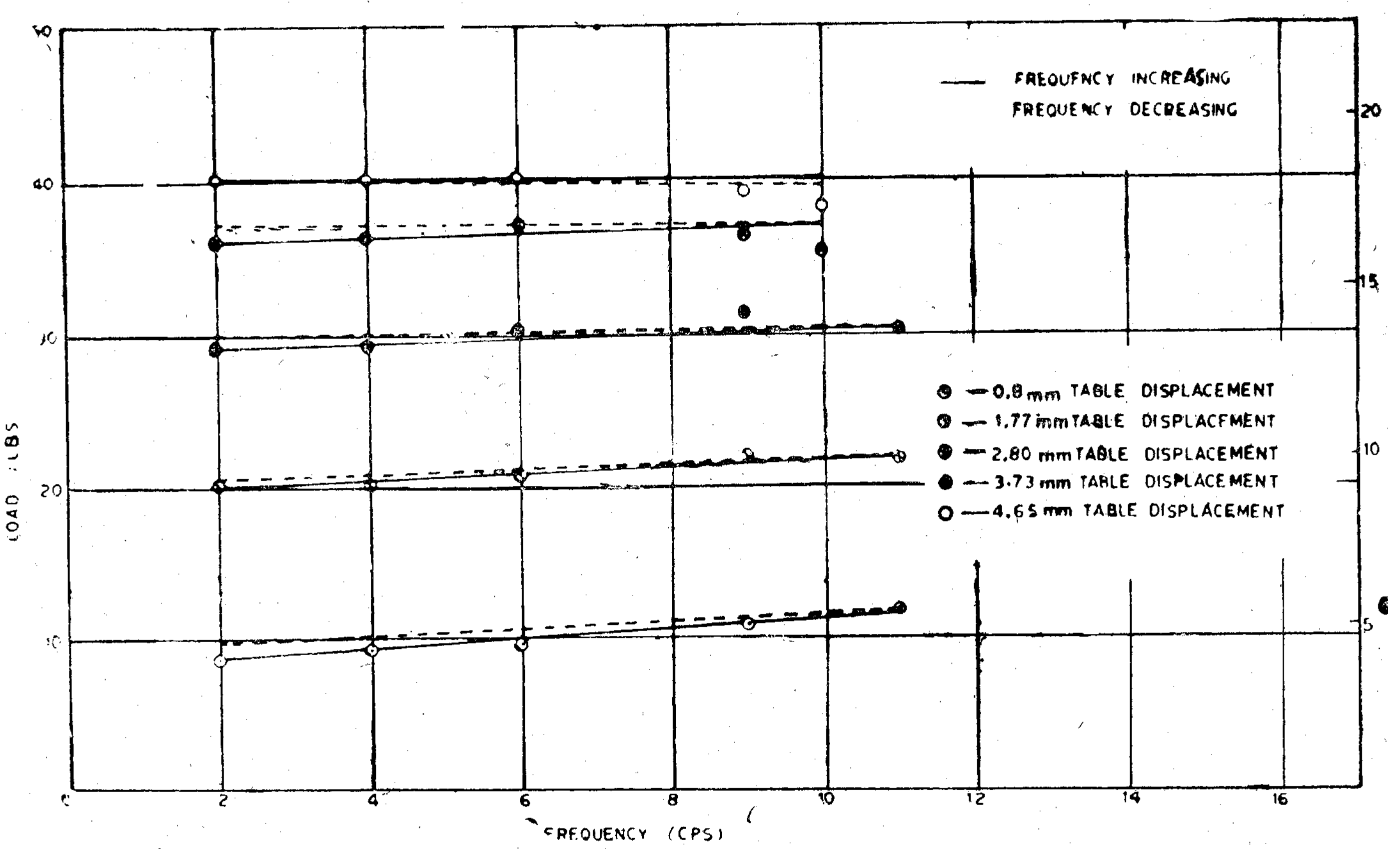

Figure 12 Load Frequency Curve (With Load at G.L.)

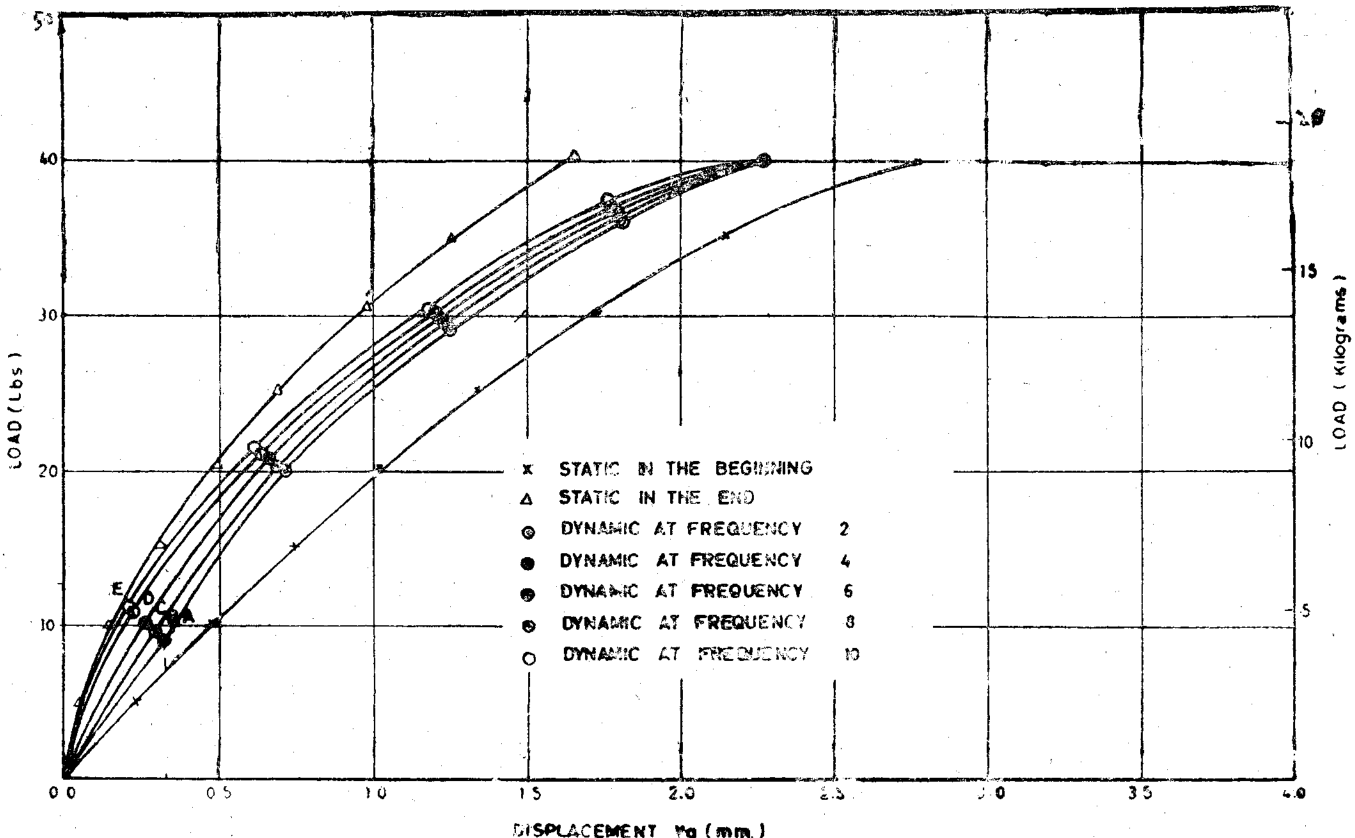

Figure 13 Load Displacement Curves with Loads at G.L. (Under Steady State Vibrations)

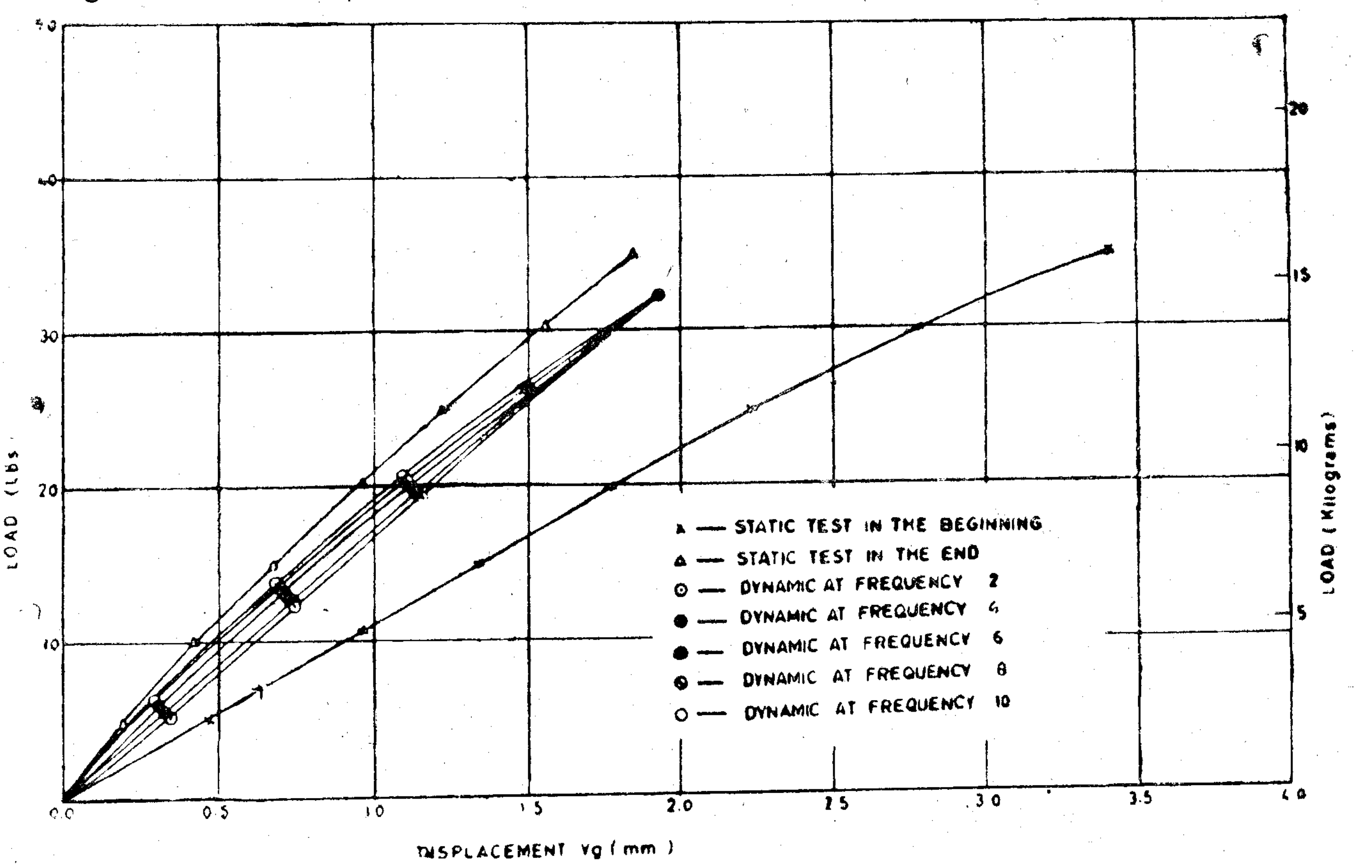

Figure 14 Load Displacement Curve with Load 5 cm High (Under Dynamic Load)

DISCUSSION OF RESULTS

Steady State Dynamic Loading—

It would appear from Figures 13 and 14 that the pile becomes stiff as frequency increases. Also it shows smaller displacement under a dynamic load than under a static load initially. It is, however, important to note that when the pile is vibrated at a fixed table

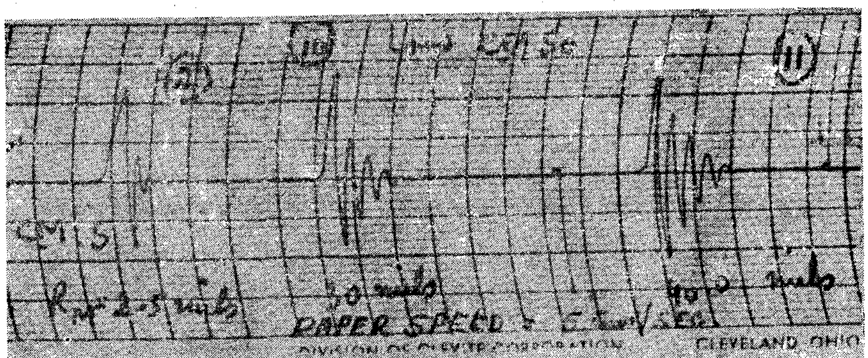

FIG. 15_RECOR OF DYNAMIC LOADS UNDER TRANSIENT LOADING

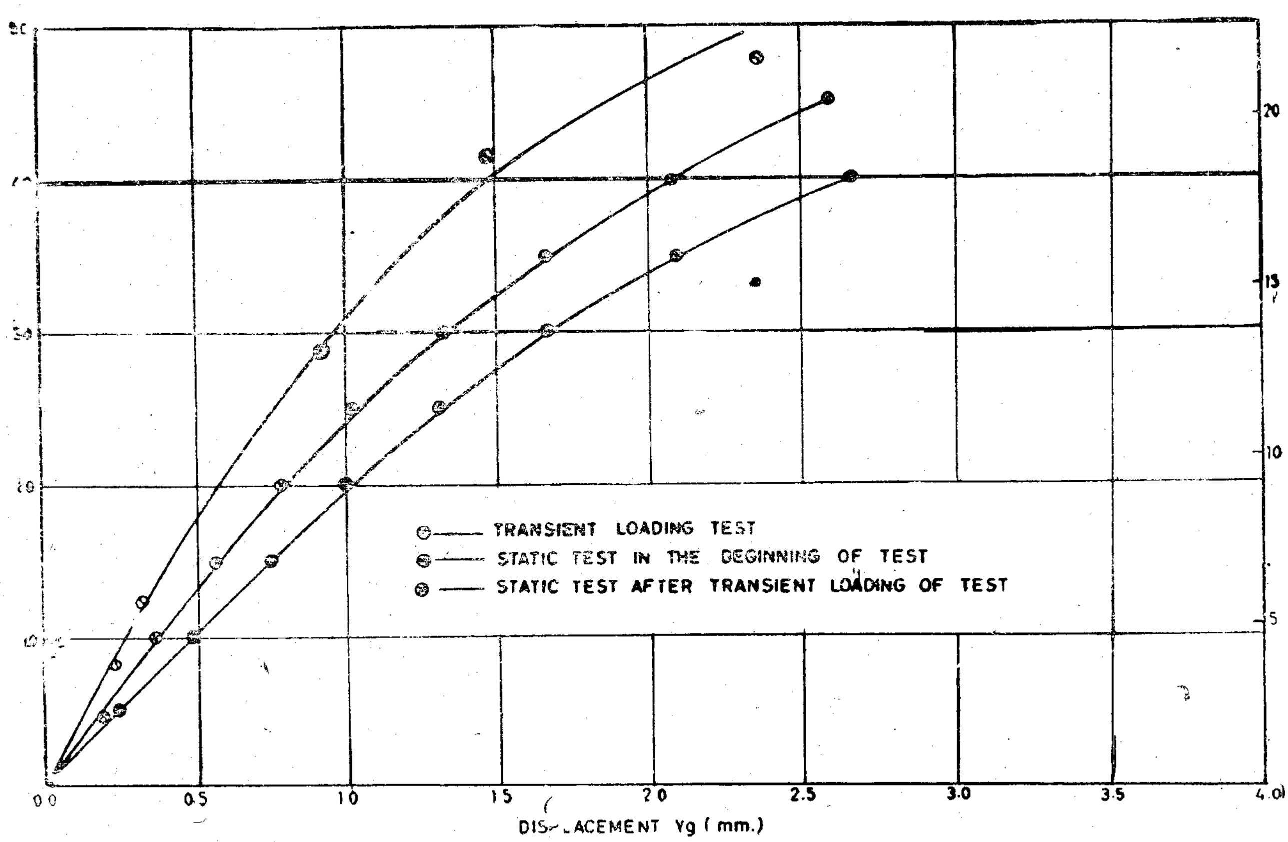

Figure 16 Load VS Displacement Curve Under Transient Loading (Loan at G.L.)

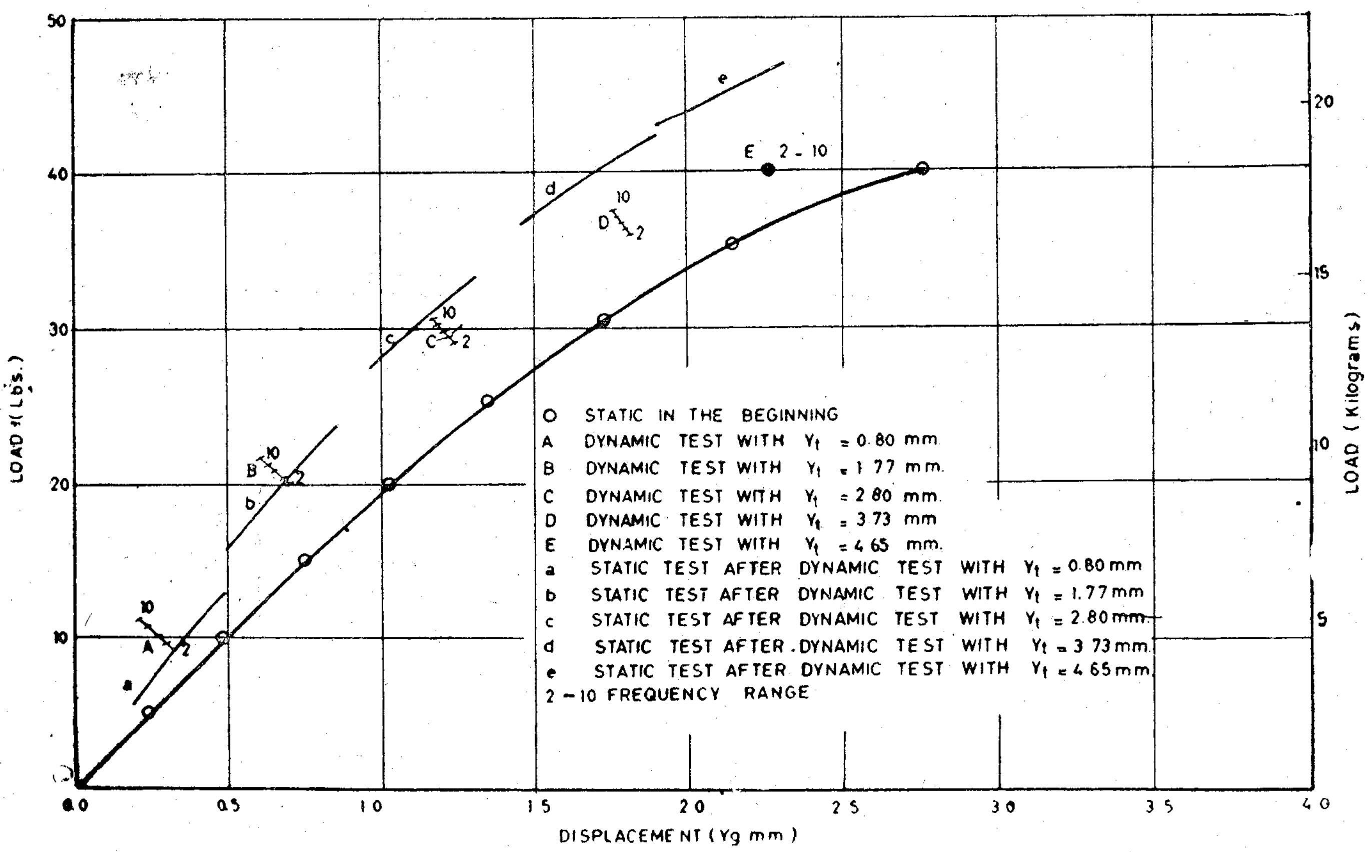

Figure 17 Load Displacement curves Under Static and and Dynamic Loads with Load at G.L.

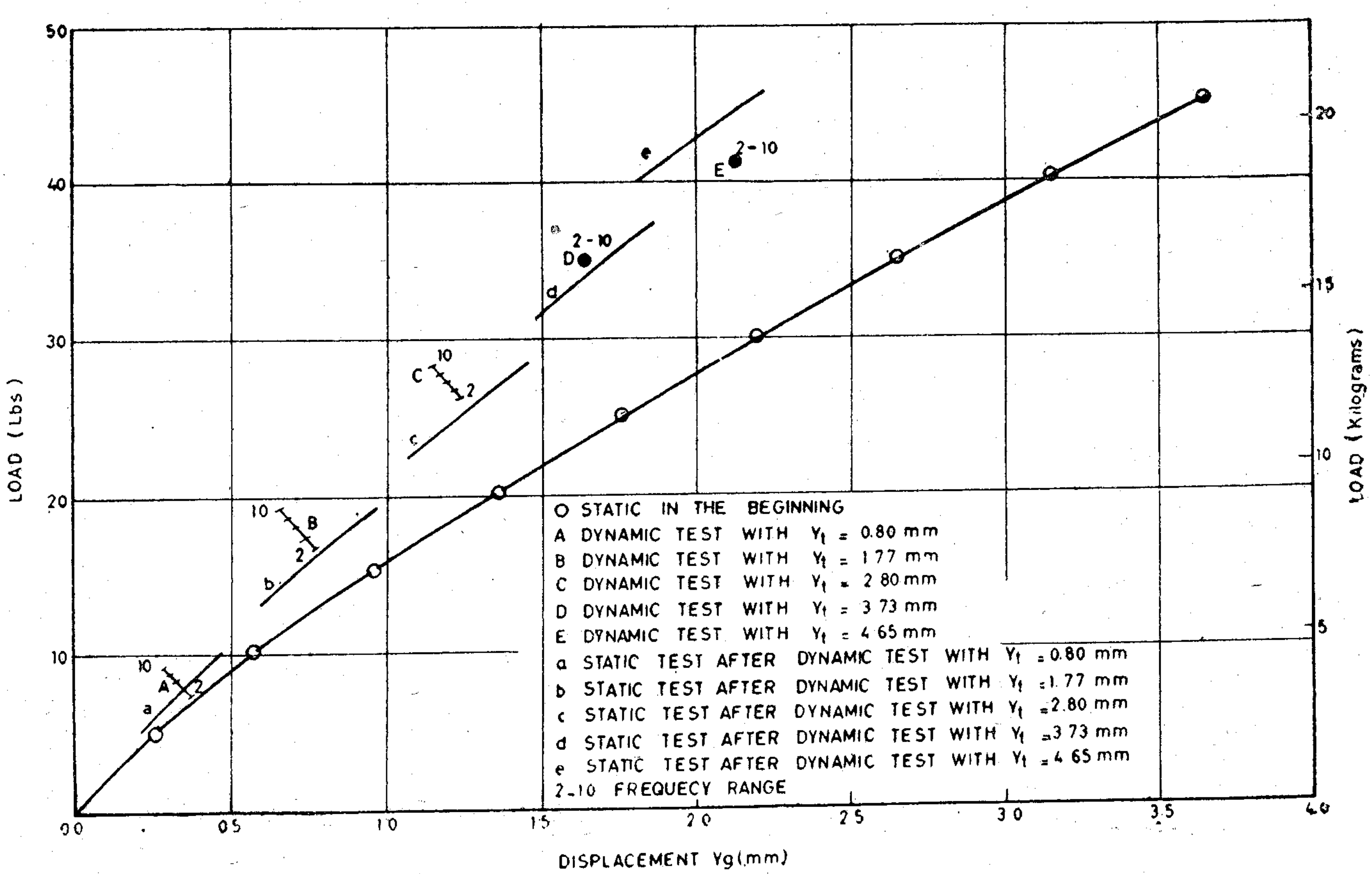

Figure 18 Load Displacement Curves Under Static and Dynamic Load with Load 2.0 cm High

displacement the load varies, usually increasing with increase in frequency (Figure 12). The charge in load is a manifestation of both the rate of loading and the increase in density of sand due to compaction.

For the same table displacement (y_t), at greater load, the deflection of the proving frame increases resulting in smaller displacement of the pile (Equation 5). In order to study this phenomenon, Figures 17, 18, 19, 20 and 21 were plotted for the load at the ground level, 2 cms, 3 cms, 4 cms and 5 cms above the ground level respectively. In these figures A, B, C, D, E represent load versus displacements of pile for table displacements of 0.8 mm, 1.77 mm. 2.80 mm, 3.73 mm and 4.65 mm respectively. The frequency interval plotted is 2 to 10 cps. Corresponding static test data after each dynamic test is shown by curves a, b, c, d and e respectively. In Figure 17, these curves are displaced up from the initial load-displacement curve, because vibration of pile causes compaction. The difference between b and a is larger than that between c and b, and the d·.ference between e and d is very small. This indicates that compaction of the soil has taken place to a large degree first and with increase in amplitude of vibration, further compaction is small. But when the load is applied above the ground level, (Figures 18, 19, 20 and 21), the soil continues to get compacted so that curve

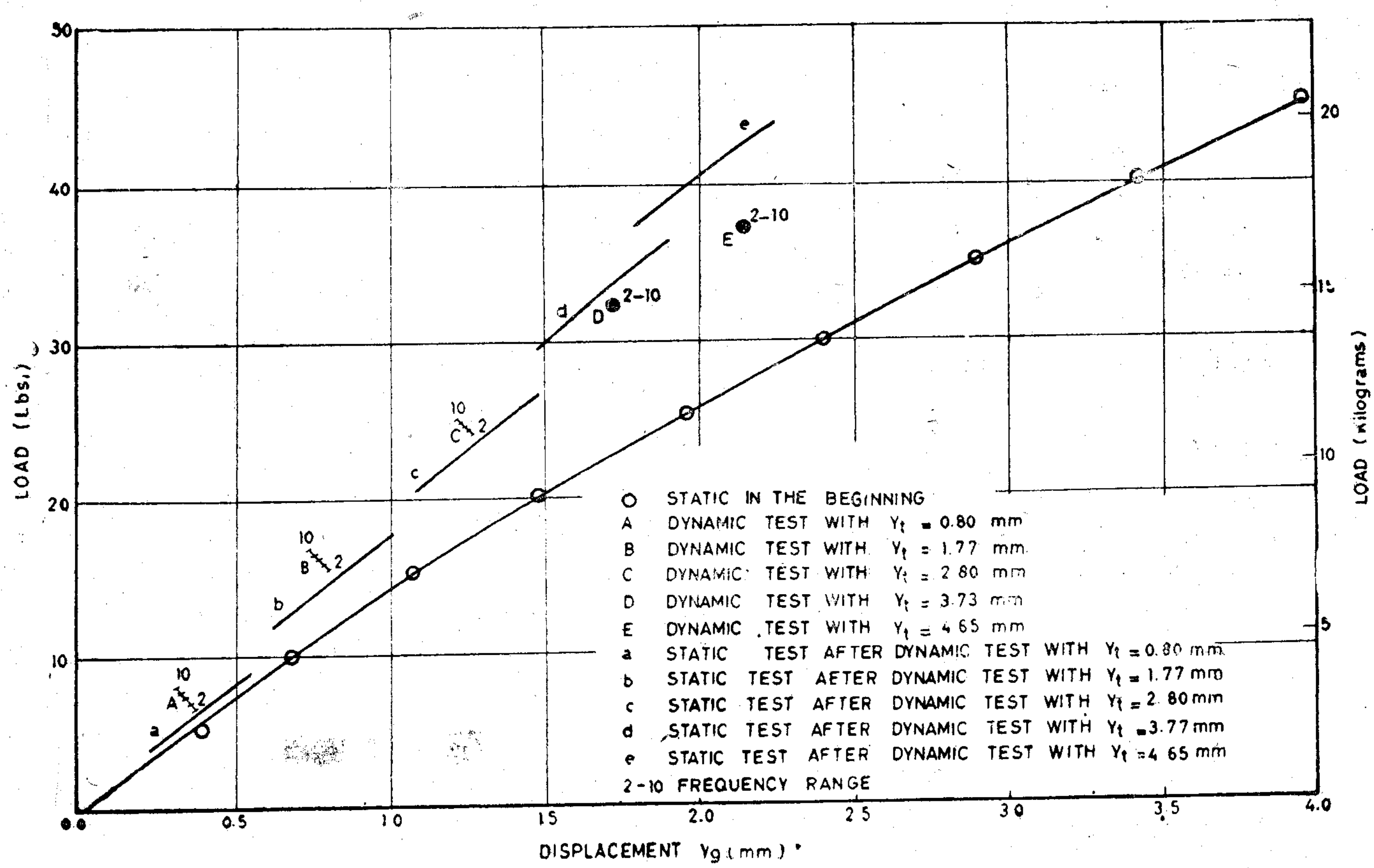

Figure 19 Load Displacement Curves Under Static and Dynamic Loads with Load 3.0 cms High

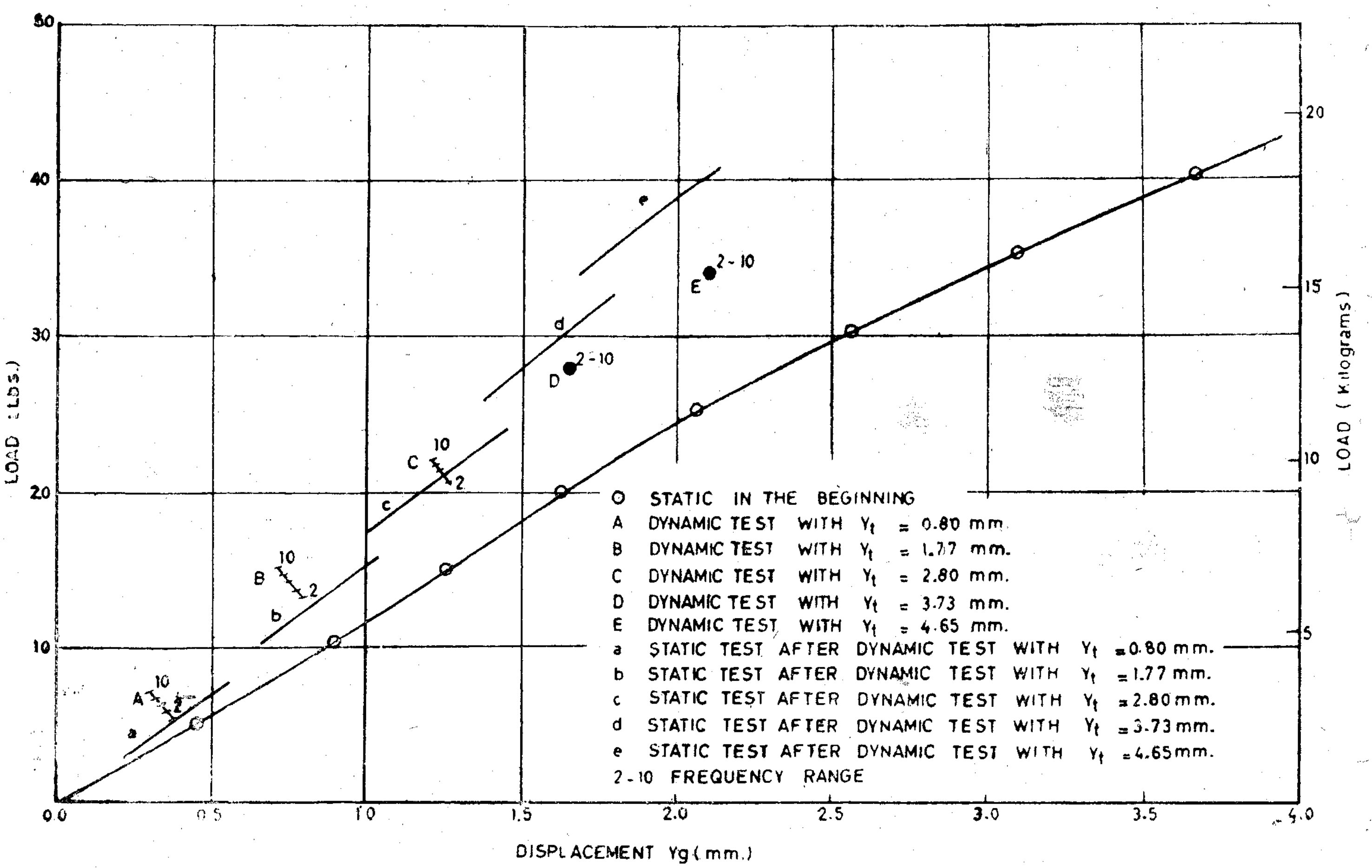

Figure 20 Load Displacement Cusves Under Static and Dynamic Loads with Load 4.0 cm High

e is displaced from d almost as much as b from a. Despite this apparent variation in the compactive behaviour of the soil, the following dynamic behaviour is exhibited in all the curves.

Curve a and b are lower than curves A and B respectively indicating that effect of compaction during vibration is more predominant. Curves c, d, e are higher than C, D, E respectively for load at ground level (Figure 17), e is higher than E for load at 2.0 cm above the G.L. in Figures 18, d and e are higher than D and E for load at 3, 4 and 5 cms above the G.L. (Figures 19, 20 and 21 respectively). The trend is that load displacement curves in static case are above the load displacement in the dynamic case after some compaction of the soil has taken place. This indicates that the effect of rate of loading is predominent. The static strength immediately after the dynamic test is greater than the corresponding dynamic strength in cases cited above. Similar explanation can be advanced if comparison is sought between load-displacement curves in the dynamic test and static test before such a test. These findings are only qualitative in nature.

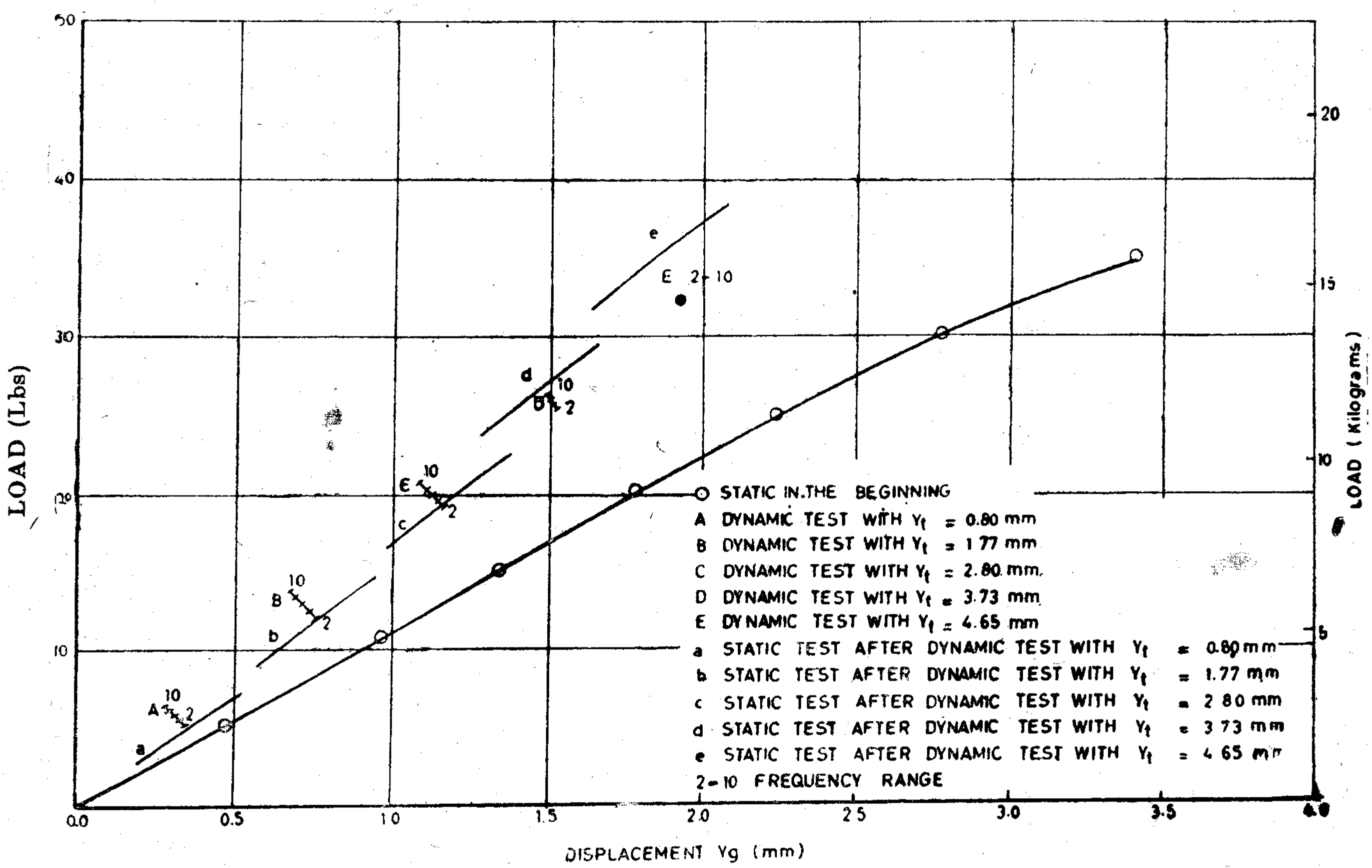

Figure 21 Load Displacement Curves Under Static and Dynamic Loads with Load 5.0 cm High

Another interesting point was that in many cases of dynamic test, only one point was obtained, E in Figures 17 and 21, D and E in Figures 18, 19 and 20. This indicates that the effect of compaction during vibrations is negligible. Thus, it would appear that the effect of dynamic loading is to make the pile soil system less stiff while the change in frequency of loading has no effect, within the range of frequencies of these tests.

Transient Loading—

Figure 16 is a plot of load-displacement at ground level under the transient test at a frequency of 2 cps. Plots of static test before and after the transient test have also been shown. It will be observed that the soil gets compacted, the plot of the static test after the transient test being higher than the one before the transient test. But the displacements in transient test are smallest of all the tests.

Comparison of Static, Steady State and Transient Loading—

Figure 22 is a plot of load displacement for the initial static case and steady state and transient loading, both at 2 cps applied at G.L. The displacements in transient test are the

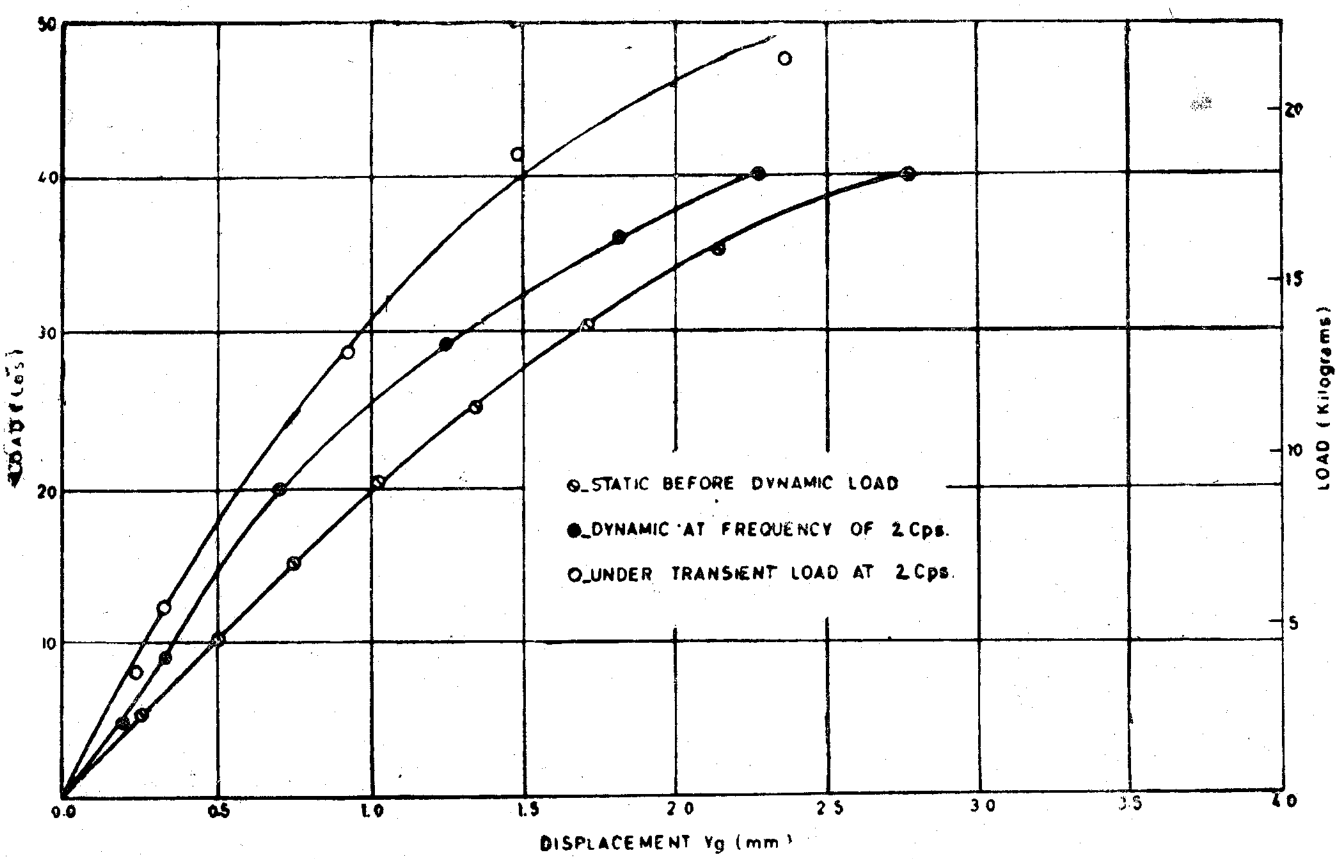

Figure 22 Load Displacement Curves Under Different Kinds of Loads

smallest, while under steady state loading, these are smaller than the initial displacements under a corresponding load.

Soil Modulus and Damping

The values of the soil modulus are shown in Table 1.

The natural frequency record of the soil-pile system afforded an opportunity to calculate the damping of the system, which was about 3 percent.

CONCLUSIONS

1. Under steady state dynamic loading, the soil around the pile gets compacted. In initial stages, the effect of compaction is more predominent.

2. The initial static strength of the pile is less than the dynamic strength. After the soil gets sufficiently compacted the static strength is greater than the dynamic strength.

3. Under transient loads, the soil around the pile also gets compacted and the static strength obtained before and after transient testing is less than the transient strength.

4. The transient strength of a pile is greater than the steady state dynamic strength which in turn is greater than the initial static strength.

5. The zone of influence of a dynamically loaded pile extends to a considerably greater distance then that for a statically loaded pile.

6, The damping of soil-pile system is about 3 percent.

ACKNOWLEDGEMENTS

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APPENDIX A—NOTATIONS

The units are in Force (F), Length (L) and Time (T)

Symbol			Units
A_y	—	Non-dimensional deflection coefficient corresponding to horizontal shear	
B	—	Width of pile	L
B_y	—	Non-dimensional deflection coefficient corresponding to Moment	
D	—	Diameter of pile	L
E	—	Modulus of Elasticity of pile	FL^{-2}
EI	—	Flexural Stiffness of Pile	FL^2
f	—	Frequency	T^{-1}
I	—	Moment of intertia	L^4
k	—	Modulus of subgrade Reaction	FL^{-2}
k_x	—	Value of k at any depth x	FL^{-2}
L_s	—	Embedded length of pile	L
M	—	Moment	FL
M_g	—	Moment at Ground level	FL
M_s	—	Moment produced by rotational restraint	FL
n	—	Empirical coefficient	
n_h	—	Constant of modulus of subgrade reaction	FL^{-3}
Q_{hg}	—	Horizontal shear at G.L	F
T	—	Relative stiffness factor $(EI/n_h)^{1/5}$	L
y_g	—	Displacement of pile at G.L	L
y_l	—	Displacement of pile at load level	L
y_{pe}	—	Number of chart lines deflected in pen recorder according to calibration	
y_{pf}	—	Deflection of proving frame	L
y_t	—	Displacement of shaking table	L
Z_{max}	—	Non-dimensional depth coefficient	
	—	L/T	
ε	—	Strain	

SEISMOLOGICAL NOTES

(India Meteorological Department, New Delhi)

Earthquakes felt in and near about India during April–September, 1965.

Date 1965	Origin time (G.M.T.) h. m. s.	Epicentre Lat. (°N)	Long. (°E)	Region	Approx. depth (km.)	Magnitude	Remarks
1	2 3 4	5	6	7	8	9	10
April 2	22 26 47.3 (CGS)	36.8	66.6	Hindukush	38	5.1 (CGS)	—
April 10	14 11 10.0 (SHL)	38.0	73.0	Pamir Plateau	—	5.7 (NDI)	Recorded at many Indian Observatories.
	14 11 22.0 (CGS)	37.6	73.4	Tadzhik SSR	33	4.3 (CGS)	—
April 11	22 33 05.9 (CGS)	26.7	92.3	Eastern India	70	5.1 (CGS)	
April 13	06 34 15 (NDI)	31.3	77.1	Himachal Pradesh	—	2.8 (NDI)	Felt in Mandi, Sundernagar and Jogindernagar.
April 17	14 50 53 (NDI)	30 Km NW of Delhi			—	2.3 (NDI)	—
April 20	05 15 24 (CGS)	34.1	82.0	Tibet	33	—	—
April 24	20 01 55.5 (CGS)	35.9	65.3	Hindukush	33	5.0 (CGS)	—
April 25	02 27 45.5 (CGS)	36.3	70.5	Hindukush	231	4.7 (CGS)	—
April 26	03 24 34.3 (CGS)	36.3	70.2	Hindukush	239	4.6 (CGS)	—
	13 27 09.8 (CGS)	11.2	94.2	Andaman Islands	33	5.2 (CGS)	

1	2 3 4	5	6	7	8	9	10
April 27	00 53 41.0 (CGS)	36.8	73.1	North Western Kashmir	—	—	—
April 30	07 13 23.1 (CGS)	28.3	96.0	India-China Border	33	4.4 (CGS)	—
May 13	04 30 41.0 (CGS)	37.0	71.4	Afghanistan-USSR BORER	90	—	—
	10 51 10 (SHL)	30.0	81.0	Nepal-India Border	—	5.0 (NDI)	Recorded at many Indian Observatories.
	10 51 15.5 (CGS)	29.8	80.5	Nepal-India Border	33	5.1 (CGS)	—
May 20	14 00 50 (CGS)	2.2	99.6	Northern Sumatra	75	—	—
	14 06 55.6 (CGS)	1.8	99.1	Northern Sumatra	75	4.8 (CGS)	—
	14 35 53 (CGS)	2.3	99.5	Northern Sumatra	75	4.4 (CGS)	—
May 26	08 47 22.0 (CGS)	36.3	70.5	Hindukush	229	4.6 (CGS)	—
May 28	09 31 19.1 (CGS)	36.7	70.1	Hindukush	286	5.0 (CGS)	—
May 30	08 48 17.9 (CGS)	26.0	95.8	Burma-India Border	88	5.8 (CGS)	—
	11 38 41.4 (CGS)	36.5	70.1	Hindukush	220	4.9 (CGS)	
May 31	02 04 42.9 (CGS)	32.6	78.2	Kashmir-Tibet Border	33	5.3 (CGS)	—
	02 04 54 (NDI)	31.5	77.2	Himachal Pradesh	—	5.4 (NDI)	Felt in Mandi, Sundernagar and Jogindernagar.
June 1	04 32 45.3 (CGS)	20.2	94.9	Burma	57	— (CGS)	—
	04 32 45.0 (SHL)	20.0	95.0	West Burma	—	5.5 (NDI)	Recorded at a few Indian Observatories.
	07 52 26.1 (CGS)	28.5	83.2	Nepal	33	5.2 (CGS)	—
	07 52 30 (SHL)	27.0	83.0	Nepal-India Border	—	5.7 (NDI)	Recorded at many Indian Observatories.

1	2 3 4	5	6	7	8	9	10
June 4	03 37 12 (NDI)	17.0	73.4	South Maharashtra	—	5.4 (NDI)	—
June 6	20 29 52 (CGS)	36.7	71.2	Afganistan-USSR Border	33	4.6 (CGS)	—
June 10	05 49 00 (CGS)	35.9	70.5	Hindukush	125	5.8 (CGS)	—
	05 48 55.0 (SHL)	36.0	71.0	Hindukush	—	—	Recorded at a few Indian Observatories.
June 11	15 43 09.0 (CGS)	24.8	95.5	Burma	123	5.3 (CGS)	—
June 13	04 21 29.0 (CGS)	33.5	69.5	Afghanistan	57	4.8 (CGS)	—
June 14	13 17 01.7 (CGS)	32.0	87.7	Tibet	37	5.1 (CGS)	—
June 15	07 59 20.4 (CGS)	29.7	95.3	India-China Border	33	—	—
June 16	23 49 04.0 (CGS)	32.0	87.6	Tibet	33	5.0 (CGS)	—
June 17	20 15 10 (SHL)	30.0	87.0	Tibet	—	5.5 (NDI)	Recorded at many Indian Observatories.
	20 14 48.6 (CGS)	32.0	97.8	Tibet	8	5.4 (CGS)	—
June 18	01 18 35.2 (CGS)	32.0	87.7	Tibet	19	5.2 (CGS)	—
	08 17 37.6 (CGS)	25.0	93.8	Eastern India	46	5.9 (CGS)	—
June 22	05 49 18.9 (CGS)	36.3	77.7	Kashmir-Sinkiang Border	28	6.1 (CGS)	—
June 25	09 21 00 (CGS)	1.2	99.1	Northern Sumatra	33	—	—
June 27	01 04 23.8 (CGS)	9.2	94.1	Nicobar Islands	—	5.2 (CGS)	—
June 28	12 14 49 6 (CGS)	27.2	66.9	West Pakistan	34	—	—

1	2	3 4	5	6	7	8	9	10
July	3	11 26 11.6 (CGS)	22.6	101.4	Burma-China Border	33	5.2 (CGS)	Recorded at a few Indian Observatories.
July	3	17 10 19 (CGS)	31.9	82.7	Tibet	41	4.5 (CGS)	—
July	5	23 41 43 (CGS)	21.1	94.1	Burma	43	—	—
July	6	13 21 07.2 (CGS)	36.6	73.0	North Western Kashmir	165	4.6 (CGS)	—
July	7	04 51 24 (CGS)	38.6	74.8	Tadzhik–Sinkiang Border	33	5.2 (CGS)	—
July	12	13 52 39.5 (CGS)	36.3	70.6	Hindukush	224	5.1 (CGS)	—
July	13	23 09 49.3 (CGS)	36.5	70.8	Hindukush	158	5.1 (CGS)	Recorded at a few Indian Observatories
July	19	09 07 16.0 (CGS)	3.0	97.1	Northern Sumatra	59	4.9 (CGS)	—
		09 09 40.3 (CGS)	3.0	97 1	Northern Sumatra	33	5.2 (CGS)	—
July	21	22 40 29.8 (CGS)	36.3	71.4	Afghanistan–USSR border	141	4.5 (CGS)	—
July	23	21 29 34.6 (CGS)	26.2	65.1	West-Pakistan	33	4.7 (CGS)	—
July	24	17 57 42.2 (CGS)	36.4	71.2	Afghanistan–USSR border	234	4.9 (CGS)	—
July	25	03 40 40.4 (CGS)	2.0	99.3	Northern Sumatra	98	5.3 (CGS)	—
		03 40 25 (SHL)	2.0	100.0	Sumatra	—	—	Recorded at many Indian Observatories.
July	26	00 38 32.0 (CGS)	41.9	70.9	Kirgiz SSR	33	4.6 (CGS)	—
July	31	16 36 53 8 (CGS)	32.7	93.2	Tibet	33	4.9 (CGS)	
		17 07 52.6 (CGS)	32.7	93.1	Tibet	33	4.7 (CGS)	Recorded at a few Indian Observatories.

1	2 2 4	5	6	7	8	9	10
	19 01 09.4 (CGS)	32.8	93.0	Tibet	33	4.4 (CGS)	Recorded at a few Indian Observatories.
	21 44 47.8 (CGS)	32.7	93.1	Tibet	21	4.9 (CGS)	—
Aug. 1	14 14 01.7 (CGS)	32.6	93.6	Tibet	33	5.5 (CGS)	—
	20 09 17.9 (CGS)	32.6	93.3	Tibet	32	5.3 (CGS)	—
Aug. 2	17 49 47.2 (CGS)	32.8	93.3	Tibet	33	4.8 (CGS)	—
Aug. 3	07 02 41.9 (CGS)	36.2	69.5	Hindukush	98	4.9 (CGS)	—
	07 35 21.8 (CGS)	33.3	91.1	Tsinghai province, China.	44	5.1 (CGS)	—
Aug. 8	16 16 53.8 (CGS)	28.9	69.1	West Pakistan	36	4.3 (CGS)	Recorded at a few Indian Observatries
Aug. 14	17 14 49.4 (CGS)	37.3	72.0	Afghanistan USSR broder.	220	4.7 (CGS)	Recorded at a few Indian observatries
Aug. 15	05 59 47.7 (CGS)	36.4	71.1	Afghanistan USSR broder.	213	4.8 (CGS)	Recorded at a few Indian observatires
Aug. 17	10 35 04.1 (CGS)	5.3	96.2	Northern Sumatra	33	5.3 (CGS)	—
	12 52 45 (CGS)	5.3	96.2	Northern Sumatra	99	4 9 (CGS)	—
Aug. 21	01 05 32.8	37.4	96.7	Tsinghai Province, China.	33	4.5	—
Aug. 31	03 45 50.0 (CGS)	7.9	94.0	Nicobar Islands	25	5.1 (CGS)	Recorded at a few Indian observatories.
Sept. 9	23 31 24.5 (CGS)	36.3	70.6	Hindukush	245	4.6 (CGS)	Recorded at a few Indian observatories.
Sept. 13	16 02 41.0 (CGS)	4.7	96.2	Northern Sumatra	33	4.7 (CGS)	—
Sept. 14	18 57 27.0 (CGS)	36.4	70.1	Hindukush	220	4.9 (CGS)	—

1	2 3 4	5	6	7	8	9	10
Sept. 16	23 57 55 (CGS)	36.1	70.2	Hinduksush	117	4.4 (CGS)	—
Sept. 22	04 24 47.8 (CGS)	20.8	99.3	Burma	35	5 5 (CGS)	—
	04 24 50 (SHL)	21.0	99.0	Eastern Burma	—	—	Recorded at many Indian observatories.
Sept. 24	20 38 07.6 (CGS)	5.2	96.1	Northern Sumatra	33	5.2 (CGS)	Recorded at a few Indian observatories.
Sept. 25	15 47 58.4 (CGS)	41.3	74.9	Kirgiz SSR	33	5.6	Recorded at a few Indian observatories.

ABSTRACT OF PAPERS

American Society of Civil Engineers

Goodman, L.J., A.R. Aidun and C.S. Grove (1965), "Soil Surface Compaction with a Foam–Type Explosive", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, No. SM 1, January 1965, pp. 143–165.

"A proposed new method for soil surface compaction based on the principle of pressure waves as a source of energy is described. This method, which has been termed the foamed propellant method for soil compaction, has a formulation containing hydrazine, ammonium perchlorate, and a foaming agent, hydrolized protien base foam liquid. The mechanics of this new compaction technique are relatively simple, with a blanket of the foamed propellant system placed on the surface of the soil to be compacted and then detonated. The results of using the foamed propellant system to compact loose cohesionless soils are presented and evaluated. This includes some preliminary investigation of the influence of both surface area and multiple detonation on compaction control parameters. The results of this investigation to date (1964) show conclusively that this new compaction is effective in increasing the density of the soil to depths of as much as 15 inches below the surface for sandy soils, sand-gravel mixtures, and predominantly silty soils containing little clay. Laboratory blast pressure measurements are also examined and presented".

Selig, E. T., and E. Vey, (1965), "Shock-Induced Stress Wave Propagation in Sand", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, No. SM3, pp. 19-49.

"Experiments were conducted to investigate air shock-induced stress waves in bars of dry sand confined under constant lateral pressure. Stresses were measured at a number of cross sections by means of embedded gages. Information was obtained on wave velocity, peak stress attenuation, and reflected stresses. The observed change in wave shape was bending over of the front and a lengthening of the period. It was demonstrated that the nonlinear, inelastic theory based on the static stress-strain characteristics of the sand could be used to predict all of the significant features of the observed wave, but to provide correct quantitative prediction of change in shape and wave velocity it was necessary to hypothesize some time-dependent stress-strain behavior."

Broms, B. B. (1965), "Design of Laterally Loaded Piles", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, No. SM3, pp. 79-99.

"Methods are presented for the design of laterally loaded piles. These methods are based on the concepts that the deformations at working loads should not be excessive and that complete collapse of the pile group should not occur even under the most adverse conditions. Lateral deflections have been calculated using a coefficient of subgrade reaction. The ultimate lateral resistance has been determined assuming that either the lateral resistance of the soil or the ultimate strength of the pile section has been exceeded. The proposed design method is illustrated by a numerical example."

Stoll, R. D. and I. A. Ebeido (1965), "Shock Waves in Granular Soil", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, No. SM4, pp. 107-125.

"Experimental and theoretical studies of shock wave propagation in an elastically confined, cylindrical sand column are described. Excitation is produced by an air shock from a pneumatic shock tube, and a special reinforced specimen container is used to restrict radial strains while allowing unimpeded axial motion. Measurements of wave velocity and pressure are made at various stations along the specimen by stress gages embedded in the sand. Test results indicate development of a stable shock preceded by a decaying precursor. Two characteristic velocities are observed : (1) The velocity of the decaying precursor, which depends on confining pressure and relative density, and (2) the velocity of the trailing steady wave form, which depends on dynamic stress amplitude and confining pressure. Theoretical interpretation of test results is presented taking into account the effects of such material characteristics as stress relaxation, a nonlinear stress-strain relationship, and differences between initial loading path and the path of subsequent load cycles."

Blanchard, K. E., and A. B. Schultz, (1965), "On the Shocks Response of Inelastic Beams", Journal of the Engineering Mechanics Division, ASCE, Vol. 91, No. EM 5, October 1965, pp. 27-35.

"Response of a uniform clamped-free beam driven into the inelastic range by half-sine, triangular, and rectangular ground acceleration pulses is investigated. Shock spectra are presented for beams possessing bilinear moment-curvature properties. Limited data are presented for a supported-supported beam. Beams are represented by limped parameter models and equations of motion are solved on an analog computer. Effects of pulse shape and duration are evaluated".

Shinozuka, M. and L. Henry (1965), "Random Vibration of a Beam Column", Journal of the Engineering Mechanics Division, ASCE, Vol. 91, No. EM5, October, 1965 pp. 123-143.

"The lateral displacements of a vertical cantilever beam subjected to an axial load acting at the free end and under a random horizontal motion of the clamped end is evaluated. This is one of the beam-column problems on which little work has been done for random loading on random boundary conditions. The axial load is assumed to be less than the fundamental buckling load of the column. For the random motion of the clamped end, a particular form of the random process is considered possibly appropriate for motion resulting from earthquake acceleration. Hence, the problem might be considered as a simplified model of the tower of a suspension bridge under horizontal earthquake acceleration".

Ward, H. S. (1965), "Analog Simulations of Earthquake Motions" Journal of the Engineering Mechanics Division, ASCE, Vol. 91, No. EM5, October, 1965, pp. 173-190.

"Two analog methods of simulating earthquake motions are presented. One method uses sections of a gaussian random function with a given power spectrum. The second method uses a hybrid computer that permits any given type of ground motion to be simulated. Linear single-degree-of-freedom systems were excited by the simulated earthquakes and their responses were measured. Base shear was used as a measure of the responses and empirical formulae were developed for base shear as a function of the mass and period of the system and the magnitude of the ground acceleration".

Anderson, R. H. (1965), "Two Degree-of-Freedom Elastoplastic Response to a Step Velocity Pulse", Journal of the Engineering Mechanics Division, ASCE, Vol. 91, No. EM5, October, 1965, pp. 191-213.

"The response of a two-degree-of-freedom elastoplastic system subjected to an instantaneous change in velocity is presented. The specific excitation considered is of interest in problems involving severe shocks of short duration, and specifically, in problems of vehicular collision, package cushioning of sensitive equipment, air delivery, and building frames subject to ground shock. The approach used consists in studying first the response on the assumption that one of the two springs remains elastic while the other spring deforms beyond the limiting elastic range. For example, it was found that when only base spring yielding is allowed, the law of variation of the maximum deformation of the base spring conforms to that of a single-degree-of-freedom elastoplastic system. Next, consideration is given to systems for which both springs can yield, and for certain combinations of the parameters, approximate relationships are presented for defining conditions under which yielding is likely to be localized in one of the springs".

American Society of Mechanical Engineers

Jones, J.P. and P.G. Buta, (1964) "Response of Cylindrical Shells to Moving Loads", 63-WA-173 ASME, March 1964.

"The purpose of this paper is to investigate the resonant speeds and to solve the problem of the dynamic response of an infinitely long cylindrical shell subjected to a ring load moving with constant velocity, taking into account the accelerations in both the radial and the axial directions. It is shown that even when longitudinal and transverse coupling effects are considered, the resonance from bending occurs at a lower velocity than the one due to axial motion.

One of the contributions of this paper lies in the observation of the non-existence of the integrals defining the solutions and in the interpretation of such non-existence of solutions for certain velocities. It is shown that such non-existence physically implies resonant load speeds. This technique can also be used to determine the resonant speeds in other problems. The advantage of the technique is that it is not necessary actually to carry out inversion operation if one is interested only in obtain ng the resonant speeds".

Klotter, K. and E. Kreyazig, (1964) "On a Nonlinear Vibrating System Having Infinitely Many Limit Cycles", 63-WA-169 ASME Journal, March 1964.

"At the present time there are two trends in the field of non-linear vibrations. One is concerned with a step-wise extension of the general theory and the other with the creation of methods for special equations. The latter approach results primarily from practical problems.

The intention is to obtain detailed equations reducible to linear first-order equations, include types that correspond to oscillations having a limit cycle.

This paper is devoted to such an equation, which is non-linear with respect to both the damping and the restoring force, and governs oscillations having infinitely many limit cycles. Bounds and approximations for the corresponding limit amplitudes are obtained from suitable estimates of first integrals".

Seelig, J.M. and W H. Hoppmann, (1964) "Impact on an Elestically Connected double beam System" 64-APM-28, Applied Mechanics Conference, Journal ASME, December 1964.

"The development and solution of the differential equations of motion of an elastically connected double-beam system subjected to an impulsive load is presented. In addition to the theory there is given a description of impact experiments on double beams. Theoretically and experimentally determined strains as functions of time are compared in the form of curves. The agreement is remarkably good for the type of load used".

Garnet, H. and J. Kempner, (1964), "Axi-symmetric free Vibrations of Conical Shells" 64-APM-24, Applied Mechanics Conference, ASME September, 1964.

"The lowest axi-symmetric modes of vibration of truncated conical shells are studied by means of a Rayleigh-Ritz procedure. Transverse shear deformation and rotatory inertia effects are accounted for, and the results are compared with those predicted by the classical thin-shell theory.

Additionally, the results are compared when either of these theories is formulated in two ways : First, in the manner of Love's first approximation in the classical thin-shell theory, and then by including the influence of the change of the element of arc length through the thickness".

Appl, F.C. and N.R. Byers, (1965) "Fundamental Frequency of Simply Supported Rectangular Plates with Linearly Varying Thickness" 64-WA/APM-1 Journal of ASME, March 1965.

"Upper and lower bounds for the fundamental eigen-value (frequency) of a simply supported rectangular plate with linearly varying thickness are given for several taper ratios and plan geometries.

These bounds were determined using a previously published method which yields convergent bounds. In the present study, all results have been obtained to within 0.5 percent maximum possible error".

Shoemaker, E.M. (1965), "Invariant Imbedding Applied to Eigen-Value Problems in Mechanics" 64-WA/APM-24 Journal of ASME, March 1965

"A formal method is presented for obtaining eigen-values of ordinary differential equations associated with problems of buckling and vibration. The method utilises the idea of invariant imbedding which has been applied previously to two-point boundary-value problems in transport theory and wave propagation.

The present method reduces the eigen-value problem to an initial value problem for a matrix of Riccati equations. The numerical solution of such formulations has proved to be generally more efficient than known methods".

Archer A A. and J. Famili, (1965), "On the Vibration and Stability of Finitely Deformed Shallow Spherical Shells" 64-WA/APM-11 ASME, March 1965.

"Equations describing the asymmetric vibrations of a shallow spherical shell which has suffered a finite axi-symmetric loading are studied by means of a finite difference analysis.

Results are obtained which show a continuous variation of the frequency spectrum as a function of the load parameter ranging from the previously computed frequencies and modes of the unloaded shell to the modes of the unloaded shell to the modes corresponding to vanishing frequencies which reveal the asymmetric buckling modes and loads for the shell.

Excellent agreement with earlier work is found for both limiting cases, thus providing an independent check on the asymmetric buckling loads recently computed by Huang".

Fulton R.E. (1965) "Dynamic Axisymmetric Buckling on Shallow Conical Shells Subjected to Impulsive Loads" 64-WA/APM-21 Journal ASME, March 1965.

"The method of Humphreys and Bonder is applied to the axi-symmetric snap-through buckling of a shallow conical shell subjected to an idealised impulse applied uniformly over the surface of the shell. A solution is obtained for laterally restrained boundaries for laterally restrained boundaries for both the simply-supported and fixed cases. Solutions are also obtained for laterally unrestrained boundary conditions where the radial stress vanishes.

Results are given for a degenerate case of the laterally unrestrained boundary conditions called 'stress free' where the radial and circumferential stress both vanish and where the slope is zero.

Results for the laterally restrained and degnerate laterally unrestrained cases are then compared with similar results for the shallow spherical shell".

Humphreys J.S. (1965) "Plastic Deformation of Impulsively Loaded Straight Clamped Beams" 64-WA/APM-17 Journal ASME March 1965.

"A series of tests, was conducted on flat steel beams of various sizes and material properties, using sheet explosive to provide sufficiently high uniform impulsive loading to produce significant plastic deformation. The beams were attached to a ballistic pendulum for measurement of applied impulse and were photographed with a Fastax camera during deformation.

The resulting final deformations are compared with the rigid-plastic theory of Symonds and Mentel, which is seen to give upper bounds that are in general higher by about 20-30 percent than the deformation observed. A fairly good first approximation to maximum deflection for engineering purposes is in fact obtained simply by using rigid-plastic beam theory (including axial constraints)".

Iwan, W.D. (1965) "The Steady-State Response of a two Degree-of-Freedom Bilinear Hysteretic System 64 Journal ASME March 1965.

"The method of slowly varying parameters is used to obtain an approximate solution for the steady-state response of a two-degree-of-freedom bilinear hysteretic system. The stability of the system is investigated and it is shown that such a system exhibits unbounded amplitude resonance when the level of excitation is increased beyond a certain finite limit".

Costantino, C.J. (1965) "The Strength of thin Walled Cylinders Subjected to Dynamic Internal Pressures"64–WA/APM–16. Journal ASME, March 1965.

"The equation of motion governing the response of long (infinite) cylinders to dynamic internal pressures is derived. The material is assumed to be rigid plastic, with strain-hardening being taken into account through the Ludwik power strain-hardening law.

Numerical results are presented for a range of hardening constants from 0 01 to 1.0, covering the range applicable to most materials of interest. The form of the dynamic pressure considered is an initially peaked, linearly decaying pressure pulse".

Seismological Society of America

Goldberg J. E. and E.D. Herness. (1965), "Vibration of Multistorey Buildings Considering Floor and Wall Deformations", Bull. Seism. Soc. Am. Vol. 55, No. 1, Feb. 1965.

"A generalised slope-deflection equation is used which takes into account shear deformation in addition to the flexural deformation conventionally included without increasing the number of equations. Using this to represent the action of walls, a method for calculating the modes and frequencies of stiffened multistorey buildings is described. The generalised equation is further used to represent the action of floor slabs and a method for determining the two-dimensional modes and frequencies of stiffened and unstiffened multistorey buildings with relatively long and narrow floors is presented. As illustrative examples, the modes and frequencies of a 10–storey stiffened building are calculated".

Ward, H. S. (1965), "Computer Simulation of Multistorey Structures Subjected to Ground Motion", Bull. Seism. Soc. Am. Vol. 55, No. 2, April 1965.

"A hybrid computer system has been used to investigate the dynamic responses of five-storey structures excited by ground motion simulated by (a) band limited white noise generator (b) saw tooth generator. The factors that have been considered are the distribution of stiffness and mass in the structure, the type of foundation condition and the action of viscous damping. The results show that it may be reasonable to calculate the base shear of a multi-degree-of-freedom system as a function of its fundamental period. In the case of lightly damped structure, however, the distribution of forces through the height of the building is also shown to be dependent on other dynamic characteristics of the structure, as well as the frequency content of the ground motion. Viscous damping besides reducing the forces acting on the structure, also tends to eliminate modes of vibration other than the fundamental. A pinned-end foundation is also shown to reduce the foreces acting on the lower stories of a building compared with the fixed-end condition".

Bogdanoff, J.L., J.E. Goldberg and A.J. Schiff (1965), "The Effect of Ground Transmission Time on the Response of Long Structures", Bull. Seism. Soc. Am. Vol. 55, No. 3, June 1965.

"The influence of the transmission time of a seismic-like ground disturbance upon the response of a long structure is studied. The responses are evaluated in terms of extreme value statistics. A simple bridge model was studied and it was concluded that the transmission time of a seismic disturbance should not be ignored when considering the response and safety of a long structure".

Gupta, I.N. (1965), "Note on the Use of Reciprocity Theorem for Obtaining Radiation Patterns", Bull. Seism. Soc. Am. Vol. 55, No. 2, April, 1965, pp. 277–281.

"Expressions for the horizontal and vertical displacements at the surface of an elastic half space when plane harmonic P or SV waves are incident at any given angle are already known. On the basis of the reciprocity theorem, these expressions are used to obtain "far field" radiation patterns of P and SV waves due to horizontal and vertical forces applied at the free surface."

Roy, A (1965), "On the General Solution of Elastic Wave Equations in a Homogeneous Medium with Applications to the Study of Earthquake Source Mechanisms", Bull. Seism. Soc. Am. Vol. 55, No. 2, April 1965, pp. 283–301.

"The elastic wave equation in an isotropic homogeneous semi-infinite elastic medium has been transformed by a combination of Fourier and Laplace-transforms and the surface displacements due to general force-system in three mutually perpendicular directions have been directly obtained. By suitably specifying the force-system, the surface displacements for all possible types of point sources so far used have been calculated, obtaining in particular, the results previously derived by Haskell for a di-polar point source of arbitrary direction. It has been shown that now the surface displacement for finite size of the source can be calculated for both body and surface waves. Expressions for the displacements at any point of the medium have been derived, giving in particular the displacements associated with incident wave in the medium, separately."

Leet, L. D. and J. L. Florence (1965), "The Earth's Mantle", Bull. Seism. Soc. Am. Vol, 55, No. 3, June, 1965, pp 619–625.

"It has been generally accepted for some time that the earth's mantle is "solid" (crystalline). But increasing complications arise as attempts are made to rationalize that state of matter with the growing list of properties of the mantle.

We suggest that materials of the earth's mantle are in a fourth state of matter, which we propose calling soliqueous a combination of solid, liquid, and gases. It includes elements for forming water molecules and allows expanding superheated steam to supply the principal force for elevating and distorting land masses.

Bridgman's experiments on plastic deformation of materials at very high pressures revealed that spasmodic jerky yielding is characteristic. We propose plastic rupture in shear as the primary mechanism by which energy in the earth is converted to the vibrations of earthquakes."

Chander, R. and N. B. James (1965), "Radiation Pattern of Mantle Rayleigh Waves and the Source Mechanism of the Hindu Kush Earthquake of July 6, 1962" Bull. Seism. Soc. Am. Vol. 55, No. 5 October, 1955, pp 805–819.

"The source mechanism of the Hindu Kush earthquake of July 6, 1962 (magnitude $6\frac{3}{4}-7$, focal depth 218 km) was studied by comparing the observed amplitude and phase radiation patterns of mantle Rayleign waves of 150 sec and 200 sec period with theoretical radiation patterns of Rayleigh waves from single and double-couple point sources, and by considering evidence from Love waves and the shape of P and S pulses. The Solution for the source mechanism, which is consistent with all the body wave and surface wave data available for this earthquake, is a double-couple acting as a step function in time with nodal planes oriented as determined from P wave data. Since for waves with periods greater than about 5 sec, the source appears to be an ideal point source, the radius of the equivalent source volume is estimated to be less than 10 km. For Rayleigh waves of 150 sec period, the agreement between observed and theoretical phases (for the above source model) is greatly improved by assuming regional phase velocities instead of a uniform phase velocity for all areas. It is concluded that with the accuracy currently attainable, a study of Rayleigh waves alone cannot determine the source mechanism of an earthquake uniquely."

Alterman, Z. and F. Abramovici (1965), "Propagation of a P-Pulse in a Solid Sphere" Bull. Seism. Soc. Am. Vol. 55, No. 5, October, 1965, pp. 821-861.

"An exact solution is obtained for the displacement of the surface of a uniform elastic solid sphere of radius 'a' due to an impulsive compressional pulse from a point-source situated at a distance 'b' from the centre. The duration of the source is δ a/c where 'c' denotes the Shear-wave velocity, and its time-variation is such that the surface-displacement stays finite when the time tends to infinity.

The solution is applied to a source at a distance of one-eighth of the radius below the surface, approximating a deep-focus earthquake. Theoretical seismograms, radial and angular component, are given at distance $0 < \theta < \pi$ for a source of duration 0.03a/c. Rayleigh waves are clearly seen at $\theta \leqq 45°$. Groups of reflected waves, especially predominant in the angular component, have the velocity of the lowest Airy phase in the group-velocity dispersion-curves. Diffracted waves, discussed in a previous paper, are found here again and in certain cases have an amplitude seven times larger than the amplitude of the direct pulse and also larger than any of the reflected pulses at the same distance. The transformed phases PS_n, P_2S_n have in general larger amplitude than the reflected P_n. Arrival times, initial amplitudes, reflection and convergence coefficients of pulses are obtained by steepest descents analysis and compared with the complete results."

Geophysics

Rosenbaum, J.H. (1964), "The Response of an Elastic Plate Submerged in a Liquid Half-space to Explosive Sound", Geophysics Vol. XXIX, No. 3, June-64, pp. 370-394.

"A mathematical analysis is presented for the case of a point-source explosion in the liquid layer above an elastic plate of infinite horizontal extent immersed in a liquid half-space parallel to the free surface of the liquid. An asymptotic solution, valid for long times after the explosion, is derived ; it expresses the pressure response in the liquid layer in terms of characteristic vibrations of the layered medium. Trapped and exponentially decaying modes have been investigated numerically for the Lucite plate in water. Special emphasis is placed on the description of sustained reverberation (singing). This phenomenon is described in terms of complex modes, where some energy travels back radially towards the source. At long times, singing can be described in terms of "standing" waves of non-vanishing horizontal wave number. It is also closely condected with a type of trapped wave in the liquid layer plate combination whose horizontal phase velocity is greater than the velocity of sound in the fluid, but which is completely decoupled from the liquid half-space below the plate. At very long times, however, the strongest signal is associated with an almost completely decoupled shear motion of the plate, and the horizontal wave number approaches zero. A brief discussion of the total transmission of plane harmonic sound waves through a Lucite plate in water is given The total transmission curves are used to show qualitatively that singing often may not be observed in connection with the above-mentioned trapped waves.

Hall, D.H. (964), "Magnetic and Tectonic Regionalization on Texáda Island, British Columbia", Geophysis, Vol. XXIX. No. 4, Aug. 64, pp. 565-581.

A significant correlation exists between direction patterns in magnetic and tectonic trends on the northern two thirds of Texada Island, British Columbia, Canada. The patterns were analyzed in further details by dividing the area into regions based on amplitudes of magnetic anomalies and on patterns in the magnetic trends. A technique involving smoothening and cross correlation was applied in the examination of the relationship between the patterns. The regions established on a magnetic basis also are distinct on the basis of tectonic pattern. However, correlation between the magnetic and tectonic patterns varies from region to region within the area.

The directions N 50°W, N–S, and E–W, common throughout the coastal area of British Columbia, are the most widespread in the patterns studied on Texada Island. A distinctive zone, identified as a fault zone cutting across the island with a trend of N20°W stands out in the patterns".

Miscellaneous

Krishna, Jai and A.S. Arya (1965), "Earthquake Resistant Design of Buildings" Jour. Inst. Engrs. (India), Vol. XLV, No 7, March 1265, Pt. Cl. 4.

"A vast area in India lying along the Himalayan ranges and the Kutch, has experienced a large number of earthquke shocks in the past and is continuous to do so at present. The adjoining countries. e.g. Burma, Tibet and Nepal have faulted formations which cause earthquakes close to the Indian border, resulting in damage to structures in the country. So far, a major part of this region was thinly populated, but rapid pace of industrialization is inducing large number of people to settle in this area. Hence the problem of considering as to how the present methods of construction of buildings have to be modified so that damage to life and property due to future earthquakes is minimized. An attempt is made in this paper to explain the behaviour of buildings and building frames when subjected to earthquake forces and the basic principles that should govern the improvements to be made in their design without heavy additional cost".

Licari, I.S. and E.N. Wilson (1962), "Dynamic Response of a Beam Subjected to a Moving Forcing System". Proc. 4th U.S. Nat. Cong. Applied Mech. 1962.

"A procedure is presented for obtaining exact solutions of the fundamental differential equations governing the dynamic response of a simply supported prismatic flexural member loaded by a damped spring mass system moving with constant velocity across the span. Effects of inertia, gravity, and damping of both the flexural member and forcing system are considered.

Expressions are derived for displacements and acceleration of the beam and the spring-borne mass for any position of the moving load. The procedure is general and can be extended to more complex systems with arbitrary initial and boundary conditions".

Caughey T.K. (1962), "Vibration of Dynamic Systems with Linear Hysteretic Damping (linear theory). Proc. 4th U.S. Nat. Cong. Applied Mech. 1962.

"Using a viscoelastic model, proposed by Biot, a study is made of the vibration of dynamic systems with linear hysteretic damping. Exact solutions are presented for transient and steady state vibrations, and it is shown that a rotating shaft, exhibiting linear hysteretic damping, will whirl above its critical speed".

Boyce, W.E. (1962) "Random Vibration of Elastic Strings and Bars" Proc. 4th U.S. Nat. Cong. Applied Mech. 1962.

"The paper considers the free transverse vibrations of elastic strings of random density per unit length, and the free longitudinal vibrations of elastic rods of random cross sectional area. Mathematically it is necessary to consider second order Sturm Liouville eigen-value problems having one or more random coefficients. Variational, asymptotic, and perturbation methods are employed to study the statistical relation between the eigen-values and the random material properties of the string or bar".

Nigam, P.S. (1965), "Elastic Foundations of Concrete for Turbo-Alternators", The Indian Concrete Journal, Vol. 39, No. 11 pp. 434-449.

"Until recently, the resonance method was used for the design of framed foundation for machinery. This article also explains two other methods–the amplitude method developed in the U.S.S.R. and the combined method developed in Hungary and formulates a comparison between all the three. The material is presented with particular reference to the foundations of turbo-alternators for steam power stations".

Entrican, G.C. (1965), "Detailing for Ductility in Reinforced Concrete Frames Subject to Earthquake Forces", The Structural Engineer, Vol. 43, No. 8, pp. 242.

"The paper outlines the uses that can be made of ductility in reinforced concrete frames, the enhanced properties of ductile frames and the means of achieving ductility. It also treats the particular application of ductility to earthquake design and the very close relationship between the ductility of frames and ultimate method of design".

Malhotra, A. (1965), "Designing a Forging Hammer Foundation", The Indian Concrete Journal, Vol. 39, No. 7 pp. 269–273.

"The design of the foundation for a forging hammer can be either highly mathematical or highly empirical. The purpose of this article is to present a method which is a compromise between the two and which can be easily followed in practice. The basic requirements for a well designed foundation have been outlined and the formulae and terms involved in the design procedu e are explained. A numerical example for a 5-tonne hammer foundation is given. The procedure illustrated can be used for designing almost any type of hammer foundation".

Press, F. (1965), "Resonant Vibrations of the Earth", Scientific American, Vol. 213, No. 5, Nov. 1965, pp. 28–37.

"When a major earthquake occurs, the entire earth vibrates like a ringing bell. These extremely slow "free oscillations" yield information on the structure of the earth's crust and mantle".

Prakash, S. and J.N. Mathur (1965), "A Pore Pressure Pick-up for Dynamic Studies of Soils" Jr. of Indian National Society of Soil Mechanics and Foundation Engineering, Vol. 4 No. 3 July 1965 pp. 299–312

"Measurements of Pore pressures in the laboratory and in model studies is problem which involves several difficulties especially if the pore pressures are induced by dynamic loading. Several types of pore pressure pick-ups are available for static loading. These included the piezometric, the null indicator and diaphragm types. In this paper, the requirements of a diaphragm type pore pressure pick-up for dynamic model studies have been discussed and design and performance of one such pick-up assembled in the School Workshop is presented".

NEWS OF MEMBERS

1. Dr. A.R. Chandrasekaran, Reader, School of Research and Training in Earthquake Engineering has been awarded the Khosla Research Prize of Rs. 500/- for his paper entitled "Vibration Analysis of Earth Dams".

2. Sri Sohan Lal Agarwal and Dr. Shamsher Prakash, Lecturer and Reader respectively, School of Research and Training in Earthquake Engineering have been awarded the Khosla Research Silver Medal for their paper entitled "Determination of Lateral Load Carrying Capacity of Pile Groups in field".

3. Sri Sohan Lal Agarwal, Lecturer in Civil Engineering, School of Research and Training in Earthquake Engineering has been offered teaching assistantship in the University of Texas, Austin, U.S.A. for advanced studies. He will be working under the guidance of Prof. L.C.Reese for his advanced degree in Soil Dynamics.

Proceedings of the Third World Conference on Earthquake Engineering

The Third World Conference on Earthquake Engineering was held in New Zealand from January 22 to February 1, 1965. The Proceedings of this Conference are expected to be available in June 1966, at an approximate price of £ N Z 12 per set of 3 volumes. Orders for the Proceedings may be placed at the following address:—

Mrs. J.H. Van Roekel,
Administrative Secretary,
N.Z. National Committee
on Earthquake Engineering
P.O. Box 5180,
Wellington, Newzealand.

Proceedings of the First and Second Symposia on Earthquake Engineering held at the School of Research and Training in Earthquake Engineering in Feb. 1959 and Nov. 1962 respectively are available for sale at Rs. 12/- each copy, orders may be placed at the following address:—

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      Title of work, Source (in full), volume number, page number (beginning)—page number (end).

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.

Krishna, J., S. Prakash and J.N. Mathur (1965), "Study of Liquefaction of Sands with Particular Reference to OBRA Dam" Earthquake Engineering studies, University of Roorkee, Roorkee, Aug. 1965.

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