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BULLETIN

INDIAN SOCIETY
OF
EARTHQUAKE TECHNOLOGY

ROORKEE U. P. (INDIA)

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INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY

Roorkee (U.P.) India

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A.N. Tandon,	Vice President
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Shamsher Prakash,	Editor

A 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

Bulletin

INDIAN SOCIETY OF EARTHQUAKE TECHNOLOGY

ROORKEE, U.P. (INDIA)

Vol. II | July 1965 | No: 2

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EDITORIAL

Dear Member,

It is with great satisfaction that we are presenting the fourth Bulletin (Vol. II No. 2) as per schedule. Subsequent to the registration of the Society as intimated in Third Bulletin (Vol. II No. 1), we have requested the Government of India for funds to improve the financial position of the Society. We hope that the Government of India will give us the necessary aid.

A few changes will be noticed in this issue of the Bulletin. Firstly, we have introduced reviews of papers published in other journals on problems connected with Earthquake Technology. It is proposed to include under this section the gist of the significant publications in future.

A column on "News of Members" has also been introduced. We shall know the changes in placement, status of other members, and distinctions achieved and this will assist us in maintaining better contacts.

A meeting of the executive committee was called by the Secretary on 13th March 1965 at Roorkee. A report on this meeting appears else where in this Bulletin. The School of Research and Training in Earthquake Engineering, Roorkee has announced that the Third S mposium on Earthquake Engineering will be held in November 1966. The last date for submission of complete papers with original drawings is June 30, 1966. A detailed announcement appears elsewhere.

The papers published in this Bulletin are open for discussion upto 15th December 1965. Contribution 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 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. The intention to present a paper and the possible title may please be sent by March, 31, 1966.

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.

EARTHQUAKE ENGINEERING MAKES PROGRESS

Jai Krishna*

III 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 First World Conference was held in 1956 at the University of Berkely, (U.S.A.) and the Second at Tokyo in 1960. The proceedings of the first two conferences have since been published. The proceedings of the Third Conference are being printed and could be available from the Administrative Secretary, III World Conference on Earthquake Engineering, Post Box No. 5180, Wellington, New Zealand after October 1965. The conference was attended by specialists from all countries of the world. There were about 180 foreigners and equal number of New Zealand Engineers, Seismologists and Geologists. The total number of papers that were accepted for publication were one hundred and eleven. The papers also include reports on the earthquakes that have recently occurred in North Africa, Mexico, Alaska, Iran, Yugoslavia and Kashmir.

The topics of discussion at the conference were as follows -

1. Soil and Foundation Conditions Relative to Earthquake Problems.
2. Seismicity and Earthquake Ground Motion.
3. Analysis of Structural Response and Instruments:
4. Earthquake Resistant Design, Construction and Regulations.
5. Recent Strong Motion Earthquakes and Resulting damage.

The only papers presented from India were by the Earthquake Engineering Research and Training Centre, Roorkee as follows :

1. "Dynamic Behaviour of Earth Dams" Jai Krishna and Shamsher Prakash.
2. "Structural Response Recorders" Jai Krishna and A. R. Chandrasekaran.
3. "Water Towers in Seismic zones" Jai Krishna and A. R. Chandrasekaran.
4. "Strengthening of Brick Buildings Against Earthquake Forces" Jai Krishna and Brijesh Chandra.
5. "Study of a Vertical Pile Under Dynamic Lateral Load" Shamsher Prakash and S. L. Agarwal.

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

6. Joint Rotation Effects on the Dynamics of Multisioreyed Frames" A. R. Chandrasekaran.
7. "A Pore Pressure Pick-up for Dynamic Studies" Shamsher Prakash and J. N. Mathur.
8. "Seismic and Tectonic History of Indo-Gengetic Plains" L. S. Srivastava and R. S. Mithal.
9. "Engineering Aspects of Badgam Earthquake" Jai Krishna.

Many of the papers were very interestingly discussed by the delegates and proceedings will include the discussions in detail. Besides the discussions of technical papers, the delegates visited the Seismological Laboratory and the GEYSERS at WEIRAKEI. At Weirakei there is a bubbling of hot steam from the ground and it has been utilized for generation of power.

UNESCO MEETINGS ON SEISMOLOGY, SEISMO-TECTONICS AND EARTHQUAKE ENGINEERING

The UNESCO has formed 2 Working Groups, one on Seismology and Seismo-tectonics and the other on Earthquake Engineering. On the former working group is Dr. A. N. Tondon, Vice-President of the Indian Society of Earthquake Technology, as a member and on the Working group on Earthquake Engineering is Dr. Jai Krishna, President of the Indian Society of Earthquake Technology, as a member.

During these meetings the following major topics were discussed :

(a) Standardization of Seismograph and Strong Motion Instruments for measurements of earthquakes and response of structures to earthquakes respectively.
(b) Revision of Micro-seismic Intensity Scale by providing greater detail in each item so that there is less variation in the allocation of Intensity from one description and the other. This scale was discussed at Paris in April 1964 and was published in the Bulletin of Indian Society of Earthquake Technology for the month of July 1964. This is being improved further.
(c) Providing a Skeleton draft for the Earthquake Resistant Regulations and Code for different countries in the world.
(d) Standardising the methods of preparing the seismic maps for the world and for each country and specific regions.
(e) Protective measures against Tsunamis.
(f) Education and training in Earthquake Engineering and Seismology.
(g) Research in Earthquake Engineering and Seismology.
(h) Publication of data and a directory containing centres of training and research as well as list of journals published on Earthquake Engineering and Seismology.

(i) Housing in Seismic Zones.

Dr. A. N. Tanton has been entrusted with the task of compiling a list of earthquakes that have occured in India, Afganistan, Pakistan and South East Asia.

Dr. Jai Krishna has been entrusted with the study of problem of housing in Seismic zones taking into account the prevalent construction methods and materials in different parts of the world.

In this manner UNESCO has taken very positive steps to standardize the methods of study of the problem resulting from earthquakes in the seismic countries of the world.

SUMMARY OF MEETING HELD ON MARCH 13, 1965

1. Only those of the members from Roorkee attended the Executive Commitee Meeting, and those from outside Roorkee regretted their absence.

2. There was no mention in the constitution about the quorum that is required for such meetings. For executive committee meetings it was decided to apply the same rules as is required for general meeting namely 25% of total membership.

3. The constitution requires that elections take place annually at the time of annual general meetings. Since even for Executive Committee meeting, members from out side Roorkee did not attend, it was felt that there may be very little response for annual general meetings. Consequently it is proposed to hold election by postal ballot. It was decided that only those who have paid their membership dues for the year 1965 as of May 31 (now extended upto 31 August 65), would be eligible for voting (This information is being circulated to those who have not paid so far). The Committee felt that the post of Secretary-cum-treasurer should be separated and there should be a separate Secretary and Treasurer. It was also felt that at least during the formative stages of the Society the post of Secretary, Treasurer and Editor be held from among those who are members from Roorkee. (This will also help to maintain the quorum at all Executive Committee Meetings).

Paper No. 26

A LARGE SHEAR - BOX FOR DYNAMIC TESTS ON SOILS

Shamsher Prakash* and Gopal Ranjan**

SYNOPSIS

The paper describes the design of a 30 cm × 30 cm strain controlled shear box machine for dynamic tests on soils. Any rate of loading from 0.004 cms/sec to 6.4 cms/sec can be achieved by the unit. The performance of machine is judged by tests on sand.

INTRODUCTION

Designing of dams in seismic zones stresses the need to study the stress deformation and strength charateristics of pertinent soils under dynamic loads. Besides that, in the conventional type of shear machines sample size of 6 cm × 6 cm × 2.5 cm is generally tested. These dimensions are so small that soils over a certain grain size cannot be tested accurately. Moreover, in case of high rock-fill dams the soil is subjected to high normal stress at base, and to correctly estimate the value of shear strength it is necessary that the soil is tested under corresponding load in the laboratory.

A 30 cm × 30 cm sample size strain controlled shear box is designed and fabricated, for carrying out dynamic shear tests on gravels and sand-gravel mixtures. The machine is designed for a maximum normal stress of 4.4 kg/sq. cm on the sample. The movement of the box is by the advancement of a lead screw which is driven through a gear box. The gear box reduces speed in ratios of 1/1000, 1/100 and 1/10. The speed of motor, driving the gear-box is controlled by a "speed control unit". The flexibility of the speed control unit affords to attain any rate of strain within limits of 0.004 cms/sec (0.1 in/min) to 6.4 cms/sec (150 in/min). The recording of load displacement is through proving frames with electric resistance strain gauges pasted on it.

DESIGN OF MACHINE

The machine can be devided into following four units:

SHEAR-BOX

The 30 cm × 30 cm box is made in two halves from 2 cms thick mild steel plates joined by allen screws. The box is provided with top plate with grid on one of its faces for proper

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transmission of load, the plate weighs 20 kg. The bottom plate is also provided with grids. Four spacing screws are provided with steel balls at the ends. During preparation of sample the two halves of the box are pinned together with two removable but locking screws. The box is housed in a water jacket, mounted on two trains of ball bearings, which consist of 10 steel balls 1.875 cm (3/4″) diameter in each train. Stops, guides, are provided to prevent the derailment of water jacket. Figure 1 shows the various components of box.

LOADING UNIT

The normal load on the sample is applied by means of single lever system with lever-ratio 1 : 15. The lever beam is made of I–Section weighing 13.785 kg. The loading hanger made up of (7.5 cm × 15 cm) I-section and (7.5 cm × 3.75 cm) channel section weighing 30 kg. rests on top-plate on sample. The weights are placed on the pan attached to one end of the beam.

DRIVING UNIT

The application of shear load is through a lead screw coupled to the put shaft of a gear box with gear ratios 1/1000, 1/100 and 1/10. The input shaft of the gear box is fed by

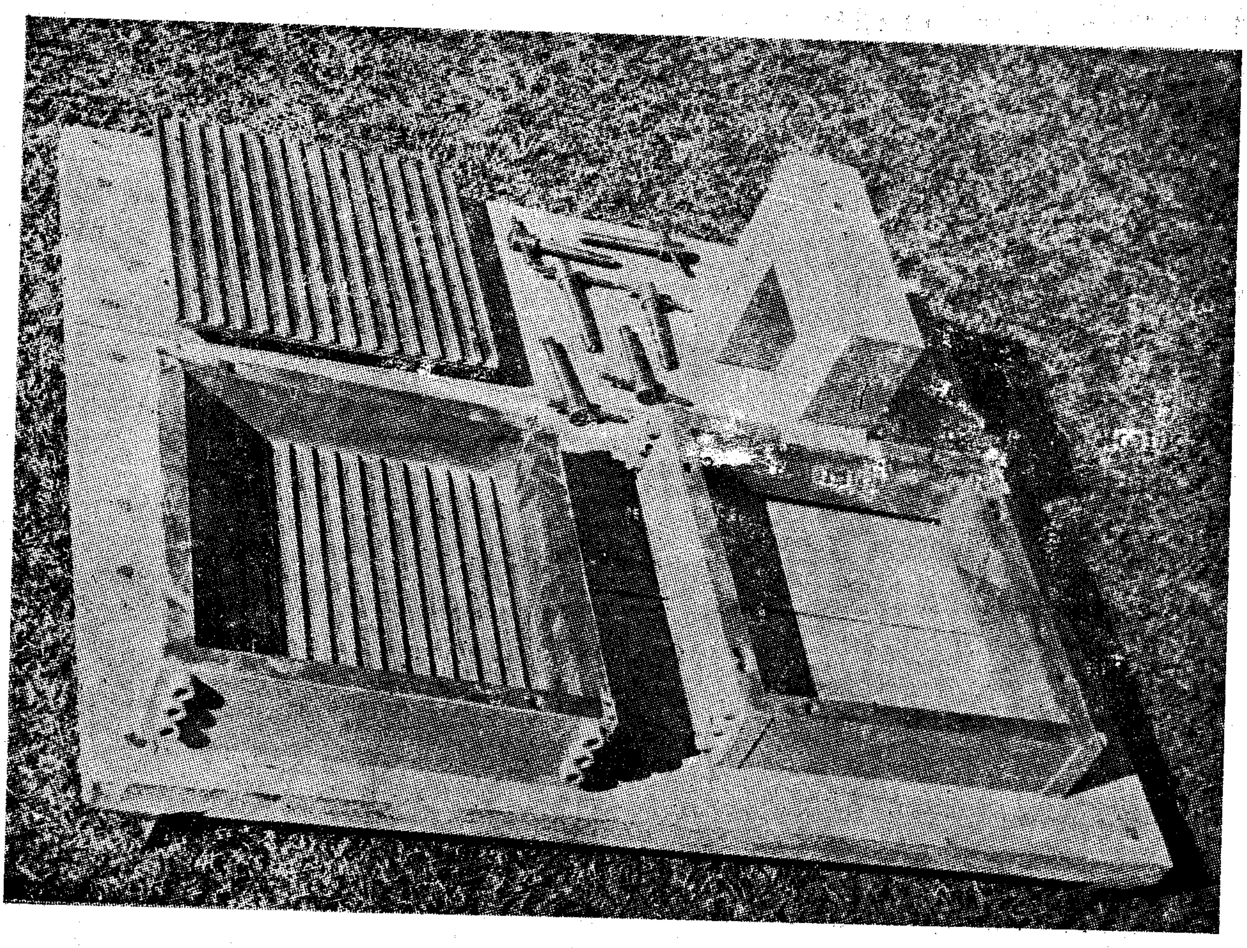

Figure 1 Components of Shear Box

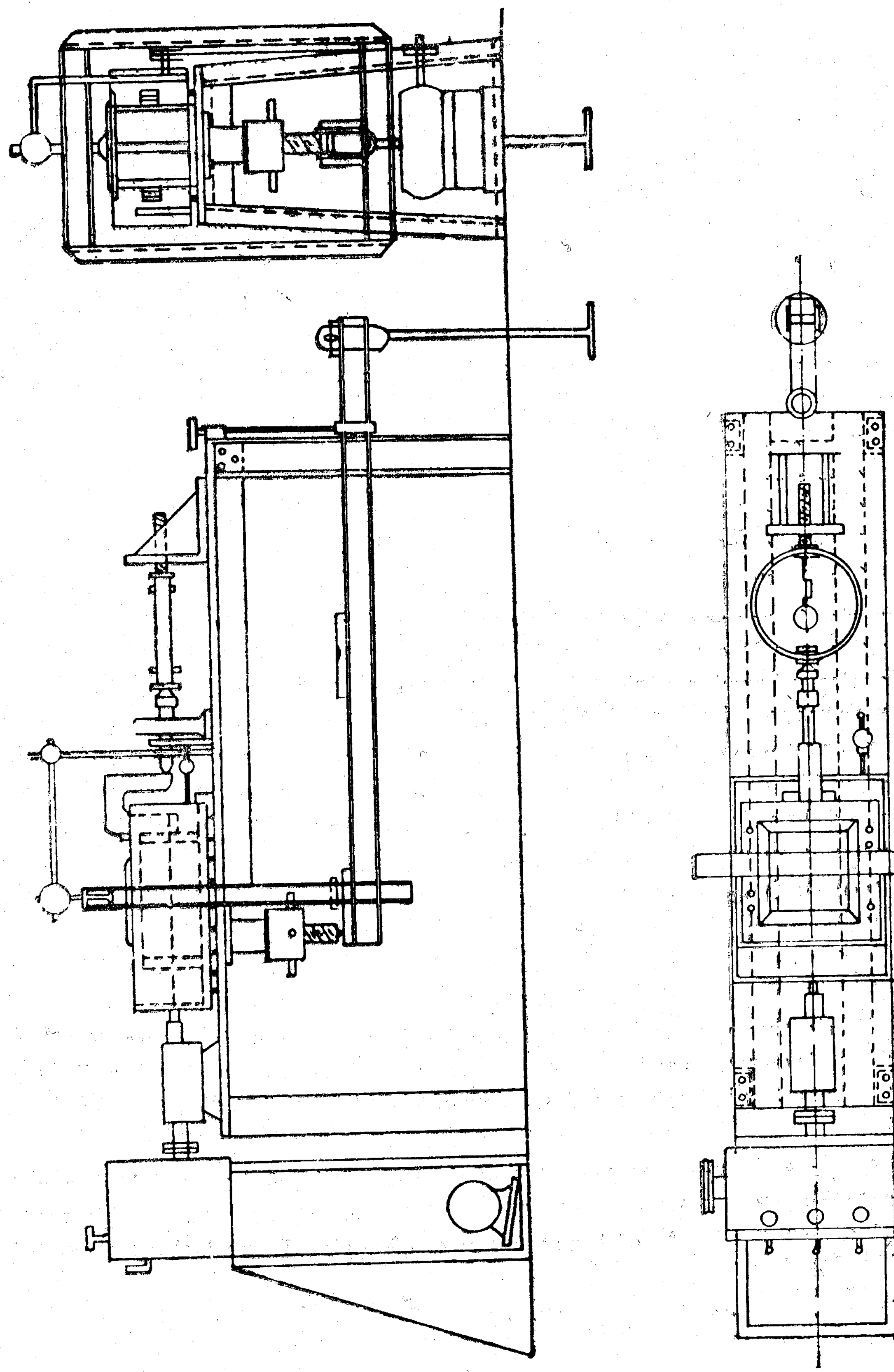

Figure 2 Assembly of Shear Box Machine.

a D.C. motor (1500 r.p.m., 440 volts) through a "speed control unit". Figure 2 shows a detailed drawing of the assembly of the machine.

INSTRUMENTATION

For measuring the load, the load-gauge is designed based on the criterion suggested by Casagrande and Shannon (1947). The proving frame used had the following dimensions :

External diameter	=	20 cms
Internal diameter	=	16 cms
Thickness	=	6 cms

Four SA-10 Rohits electric resistance strain gauges were pasted. Specifications of gauges :

Type	SA—10
Pattern	Flat
Overall size	24 × 12 mm.
Gauge length	10 × 5 mm.
Resistance in Ohms	122 ± .025%
Gauge factor	2.05

The proving frame was calibrated under compression in a 2-ton Universal Testing Machine. Calibration was done under progressive loading between deflection of 0.002 mm. dial of proving ring and number of chart lines deflected as recorded by a Universal Amplifier and Automatic pen recorder (Brush Oscillograph)*.

In addition to the design criteria of load gauge it was necessary that the deformation gauge should take minimum load so that effective shear load is not reduced. Moreover for maximum deformation it should not loose its initial geometry. With these points in view a ring was made from clock spring with these specifications:

Internal diameter	=	18.5	cms
Thickness	=	0.0356	cms
Width	=	0.87	cms
Weight	=	11.75	gms

The ring was provided with four SA-10 electric resistance strain gauges pasted on it, comprising of two tension and two compression gauges. It was calibrated using a dial extensometer to measure displacement and number of chart lines deflected as recorded by pen recorder.

A micro-switch is provided to switch off the motor under fast rates of loading and to protect the motor from being damaged. The switch is mounted on the table and is properly shielded against the inertia of box at fast rates of strain. Figure 3 shows the complete assembly of the box.

* For details, refer to manufacturer's catalogues

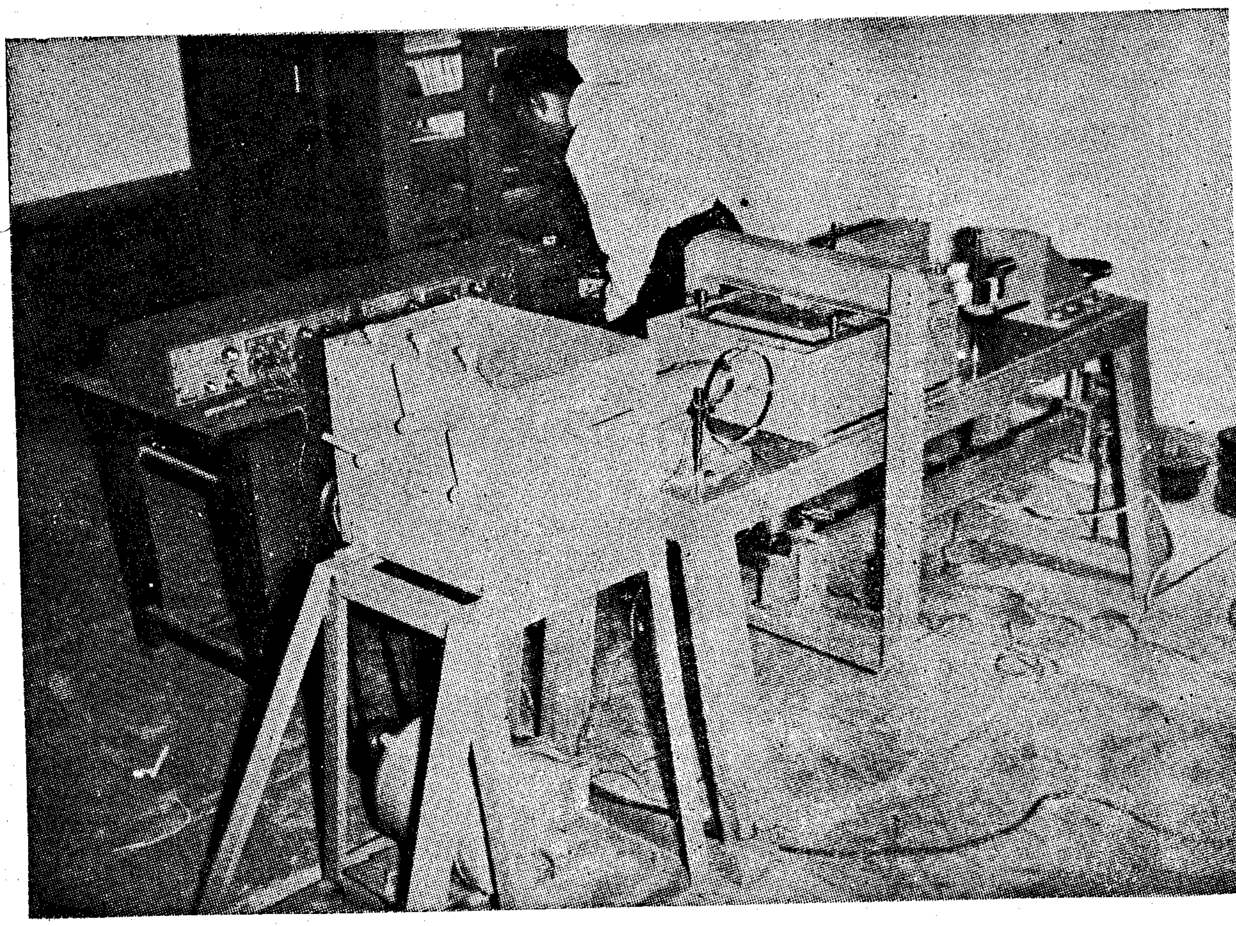

Figure 3 Dynamic Shear Test in Progress

PERFORMANCE OF MACHINE

From the tests carried out on the sand from Obra Dam site (Ranjan 1965) it was observed that the machine functioned satisfactorily for the time of loading upto two seconds. Tests on this sand were performed for rates of loading from 0.0062 cms/sec to 0.36 cms/sec. Tests on rates of loading faster than 0.36 cms/sec could not be performed due to some difficulty.

COST ANALYSIS

The large scale shear box for dynamic tests was fabricated in the workshop of the School. The box along with its components, table on which it is mounted and with loading arrangement for normal load and gear-box was fabricated at an approximate cost of Rs. 3000.00. The speed control unit along with the D.C. motor was designed and assembled at an approximate cost of Rs. 5000.00. The proving ring used for measuring load was purchased from Associated Instruments Manufacturers India Ltd. for about Rs. 1000.00.

SUMMARY AND CONCLUSIONS

The machine functioned satisfactorily for the time of loading upto two seconds. I was observed that U-frame attached to the upper half of the box causes eccentricity whict results in the tilting of the upper frame and the top plate. To minimise tilting the U-frame should be replaced by a solid rod attached to the upper frame and abutting against the proving ring.

ACKNOWLEDGEMENTS

The paper is based on Master of Engineering Dissertation of the junior author. The machine was fabricated in the workshop of the Earthquake Engineering School. Cooperation of other staff is acknowledged. The paper is being published with the permission of the Director of the School.

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Ranjan, G. (1965), "Design and Performance of a Large Shear Box for Dynamic Loads", M.E. Thesis, Deptt. of Civil Engineering, University of Roorkee, Roorkee.

Paper No. 27

FREE PERIOD OF VIBRATION OF R C.C. ELEVATED WATER TOWERS

Anand Prakash*

SYNOPSIS

In this paper a simplified formula is evolved for prediction of free period of vibration of R. C. C. elevated water towers. The formula enables to calculate the period knowing the height of staging, height of the centre of the mass, diameter of the ring beam at the top of the staging and the seismic coefficient for which the tower is intended to be designed. This will greatly simplify the aseismic design of water towers since knowing the period and assuming a reasonable value of damping coefficient the lateral force acting on the tower during an earthquake can be known with reasonable accuracy from average acceleration spectrum curves, and the tower can be designed for the lateral force so calculated.

It is also concluded that for the same magnitude of the earthquake, towers with higher staging should be designed for lower value of seismic coefficient to allow for the increased flexiblity with increase in height of the tower.

INTRODUCTION

It is very necessary that an effective water supply system be maintained during an earthquake seeing life and fire hazards generally accompanying it. It requires that the water towers be designed earthquake resistant in seismic zones. For this purpose the lateral force acting on the tower due to the expected earthquake magnitude should be accurately estimated and the tower be designed for it. The lateral force can be determined if the dynamic characteristics of the tower i.e. free period and damping coefficient are known. Therefore it would be very helpful if the free period of the tower is known before the actual design of the tower is taken up. An attempt has been made for the above purpose in this paper.

ASSUMPTION AND SCOPE

The following assumptions are made :

1. The elevated tower supporting tank is a system with a single degree of freedom with the mass concentrated at the centre of gravity of the tank.

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2. The free period T, in secs. is calculated from the following formula :

$$T=2\pi\sqrt{\delta/g}$$

where δ=the static deflection at the top of the tank under a static horizontal force equal to its own weight W acting at the centre of gravity of the tank.

g=acceleration due to gravity.

3. The staging is designed throughout of uniform strength i.e. the section of the staging at any height is provided in accordance with the forces carried by it.

4. The staging is assumed as consisting of a circular shell of uniform thickness and same ratio of longitudinal reinforcement throughout the height. The area and moment of inertia of the section is increased by increasing uniformly the diameter of the shell downward. This is clearly shown in figure 1.

5. Moment of inertia is calculated on the basis of the effective section i.e. total area of concrete section plus the transformed area of steel.

The assumptions (1) & (2) are taken from Indian Standard 1893-1962 and are reasonable.

Assumption (3) is justified both from design and economic considerations.

Assumption (4) is made for simplification of the analysis made in this paper. However, it does not disallow the use of the formula evolved for other types of stagings such as one with columns and braces since in that case, the only difference is that the section is lumped at few points instead of providing continuous along the circumference of a circle.

Assumption (5) is justified since the section will be subjected to heavy compressive load of the tank at all times and the whole section including the transformed area of steel shall be effective. Tnis is also on the safer side from the dynamic analysis point of view.

NOTATIONS

Following are the notations used in this paper.

T=Free period of vibration of the water tower.

W=The weight of the tank when full.

δ=The static deflection at the top of the tank under a static horizontal force W.

g=Acceleration due to gravity.

H=Height of the staging.

h=Height of the centre of the mass of the tank from the top of the staging.

D_1=Diameter of the supporting ring beam at the top of the staging.

D_2=Diameter of the supporting ring beam at the bottom of the staging.

D=Diameter of the circular shell at a depth x from the top of the staging.

x=Depth of the cross section under consideration from the top of the staging.

d=Thickness of the shell for the staging.

A=Area of cross-section of the staging=πdD.

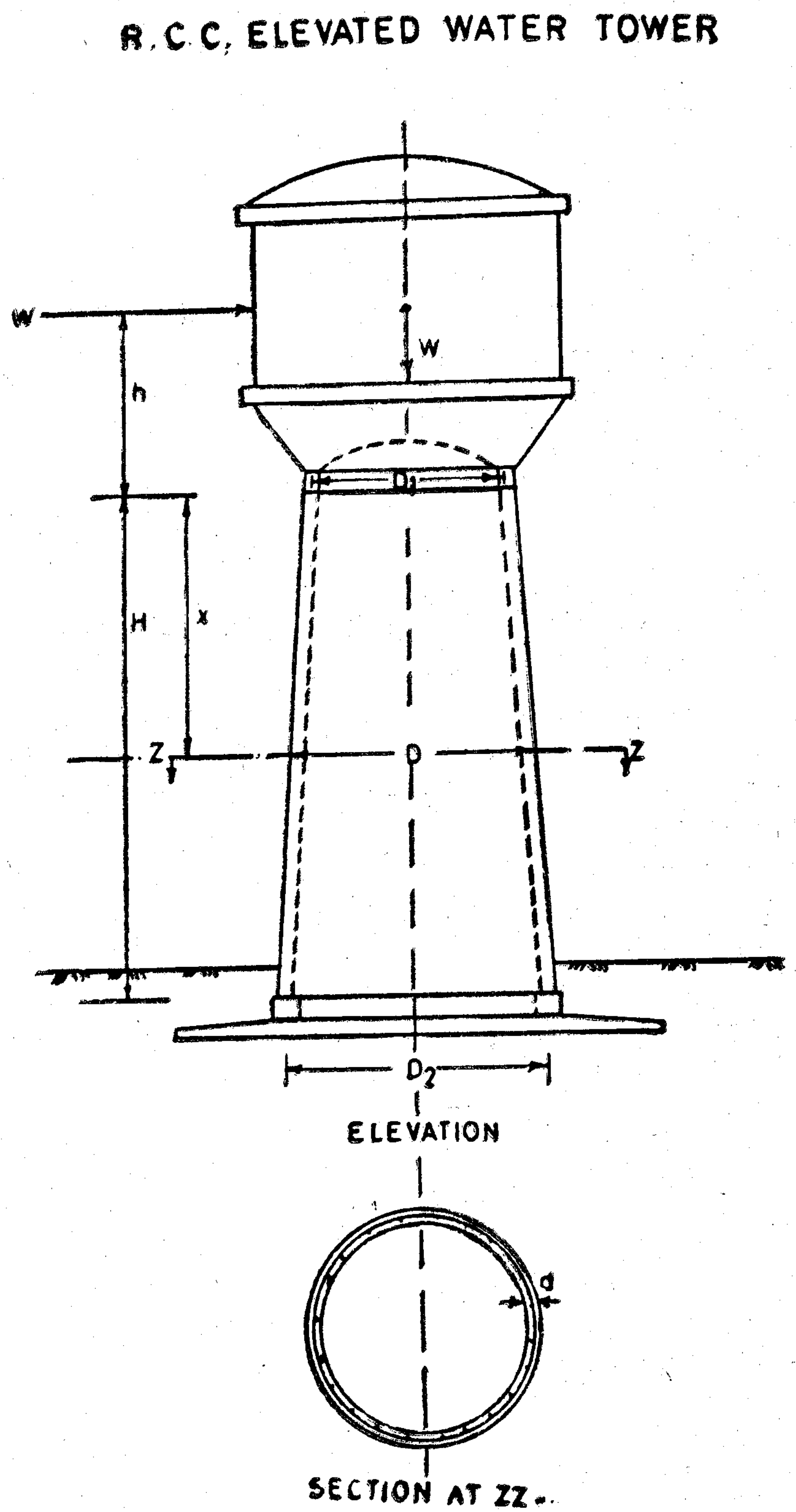
R.C.C. ELEVATED WATER TOWER
W
w
h
D1
H
x
Z
D
Z
D2
ELEVATION
d
SECTION AT ZZ.

Figure 1

A_t = Area of the longitudinal reinforcement.

$p = A_t/A$

m = Modular ratio

d_e = effective thickness of the shell $= d\,\{I+(m-I)p\}$

A_e = effective area of x-section $= \pi d_e D$

I_e = Effective moment of inertia $= \pi d_e D^3/8$

c = maximum allowable compressive stress in concrete.

α_h = Design seismic coefficient.

E = modulus of elasticity of concrete.

n = Dimensionless factor $= H/h$

a = Dimensionless factor $= h/D_1$

b = dimensionless factor $= D_2/D_1$

k = Dimensionless coefficient depending upon α_h

K = Dimensionless coefficient, a function of α_h, n and a

M = Bending moment at any cross-section.

N = Multiplying factor depending on the magnitude of the earthquake.

$\triangle$ = Maximum dynamic deflection of the tower.

METHOD AND CALCULATION

The cross-section of the staging at any depth x is subjected to a direct load W and a bending moment M given by,

$$M = \alpha_h W(x+h) \tag{1}$$

The max^m compressive stress in concrete is given by

$$c = \frac{W}{A_e} + \frac{M}{I_e}\,\frac{D}{2} \tag{2}$$

or

$$c = \frac{W}{\pi d_e D} + \frac{\alpha_h W(x+h)}{\pi d_e\, D^3/8}\cdot\frac{D}{2} \tag{3}$$

$$= \frac{W}{\pi d_e D}\left[1 + \alpha_h(x+h)\,.\,\frac{4}{D}\right] \tag{4}$$

For cross-section at the top of the staging

$$x=0,\ D=D_1$$

$$\therefore \quad c = \frac{W}{\pi d_e D_1}\left[1 + \alpha_h(0+h)\,.\,\frac{4}{D_1}\right]$$

or

$$d_e = \frac{W}{\pi c D_1}\left[1 + 4\alpha_h \frac{h}{D_1}\right] \tag{5}$$

For the cross-section at the bottom of the staging

$$x=H,\ D=D_2=bD_1$$

$$c = \frac{W}{\pi d_e D_2}\left[1 + \alpha_h(H+h)\,\frac{4}{D_2}\right]$$

From which

$$D_2=\frac{W}{2\pi d_e c}\left[1+\sqrt{1+\frac{16c\pi d_e}{W}\alpha_h(H+h)}\right] \tag{6}$$

Putting the value of d_e from (5) into (6), we get

$$D_2=\frac{D_1}{z\left(1+4\alpha_h\frac{h}{D_1}\right)}\left[1+\sqrt{1+\frac{16\alpha_h(H+h)}{D_1}\left(1+\frac{4\alpha_h h}{D_1}\right)}\right] \tag{7}$$

Now putting

$$\frac{D_2}{D_1}=b\,;\ \frac{h}{D_1}=a\,,\ \frac{H}{h}=n$$

$$\frac{H}{D_1}=\frac{H}{h}\times\frac{h}{D_1}=na$$

We get from equation (7)

$$b=\frac{1}{2(1+4\alpha_h a)}\left[1+\sqrt{1+16\alpha_h a(1+n)(1+4\alpha_h a)}\right] \tag{8}$$

Increasing the diameter of the shell uniformly from D_1 to D_2 downward

$$D=D_1+\frac{D_2-D_1}{H}x=D_1\left[1+\frac{(b-1)x}{H}\right] \tag{9}$$

The deflection δ at the centre of the mass due to a static horizontal load W is given by applying moment area method as follows

$$\delta=\int_0^H\frac{W(x+h)^2}{E\pi d_e D^3/8}dx \tag{10}$$

Putting the value of d_e from (5) and value of D from (9) into equation (10), we get on integrating,

$$\delta=\frac{8cha^2}{E(1+4\alpha_h a)}\left[\frac{2n}{b-1}\left\{1-\frac{(1+n)^2}{b^2}\right\}+\left(\frac{2n}{b-1}\right)^2\left\{1-\frac{1+n}{b}\right\}+\frac{4n^3}{(b-1)^3}\log_e b\right] \tag{11}$$

Now period T is given by

$$T=2\pi\sqrt{\frac{\delta}{g}}$$

$$T=2\pi\sqrt{\frac{8cha^2}{E(1+4\alpha_h a)g}\left[\frac{2n}{b-1}\left\{1-\left(\frac{1+n}{b}\right)^2\right\}+\left(\frac{2n}{b-1}\right)^2\left\{1-\frac{1+n}{b}\right\}+\frac{4n^3}{(b-1)^3}\log_e b\right]}$$

or $$T=2\pi\sqrt{\frac{8ch}{Eg}}\times K \tag{12}$$

where,

$$K=\sqrt{\frac{a^2}{(1+4\alpha_h a)}\left[\frac{2n}{b-1}\left\{1-\left(\frac{1+n}{b}\right)^2\right\}+\frac{4n^2}{(b-1)^2}\left(1-\frac{1+n}{b}\right)+\frac{4n^3}{(b-1)^3}\log_e b\right]} \tag{13}$$

and is dimension less

Substituting values of b from equation (8)

$$K = f\,(a,\, n,\, \alpha_h) \tag{14}$$

On actual numerical analysis it is found that values of K as given by (13) are also approximately given by the following simple equation for values of $a=0.5$ to 1.5 and $n=1$ to 10.

$$\therefore \quad K=(kn+1)\,a+(0.6n-0.3) \tag{15}$$

Where k depends upon seismic coefficient α_h and is given by Table No. 1 below

TABLE No. 1

$\alpha_h=0.05$	0.075	0.10	0.125	0.15
$k=1.23$	1.0	0.84	0.7	0.6

Taking an average value of $k=0.9$ for all values of α_h we get

$$K=(0.9n+1)\,a+(0.6n-0.3) \tag{16}$$

Therefore from equations (12) and (16)

$$T=2\pi\sqrt{\frac{8ch}{Eg}}\times\left[(0.9n+1)\,a+(0.6n-0.3)\right] \tag{17}$$

Now using 1:2:4 concrete mix with maximum allowable stress,

$c=1000$ psi (including 33% increase in stresses)

$E=1.67\times10^{6}$ psi

$g=32.0$ ft/sec^2

The period is given by

$$T=0.077[(0.9n+1)\,a+(0.6n-0.3)]\sqrt{h} \tag{18}$$

where h is in feet.

or

$$T=0.14[(0.9n+1)\,a+(0.6n-0.3)]\sqrt{h} \tag{19}$$

where h is in meters.

EXAMPLE

For a 10^5 gallon capacity Intze tank as shown in Fig. 1, say

$h=17'$; $D_1=20'$

$a=\frac{17}{20}=0.85$

Period T is given by from equation (18) on substituting the values of h and a

$$T=0.318(1.365n+0.55) \tag{20}$$

In Table No. 2 are given the values of T, average acceleration S_a for 5% damping, seismic coefficient α_h for Toft California Earthquake with $N=1.6$, Max^m dynamic reflection $\triangle$ due to the above earthquake and values of $\triangle/H$ for heights of the tower from 25 ft. to 100 ft.

CONCLUSIONS

The following conclusions can be drawn.

1. The free period of vibration of R.C.C. water tower can be found out from equations (17), (18) or (19).
2. For the same magnitude of the earthquake, the water towers with higher staging should be designed for lower seismic coefficient to allow for the inereased flexiblity with increase in height of the staging.
3. The maximum dynamic deflection due to the same earthquake in water towers of different heights bears almost the same ratio with height of the staging and is found to be with in permissible limit.

ACKNOWLEDGEMENT

The auther is thankful to Dr. Penzien, UNESCO expert at IISEE for fruitful discussions with him from time to time. He is also grateful to Dr. Jai Krishna and Dr. A. S. Arya for their guidance and encouragement in his work concerning this paper.

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TABLE No. 2

Height H in ft. of the staging	25	50	75	100
$n=\frac{H}{h}=\frac{H}{17}$	1.5	3.0	4.5	6
Free period in sec. $T=0.318\ (1.365n+0.55)$	0.83	1.48	2.13	2.8
Average acceleration cm/sec^2 For 5% damping (S_a) From spectrum curve	117.5	71	59	47
$\frac{S_a}{g}$	0.12	0.072	0.06	0.048
For Taft., California Earthquake with N=1.6 Seismic coeff. $\alpha_h = \frac{S_a}{g} \times 1.6$	0.19	0.115	0.096	0.072
Maximum dynamic deflection $\Delta=\alpha_h \times g \times \left(\frac{T}{2\pi}\right)^2$ in ft.	0.1065	0.205	0.353	0.456
$\frac{\Delta}{H}$	$\frac{1}{235}$	$\frac{1}{244}$	$\frac{1}{212}$	$\frac{1}{219}$

Paper No. 28

ON THE CAUSE OF THE OSCILLATORY CHARACTER OF GROUND MOVEMENT DURING EARTHQUAKES

Umesh Chandra*

The ground motion during an earthquake is of a very complex character, and the disturbance is recorded as a train of waves on a seismogram. The observed oscillatory motion in seismograms may be illustrated with the help of fig. 1, representing a typical earthquake record. Of course, the seismogram as such does not give a true picture of the earth movement, as the motion is amplified and complications arise from the fact that in actual case the period

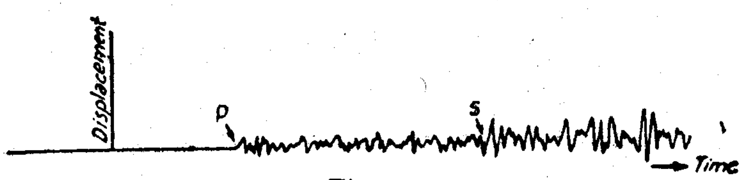

Figure 1

dependent dynamic magnification, of the seismograph and not the static, is of significance. The actual ground movement is still noted to be oscillatory in character, even though its form may not be quite similar to that of the seismogram. It has been considered by earlier workers that these oscillatory movements may be due to dispersion effects. But only the dispersion of surface waves has to some extent lent itself to mathematical formulation and no satisfactory analysis has been put forward to explain the oscillatory character of motion on seismograms attending the body waves.

Theoretical considerations show that an impulsive disturbance in a homogeneous medium should spread out as an impulsive disturbance, and not give rise to a train of oscillations. At a distant point the disturbance takes the form of three movements, namely the P, S and the Rayleigh type respectively, each consisting of a single displacement and a return. As the P and S waves, in the case of a perfectly elastic isotropic media, travel independently of each other, a consideration of a simple hypothetical earthquake sending out P and S waves through a homogeneous medium, gives the graph, displacement vs. time, of a distant particle of the form shown in fig. 2 (see also, Bullen, K.E., 1963, p. 77). The graph does not show any oscillatory movement. There is a quiescent interval between the arrivals of the P and S disturbances.

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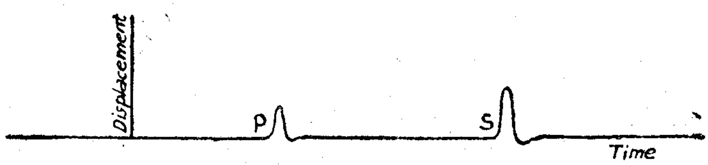

Figure 2

Evidently, the motion recorded on a seismogram following an earthquake is markedly different from that of fig. 2. Jeffreys (1931) has considered the effects of scattering, the complex initial conditions at the focus, fluctuations in the local gravity value during the passage of a disturbance, imperfections in elasticity and continuous variations in density and elastic parameters within the earth and has shown that none of these is a sufficient cause. If the original disturbance at the focus is assumed to be oscillatory, it would explain the oscillations in P and S, but in such a case the duration of the oscillation should be the same for all distances, whereas it is actually observed to increase with distance. The cause of seismic wave dispersion must therefore be sought in terms of heterogeneities inside the earth, most probably in the crust, but the precise way in which it occurs must await more detailed knowledge of the crustal structure than is yet available.

Pekeris (1955) investigated the motion of the surface of a uniform elastic half-space produced by the application at the surface of a point pressure pulse varying with time like the Heaviside unit function. He also considered the case of a burried pulse (Pekeris 1955, Pekeris and Lifson 1957). Pekeris and Longman (1958) considered the motion of the surface of a uniform elastic half-space produced by the application of a burried torque-pulse. The time variation of the torque being assumed to be represented by the Heaviside unit function. In all these analyses the computed theoretical curves do not exhibit any oscillatory movement.

Pekeris and Longman (1958) considered the problem of propagation of explosive sound in layered liquid and derived the exact solution based on ray theory. Here again the time variation of the pressure pulse was assumed to have the form of a Heaviside unit function. For a high speed bottom, the pressure pulse at large ranges, as computed by the ray theory,

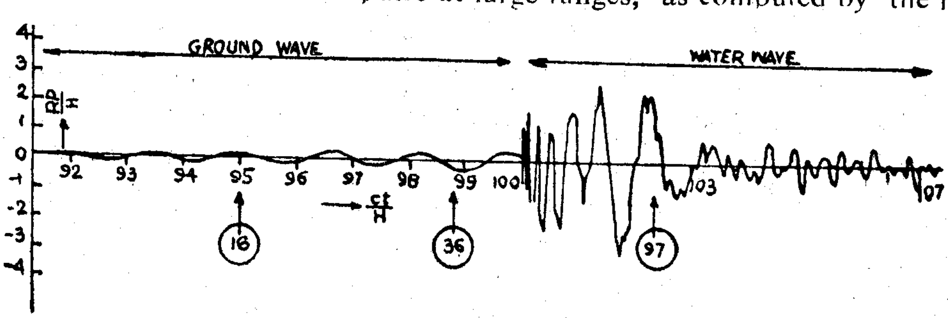

Figure 3. Theoretical Pressure Records due to an Underwater Explosion of Heaviside form in a Layered Liquid. P=pressure, H=depth of layer r=horizontal distance, c= sound velocity at depth H/2, $\rho_2=2\ \rho_1$, $c_2=1\ 1\ c_1$. The encircled figures denote the number of rays arriving before the given epoch. The logarithmic infinities have not been drawn in (After Pekeris and Longman, 1958).

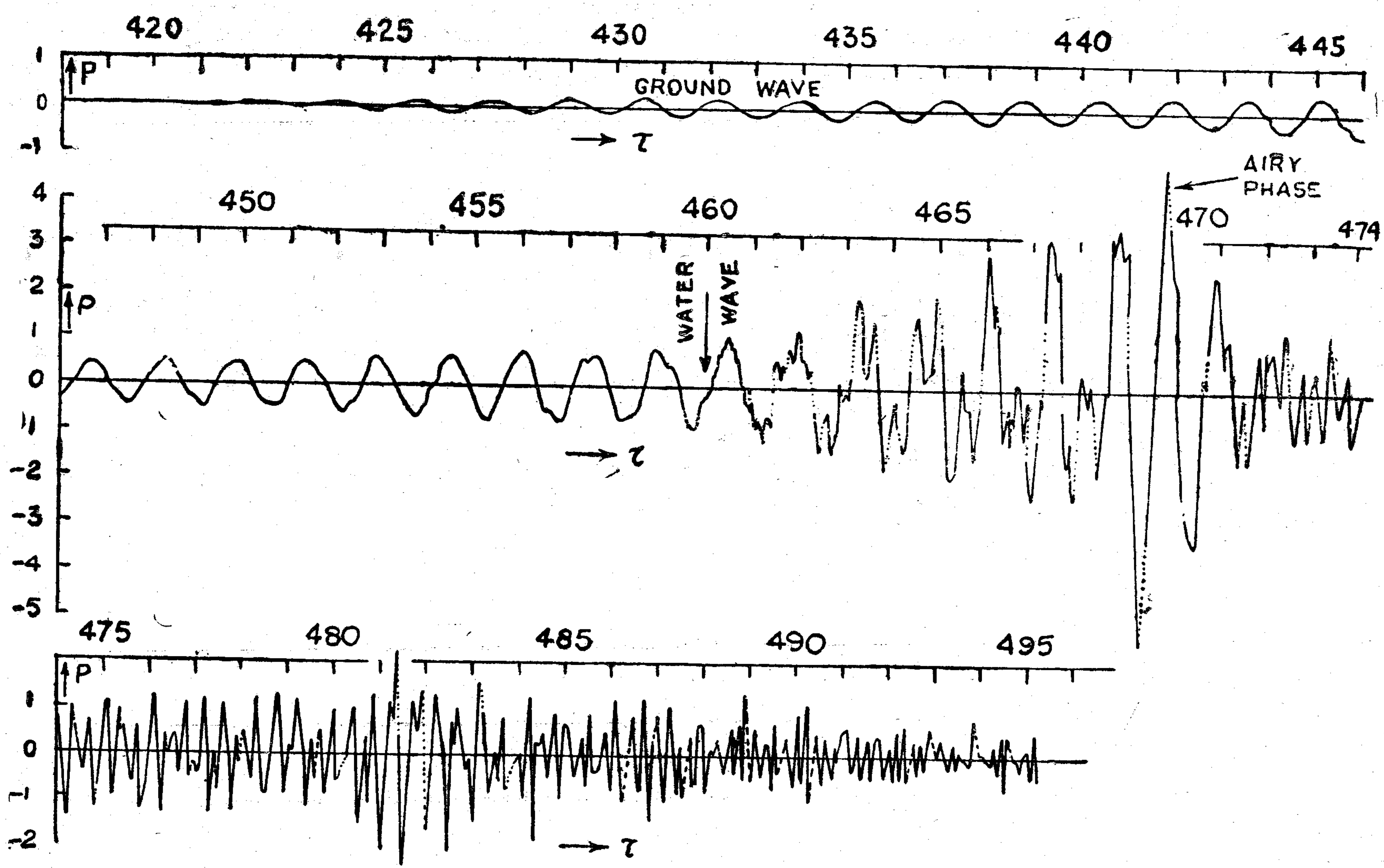

Figure 4. Exact Theoretical Pressure Record Resulting from an Underwater Explosion of Heaviside form in a Layered Liquid. Solution obtained by ray theory. Velocity of sound in bottom c_2 is 1.1 times the velocity of sound in water. Density of the bottom is twice the density of water. P=pressure, H=depth of layer, $r=460$ H, $\tau = ct/H$. The dotted portions show where logarithmic infinities have been omitted. (After Pekeris, Longman and Lifson 1959).

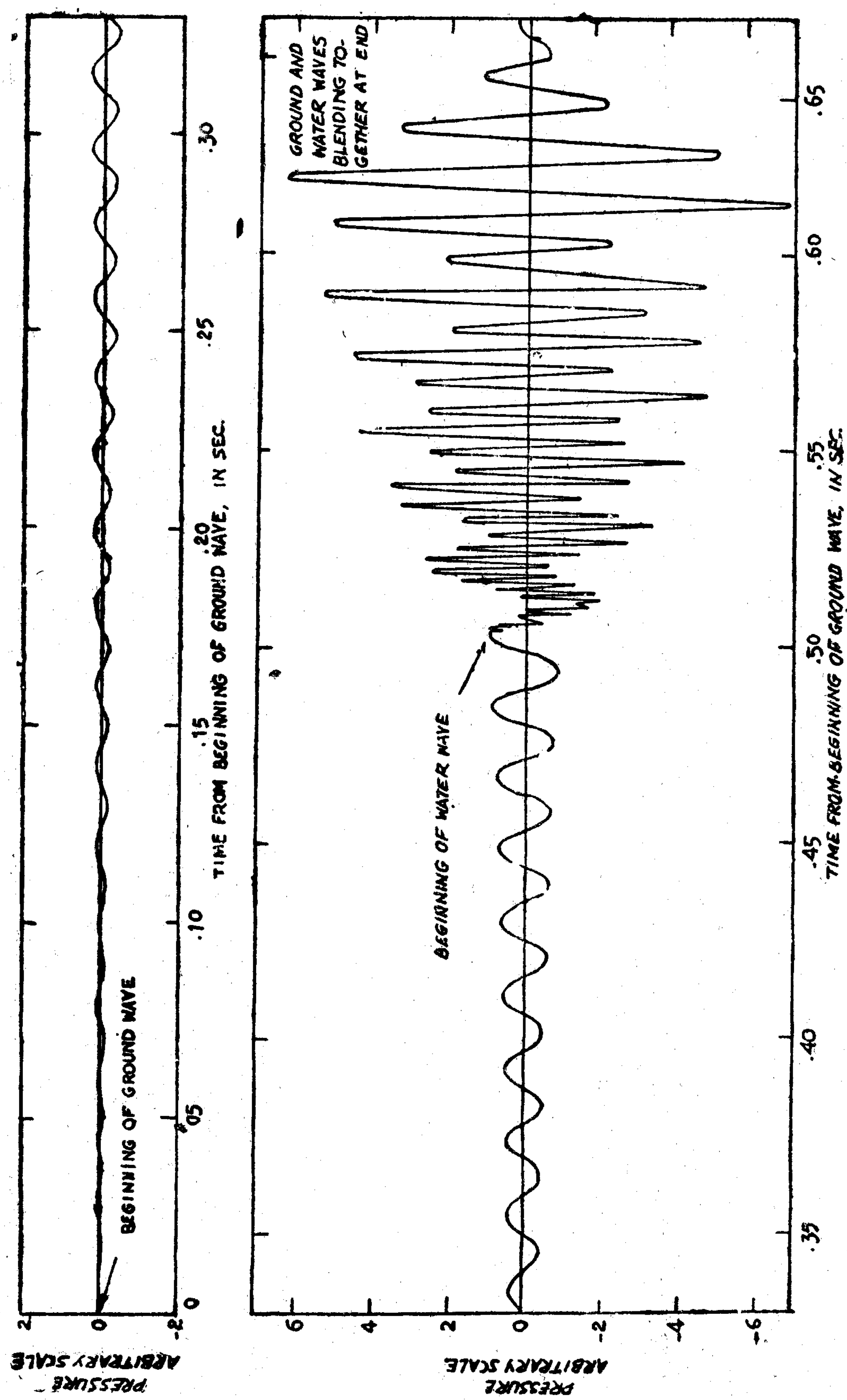

Figure 5. Theoretical Pressure Record for the same Conditions as in figure 4, computed by the Normal Mode Theory, using only the First Mode. (After Pekeris 1948).

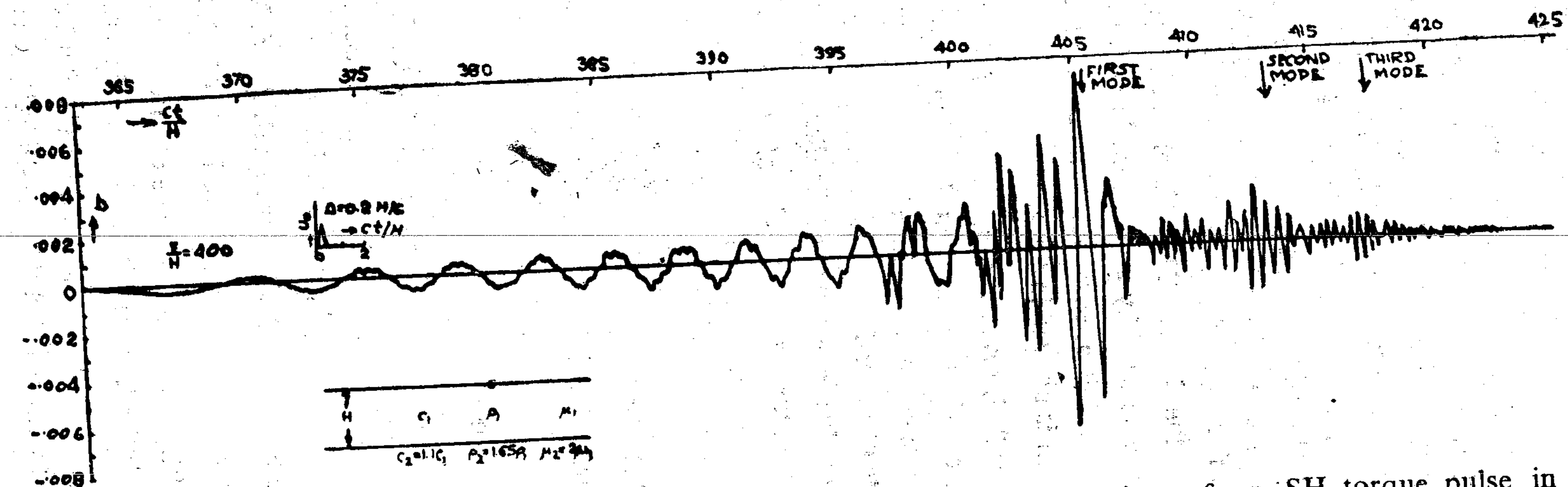

Figure 6. Horizontal displacement at the surface q caused by the application of an SH-torque pulse in a layered elastic half-space. Depth of source is H/2. H denotes thickness of layer, and r horizontal distance. The source produces a displacement u_0 which at large distances approaches the form of a sawtooth:

$$f(t)=(t-R/c)H(t-R/c)-2(t-\Delta-R/c)H(t-\Delta-R/c)+(t-2\Delta-R/c)H(t-2\Delta-R/c).$$

(After Pekeris, Alterman and Abramovici. 1963) exhibits the well known features, first observed by Ewing and Worzel (1948) and later deduced from the normal mode theory of a long period ground-wave followed by a high frequency dispersive water-wave (fig. 3). The curves obtained by them show some oscillatory character, though not quite similar to actual seismograms (Pekeris and Longman, 1958, figs. 2-5). Pekeris, Longman and Lifson (1959) applied the ray theory solution to the problem of long-range propagation of explosive sound in a layered liquid. The exact ray theory solution exhibits the characteristic features of a ground wave followed by a dispersive water wave (fig. 4). The curve computed by the normal mode theory, using only the first mode, for the same conditions is given in fig. 5, for reference. Pekeris, Alterman and Abramovici (1963) investigated the motion of the surface of a layered elastic half-space produced by a torque-pulse from a point source situated inside the layer. The axis of the torque was assumed vertical (SH) and its time variation being represented by a step-function with rounded shoulders. The displacement due to the source approaches a saw-tooth shape at large distances. The curve for one particular case given by them is shown in fig. 6.

An examination of the figures 3–6, reveals the presence of oscillatory movements. It follows, therefore, from the above considerations that layering may be considered to be the major cause for the observed oscillatory movement in seismograms. Further apart from layering, if we could include the effects of other factors influencing elastic wave propagation, such as, fluctuations in the local gravity value, curvature of the discontinuities encountered, internal friction, scattering etc., a more satisfactory result would be obtained. However, the general case of a layered solid media with arbitrary source excitation does not seem to have been investigated so far, owing of course to the very difficult nature of the problem.

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Paper No. 29

SEISMICITY AND TECTONIC HISTORY OF THE INDO-GANGETIC PLAINS

L. S. Srivastava* and R. S. Mithal**

SYNOPSIS

The seismicity of an area depends to a large extent on its tectonic history. Areas of recent tectonic movements show higher activity. An attempt has been made in this paper to describe the tectonic history of the development of the Indo-Gangetic Plains which indicates that the Bengal basin and the Ganga basin, which show great seismic activity, probably developed during the last phase of the Himalayan orogeny. These movements are still continuing and cause frequent earthquakes in the region.

INTRODUCTION

The northern parts of the Indian sub-continent, including the Himalayan region, are frequently affected by earthquakes at short intervals. It is a common observation that the frequency of earthquake is very high in areas of active orogenesis.

The Indo-Gangetic Plains, covering an area estimated at over 250,000 sq. miles, forms a monotonous level surface filled with alluvium covering the rocks and structures below them. Thus the geology and the tectonic history of this region has remained a matter of speculation. This was mainly due to an apparent lack of interest as no economic mineral deposits were expected in this region. With the launching of the Five Year Plans, the establishment of the Oil and Natural Gas Commission, and construction of the Multipurpose projects in the highly seismic zones, renewed interest has been created to unravel the hidden mysteries below the alluvium and the Siwaliks. On the basis of the available data a possible explanation on the "Geotectonic Position and Earthquakes of the Gango-Brahmputra region" was given by Mithal and Srivastava (1959). This aroused a great interest amongst the Indian Geologists and Geophysicists and a number of papers based on actual field data have recently been published.

The interpretations put forward in this paper are based on records of the famous Indian earthquakes and other available geological and geophysical data. Unfortunately most of the results of the gravity, magnetic and seismic prospecting carried for oil exploration

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have not been published so far and hence the available data for such an extensive subject dealing with so vast an area is so meagre that it permits only to draw a tentative conclusions, which may be modified in future.

TECTONICS

The Indian sub-continent including India, Pakistan and Ceylon is composed of three distinct units : Deccan Peninsular shield, Himalayan belt and Sedimentary alluvial basins of the Indo-Gangetic Plain. The Deccan shield with the island of of Ceylon forms a single crustal block and has remained as a stable triangular plateau having suffered little or no folding since the Pre-Cambrian times. It has however been subjected to fracturing resulting in a set of crustal blocks (Fig. 1). These blocks are considered stable and practically aseismic but as these are bounded by fault zones, their shaking due to sympathetic vibrations and marginal adjustments are possible. It is also

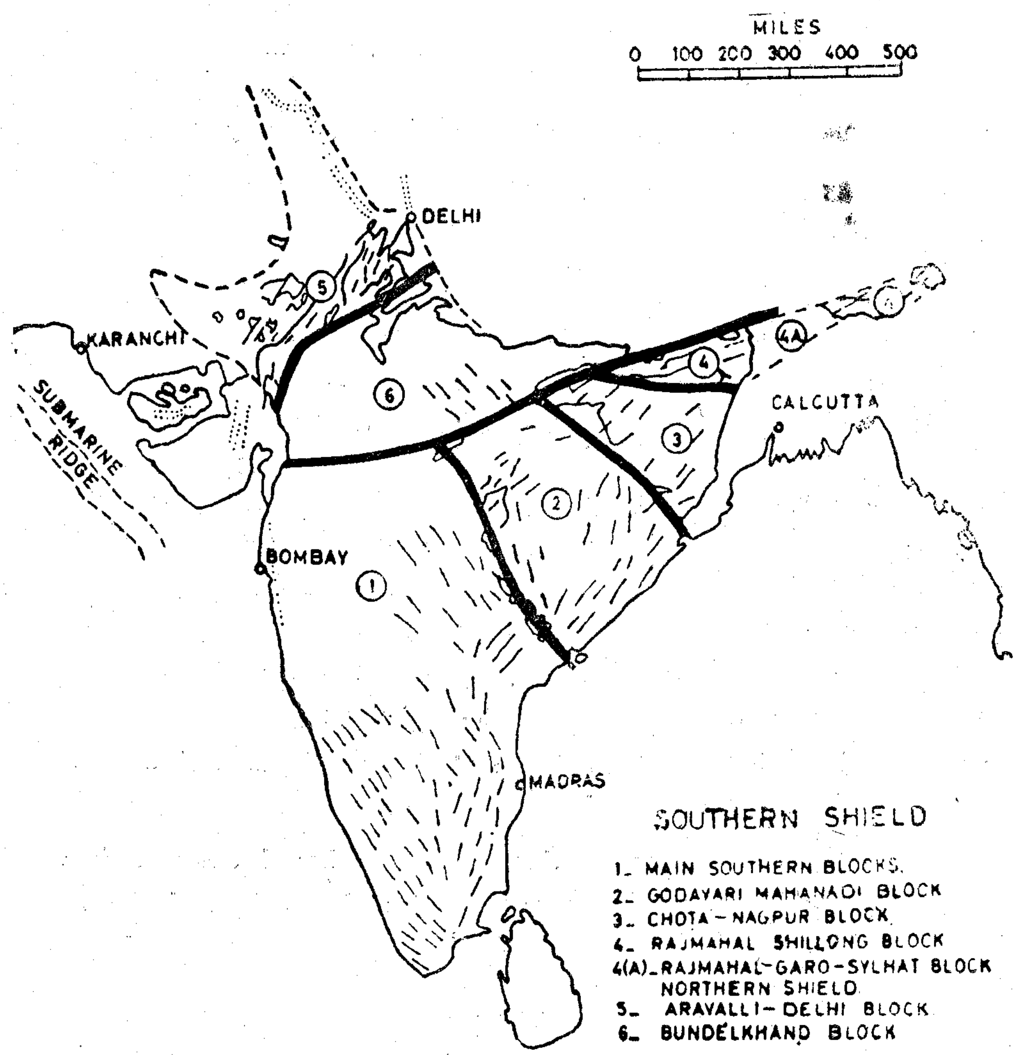

Figure No. 1 Structural Blocks of Peninsular Shield considered that the dynamic processes which separated India from Africa and Australia, and which have perhaps not yet entirely ceased are responsible for the earthquakes in the Peninsular shield area.

No detailed geological work has been carried out in the Himalayan region and the sedimentary alluvial basins. However, the origin of the two is intimately connected and have been discussed by several authors. The Himalayan mountains and the sedimentary alluvial basins are considered to have formed due to the same tectonic movements. This was initiated by the uplift of the Tethys floor to form the inner Himalayas, breaking up of the Gondwana land and the eruption of the Deccan lava. The later upheavels (during post Eocene times) again raised the remnants of the Tethys with folds of the outer Himalayas followed by the post-Miocene movements raising the lesser Himalayas. During the end phases of this great Tertiary orogeny were formed the outer most sub-Himalayas. All these movements are well marked by the transverse ranges separated from each other by parallel reverse faults. The faults, north of the "Great Boundary Fault" and the "Krol Thrust" appear to be related with the first two uplifts of the Himalayas and those in the south with later orogenic movements parallel and sub-parallel to the "Gango-Brahmputra Rift." This also indicates that the fracture pattern in the northern part of the Peninsular shield have controlled and influenced the later movements in the outer Himalayan zone.

The knowledge of the base and the basement rocks of the Indo-Gangetic Plain is mostly a matter of conjecture and unless more data is made available any explanation offered may remain doubtful. Agocs (1956) on the basis of the aeromagnetic survey, of a large part of the Indo-Gangetic plain in Uttar Pradesh, Bihar and parts of Punjab indicated that the basement floor varies from 500 to more than 30,000 feet deep, increasing in thickness towards the foot of the Himalayas. He also suggested that the basin may be segmented into several transverse sections bounded by major faults. Later geophysical surveys have confirmed most of the observations of Agocs.

On a study of the various informations available Mithal and Srivastava (1959) suggested that the floor of the Indo-Gangetic Plain is gently sloping towards the Himalayas and the thickness of the sediments varies from 5000 feet or more at the southern margins. Along the northern fringes at the foot hills it may be 10,000 feet or more in the Uttar Pradesh and Bihar, 4000 feet to 12,000 feet or more in Bengal basin, and 20,000 feet or more in the Upper Assam Valley. Considering the above and the tectonic pattern the alluvial tracts of India have been Sub-divided into the following basins (Fig. 2).

(i) Cambay and Cutch basin,
(ii) Jaisalmer basin,
(iii) Punjab basin,
(iv) Ganga basin,

(v) Assam basin,

(vi) Bengal basin.

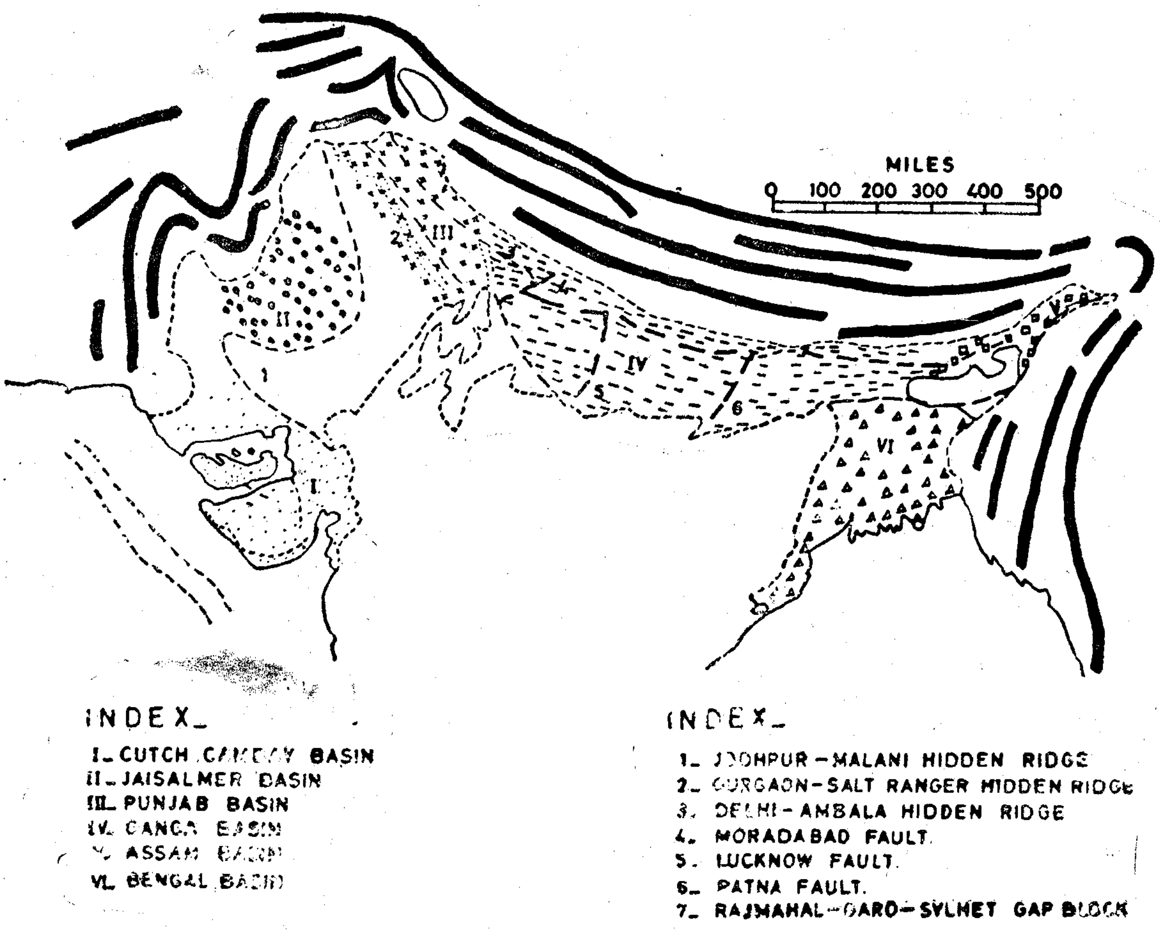

Figure 2 Sedimentary Alluvial Basins of Indo Gangetic Plains

The Penisular shield during the late Mesozoic was composed of seven Blocks. These blocks were bounded by the coastal faults, Gondwana rifts and other faultings, and were responsible for the present shape of the southern margins of the shield. The Indo-Gangetic Plains were then probably occupied by the northern sloping Aravalli-Delhi and Bundelkhand blocks with extensions of the present Aravalli, Satpura, Kaimur and Vindhyan ranges. Subsided and hidden ramifications of these ranges have been investigated and a few of these have also been confirmed by geophysical investigations followed by drilling by the Oil and Natural Gas Commission and the Exploratory Tube Wells Organisation of India.

The Aravalli-Delhi block, thus appears to have broken up during the first Himalayan orogeny and the subsided grabens, between the hidden horsts (ridges) extending along (i) Delhi-Ambala, (ii) Gurgaon-Sargodha-Salt Range, (iii) Jodhpur-Malani, and (iv) the Submarine ridge sub parallel to Sindh-Cutch coast line seem to be responsible for the development of the Punjab, Jaisalmer and Sindh-Cutch-Cambay basins.

The Punjab basin, lying within the Aravalli and Delhi strikes on the southeast and the Himalayan trends on the north, forms a "nuclear basin" with 'anisochronous' frame work. The Jaisalmer, Cutch and Cambay basins are "discordant" in relation to the tectonic pattern. The Assam basin also seems to have subsided during the first uplift of the Himalayas and formed a 'nuclear basin' with 'isochronous' frame work.

The subsidence of these basins on southern margins of the rising Tethys geosyncline formed depressions popularly known as the "fore-deep" of Suess. Such subsidences on the margins of geosynclines are a common phenomena and are noted on other parts of the world e.g. Scandia (Scandinavia) got subsided during upper Tertiary with simultaneous uplifts, and volcanic activity in the Norweogian Geosyncline, and the submergence of Cascadia after the upper Jurassic foldings of Sierra Nevada in North America.

Simultaneously, with the second uplift the northern fringes of the Bundelkhand Block started sinking along the southern margins of the Gango-Brahmputra rift or the Neogene Siwalik Trough. Concomitantly, the areas south of the rift in the Ganga basin, which was probably a land mass suffered shearing and fracturing along the NE–SW and NNE–SSW directions. Due to these shears the Ganga basin got splitted in four blocks bounded by the Delhi–Ambala ridge in the West and the Rajmahal Shillong gap block in the east with Moradabad, Lucknow and Patna faults in between, and each of the four blocks appear to be further segmented by criss cross en echelon faults within them. Like the Aravalli-Delhi Block, hidden extensions of the Vindhyan platform and the Kaimur ridges seem to be present underneath the alluvium in these blocks. This is partly confirmed by the reported occurrence of the orthoquartzites (sedimentary origin) at a depth of nearly 13,000 feet in the Terai zone of northern Bihar and at about 4500 feet in the central part of Uttar Pradesh.

The Ganga basin, south of the Gango-Brahmputra rift (occupied by the then "Indo-brahma" river) started sinking by "differential earth movements" during the third uplift of the Himalayas upto the close of the Siwalik sedimentations. It is also likely that later deposits in this basin rest on the Vindhyans. The reported limestones and quartzites met in the deep bore holes below the alluvium near Barielly and Shahjahanpur if proved to be of Vindhyan age will support the above idea.

During the fourth phase of the Himalayan orogeny, the sinking of the already sheared Ganga basin in the south ultimately resulted in the uplift of the Siwalik ranges coupled with the dismemberment of the "Indo-brahma" into three separate river systems of the Indus Ganga and Brahmputra. Simultaneously the northern half of the Bengal basin also collapsed along the earlier formed longitudinal shears forming the Rajmahal-Garo-Sylhet Gap in the Rajmahal-Shillong block of the Peninsula rshield. This gap has helped in the draining of the Ganga and Brahmputra rivers into the Bay of Bengal. The disturbances at this stage along the Delhi-Ambala Ridge were probably responsible for the obliteration of the south westerly flowing legendry river Saraswati into Rajasthan and Cutch.

From the above account it is evident that the sinking of 'blocks' and 'basins' into graben structures have played a greater role in the development of the present configuration of the Ponisular Shield and the Indo-Gangetic plain. It is also worth noting that similar subsidences are also making transverse ridges, valleys or doons in and along the Himalayas. On the basis of detailed structural analysis, Krishnaswamy (1963) indicated a system of block faulting in the Punjab outer Himalayas. Likewise, as a result of studies in connection with the last Kashmir earthquake of 2nd September 1963, Srivastava, et al (1964) have shown that the valley of Kashmir has developed as a sunken block. The Doons and the Kathmandu Valleys also seem to be of similar origin.

SEISMICITY

The Indo-Gangetic Plains have been affected by a number of severe earthquakes in the historic past. The Narbada and Tapti basins of the Peninsular shield and the Cambay and Cutch basins have been affected frequently by earthquakes. The last named area suffered from major shocks (in 1889 and 1956) with great damage to life and property. The Jaisalmer basin does not show activity on its eastern margin but earthquakes have been reported on its western side along the Indus river.

The Punjab basin is composed of sands, silts and clays brought by the Indus and its tributaries. It is noted (Mathur and Kohli, 1959) that the floor of the basin slopes in general at an angle of 1° to 2° towards the Himalayas, and the sediments overlying it from homoclinally dipping structures parallel to the Himalayan ranges. The floor of the central and southern parts of this basin does not show any major active fault zones and is comparatively less susceptible to seismic activity. The Siwalik belt in the north and the Delhi rocks in the east of this basin show considerable seismic activity and the 1905 Kangra earthquake did considerable damage in the northern parts. The earthquakes along the Delhi border in the east are of frequent occurrence indicating that movements are still operative along the the Delhi-Ambala ridge.

The Ganga basin, with an approximates area of 1,40,000 square miles, extending from Delhi to Rajmahal and continuing upto the the Garo Hills represents the northern slope of the Peninsular Shield. The thickness of sediments in this basin appears to be gradually increasing towards north, and towards the foot hills it becomes enormous due to the development of the 'Gango-Brahmputra Rift' at the fringes of the Himalayas. As stated earlier the floor of the basin lying between the Delhi-Ambala hidden rigde and the Rajmahal-Garo Sylhet gap block appears to be further subdivided into four block by en echelon subterranean faults – i.e. the Moradabad fault with ENE trend, Lucknow fault with NE to NNE trend and the Patna fault with NE–SW trend. Thus the Ganga basin appears to be composed of huge blocks bounded by the hidden ridges and the faults and the boundaries of these blocks have shown seismic activity in the past. The 1505 earthquake at Agra, the 1803 earthquake at Mathura and the 1720, 1825, 1830, 1831, 1842 and other recent earthquakes at Delhi indicate that the eastern side of the Delhi-Ambala ridge is still active and is the cause of occassional earthquakes in the region. The Moradabad fault appears to have released energy in the past, which caused the 1803 Upper Ganga earthquake and the Meerut earthquakes of 1833 and 1852. The earthquake of 1808 at Banda, and of 1825 and 1864 at Lucknow indicate activity along the Lucknow fault. The region around Patna fault have been subjected to great activity and 1833 and 1934 Bihar-Nepal earthquakes in the area were of very high magnitudes, responsible for great damage to life and property. The Himalayan foot hills forming the northern borders of the Ganga basin is an area of moderate seismic activity but the southern margins have not shown much activity in the recent past.

The Rajmahal-Garo-Sylhet-gap block, bordering the Ganga basin in the east, is a subsided burried mass, with cross shears extending towards south upto the coast of Bay of Bengal. Earthquakes are of frequent occurrence in the Bengal Basin (includiug east Pakistan) and the epicentres appear to be mostly concentrated near the eastern and western boundaries, though earthquakes have also been recorded from within the basin. All the epicentres of the earthquakes in this basin appear to be closely related with the shears of the subsided block and the less conspicuous subterranean ridge extending from Rajmahal to east of Cooch Bihar.

Geologically the Assam region, the most seismic region of India has been worked out for almost 75 years. The knowledge gained is highly valuable and is based on geological mapping, drilling and geophysical survey. Little is known about the geology of the Assam Himalayas which are believed thrusted towards this basin from the north. The Shillong plateau, Mikir hills, and the thrusted Naga hills form its southern and eastern margins. The Assam region has witnessed many severe earthquakes in the 19th and the 20th century, which have caused great damage in the area.

During the 1897 great Assam earthquake the accelerations are estimated to have exceeded gravity. Among the innumerable other earthquakes, the 1843, 1845, 1930, 1943 and 1950 Assam earthquakes were very destructive in intensity. The epicentres of most of the destructive earthquakes lie in sorrounding mountainous belt outside the alluvial tracts, and the movements are more connected with the last phase of the Himalayan orogeny; and the basement below the valley appears to be have adjusted most of its strain due to block faulting.

CONCLUSIONS

The evolution of the Indo-Gangetic Plains has passed through different geotectonic stages. The first stage of movements, concomitant with the rise of Himalayas, appear to have produced the Sindh -Cutch-Cambay basin, Jaiselmer basin and the Punjab basin by the breaking up of the Aravalli-Delhi block of the Peninsular shield. The Assam basin also appear to have developed during this phase of the movements. The Gango-Brahamputra Rift (or the Neogene Siwalik trough) got developed by the sinking of the northern fringes of the Bundelkhand block during the second uplift of the Himalayas. The Ganga basin was then probably a land mass extending upto Shillong Plataeu, and the southern half of the Bengal basin formed a continental shelf of the Bay of Bengal.

The Ganga basin, south of the Gango-Brahmputra rift, got dismembered into four blocks during the third uplift of the Himalayas, and started sinking culminating in subsidence of the Rajmahal-Garo-Sylhet gap in the fourth uplift of the Himalayas. This ultimately established the depression for the development of the present configuration of the rivers of the Indo-Gangetic Plains.

The Indo-Gangatic plains show more activity along the boundaries of the subsided Bengal basin and the block of the Ganga basin, indicating that the shears have not adjusted their strains upto the present day, and are the cause of very frequent earthquakes in the regions. The activity in the other basins is more related with the instability in the surrounding mountainous belts and the hidden ridges, while the basements have almost adjusted themselves except the Cutch and Cambay basin where adjustments are still taking place.

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Paper No. 30

COMPARATIVE STUDY OF PREDICTING NATURAL FREQUENCY OF FOUNDATION-SOIL SYSTEM

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SYNOPSIS

Design of machine foundations where the loads are repetitional is complex and is commonly met with by foundation engineers. Prediction of natural frequency of the foundation-soil system is necessary since a major criterion is to design the foundation such that its natural frequency lies outside the range of the operating frequency. The many factors influencing the natural frequency are listed and a review of the well known methods of predicting natural frequency presented. The influence of the various parameters and relative merits of individual methods are discussed. Ford and Haddow's method is recommended for its simplicity and Sung's for its rigorousness.

INTRODUCTION

Design of foundations for machines where the loads are repetitional in nature, is one of the usual problems met with by the foundation engineer. This problem is much more complex than the design of foundations which support only static loads. Large machines are usually supported directly on the soil in a manner that permits a direct transmission of these periodic impulses into the soil, involving the study of soil dynamics. Review of literature shows that considerable importance is being given to the study of vibrations of massive foundations and to the study of exciting loads created by various machines since they are of great importance in engineering practices. Vibrations of machine foundations are harmful to the operation of machine itself and have harmful effect on the foundations themselves.

When out of balance machines are mounted on foundations built on the ground, the problem arises as to whether resonance of the foundations can occur. When the natural frequency of the soil foundation system coincides with the operational speed of the machine, excessive vibration amplitudes might occur leading to structural damage or even failure of the foundation.

It has been found that the natural frequencies of most foundation-soil systems are less

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than 30 c.p.s., consequently there is the possibility of resonance occurring if the operating frequency of a machine is less than 1800 r.p.m. Many machines, especially, stationary reciprocating machines, have rotational speeds less than 1800 r.p.m. Prevention of resonance in the machine foundation system is thus one of the major criteria in their design. Therefore it is often required to design the machine foundation in such a way that the natural frequency will lie outside the range of the operating frequency of the machine.

Most of the published literature is centred round the vertical vibration of the foundation, generated either by a single blow resulting in free oscillations, or by continuously applied sinusoidal force giving forced oscillations. Vibrations so generated are the ones most commonly met with in practice.

The earliest approach to the problem of foundation vibration was made by Degebo (1) by considering the vibrating system to behave as a single mass supported by a weightless spring. As a result of their extensive series of tests, it was necessary to consider the spring to have an effective mass.

The second approach was to consider the problem as the vertical motion of an oscillator resting on a semi-infinite isotropic, homogeneous, elastic body. Most of the existing works are based on either of these two approaches. Interesting review can be found in the publications of Weil (1963) and Prakash and Bhatia (1964). It is the purpose of this paper to examine and compare some of the well known works and discuss the relative importance of various parameters affecting the natural frequency of foundation soil systems.

FACTORS INFLUENCING THE NATURAL FREQUENCY OF FOUNDATION SOIL SYSTEM

There are too many factors which influence the natural frequency of foundation soil system to list them all, but the most important are (i) the static load which the foundation carries (ii) the shape and size of the foundation (iii) the exiting force (for forced oscillation), (iv) the type of contact pressure distribution (v) the depth of embedment of foundation and (vi) the soil type.

THE SOIL SPRING ANALOGY

The first approximation made in the study of foundation vibrations was by considering the system as a single mass supported by a weightless spring. Later, it was proposed that a certain soil mass underlying the foundation should be considered to oscillate together with the machine foundation, in which case the natural frequency 'f_n' is given by

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k.g}{W_v + W_s}} \tag{1a}$$

or

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k'Ag}{W_v + W_s}} \tag{1b}$$

where, A is area of contact
k dynamic Spring constan
k' dynamic soil modulus
W_s weight of soil actively participating with the oscillation or equivalent weight of soil. (It has no clear physical boundaries).
W_v total static load.
g acceleration due to gravity

This leaves the designer the difficult task of estimating the dynamic soil spring constant k' and the value of W_s. No satisfactory method has so far been developed for the determination of these soil coefficients.

Tschebotarioff and Ward (1948) modified the equation (1) and introduced a new parameter 'Reduced Natural Frequency' in order that the modified equation could be useful in comparing foundation of different areas and subject to different pressures. An ampirical relationship was established between the area of contact of the foundation and the parameter 'reduced natural frequeney' denoted as f_{nr} which bears the following relationship with f_n :

$$f_{nr} = f_n \times \sqrt{p} \tag{2a}$$

where p is the intensity of load.

$$\text{Hence } f_n = \frac{1}{2\pi}\sqrt{\frac{A}{W_v}} \cdot \sqrt{\frac{k' g}{(1+W_s/W_v)}} \tag{2b}$$

In a later publication Tschebotarioff (1953) further confirmed the relationship between area and f_{nr} with some more data on the performance of full scale foundations. The relationship between f_{nr} and area is shown in Fig. (1).

Number of limitations can be cited for the above work. Tschebotarioff himself has listed down some of them. The resulting lines in Fig. (1) are based on a limited number of performance records and several inconsistencies can be noticed. Furthermore, both horizontal and vertical unbalanced forces and their combinations were treated together. The ratios of height to width of the foundations varied quite widely, and the natural frequencies determined by several different methods. The plotting of results on a log-log scale is likely to mask the actual behaviour The f_{nr} vs. area relationship is independent of static intensity, shape of the area of foundation, depth of embedment and the exciting force (for forced oscillations).

All these factors considered above are bound to affect the natural frequency and possibly some of the factors may cancel with each other and some may be cumulative. However, keeping constantly in mind, all the limitations and uncertainties involved the empirical relationship between f_{nr} and area established may still serve as a guide to help the engineer reasonably, especially since the relationship is from valuable field records.

PAUW'S METHOD

Pauw (1953) assumes the stresses to be distributed uniformly and in an effective zone

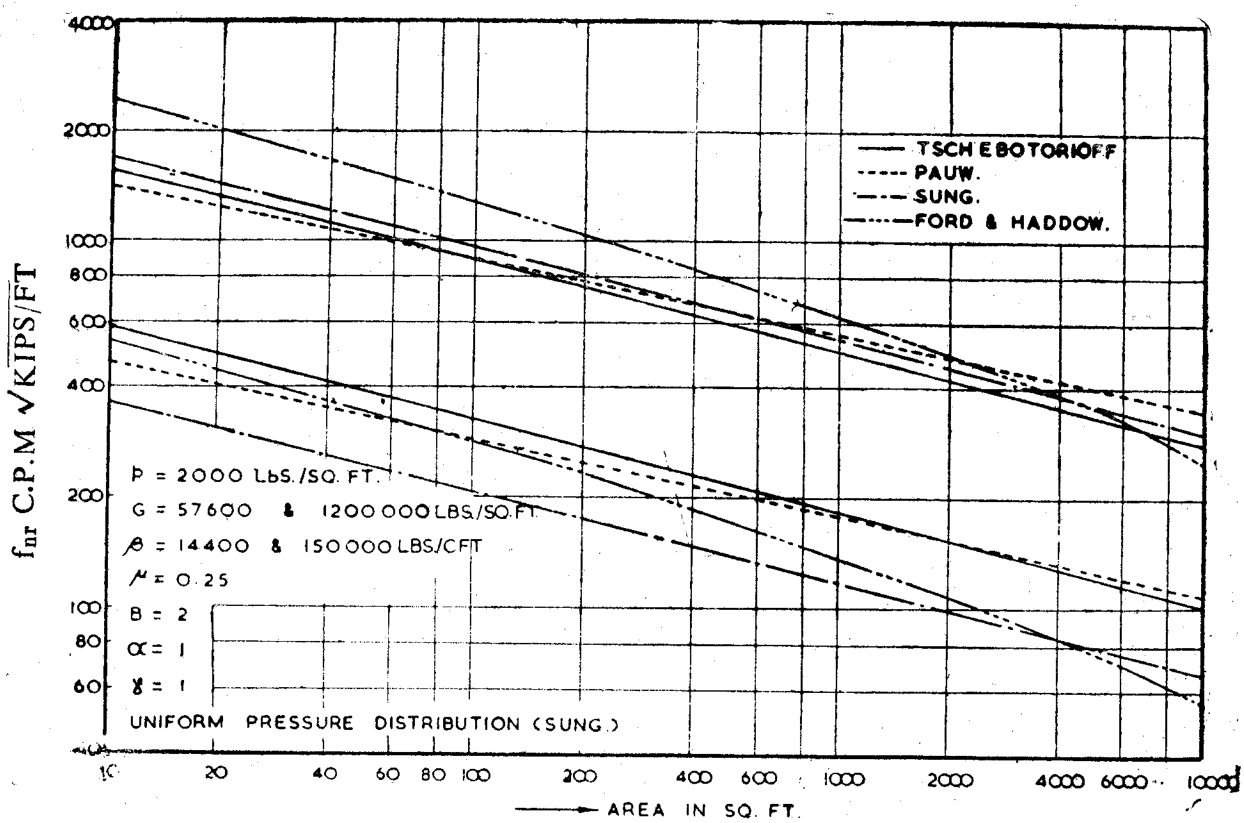

Figure 1

defined by a truncated cone or pyramid, the sides of which slope away at an angle whose tangent is $\alpha/2$. Further the modulus of elasticity is assumed to be constant for cohesive soils and proportional to the effective depth for cohesionless soils. Thus

$$E_z = \beta (h + z) \qquad \text{(for cohesionless soils)} \qquad (3a)$$

and $$E_z = E_0 \qquad \text{(for cohesive soils)} \qquad (3b)$$

where, E_z is the modulus at any depth z,

β rate of increase of modulus with depth,

h equivalent height $= \dfrac{\text{Intensity of static load}}{\text{unit weight of soil}}$

E_0 modulus at the surface.

With these assumptions the spring constant is determined by integrating, over the depth of the pyramid, the displacement of each infinitesimal layer due to a given load on the foundation.

The equation for equivalent weight of soil oscillating with vibration has been derived by equating the kinetic energy of the affected zone to the kinetic energy of a mass assumed to be concentrated at the base of the foundation. Substituting the values for spring constant and equivalent weight of soil in equation (1a) the natural frequency can be found. The various parameters considered by Pauw are (1) shape and size of foundation, (2) the load dispersion angle or the slope of the pyramid, (3) the density of the soil, (4) the soil type represented by β and (5) the intensity of static loading. The use of this approach does not require the soil to be homogeneous.

In order to use the above method one must know the rate of increase with depth and value at the surface of the modulus of elasticity and the value of α. No satisfactory method has been recommended to obtain the value of β and α. According to this anology the value of 'k' increases with the static intensity which is not in conformity with the existing knowledge. The modulus of subgrade reaction 'k' (i.e. spring constant) reduces as the static load increases. Another limitation of this method would be the non-convergence of the infinite integral to a finite limit for the determination of the equivalent soil mass in the case of cohesive soil.

SUNG'S METHOD

Reissner (1936) presented an analytical solution for the vertical motion of an oscillator resting on an elastic homogeneous isotropic semi-infinite continuum. This analysis assumes the vertical periodic, pressure forces to be uniformly distributed. The resonant frequency, amplitude of oscillation and power requirements of the vibrating soil system were determined as functions of radius of loaded area, static weight of vibrator, weight and frequency of the vibrating mass, and material properties of soil.

Sung (1953) and Quinlan (1953) have extended Reissner's treatment to cover different contact pressure distributions between the oscillator and elastic body obtaining essentially the same results.

Sung (1953) has considered three different axially symmetrical pressure distributions i.e. uniform, parabolic and that produced by a rigid footing on a clayey soil. The basis of the Sung's theory is Navier's displacement equilibrium equations neglecting damping. The various parameters considered by Sung are (1) the radius γ_o of the loading area, (2) the total mass m_o, (3) the amplitude of dynamic force applied, (4) the distribution of contact pressure on the base, (5) Poissons' ratio, μ, (6) mass density ρ, and (7) shear modulus G of the foundation material. The results of analysis are presented graphically, with the dimensionless maximum amplitude and dimensionless frequency at maximum amplitude plotted against μ, the Poisson's ratio for different values of 'b' (a significant parameter referred as the mass ratio) for all the three pressure distributions analyzed.

In order to adopt Sung's method the designer must know among other things, the shear modulus G, Poisson's ratio μ and the type of pressure distribution. Indeed Sung suggests small scale dynamic tests to find the soil constants and the type of pressure distribution. An uncertainty introduced by the Sung's approach pertains to the type of pressure distribution involved which is significantly affected by the magnitude of oscillating force and the static intensity Weil (1963). Sung also makes a questionable assumption that the shape has no influence on 'f_n'.

FORD AND HADDOW'S METHOD

Ford and Haddow (1960) have suggested a simple method of predicting the natural frequency of machine foundation based on Raleigh's principle. In this method the maximum strain energy of the system has been equated to the maximum kinetic energy of the system to derive an expression for the natural frequency.

Further, the dynamic stress at any depth 'z' is uniformly distributed over a section of the solid, parallel to the 'x y' plane that is, parallel to the base of the foundation and the whole system has been considered as conservative.

With these assumptions and using Raleigh's principle, Ford and Haddow arrive at the expression for natural frequency.

$$f_n = \frac{1}{2\pi}\sqrt{\frac{E'\beta' ab}{\frac{\rho ab}{\beta'} + M}} \tag{4a}$$

$$\text{i.e.}\quad f_n = \frac{1}{2\pi}\sqrt{\frac{2G(1+\mu)\beta' ab}{\frac{\rho ab}{\beta} + M}} \tag{4b}$$

where, M = mass of the foundation and machine

ρ = mass density of the soil

a, b = two sides of the foundation block

E' = dynamic modulus of elasticity of soils

β' = is the decay factor which is defined from

$$W_o = W_{of}\, e^{-\beta' z} \tag{5}$$

where, W_{of} = amplitude of vibration at the surface,

W_o = amplitude of vibration at a depth z.

The decay factor β' is given by an expression

$$\beta' = \frac{B}{m'\sqrt{A}\,(1-\mu^2)} \tag{6}$$

where, B = a soil constant (varies between 1.5 for clays and 2.0 for sands).

m' = shape factor — a constant

A = area of foundation

μ = Poisson's ratio

To adopt this method in practice one must know the soil constants such as E' or G, μ and m' from dynamic tests.

Among other things the expression for the decay factor forms an uncertainty. How far this expression is valid for soils needs confirmation. Though uniform distribution of dynamic stress is inaccurate, any other form of assumptions will make the analysis further complex with a negligible gain in accuracy. This procedure also neglects the losses such as damping.

DISCUSSION

In the foregoing paragraphs four well known methods of predicting the natural frequency of foundation soil system have been presented. None of these methods considers all the parameters which have been listed earlier in this paper which affect the performance of an oscillator. One or more factors have been ignored by each method giving importance to other parameters. A study of the influence of each parameter as given by each method and a comparison between these methods will certainly help in better understanding the vibrations of foundation soil system.

The authors have computed the values of 'f_n' and 'f_{nr}' using the above three analytical methods, for a set of foundations of different shapes and sizes and for different values of G, μ, β, B, α, p and different types of pressure distribution, to compare with Tschebotarioff's

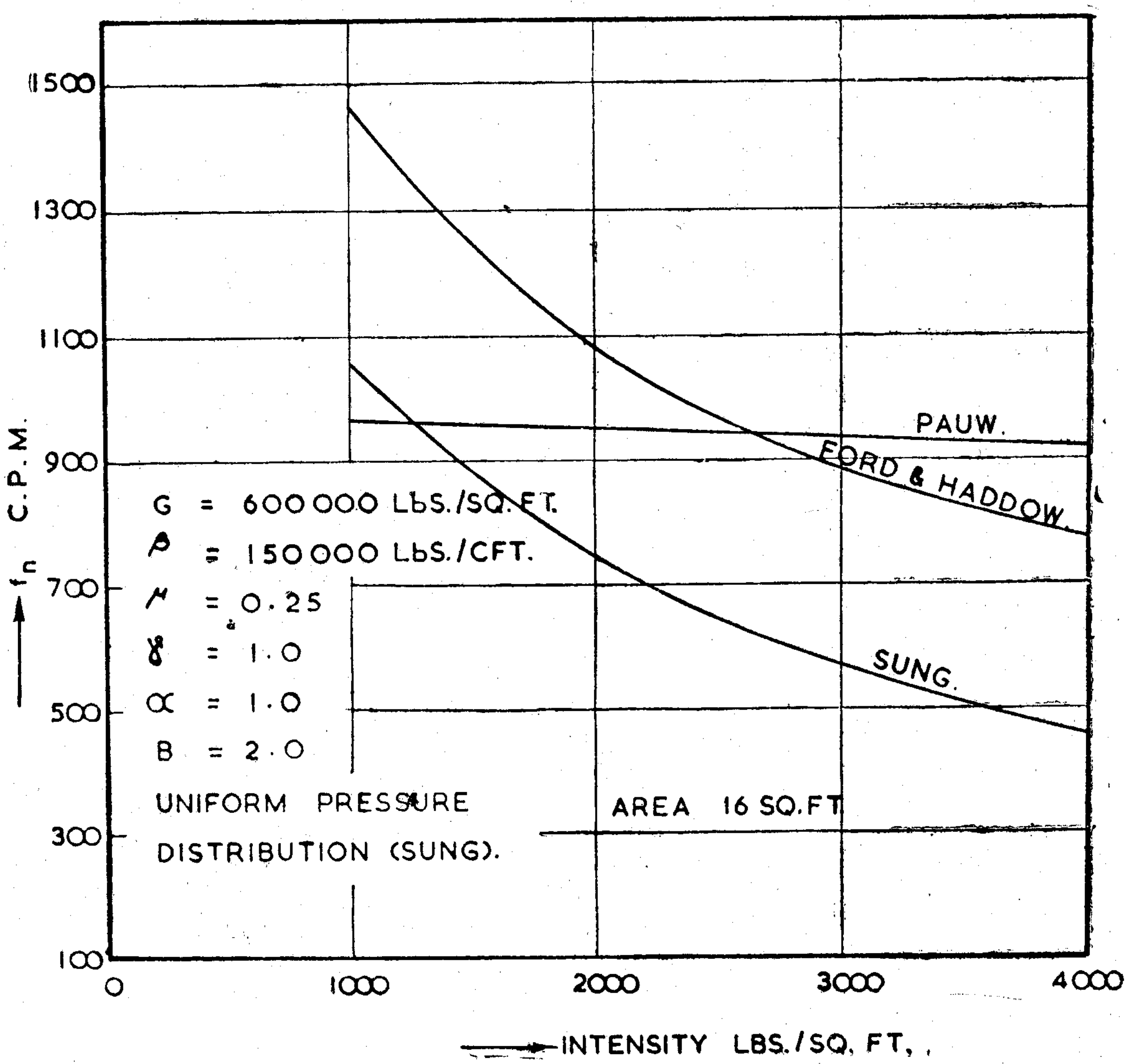

Figure 2 (a)

empirical relationship. For the purpose of comparison, suitable values for the soil constants such as G, μ, β, B and the physical properties of foundation soils have been assumed and the f_n and f_{nr} have been evaluated. These parameters have been varied quite widely and only limited results have been presented for conciseness. The values used for various parameters have been given in each figure. The following relationship between β (Pauw) and G has been made use of to have comparative values in all the three methods.

$$E_o = \beta\,(h) \quad \ldots \quad (Z \text{ being zero})$$
$$= 2G\,(1 + \mu)$$

$$\text{or } \beta \left(\frac{p}{\rho}\right) = 2G\,(1 + \mu)$$

$$\text{or } \beta = \frac{2G\,(1 + \mu)\,\rho}{p}$$

where ρ is the unit weight of the soil in lbs/cft.

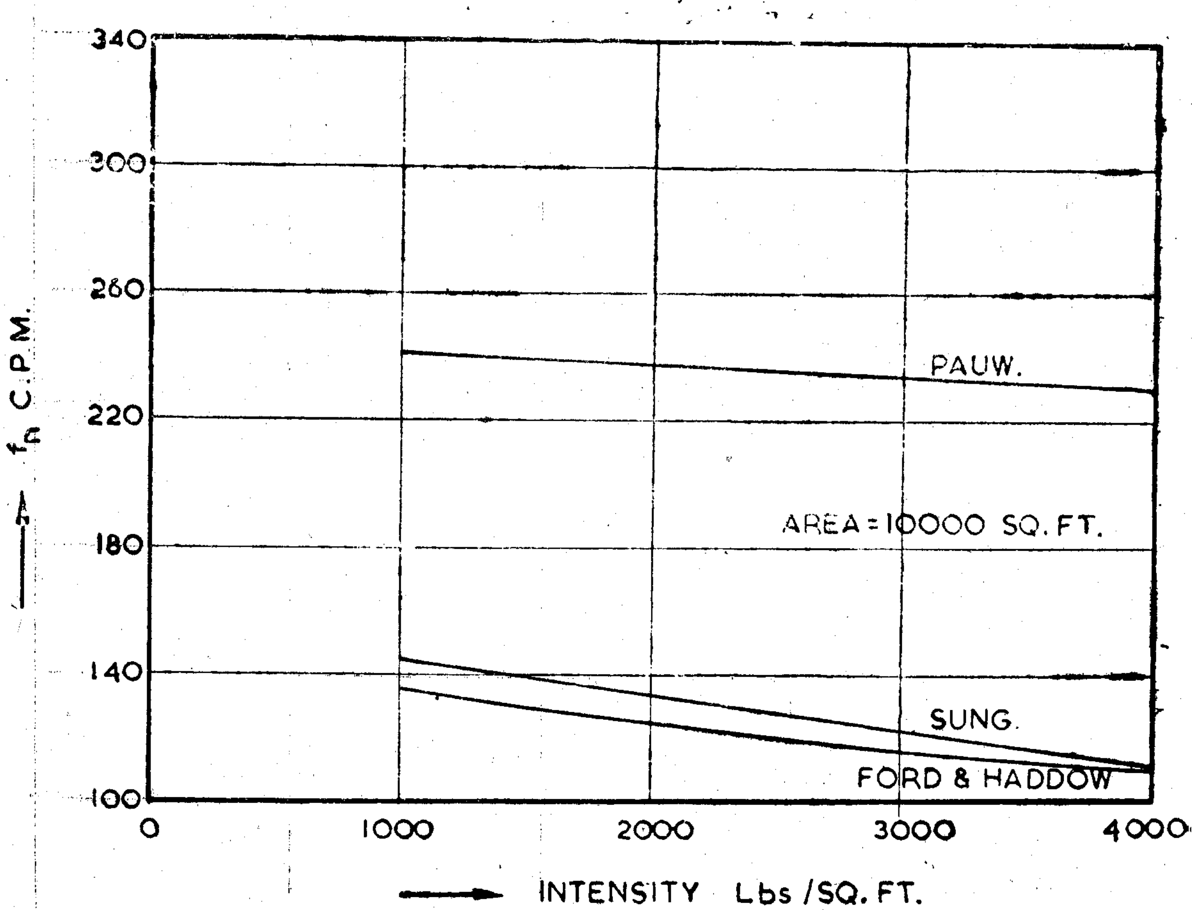

Figure 2 (b)

All the three methods i.e. of Sung, Pauw and Ford and Haddow consider the influence of laod intensity on the natural frequency. It is an accepted fact that an increase in the

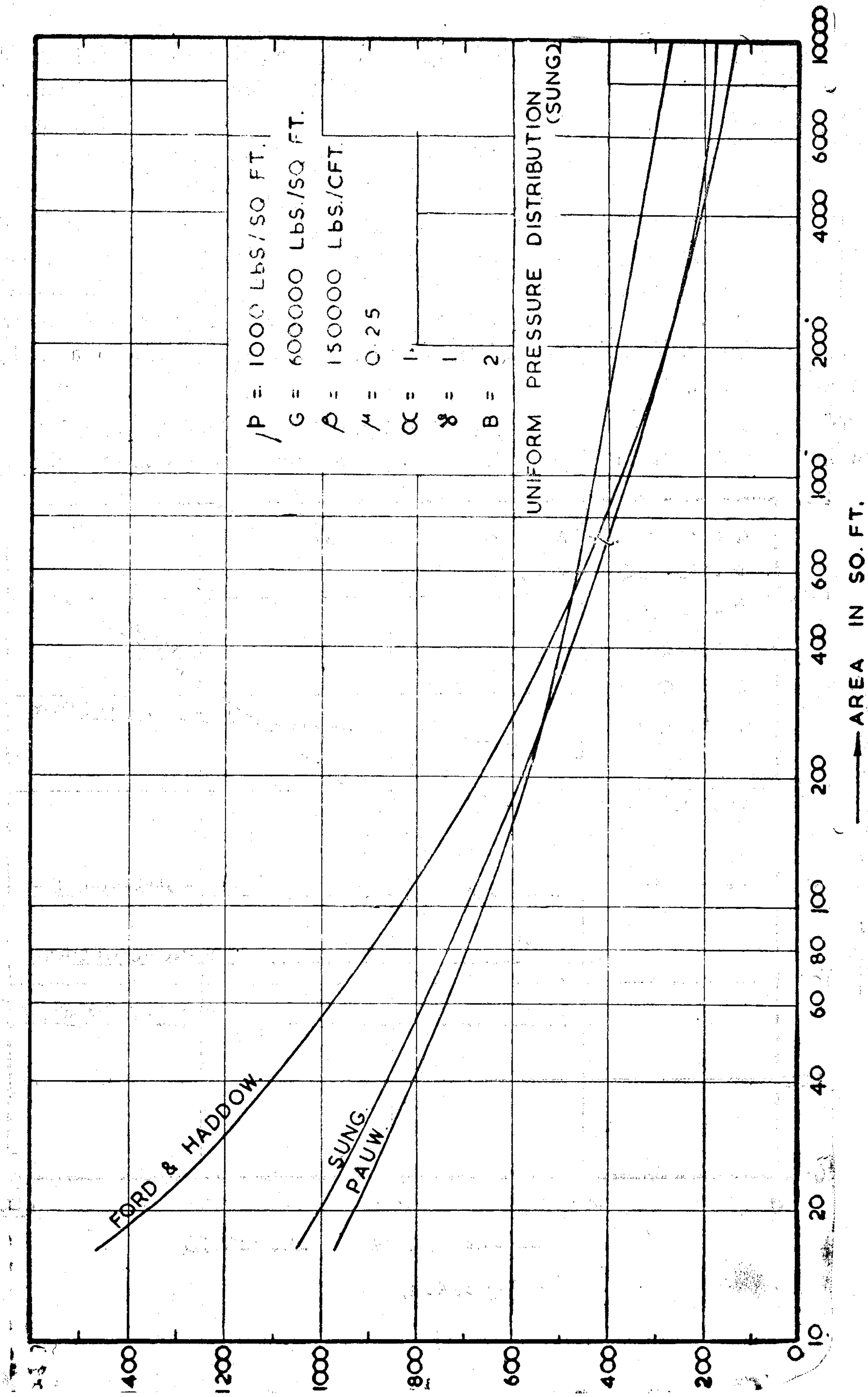
P = 1000 LbS/SQ FT.
G = 600000 LbS/SQ FT.
ρ = 150000 LbS/CFT.
μ = 0.25
α = 1
γ = 1
B = 2
UNIFORM PRESSURE DISTRIBUTION (SUNG)
FORD & HADDOW
SUNG
PAUW
AREA IN SQ. FT.
10
20
40
60
80
100
200
400
600
1000
2000
4000
6000
10000
0
200
400
600
800
1000
1200
1400

Figure 3

intensity of loading decreases the natural frequency. Many published records (East Wood 1953and Ford and Haddow 1960,) show the static intensity to have a great influence on natural frequency and that the natural frequency decreases to an extent of 50 to 60%. Fig. 2a and 2b show the relation between static intensity and the natural frequency for two extreme areas of contact. The rate of decrease of natural frequency with load intensity is almost same for Sung and Ford and Haddow methods whereas Pauw's method in contrast shows a neglegible decrease in f_n with increase in intensity. It is mainly due to the questionable assumption that 'E' increases with intensity of static loading. The effect of Area on 'f_n' for three different methods is shown in Figure 3. It can be discerned from the same that at smaller areas the difference between Sung and Ford and Haddow methods is considerable which decreases to a negligible value as area increases.

The effect of intensity on f_{nr} is considered in figures 4a, 4b, 4c and 8. Tschebatorioff's empirical plot exhibits that f_{nr} is not affected by the intensity. Sung's method also shows

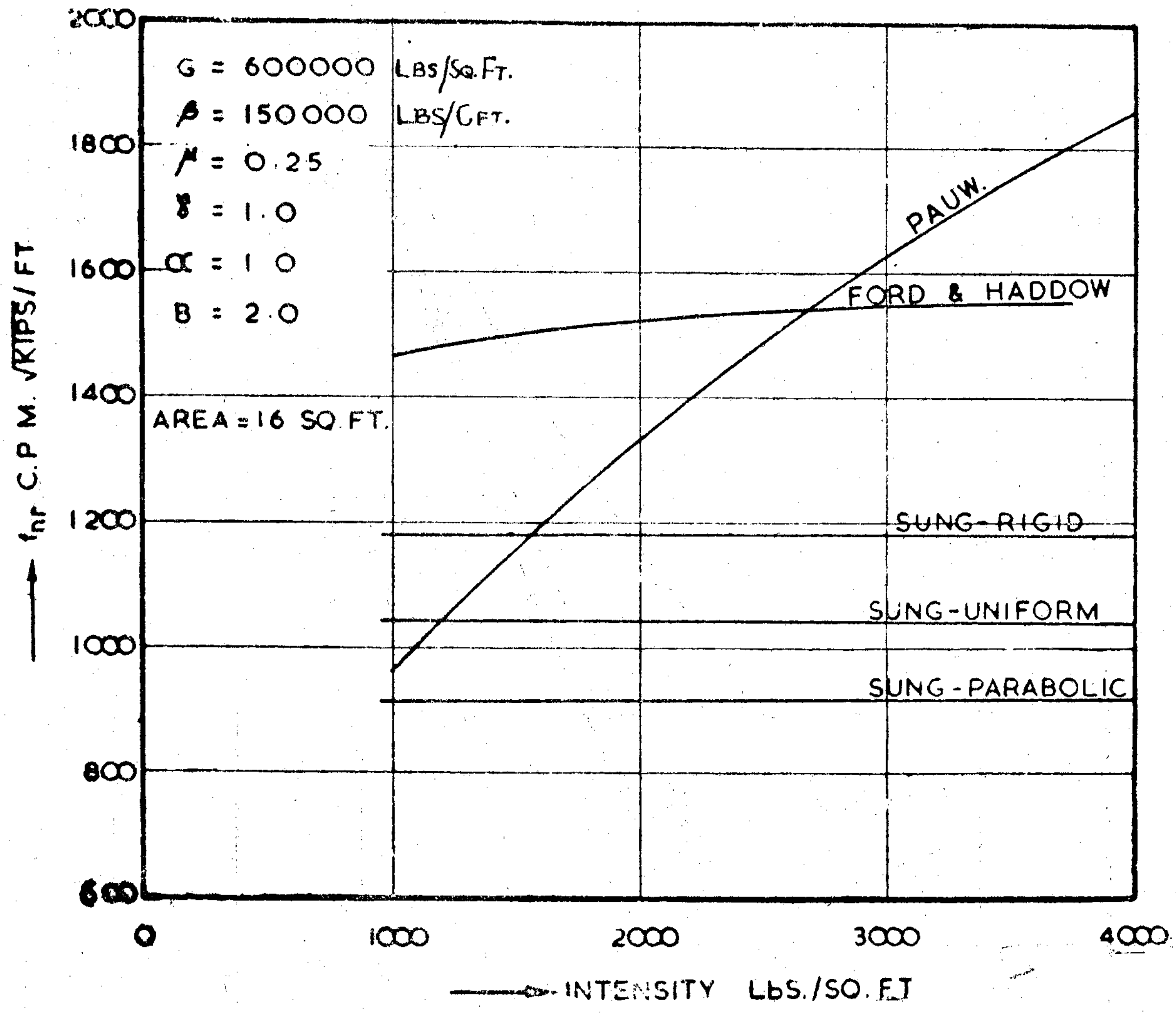

Figure 4 (a)

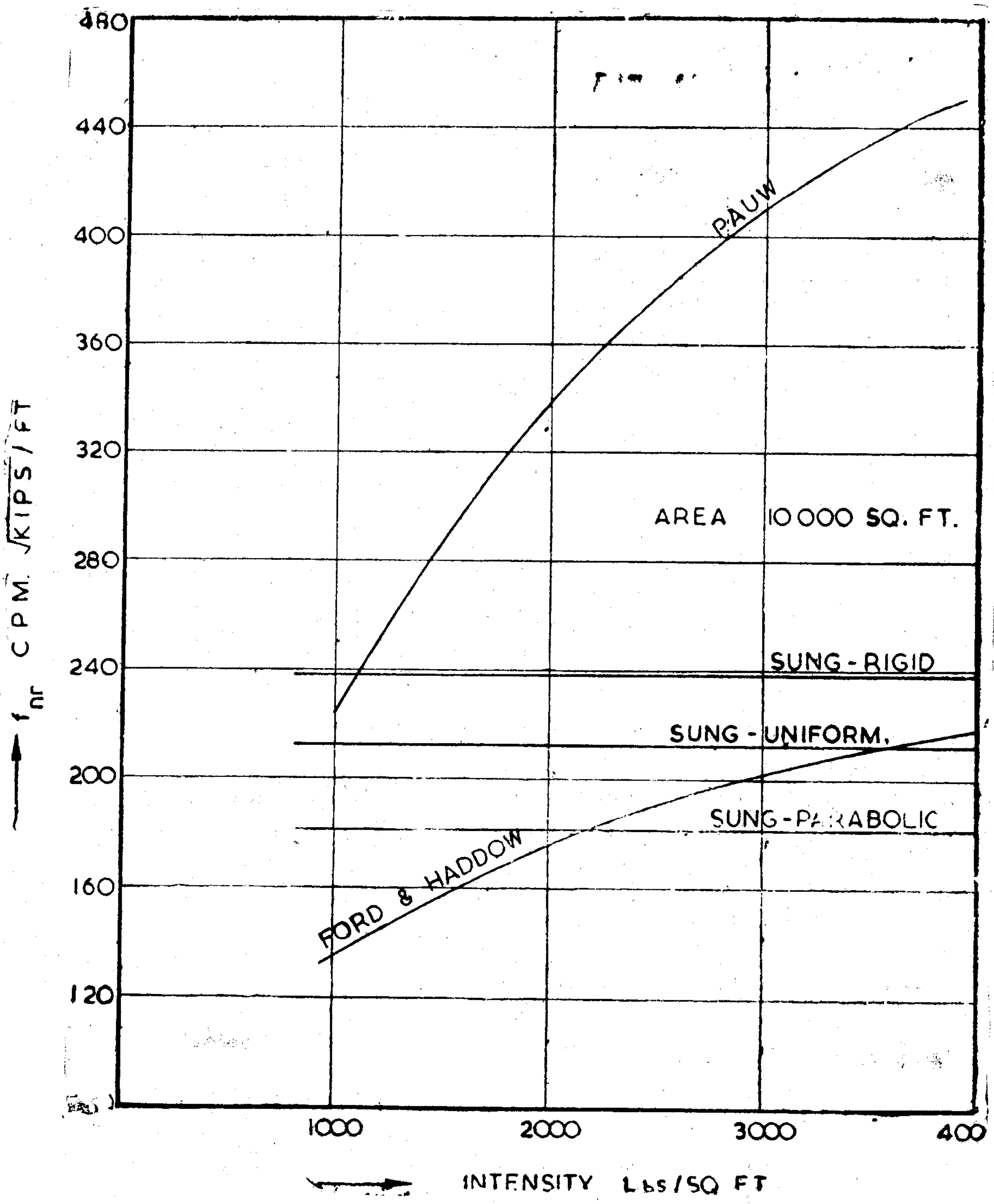

Figure 4 (b)

that the intensity has no effect on f_{nr} for all the three types of pressure distributions and Ford and Haddow's method shows a slight increase in f_{nr} with intensity at smaller areas, increasing for larger areas. The difference in f_{nr} for various intensities increases with area by Ford and Haddow's method (Fig. 4c) and this is always much less than that shown by Pauw's method

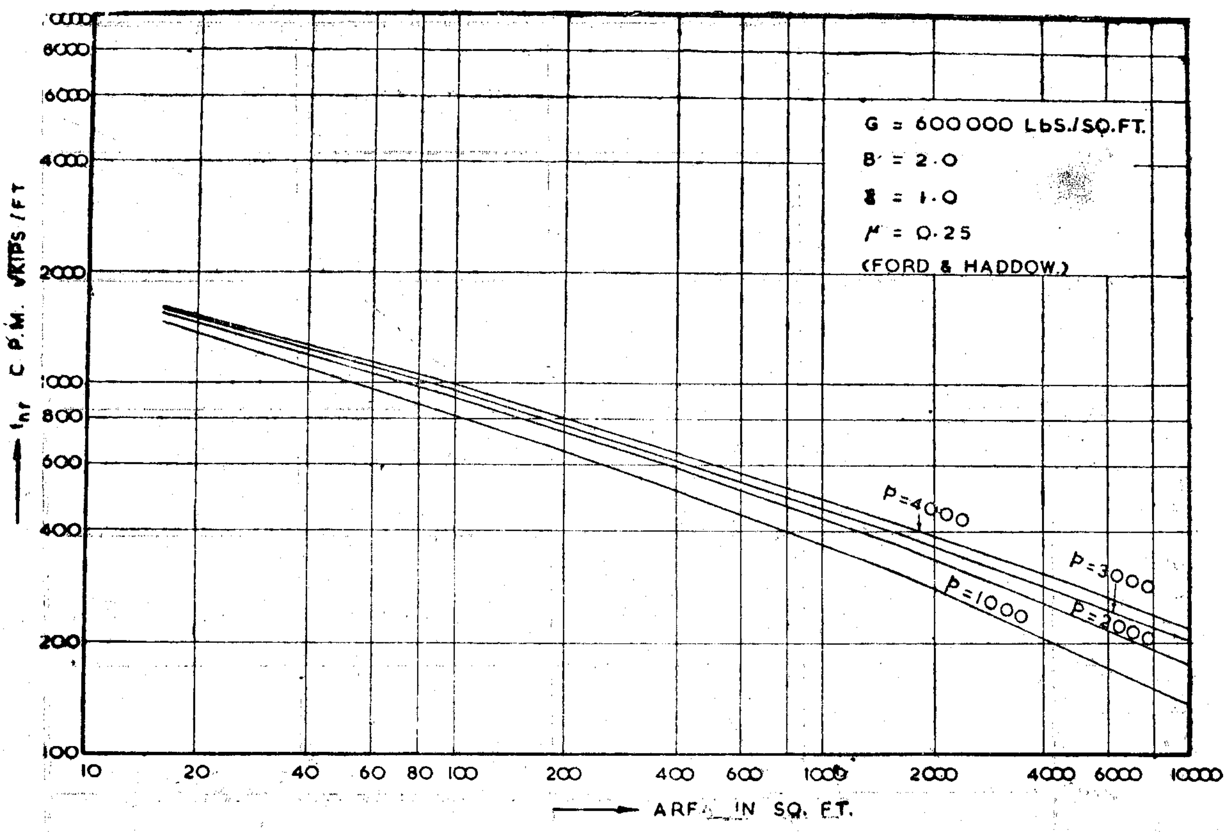

Figure 4 (c)

(Fig. 8). The f_{nr} increases significantly with intensity for all areas of contact in Pauw's method which is not consistant with Tschebotarioff's plot and the other two methods. This marked difference in the behaviour is mainly due to the basic assumption that 'E' increases with static intensity, as stated earlier.

The influence of shape has been taken into consideration by Ford and Haddow and Pauw's methods only. The other two methods ignore the shape effect.

Table 1 shows the effect of shape on f_{nr}. There is a general agreement between the Pauw and Ford and Haddow methods to the extent that the f_{nr} increases with the change in shape from circular to rectangular. The increase in f_{nr} is in the range of 6–12% being larger for larger areas. With the complexity in the behaviour of subgrades under dynamic loading and other vagaries in assessing values for the parameters, differences of less than 10% are unimportant.Hence considering all factors it can be said that the shape of the foundation block has neglegible effect on f_{nr} and f_n of the system.

Figs. 5a and 5b show the influence of μ on f_n for two different areas. The Sung and Ford and Haddow methods show consistent increase in f_n or f_{nr} and the effect of small change in μ on f_n or f_{nr} is phenomenal.

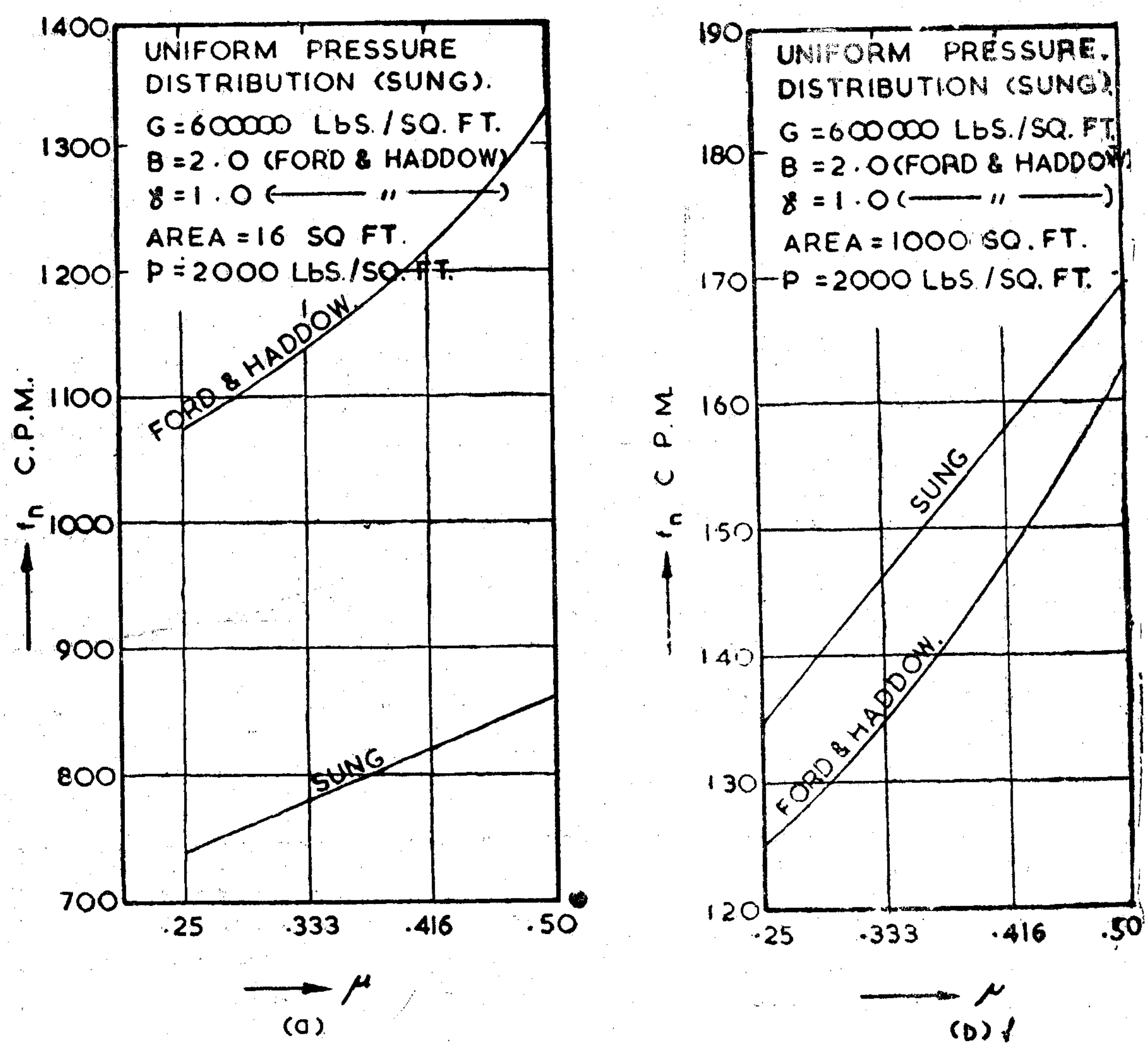

Figure 5 (a,b)

Fig. 6 demonstrates the effect of different types of pressure distribution on f_{nr}. This clearly brings out the necessity of the true knowledge about the pressure distribution if Sung's method is to be adopted. For larger areas the effect is not marked.

Fig 7 exhibits the effect of B values on f_n. The increase in the values of f_n with B is obvious and the effect of 'B' on f_n is phenomenal. Larger the value of B, the more will be the percentage increase of f_n with area. This necessitates proper judgements in the values of B.

Fig. 8 illustrates the effect of 'α' the dispersion angle, in Pauw's method on f_{nr}. A decrease in α from 1.75 to 0.875 reduces f_{nr} to an extent of about 35%. In the absence of a positive method to determine the value of α the assessment of α from experience and judgement may pose a difficult problem.

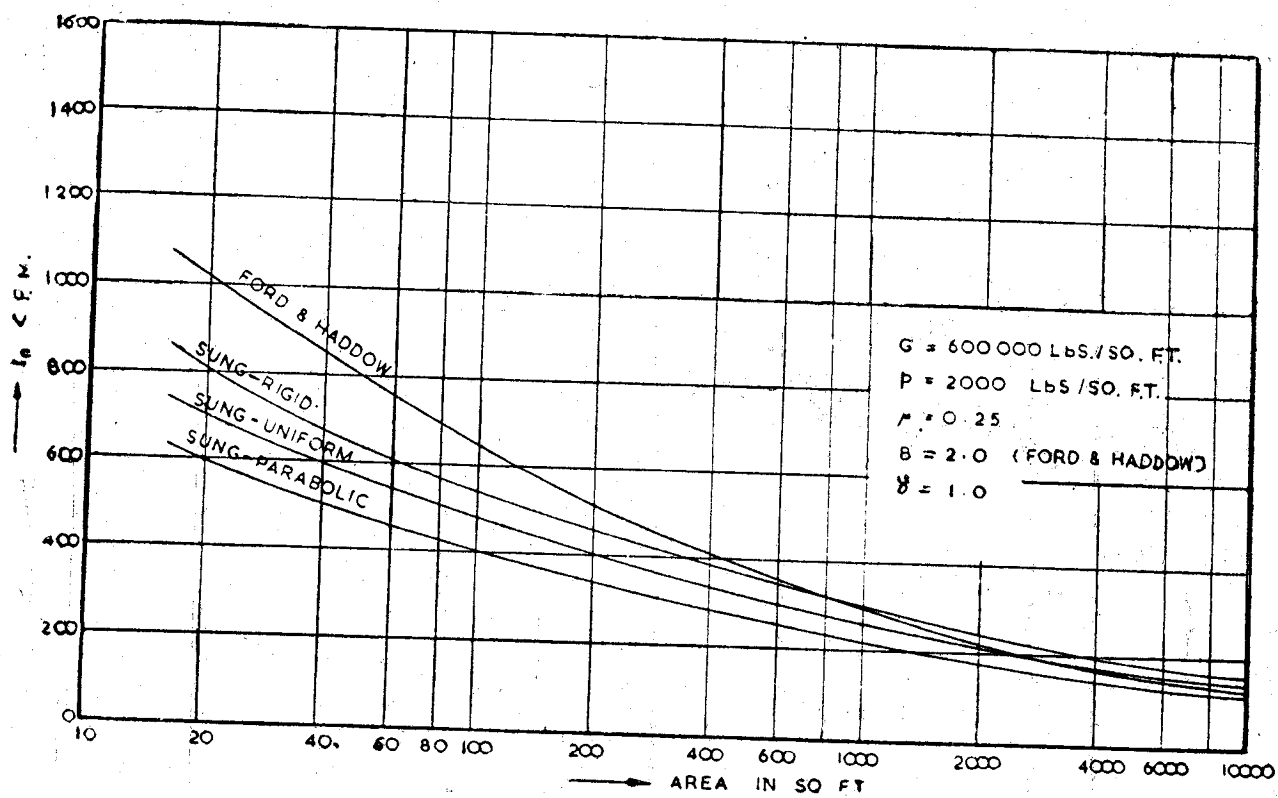

Figure 6

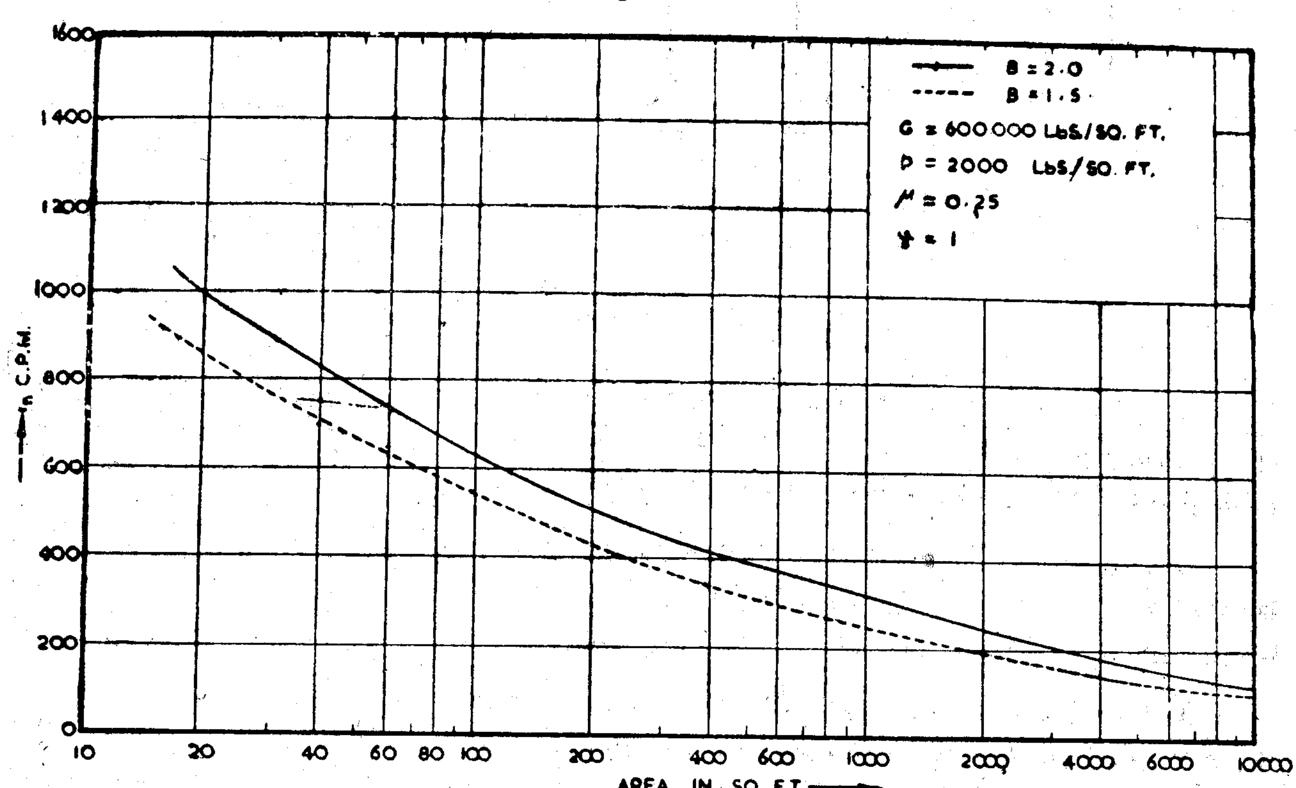

Figure 7

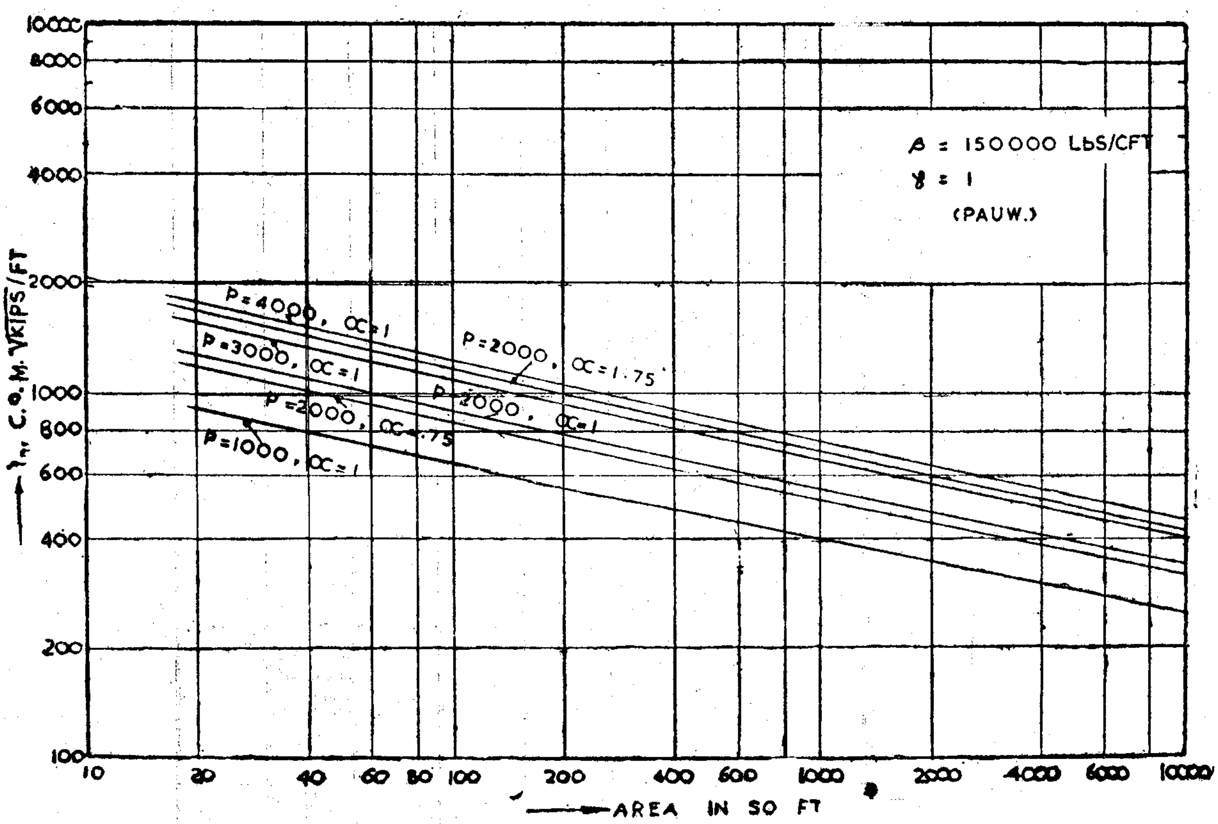

Figure 8

Fig. 9 shows the relation between $\sqrt{G}$ or $\sqrt{\beta}$ with f_{nr} for three different areas. Since f_{nr} is directly proportional to $\sqrt{G}$ or $\sqrt{\beta}$ a straight line relationship is obtained. It is rather interesting to note that all the three methods i.e. Pauw's, Sung's and Ford and Haddow's derive the expression for f_n such that the influence of p, shape, size, μ, B, α and type of pressure distribution is not at all affected by the value of G or β. In other words, for example, the rate of decrease of f_n must be same either for a clayey soil or a sandy soil irrespective of their values of G or β. Experimental confirmation is essential to this effect.

Fig. 1 shows the relation between f_{nr} and Area for two limits of G and β values, for all the three methods. Tschebotarioff's boundary lines i.e. line for sand stone and line for peats have also been included in the plot. The values of G and β were chosen such that they represent the extreme limits for soils. The values for other parameters are also given in the figure. Since the influence of these parameters has been shown earlier, the readers can easily imagine the effect of the possible changes of these parameters in shifting the lines. It is seen that there is a basic agreement between all the four methods especially with Sung's and Ford and Haddow's and Tschebotarioff's plot. The intensity of load for this plot has been taken as 2000 lbs./sq.ft. A change in the value of load intensity alters the position of Pauw's boundary significantly, Ford and Haddow's neglegibly and Sung's none at all. It is evident

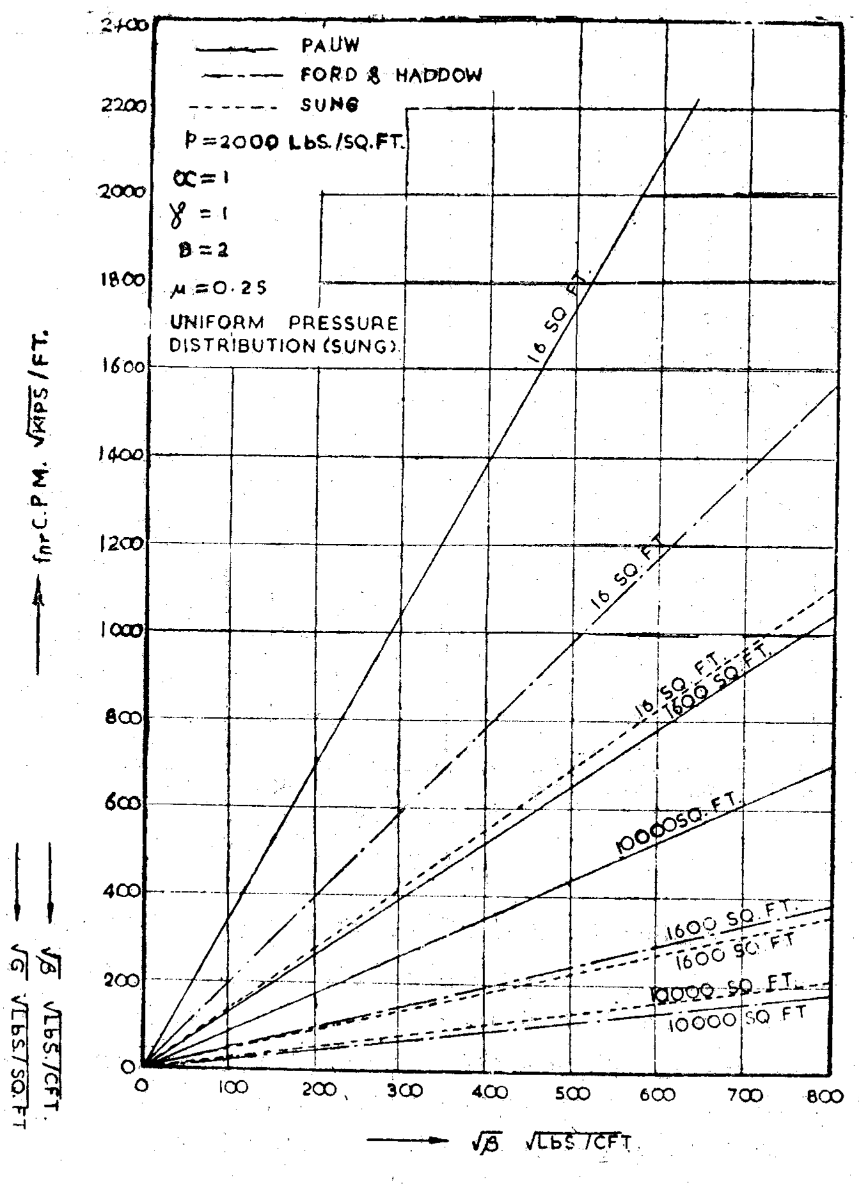

Figure 9

from this illustration that Tschebotarioff's plot almost fixes up the boundaries of danger zone for the occurrance of resonance in machine foundations.

CONCLUSIONS

From the above comparative study, the following conclusions can be drawn for foundations subjected to vertical oscillation.

With different basic approaches Sung's and Ford and Haddow's methods show a similarity with Tschebotarioff's plot between area and f_{nr}.

Ford and Haddow's method is recommended for its simplicity and Sung's for its rigourousness if the type of pressure distribution is known.

Owing to the complex factors involved in the design of foundations subjected to vertical oscillating forces it is hard to expect, any single method to satisfy all the requirements. Hence Sung's and Ford and Haddow's analyses may be applied for any problem and the probable value judged on individual merits.

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TABLE 1

Values of Natural Frequency for different shapes and sizes

p = 2000 lbs./sq. ft. β = 150,000, G = 600,000 μ = .25

B = 2.0 a = 1.0

γ = Ratio of Length to Breadth.

Sl. No.	Area	Ford and Haddow				Pauw			
		Circular	$\gamma=1$	$\gamma=2$	$\gamma=4$	Circular	$\gamma=1$	$\gamma=2$	$\gamma=4$
1	16	1064	1068	1085	1128	900	910	924	940
2	100	636	644	660	686	595	605	617	631
3	400	414	419	428	448	444	452	462	474
4	1600	254	259	267	280	326	334	343	353
5	4900	166	170	173	184	263	270	278	288
6	10000	124	126	129	132	222	229	237	247

Paper No. 31

DYNAMIC RESPONSE OF RECTANGULAR PLATES ON ELASTIC FOUNDATION

P.C. Sharma*

SYNOPSIS

Navier type solution for the dynamic response problem of rectangular plates, simply supported all arround, is presented here. Case of a plate supported on Winkler type foundation and also having damping, is considered and solution obtained. The solutions thus obtained can be useful in the dynamic response analysis of more complicated cases.

INTRODUCTION

The dynamic theory of plates finds many applications in modern technology such as the analysis and design of buildings, aircrafts, ship hulls and pavements. Except for a few exceedingly simple cases, an exact mathematical analysis of such problems is practically impossible. This is even more so for the case of plates on elastic foundation which is important for example in rigid pavement design.

From engineer's standpoint, both frequency and displacement are significant quantities. Bending moment responses can be easily obtained from displacement responses. Therefore it is important to get the displacement response. Unfortunately not much is found under dynamic response of plates, even for simple cases where the solution is straight forward. Since engineers do not have time to devote to routine mathematical manipulations and derivations, it is of some significance to have these results available. Therefore the purpose of this paper is two fold:

1. To demonstrate the use of Navier's method of analysis for dynamic response problem;
2. To make available a few basic results which can be of further use in the dynamic response analysis of plates on elastic foundation.

DIFFERENTIAL EQUATION

The governing differential equation for the small deflections of an elastic thin plate subjected to a lateral loading q (x, y) is given by:

$$\nabla^4 \omega = \frac{\partial^4\omega}{\partial x^4} + \frac{2\,\partial^4\omega}{\partial x^2 \partial y^2} + \frac{\partial^4\omega}{\partial y^4} = \frac{q\,(x, y)}{D} \tag{1}$$

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in which w is the diflection, x, y are space coordinates and D is the flexural rigidity of the plate. (see fig. 1)

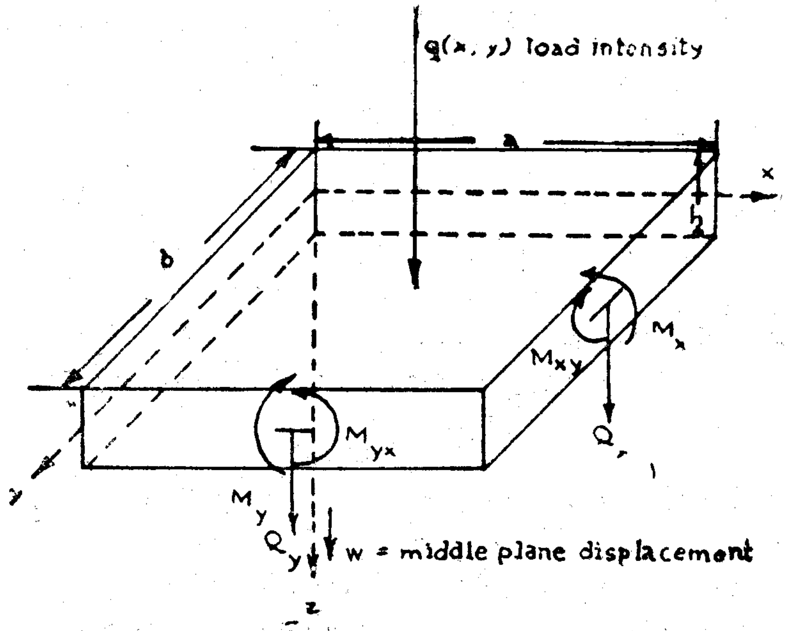

Figure 1 Plate Notation

For the case of dynamic loading and Winkler type elastic foundation with viscous damping, the quation of motion is obtained by replacing q (x.y) in Equation 1 by$(m\frac{\partial^2 w}{\partial t^2} + c\frac{\partial \omega}{\partial t} + kw) + P(x, y, t)$, where $m\frac{\partial^2 \omega}{\partial t^2}$ is the inertia force and $c\frac{\partial \omega}{\partial t} + kw$ is the reaction of the foundation including the effect of viscous damping. Also, P (x, y, t) is the forcing function. Thus Equation 1 becomes:

$$\nabla^4 \omega + \frac{kw}{D} + \frac{c}{D}\frac{\partial \omega}{\partial t} + m\frac{\partial^2 \omega}{\partial t^2} = \frac{P(x, y, t)}{D} \tag{2}$$

This equation together with the appropriate boundary conditions governs the dynamic respo-n e of the plate system to the dynamic loading P (x, y, t).

Mathematically speaking, this partial differential equation is of the parabolic type, and is referred to as a propagation problem in two space dimensions. The solution "marches" in the time domain starting with the initial conditions and confined in space by the boundary conditions. In other words, for the case of rectangular plates, considered in this paper, the solution has to march inside a box (as shown in Fig. 2) whose base is made up of the initial conditions and all the four sides are made up of the boundary conditions, the top being open.

NAVIER TYPE SOLUTION

The double sine series solution of the problem of forced vibration of a simply suppor-

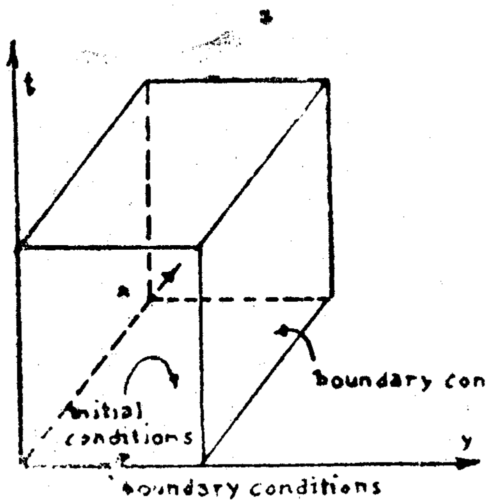

Figure 2 Pictorial Representation of Propagation Problem

ted rectangular plate on an elastic foundation is presented here, which is based on the Navier's solution for the static case. Referring to Equation 2 the solution using the technique of separation of variables, may be written as :

$$w = \sum_{i=1}^{\infty} \sum_{j=1}^{\infty} S_{ij}\, T_{ij} \tag{3}$$

where $S_{ij} = \mathrm{Sin}\,\frac{i\pi x}{a} \,.\, \mathrm{Sin}\,\frac{j\pi x}{b}$ and T_{ij} is a function of time only. Also assuring the loading function be given as :

$$P\,(x, y, t) = G\,(x, y)\, F\,(t) \tag{4}$$

where G (x, y) is a function of the space coordinates 'x' and 'y' only and F (t) is a function of time 't' only.

Let G (x, y) be expanded in a double sine series:

$$G\,(x, y) = \sum_{i=1}^{\infty} \sum_{j=1}^{\infty} g_{ij}\, \mathrm{Sin}\,\frac{i\pi x}{a}\, \mathrm{Sin}\,\frac{j\pi y}{b} \tag{5}$$

in which

$$g_{ij} = \frac{4}{ab} \int_0^a \int_0^b G\,(x, y)\, \mathrm{Sin}\,\frac{i\pi x}{a}\, \mathrm{Sin}\,\frac{j\pi y}{b}\, dxdy \tag{6}$$

Substituting the preceding expressions 3, 4 and 5 into the equation of motion (Equation 2), the following equation is obtained

$$\sum_{i=1}^{\infty} \sum_{j=1}^{\infty} \left[\left(\frac{i\pi}{a}\right)^4 S_{ij}\, T_{ij} + \frac{2(i\pi)^2(j\pi)^2}{ab}\, S_{ij}\, T_{ij} + \left(\frac{j\pi}{b}\right)^4 S_{ij} T_{ij} + \frac{k}{D}\, S_{ij} T_{ij} + \frac{c}{D}\, S_{ij} \dot{T}_{ij} + \frac{m}{D}\, S_{ij}\, \ddot{T}_{ij} - \frac{g_{ij}}{D}\, S_{ij}\, F\,(t) \right] = 0 \tag{7}$$

Since S_{ij} is not identically zero, one obtains :

$$\ddot{T}_{ij} + 2r\,\dot{T}_{ij} + p_{ij}^2\,T_{ij} - \frac{g_{ij}}{m}F(t) = 0 \tag{8}$$

where $r = \frac{c}{2m}$ and p_{ij} is the natural undamped circular frequency of the $(i, j)^{th}$ mode of the plate :

$$p_{ij} = \sqrt{\frac{D}{m}\left[\left\{\left(\frac{i\pi}{a}\right)^2 + \left(\frac{j\pi}{b}\right)^2\right\}^2 + \frac{k}{D}\right]} \tag{9}$$

For the case of zero initial displacement and velocity the solution of Equation 8 may be written as :

$$T_{ij} = \frac{g_{ij}}{mq_{ij}} \int_0^t F(\tau)e^{-r(t-\tau)} \sin q_{ij}(t-\tau)\,d\tau \tag{10}$$

in which q_{ij} is the damped natural circular frequency given by

$$q_{ij} = \sqrt{p^2_{ij} - r^2} \tag{11}$$

the "Critical damping" c_{cr} for the system can be obetained by setting $q^2_{ij}=0$ thus—

$$c_{cr}(i, j) = 2\sqrt{km}\sqrt{\left[1 + \frac{D}{k}\left\{\left(\frac{i\pi}{a}\right)^2 + \left(\frac{j\pi}{b}\right)^2\right\}^2\right]} \tag{12}$$

It may be observed that the factor inside the bracket shows effect of the flexural rigidity of the plate and the mode shapes (value of i and j) or the value of the critical damping for the $(i, j)^{th}$ mode.

For any given loading the solution can be obtained from Equation 3 by use of Equation 6 and 10.

1. Triangular Pulse Loading—In this case $P(x,y,t) = P\left(1 - \frac{t}{t_1}\right)$ is constant over the entire place and taking $r=0$ (no damping), for a square plate Equation 6 yields—

$$g_{ij} = \frac{16P}{\pi^2\, ij}$$

Equation 10 yields—

$$T_{ij} = \frac{16P}{\pi^2 m_{ij} p^2_{ij}} \int_0^t \left(1 - \frac{\tau}{t_1}\right) \sin p_{ij}(t-\tau)\,d\tau$$

$$= \frac{16P}{\pi^2 m_{ij}\, p^2_{ij}}\left(1 - \frac{t}{t_1} - \cos p_{ij}t + \frac{\sin p_{ij}\,t}{t_1\, p_{ij}}\right)$$

Finally substituting into Equation 3 one has

$$w(x,y,t) = \frac{16P}{\pi^2 m} \sum_{i=1}^{\infty} \sum_{J=1}^{\infty} \frac{i}{ij\, p^2_{ij}}\left[1 - \frac{t}{t_1} + \frac{\sin p_{ij}\,t}{t_1\, p_{ij}\,t} - \cos p_{ij}\,t\right] \sin\frac{i\pi x}{a}\, \sin\frac{j\pi y}{b}$$

for $t < t_1$ (13)

2. **Partial loading**—The loading is the same as preceding one except that it is applied over an area uxv whose centre is located at (ξ, η). In this case Equation 6 yields—

$$g_{ij} = \frac{16P}{\pi^2 ij\ uv} \text{Sin} \frac{i\pi x}{a} . \text{Sin} \frac{j\pi n}{b} \text{Sin} \frac{i\pi u}{2a} \text{Sin} \frac{j\pi v}{2b}$$

When the load is concentrated, i.e. u, v→O and P. u. v,→F (a constant), the above equation yields.

$$g_{ij} = \frac{4F}{ab} \text{Sin} \frac{i\pi\xi}{a} \text{Sin} \frac{j\pi\eta}{a} \tag{15}$$

complete solution can be written as before

3. **Rectangular Pulse**—The load P is applied over the plate. In this case the effect of the foundation damping is also included. From Equations 6 and 10 one obtains respectively—

$$g_{ij} = \frac{16P}{\pi^2{}_{ij}} \qquad [16\ (a)]$$

$$T_{ij} = \frac{16P}{m\pi^2 ij\ q_{ij}} \int_0^t e^{-r(t-\tau)} \text{Sin}\ q_{ij}\ (t-\tau)\ d\tau$$

$$= \frac{16P}{\pi^2 ijm} \frac{1}{r^2+q^2_{ij}} \left\{1 - \frac{e^{-tr}}{q_{ij}} (\ r\ \text{Sin}\ q_{ij}\ t + q_{ij}\ \text{Cos}\ q_{ij}\ t\ )\right\} \qquad [16\ (b)]$$

The complete solution is given by Equation 3 as :

$$w\ (x, y, t) = \frac{16P}{m\pi^2} \sum_{i=1}^{\infty} \sum_{j=1}^{\infty} \frac{1}{ij(r^2+q^2_{ij})} \left[1 - e^{-rt} \left(\frac{r}{q_{ij}} \text{Sin}\ q_{ij}\ t + \text{Cos}\ q_{ij} t\right)\right]$$

$$\text{Sin} \frac{i\pi x}{a} \text{Sin} \frac{j\pi y}{b} \text{ For all } t > 0 \text{ and } c_{ij} < c_{cr}\ (i, j) \qquad [16\ (c)$$

NUMERICAL EXAMPLE

As a numerical example, a simply supported concrete slab $12' \times 12' \times 1'$ $(E=2\times10^6$ psi) resting over firm soil ($k = 614.4$ lbs/in^3 and $c=0$) is considered. The first fundamental period ($i=1$, $j=1$) for such a plate is found to be .0093 sec., using Eq. 9 and similarly the value of critical damping for the first mode is found to be c_{cr} (i, j) $= 3.51$ lbs/in^3/sec. It is of interest to note here, that the first fundamental period of the plate corresponding to the first mode ($i=1, j=1$) for $k=0$, $c=0$ is 1.405 times larger i.e. it is .01335 sec. Hence the effect of foundation stiffness is to reduce the period which follows also from mass-spring analogy.

The centre point response is shown in Fig. 3, for the case of a rectangular pulse loading of magnitude 10 psi acting uniformly all over the plate ($t_1=0$) and for $k=614.4$ lbs/in^3 and $c=0$. In the same figure is shown the response curve for same 'k' value but $c=3.50$ (slightly less than the critical damping for the first mode). It may be observed that the influence of damping in reducing the magnitude of centre point response is quite pronounced. It may

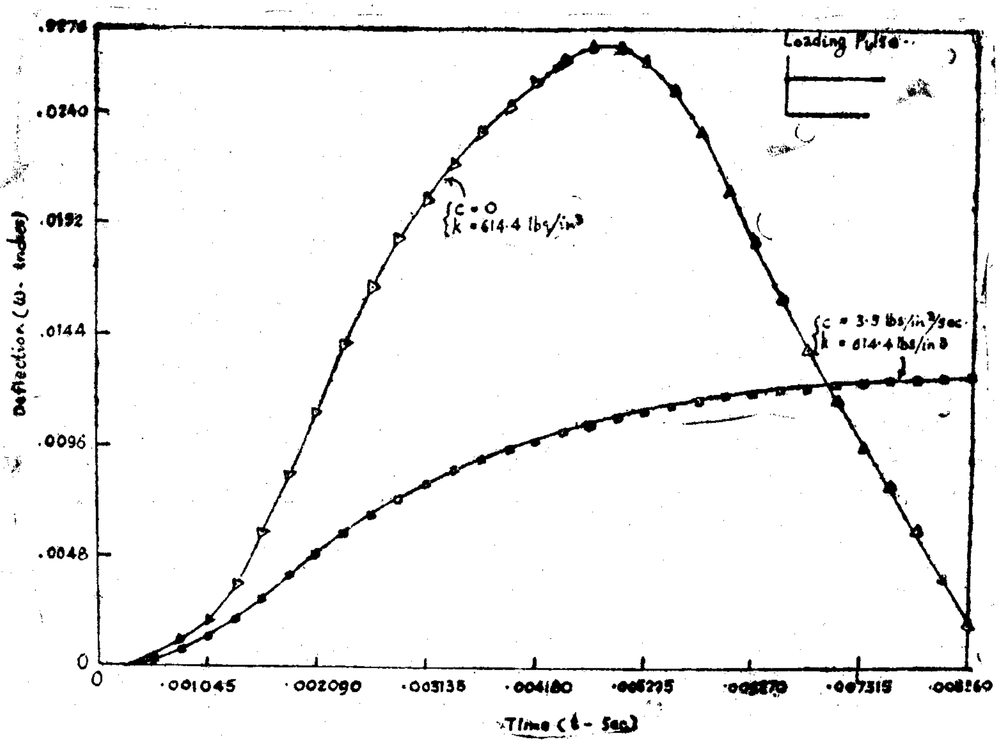

Figure 3 Centre Point Deflection Response

further be noted that for value of $c=3.5$ lbs/in^3/sec, the dynamic response approaches the static response (centre point static deflection=.012971 in.). This would point to the possibility of obtaining static solutions by use of dynamic analysis ("pseudo dynamic approach").

CONCLUSIONS

Navier type solution is convenient and straight forward for solving dynamic response problems of simply supported rectangular plates. However if the boundary conditions are other than those described above this solution will not be a suitable one as is true for the static analysis also Here the vertical reaction of soil has been taken proportional to displacement at that point only which is only an approximation. To consider complex behaviour of soil numerical methods may be used.

ACKNOWLEDGEMENT

This paper is based on the part of author's thesis submitted to Michigan State University, East Lansing Mich. U.S.A. in partial fulfilment of the requirement for the Ph. D degree under guidance of Dr. R.K. Wen, Associate Professor, Department of Civil Engineering.

APPENDIX

NOTATIONS

a=length of the longer side of the plate;

b=length of the shorter side of the plate;

c=foundation damping coefficient;

c_{cr} (i, j)=critical damping for the mode (i,j);

$D=Eh^3/12\,(1-\upsilon^2)$, flexural rigidity of the plate;

E=modulus of elasticity of the plate material;

$F(t)$=time dependent part of the forcing function $P(x,y,t)$;

$G(x,y)$=space function part of the forcing function $P(x,y,t)$;

g_{ij}=Fourier coefficient for $G(x,y)$;

h=plate thickness;

i,j=variable subscripts to denote number of terms in the infinite series;

k=foundation stiffness constant;

m=mass per unit area of plate;

P=magnitude of forcing function;

$P(x,y,t)$=forcing function;

p_{ij}=natural circular frequency (undamped) of the (i, j)th mode of the plate;

$q_{ij}=\sqrt{p^2_{ij}-r^2}$, damped natural circular frequency corresponding to the (i, j) mode;

$q(x,y)$=static loading function acting over the plate;

$r=\dfrac{c}{2m}$, viscous damping parameter;

$S_{ij}=\text{Sin}\,\dfrac{i\pi x}{a}\;\text{Sin}\,\dfrac{j\pi y}{b}$ space function;

T_{ij}=time function;

t=time;

t_1=duration of loading pulse;

u=width along x coordinate direction of the partially loaded area;

v=width along y coordinate direction of the partially loaded area;

w=deflection of plate;

x=space coordinate;

y=space coordinate;

Δ^4=biharmonic operator;

ξ=x coordinate of the centre of loaded area;

η=y coordinate of the centre of loaded area; and

ν=Poisson's ratio.

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SFISMOLOGICAL NOTES

(India Meteorological Department, New Delhi)

Earthquakes felt in and near about India during January–March, 1965.

Date 1965	Origin time (G.M.T.) h. m. s.	Epicentre Lat. (°N)	Epicentre Long. (°E)	Region	Approx. depth (km.)	Magnitude	Remarks
1	2 3 4	5	6	7	8	9	10
Jan. 12	13 32 25 (Shillong)	27	87	Near Chatra, Nepal	—	6.1 (DLH)	Recorded by all Indian observatories.
	13 32 24 (USCGS)	27.6	88	Nepal	33	6.1 (CGS)	
15	18 10 40 (DLH)	22 kms. south west cf Delhi		—	—	—	Felt in Delhi.
21	13 31 29.4 (USCGS)	34.6	86.9	Tibet	33	5.0 (CGS)	Recorded at a number of Indian observatories.
29	20 06 00 (Shillong)	36	74	Karakoram Mountain	—	—	Recorded at a number of Indian observatories.
	20 06 02.4 (USCGS)	35.6	73.6	N-western Kashmir	33	5.7 (CGS)	
Feb. 1	11 14 08 (DLH)	41 kms. North west of Delhi		—	—	—	Felt at Sonepat.
2	15 56 44 (Shillong)	38	74	Pamir Plateau	—	5.9 (DLH)	Recorded at all Indian observatories.
	15 56 51 (USCGS)	37.5	73.4	Tadzik SSR	33	5.8 (CGS)	
13	04 13 32 (Shillong)	45 kms. away from Shillong		—	—	—	Felt at Shillong.
15	01 19 16 (DLH)	40 kms. NW of Delhi		—	—	—	Felt at Sonepat.
	10 03 41 (DLH)	41 kms. NW of Delhi		—	—	—	Felt at Sonepat.
16	20 46 37.4 (USCGS)	36.3	70.8	Hindukush Region	190	5.3 (CGS)	Recorded at a few Indian observatories.
18	04 26 40 (Shillong)	25	94	Manipur State	—	6.2 (DLH)	Felt in large parts of Assam, Manipur etc.

(Continued)

1	2	3	4	5	6	7	8	9	10
	04	26	33.5 (USCGS)	25	94.3	Burma India border Region	36	5.4 (CGS)	
25	10	34	15 (Shillong)	23	93	Lushai Hills	—	—	Felt upto Shillong.
	10	34	06.1 (USCGS)	23.8	94.8	India-Burma border Region	87	5.4 (CGS)	—
Mar. 13		—		—	—	—	—	—	Felt at Calcutta at 03 h. 30 mts. GMT.
14	15	53	00 (Shillong)	37	70	Hindukush Region	—	7.4 (DLH)	Widely felt in all over North west India.
	15	53	06.6 (USCGS)	36.3	70.7	Hindukush Region	219	6.6 (CGS)	Light damage in Afghanistan & W. Pakistan.
18	02	41	27.6 (USCGS)	29.9	80.3	Nepal India border Region	33	5.2 (CGS)	Recorded at a number of Indian observatories.
26	10	04	12 (Shillong)	24.4	70	Rann of Kutch	—	5.3 (DLH)	Recorded at a few Indian observatories.

ABSTRACTS OF PAPERS

Barry, R. (1965), "Dynamic Changes of Mode in Rigid Plastic Structures," J. Engg, Mechanics Div. ASCE, April 1965.

"An account is given of some properties of structures with rigid plastic characteristics, when acted on by time-dependent loads producing accelerated motion of the members. Mode changes are studied and conditions for changes are examined, both for the case in which the number of degrees of freedom increases and for the case in which it decreases."

Hardin, Bobby O. (1965), "The Nature of Damping in Sands," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, No. SM1, Proc. Paper 4206, January, 1965, pp. 63–97.

"The problem of determining soil properties in the dynamic range of loading is discussed. Analytical solutions for two models, closely representing some of the systems that have been used to study the dynamic properties of soils, are presented, where the Kelvin-Voigt model (viscous damping) has been assumed to represent the material. Although sand is a complex material and no simple model will completely describe its behaviour under all loading conditions, the Kelvin-Voigt model can be a useful tool for describing the behaviour of dry sand subjected to small amplitude sinusoidal vibration, over a large frequency range. Experimental results, obtained from steady state and free vibration and static torsion tests, have been presented to show that for shear strain amplitudes in the order of 10^{-6} to 10^{-4} and for confining pressures between 500 and 3,000 psf and for frequencies less than 600 cps, the viscosity should be assumed to decrease with frequency such that the ratio, viscosity x frequency / shear modulus, is constant with frequency, in order to use this model. Values of this ratio for dry sand are given".

Prakash, S., (1965). "Review and Trends in Soil Mechanics Research in India", J. of Indian Society of Soil Mech: and Found. Engg. Jan. 1965 Vol. 4 No. 1.

"A brief summary of significant research in the field of Soil Mechanics carried out in India upto June 1964 has been presented along with trends of future work. Under the conditions of static loading, the problems of Determination of Atterberg limits, Shrinkage, Swelling and consolidation Characteristics of Black Cotton Soils, Bearing Capacity of Shallow Footings, Model study, Penetration tests, Footing in Expansive Soils, Pile Foundations including under-reamed pile foundations, Well foundations, Earthdams and Seepage studies, Instrumentation for soils study etc. have been dealt in great details. Under dynamic conditions, the behaviour of soils under dynamic loads, machine foundations, dams subjected to seismic forces, instruments for dynamic measurements like pore-pressure pick-up, load gauges, deformation gauges, dynamic triaxial equipment etc. and pile foundations subjected to dynamic lateral loads have been presented. It is recognised that there may be some omissions which are expected to be forth coming as discussions".

Choudhary, H.M. (1965), "Sensitivity Adjustments of Electromagnetic Seismograph" Indian Journal of Meteorology and Geophysics (Quarterly) Vol. 16 No. 1, Jan. 65.

"The paper presents a study of the limitations impossed by Seismometer and Galvanometer on the attenuation factor of the circuit. It is seen that the full range from zero to one of this factor can be used only if (i) $Z_{11}=Z_{22}$, (ii) R_1/Z_{11} and R_2/Z_{22} are both less than or equal to $\frac{1}{2}$. The special case of a Seismograph satisfying the Galitzin candition has been studied in the light of the above limitations and a few possible ways of adjusting the various parameters of the seismometer to obtain the optimum magnification considered".

Chandra, U. (1965), "Earthquake Energy Determinations" Journal of the Indian Geophysical Union, Vol. 2, No. 1, pp. 29–37, 1965

"Various methods for estimating the order of energy released in an earthquake have been discussed. These include the use of macroseismic observations as well as seismographic records. Both, body waves and surface waves have been used. The equations used are based on many simplified assumptions, which have been discussed at some length".

IDNIAN SOCIETY OF EARTHQUAKE TECHNOLOGY
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2. Manuscripts must be type written in English or Hindi with two-line spacing on one side of the paper only.
3. The use of the first person should be avoided, the writer being referred to as "the Author".
4. All mathematical symbols should be defined, where they appear first in the text.
   Drawings or sketches should not be included in, or pasted on to the pages of the manuscript, but should be entirely separate.
5. Each article should be accompanied by a "Synopsis" of its subject matter, with special reference to any conclusions, and it should not exceed 300 words.
   A set of conclusions must be given at the end of the article.
6. Bibliographical references should be standardized as follows:—
   (a) In the text the auther's name and the year of publication should appear in parentheses as (Krishna 1960) or Krishna (1960).
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   Name, initials, year of publication.
   Title of work, Source (in full), volume number, page number (beginning)—page number (end).

Example:—

Chandrasekaran, A.R., (1962) "Effect of Joint Rotation on the Dynamics of Multistoreyed Frames", Proceedings, Second Symposium on Earthquake Engineering, University of Roorkee, Roorkee, Nov. 10–12, 1962, pp. 305–316.

Prakash, S. and U.K. Bhatia (1964), "A Review of Machine Foundation Behaviour" Bulletin, Indian Society of Earthquake Technology, Vol. I No. 1, January, 1964, pp 45–64.

ILLUSTRATIONS

1. Drawing should be made on tracing linen or paper in dense black drawing-ink, the thickness of lines being consistent with a reduction to one half or less in the process of reproduction, details shown should represent the minimum necessary for a clear understanding of what it is desired to illustrate.
2. The maximum final size of a single drawing or of a group of drawings which are intended to appear on the same page, is 7½ inches (19 centimeters) by 5 inches (13 centimeters). Drawings should be submitted larger than final size, the ideal being twice final size i.e. upto 15 inches (38 centimeteres) by 10 inches (26 centimeters).
3. It should be ensured that printing of caption in the illustration is large enough so that it would be legible after reduction to one half linear size. 3/16″ (0 5 cm.) size letters are recommended.
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5. MSS. may also be accompanied by photographs (glossy prints) which should, however, represent the minimum number essential to a clear understanding of the subject. No lettering of any kind should be added to the face of a photograph: the figure number and caption being printed lightly on the reverse side or upon the front of the mounting if mounted.
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EDITOR

NEWS OF MEMBERS

1. **Dr. Jai Krishna,** Professor and Director, School of Research and Training in Earthquake Enineering has been invited to join UNESCO working group on Earthquake Engineering as a representative of India. He attended the first meeting of the group held in Tbilisi (U.S.S.R.) in April 1965.

2. **Dr. Jagdish Narain,** Professor of Civil Engineering University of Roorkee has been honoured by the American Society of Civil Engineers with Norman medal for the year 1965 for his paper entitled "Flexibility and Cracking of Earth dams".

3. **Dr. V. J. Patel,** left Civil Engineering Department of Indian Institute of Technology, Kanpur and has joined as Professor and Head of Civil Engineering Department, Government Engineering College, Jabalpur.

4. **Dr. Shamsher Prakash,** has been invited to join the Sectional Committee B D C 23, by the Indian Standards Institution. He has also been elected as member of the Executive Committee of Indian National Society of Soil Mechanics and Foundation Engineering for the period, 1965–1967.

5. **Sri J. N. Mathur,** Lecturer in Civil Engineering, School of Research and Training in Earthquake Engineering has been offered teaching assistantship in the University of California, Berkeley for advanced studies. He will be working under the guidance of Prof. H. B. Seed for his advanced degree in Soil Dynamics.