DYNAMIC RESPONSE OF STRUCTURES ON PILES

A Dissertation
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of the requirements for the Degree of
MASTER OF ENGINEERING
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EARTHQUAKE ENGINEERING
with Specialization in Structural Dynamics

By

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C E R T I F I C A T E

Certified that the thesis entitled "Dynamic Response of Structures on Piles", which is being submitted by Sri Piyush Kumar in partial fulfilment for the award of Master of Engineering in "Earthquake Engineering" with specialisation in "Structural Dynamics" of the University of Roorkee, Roorkee is a record of student's own work carried out by him under our supervision and guidence. The matter embodied in this thesis has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for an effective period of 9 months from January 1971 to September 1973 for preparing this thesis for Master of Engineering Degree at this University.

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SYNOPSIS

The analysis of the response of structures supported on flexible pile foundations, subjected to dynamic loads, involves the problem of interaction between soil, pile and superstructure systems. Various authors have proposed methods for the analysis of such systems. These methods, involve tedious mathematical operations and requires elaborate field testing arrangements, and are therefore not suited for the day to-day design problems.

In the work reported here-in, an attempt has been made to evolve a simple method to analyse the response of such systems under dynamic lateral loads. The complex system has been assumed to be represented by an equivalent two degree freedom system. The soil and pile have been assumed to be represented by a single degree freedom system, which is coupled to an another single degree freedom system, representing the super-structure. The effect of various parameters, that is, natural frequency of foundation, natural frequency of Super-structure and the mass ratio between the super-structure and foundation systems on the response of system, when exited by earthquake motion has been

theoretically analysed.

Experimental studies have also been conducted by mounting a single degree mass-spring system, on an aluminium pile head, embedded in sand. The response of the system hasbeen actually observed by giving it Sinusoidally varying vibrations. The above model has been theoretically analysed, by experimentally evaluating the various constants for it. The results obtained by experimental observations and those obtained by theoretical analysis compares well.

It is concluded that,

A structure supported on flexible piles can be reasonably assumed as an equivalent two degree freedom system i.e. a single degree freedom system representing the super-structure coupled with another single degree freedom system representing the foundation and soil.

It is observed that, there is a particular range of Sub-structure natural frequencies at which, the forces developed in the super-structure are larger, for a particular natural period of superstructure.

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NOTATIONS

The notations are defined wherever they first appear. Here they are listed in alphabetical order for convenience of reference.

A	=	An Emperical Constant in Spectra Equation.
(A)	=	Mass Matrix.
A_y	=	Non Dimensional Deflection Coefficient.
(B)	=	Damping Matrix
(C)	=	Stiffness Matrix
C_f	=	Damping in Soil- foundation System.
C_s	=	Damping in Super-structure System.
E	=	Modulus of Elasticity.
F_f	=	Natural Frequency of Soil-foundation System.
F_s	=	Natural Frequency of Super-structure System.
g	=	Acceleration due to Gravity.
I	=	Moment of Inertia.
$K(x)$	=	Subgrade Modulus at Depth x.
K_f	=	Stiffness of Soil-foundation System for Lateral Loads.
K_s	=	Stiffness of Super-structure System for Lateral Loads.

LIST OF ILLUSTRATIONS

L_s = Embedded Length of Pile.

m_f = Mass of Soil-foundation System Active in Vibrations.

m_s = Mass of Super-structure System Active in Vibration.

Q_{hg} = Horizontal Force Applied on Pile at Ground Level.

S_v = Spectral Velocity.

S_d = Spectral Displacement.

T = Natural Period/Relative Stiffness Factor.

T_a = Natural period of Absorber System.

T_p = Natural Period of Parent System.

W = Effective Weight of Soil-foundation System Active in Vibrations.

X = Displacement

Y_g = Lateral Displacement of Pile at Ground Level.

Z = R.M.S. Value of Maximum Inter storey Shears in Different Modes.

α = Ratio of Mass of Super-Structure to Sub-structure.

β = Response Factor.

w = Modal Frequency.

ξ = Critical Damping.

H = Coefficient of Subgrade Modulus.

CHAPTER-I

INTRODUCTION

Structures supported on pile foundations are often subjected to dynamic lateral loads. Waterfront structures can be cited as a good example of such cases as they experience the impact, mooring forces of ships and wave forces. Earthquake excited ground motions also cause dynamic lateral forces in structures.

The procedure most resorted to, in the earthquake resistant design of such structures, is the so called equivalent static method in which a horizontal force equal to $\alpha_h W$ is taken to act laterally on the structure, α_h being the horizontal seismic coefficient and W the total weight of the structure including the surcharge load. In such cases, the pile groups are designed for the combined lateral resistive force equal to the static horizontal force transferred from the super structure to the top of foundation. In the analysis, the structure and foundations are considered separately and the influence of one over the other is ignored.

Though the design procedure discussed above is simple and handy, so far as practical design problems are concerned, the theory is usually based on a crude assumption that the soil, the pile and the super structure remain uncoupled during vibrations, and independent of each other's influence. In fact such a problem involves interaction between the three elements of the system as mentioned above.

The field data show a different response as compared to structures supported on rigid bases for the same magnitude and nature of applied dynamic load. Murphy[4] has discussed the inter-relationship of the earthquake ground-structure systems. The effect of interaction between the foundation and structure is primarily to modify the natural periods of vibrations of structures[3,5].

P-ensien[10] has proposed a rigorous and systematic procedure for the analysis of dynamically coupled pile-structure systems. A rational analytical procedure has been proposed by assuming the pile-structure as a continuous system. The interaction between the pile and soil has been taken into account by superimposing the pile displacements, which in turn are obtained by a separate analysis of the medium. Though the procedure suggested by Penzien is systematic and exact, it involves tedious mathematical operations and requires elaborate field testing arrangements and therefore is not suited for use in design offices.

In the present studies an attempt has been made to analyse the response of coupled structure pile systems, subjected to lateral dynamic loads. As a result of the experimental studies, it has been possible to evolve a simplified analysis, suitable for use in field problems.

In the theoretical studies reported here the structure foundation system has been assumed to be represented by a system having two degrees of freedom. Thus a single degree freedom system representing the superstructure is coupled with another single degree freedom system representing soil medium and foundation. Experimental studies have been conducted by mounting a single degree mass spring system, on a 26" long aluminium pile, embedded in the sand. The actual response of the mounted mass has been observed by exciting the system.

The response of the model structure supported on the pile embedded in sand has been obtained theoretically by assuming a two degree freedom to represent the system. The effective mass, spring and damping constant for the equivalent two degree freedom system, have been found experimentally. The computed response of the model and

C H A P T E R II

REVIEW OF THEORETICAL AND EXPERIMENTAL WORK

2.1 A brief review of the theoretical and experimental work on structures supported on pile or flexible foundations with a special reference to their dynamic behaviour is discussed in this chapter.

2.2 Dynamic Coupling Between the Foundations and Superstructure Systems

The superstructure and the foundation ~~which~~ form two integerated parts of a complete structural system. The response of the superstructure to the applied loads, not only depends on its structural properties, but the foundation also plays an important role, in the structural behaviour.

The problem of such coupled systems was analytically studied by Fleming J.F. and Screwalla F.N.[9], by considering a typical multistoreyed structure supported on a flexible foundation. The effect of the flexible foundations on the structural response was studied by analysing the system, assuming the foundation to have different flexiblities. The flexibility of the foundation was found to have a significant effect on the inter-story displacements of the superstructure.

The model which was considered in the analysis is shown in figure 2.1. Figure 2.1(a) shows a two storeyed building frame resting on the ground. The mass of the foundation is assumed to be lumped at the ground floor level. The flexibility of the foundation was incorporated in the model, by assuming the superstructure to be connected to the rock, by a massless flexible member, The rotational stiffness of the foundation is shown by a torsional spring at the ground floor level.

The three humped weights discussed above were assumed to be 40,000 lbs each. The flexible member was assumed to have the linearly elastic force-displacement relationship for the lateral loads. The system was analysed for the different flexibilities of the flexible member as 1,000, 100,000, 1000,000 p.s.i. and for a perfectly rigid foundation condition. The results of the analysis with different foundation flexibilities are shown in figure 2.1(b), (c), (d).

It is observed from the above memtioned figures that when the foundation is sufficiently flexible to cause more or less a rigid body movement of the whole frame, the interstory deformations in frame are small, thus causing low stresses in the columns. Relative displacements of masses are small even though the absolute displacements are large. With the stiffening of foundation, the masses are subjected to higher interstorey displacements. It is interesting to note that the displacement of the masses are of the same order in the case of a rigid base and for the stiffness of foundation as 100,000 p.s.i. Therefore for this particular structure, the foundation stiffness of 100,000 p.s.i. represents a case of rigid base.

From the above results it can be concluded that the flexibility of the foundation relative to superstructure in an impbrtant factor to be considered in the analysis. Any rigorous method adopted to analyse the structural resposse, can lead to erroneous conclusions if the interaction between the foundation and super structure is ignored.

It is therefore important that the behaviour of the foundation to the dymamic loads must also be considered in the analysis of structural systems. A brief review of the behaviour of piles to the dynamic as well as to static lateral loads is discussed in 2.3.

2.3 Behaviour of piles Under Lateral Loads

2.31 Behaviour of piles under static lateral loads

The lateral load on pile tends to deflect it in its direction of application. The response of the pile to lateral load depends on the relative stiffness of pile with respect to ground and the embedded length in the supporting media. Depending on the structural behaviour of the pile, it can be classified in two groups, as described below.

(i) Rigid piles

The pile, when embedded to a small depth, behaves as a rigid pole. In such a case, the bending strains do not develope and only the shear strains are predominant. A pile acting as a pole affers a weaker resistance to the lateral loads.

(ii) Flexible piles

As the embedded length increases, the bottom tip of the pile starts to develop restrain against the lateral displacement and rotations. The pile starts behaving like a cantilever (or beam) on elastic foundation, by resisting the strain by its flexural stiffness. With the increase of pile length in soil, the fixity at the tip of pile increases and a stage is reached when further embedment of pile does not cause any change in the pile behaviour. This particular embeded length of the pile is termed as the characteristics length.

Figure 2.2(a) shows the deflected shape and soil reaction in a laterally loaded semi flexible pile.

Load Deformation Relationship for a Laterally Loaded Pile

As explained above, the pile starts deflecting, when a lateral load is applied on it, compressing the soil, which comes in contact with it. A typical load-deformation curve for a point along the embedded length of the pile, is shown in figure 2.2(b). It is observed in general, that upto the limit of load less than about half of the ultimate load, the deflection remain almost in direct proportion to the load acting at the particular location. The slope of this line is termed as "Modulus of subgrade reaction" with the further increase in the load, the curve follows a non-linear relationship. In this range of non-linearity, the soil modulus is defined by the Secant modulus of the curve.

Soil modulus is effected by the type of the soil and the size of the loaded area, and the depth of the location considered.

Terzaghi[16] found that the subgrade modulus remains almost constant in the over consolidated clays. For sands this value may be taken to be linearly varying with the depth. Figure 2.3 (a),(b), and (c) shows different cases of the variation of subgrade modulus with the depth of pile. In Figure 2.3(d) the generally assumed and probable variation of subgrade modulus with depth is shown for the stiff clays. Figure (e) shows for the sands.

A general expression for the subgrade modulus at depth x is expressed as

$$K(x) = K\left(\frac{x}{L_s}\right)^n$$

where K is the maximum value of K at the bottom tip of the pile. L_s is the embedded length of the pile n is an emperical coefficient.

For stiff clays n may be assumed be zero hence the expression becomes as

$$K(x) = K$$

And for the sands n may be assumed to be unity, hence the expression may be taken as

$$K(x) = K/L_s x$$

or $$K(x) = \eta_H x$$

where η_H is a constant and termed as "coefficient of modulus of subgrade reaction".

Significant investigation for the laterally loaded piles under static loads, for subgrade modulus varying linearly with depth were made by Reese and Matlock[17]. They ignored the affect of vertical loads acting on the pile. A brief review of their work is given below;

It was observed that the behaviour of the laterally loaded pile (under static loads) primarily depends on the type of soil and its properties, variation of subgrade modulus with the depth, properties of the pile, forces acting at the pile head and end conditions of the pile.

Taking into account the above factors, a theory was developed for the computations of the deflections of a laterally loaded flexible pile.

As described above the behaviour of the pile depends on the relative stiffness of the pile to that of soil, a relative stiffness factor T was expressed as

$$T = \sqrt[5]{EI/\eta_H} \quad \text{(For K linearly varying with depth)}$$

Further a nondiemensional factor (Z) was defined as

$$Z = L/_T \quad \text{(for K linearly varying with depth)}$$

where EI is the flexural stiffness of the pile

L is thedepth of the point along the embedded length of the pile.

It was suggested that for a flexible pile embedded in sand the minimum value of Z_{max} or ($Z_{max} = L_s/T$ where L_s be the embedded length of the pile) should be greater than 5.

A theory was developed to compute the deflections of laterally loaded pile (static loads), by solving the flexural differential equation in terms of nondiemensional parameters. This has been explained in greater detail in chapter VI.

2.3.2 Behaviour of Pile Under Lateral Dynamic Loads

Very little literature is available on the experimental work on dynamically loaded prototype piles, probably because of cost considerations. Experiments were performed by Gaul(11) on the dynamically loaded model of piles. The work was done on a dimensionally scaled model of a vertical pile, embedded in Bentonite clay.

Gaul reported that,

(1) Pile vibrates in a form of tanding wave, which is in phase with the oscillating load. Velocity of the pile, and hence the damping capacity of soil was negligible.

(2) At relatively low frequency of load oscillation, no dynamic load factor is required to produce the same maximum bending moment.

(3) Soil modulus is constant for Bentonite clay under dynamic loads.

(4) Location of the point of maximum bending moment is independent of the magnitude of lateral load, unless the pile displacements are enough to stress the soil beyond elastic range.

(5) Over burden reduces bending moment, but the shape and the location of maximum bending moment remains the same.

The above study was perfoemed on only one type of the clay (Bentonite) and the dynamic load was applied at only one frequency (1 cps), therefore no general conclusion can be drawn from this study.

Further work in this field was done by Hayashi and Miyajima[13]. They conducted tests on steel H piles embedded vertically in sand and subjected to lateral, verticall and static loads. Dimensions of the piles used were 300x305x15 mm lengths 14 meters and 16 meters. Four kinds of artificially conditioned subgrade with different relative density were prepared. Methods of loading were (i) Free vibration of pile caused by Sudden release of initial tension by cutting the wire rope (ii) Forced vibration by vibration generator installed at the pile head.

From the experiments is was concluded that

(1) Natural frequencies and the resonance curve of single vertical pile could be calculated by conceiving a simplified system of vibration and the results of calculations agreed with those obtained by the free and forced vibration tests.

(ii) Damping co-efficient measured in free vibration tests depended on the relative density of the subgrade and length of the free part of pile.

Aggarwal, S.L.[14] conducted static and dynamic tests for a laterally loaded model piles. The piles were of aluminium pipe having 15 mm outer diameter and 10 mm inner diameter. The soil medium was sand. Study revealed that;

(i) The pile vibrates in the form of standing wave probably in phase with the oscillating load.

(ii) At lower pile displacements, increase in frequency results increase of load. As the pile is vibrated at different displacements, the soil vibrates in greater quantity, compacting the soil completely. After complete compaction of soil, the frequency does not have any effect on load displacement curves.

(iii) The zone of influence of dynamically loaded pile extends to a considerable distance than that of a statically loaded pile.

2.4 Systematic Analysis of Dynamically Coupled Structure-Foundation System

A systematic and rigorus analysis of a coupled structure, subjected to dynamic lateral loads, was presented by Penzien(10). The pile-structure system is assumed as continuous system. The interaction between the pile and soil has been taken into account by superimposing the pile displacements, which in turn are obtained by a separate analysis of the medium, exciting it by forced ground motion.

The proposed analysis takes into account all the relevent soil properties, i.e. creep, damping and remoulding of soil due to the vibrations.

One of the important findings of Penzien is that the effect of interaction between the soil and the pile is very small. There is negligible effect in the response of the system, by considering the soil mass participating in the vibrations. This is mainly because of the greater stiffness of the pile material as compared to that of the surrounding media.

Though the procedure suggested by Penzien is systematic and exact, it involves tedious mathematical operations and requires elaborate field testing arrangements.

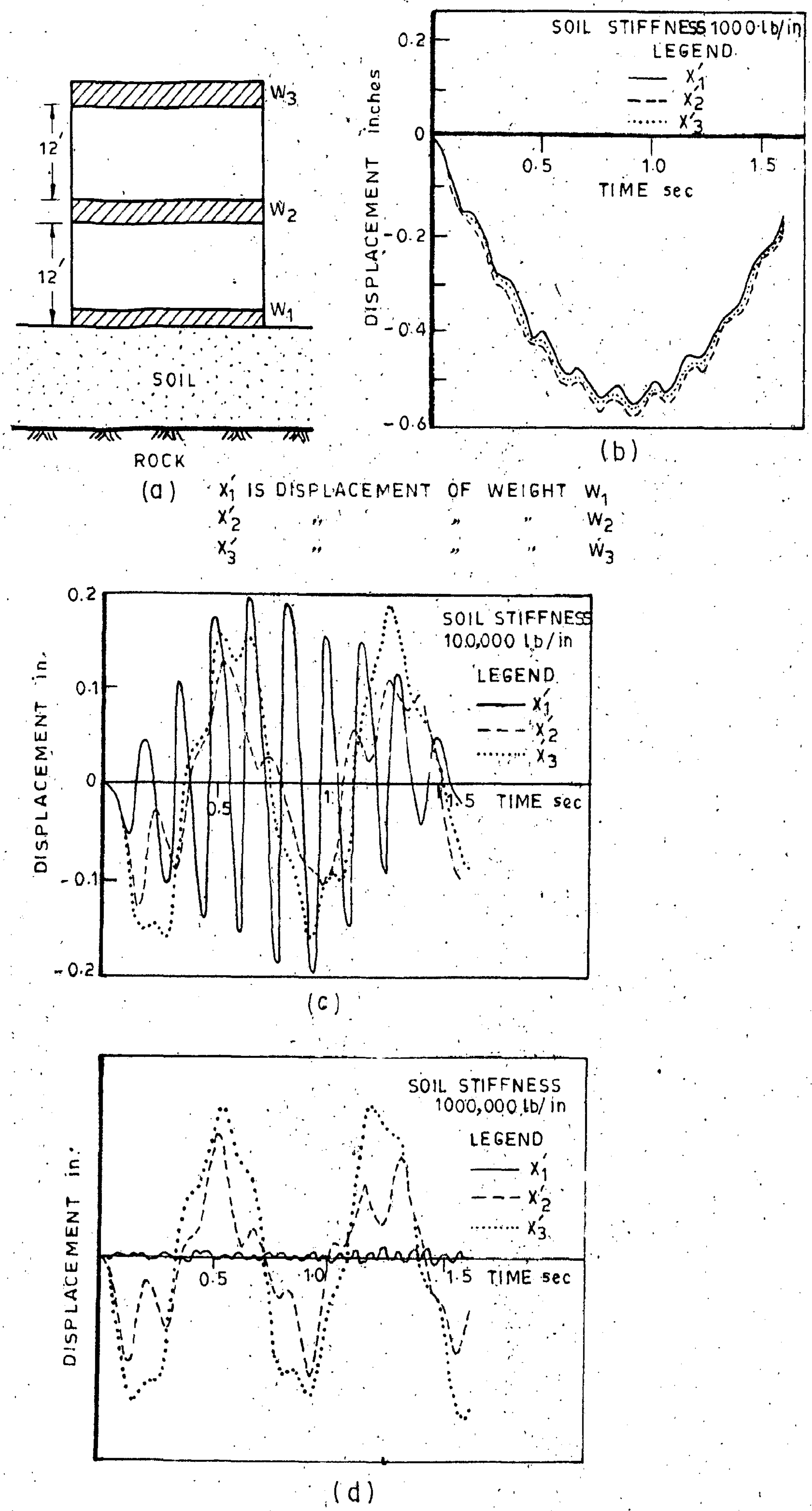
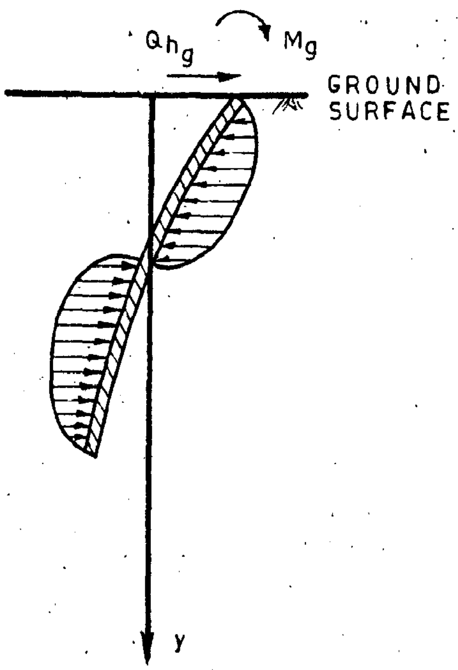
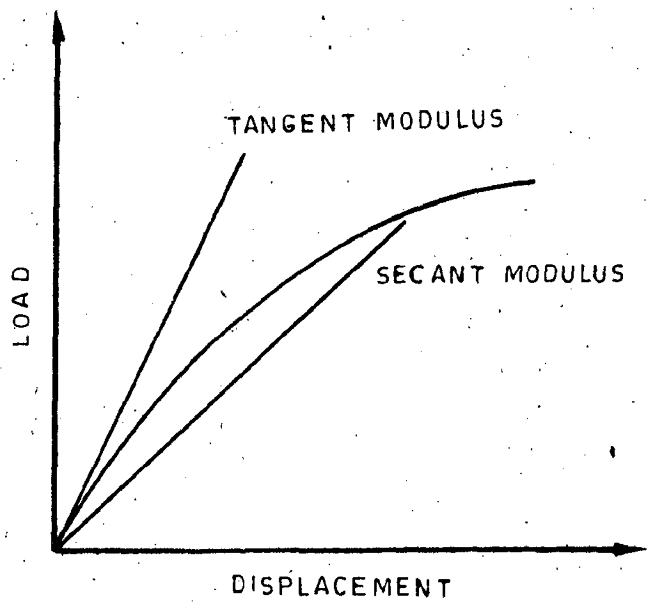

W3
W2
W1
12'
12'
SOIL
ROCK
(a)
SOIL STIFFNESS 1000 lb/in
LEGEND
X1'
X2'
X3'
DISPLACEMENT inches
TIME sec
(b)
X1' IS DISPLACEMENT OF WEIGHT W1
X2' " " " W2
X3' " " " W3
SOIL STIFFNESS 100,000 Lb/in
LEGEND
DISPLACEMENT in.
TIME sec
(c)
SOIL STIFFNESS 1000,000 lb/in
LEGEND
DISPLACEMENT in.
TIME sec
(d)

FIG. 2.1



a – DEFORMATED SHAPE OF A LATERALLY LOADED SEMIFLEXIBLE PILE



b – LOAD DISPLACEMENT CURVE FOR A POINT ALONG THE PILE

FIG. 2.2

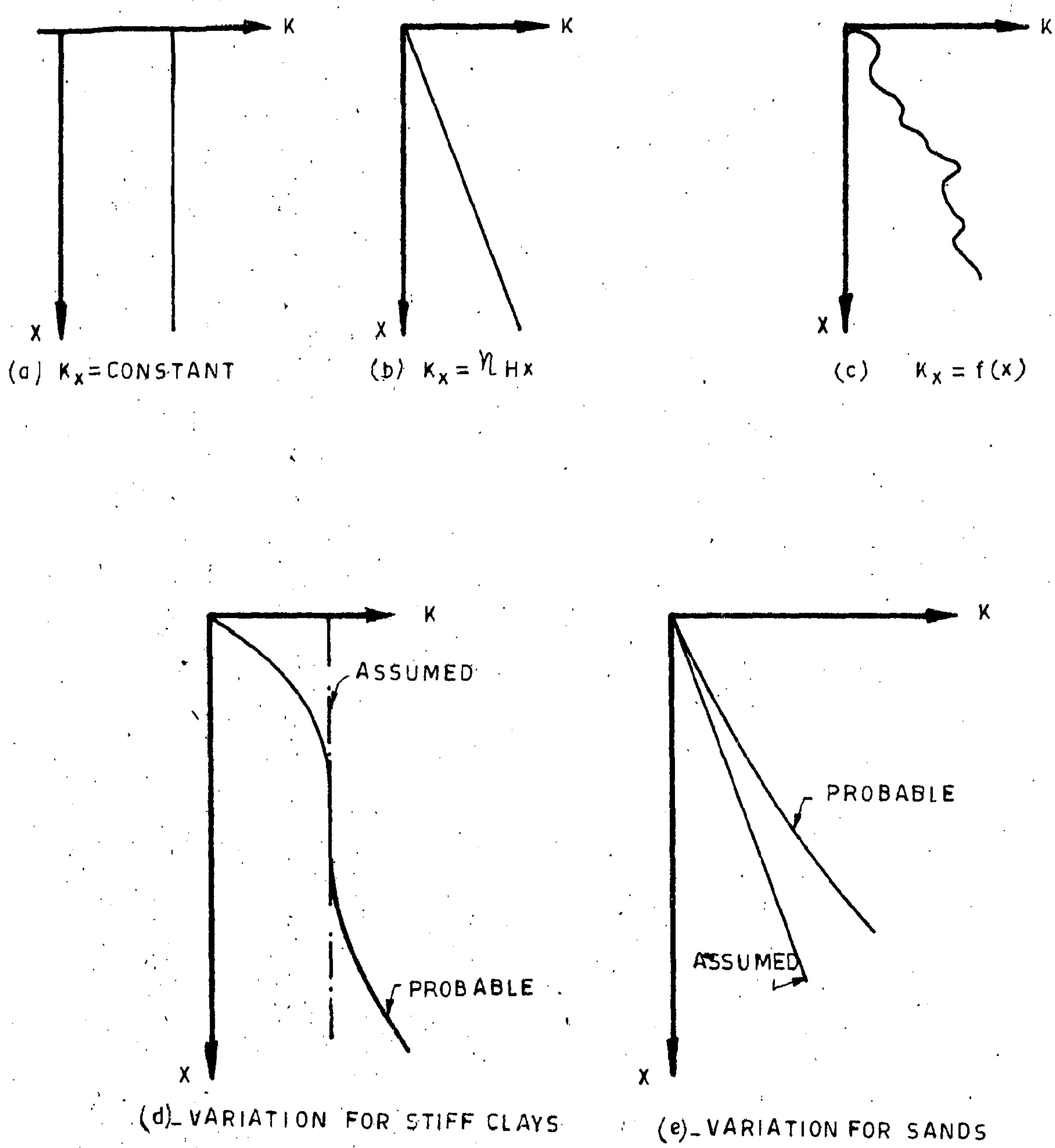


FIG. 2.3_VARIATIONS OF COEFFICIENT OF SUBGRADE MODULUS ALONG THE EMBEDDED LENGTH OF PILE

CHAPTER III

THEORETICAL ANALYSIS

3.1 Approach to the Problem

The exact analysis of a multistoreyed building resting on piles under dynamic load is a complicated problem. In practice the static design of multistoreyed buildings is carried by assuming the base to be rigidly fixed at the pile cap level, while under static lateral loads, there is some displacement at pile cap level and is not taken in account, since it complicates the problem to a great extent. The dynamic load imposed on such system further complicates the analysis and the exact solution for such cases are not yet available.

Such a system can be assumed to be divided in two components, one the superstructure and other the foundation system. Much of the work is available as far as the superstructure portion is concerned. The superstructure based on rigid foundations offers more or less well defined physical properties such as stiffness and damping and thus enables exact and rigorous solutions for its analysis. Hosever, the problem is not so simple in case of pile foundations.

The behaviour of pile under dynamic lateral loads is a complicated problem. A vertical pile under lateral loads behaves as a beam on elastic foundations (offered by assumed elastic foundation media supporting the pile). When subjected to dynamic loads, some part of the soil also becomes active in vibration with pile and thus changing the net effective mass and stiffness of the foundation system. Thus in true sense such a foundation system will be a problem of infinite degrees of freedom and shall involve the complex phenominon of interaction between the soil and pile.

In the present studies an attempt has been made to analyse the problem in a simplified manner by assuming the whole system to be represented by a two degrees of freedom system. The superstructure has been assumed to be represented by a single degree freedom system, mounted on another single degree system representing the foundation of the structure. The problem of infinite degrees of freedom has been thus reduced to a problem of two degrees of freedom system.

The effect of different parameters on the interstorey shears has been studied. The parameters which effect the response, have been widely varied in normal range of such prototype systems. The different parameters studied are as below.

(i) Natural period of superstructure (when it is supported on rigid foundations).

(ii) Natural period of substructures system. (This is assumed to include foundation and soil system both).

(iii) Mass ratio of superstructure to foundation.

3.2 Formulation of the Problem

An example of a multistoreyed building resting on pile foundation is shown in figure 3.1(a). A multistoreyed system of any number of degrees of freedom is shown to be resting on pile cap which itself is supported on a system of group of piles. For the purpose of present studies this generalized form of complex structure (having infinite degrees of freedom and complex phenominon of coupling between different elemtns of the system) has been assumed to be represented by a simple system of two degrees of freedom. This assueed idealisation is shown in figure 3.1(b). The top mass-spring system is assumed to be representing the superstructure portion (above pile cap level), while the bottom oneis representing the foundation - soil system. The different parameters shown in the figure are as explained below

m_s - is the effective mass of superstructure

m_f - is the effective mass of the foundation-soil system.

K_s - is the effective spring constant of superstructure for lateral movements

K_f - is the effectivd spring constant of foundation-soil system for lateral movements.

x_s and x_f are the absolute dynamic displacements of superstructure and foundation masses respectively, produced by superimposed ground displacement x_g at the particular instant of time.

In figure 3.1(c), the same superstructure system is shown to be idealised by a single degree mass-spring system, resting on a rigid- non flexible base.

In order to study the effect of various parameters on the interstoreyed shears, a term Response Factor (β) has been defined as in equation 3.1.

$$\text{Response factor } \beta = Z/Z_s \qquad \ldots(3.1)$$

Where Z represents the root mean square values of maximum interstorey modal displacements for the mass m_s in figure 3.1(b) and Z_s the maximum interstorey model displacement of mass m_s in figure 3.1(c).

The various parameters in the problem have been varied and thus their effect on the response factor β has been studied. The parameters are varied in ranges as explained below;

(i) Natural frequency of soil-foundation system (when considered alone) is varried between cps. and 12 cps.

(ii) Natural frequency of superstructure system alone (when comsidered to be fixed with rigid foundation) is varied between 2 cps. and 10 cps.

(iii) The ratio of effective masses of superstructures to substructure (defined as $\alpha = m_s/m_f$) is varied between 0.8 to 3.2.

The response factor β would depend on the nature of ground motion applied to the system. The velocity spectra (suggested by Housner[18]) has been assumed to indirectly represent the ground motion. The advantage of this spectral curve is that it can be represented by a simplified exponential function of time period T in the following manner,

$$S_v = A\,(1-e^{-2T}) \qquad \ldots(3.2)$$

Where S_v is the average spectral velocity of the system having time period T. A is the multiplying factor which takes into account the damping effect.

3.3 Analysis of the Model

The systems shown in figure 3.1(a) and (b) are assumed to be excited by the ground motion as explained earlier. System shown in figure 3.1(a) has two degrees of freedom for motion and therefore shall have two modes of vibrations. For the computations of the maximum interstorey shears (or displacements) for this system, the method of modal analysis has been applied and thus the displacements in two modes have been superimposed on each other. For the system shown in figure 3.1(c), which is a single degree freedom system, the maximum interstorey shear (or displacement) has been obtained directly from the spectral displacement equation.

The complete analysis for the two systems and thus the method of computation of response factor β is explained below.

For a two degree freedom system as shown in figure 3.1(a) are written as follows

$$m_f \ddot{x}_f + K_f x_f + K_s (x_f - x_s) = 0 \qquad \ldots (a)$$

$$m_s \ddot{x}_f + K_s (x_s - x_f) = 0 \qquad \ldots (b)$$

$$\ldots (3.3)$$

From the above free vibration equations, the modal frequencies are derived by obtaining the non-trival solution of equation 3.3, which is given by equation 3.4.

$$w^4 - \left\{\frac{K_f + K_s}{m_f} + \frac{K_s}{m_s}\right\} w^2 + \frac{K_s K_f}{m_s m_f} = 0 \qquad ...(3.4)$$

The equation 3.4 can be expressed as

$$w^4 - (F_f^2 + F_s^2 (1+\alpha)) \; w^2 - F_f^2 F_s^2 = 0 \qquad ...(3.6)$$

where $F_f^2 = \frac{K_f}{m_f}$ and $F_s^2 = \frac{K_s}{m_s}$

(F_f denotes the natural frequency of foundation system when superstructure is considered to be removed from it and F_s is the natural frequency of superstructure when it is considered to be founded on the perfectly rigid base) also α denotes the ratio of effective mass of superstructures to that of foundation.

$$\alpha = m_s/m_f$$

The frequencies in the two different modes are given by the square roots of non zero solutions of equation (3.6), as expressed below

$$w^2 = \frac{(F_f^2 + F_s^2 (1+\alpha)) \pm \sqrt{(F_f^2 + F_s^2 (1+\alpha))^2 - 4F_f^2 F_s^2}}{2}$$

...(3.7)

From the above equation normalised modal displacements for the masses m_s and m_f are computed by substituting this (eq. 3.7) in equation 3.8.

$$\Phi_s^{(1)} = \Phi_s^{(2)} = 1.0 \qquad \text{... (a)}$$

$$\Phi_f^{(1)} = \frac{F_s^2 \alpha}{F_f^2 + F_s^2\alpha - w_1^2} \qquad \text{... (b)}$$

$$\Phi_f^{(2)} = \frac{F_s^2 \alpha}{F_f^2 + F_s^2\alpha - w_2^2} \qquad \text{... (c)}$$

$$\text{... (3.8)}$$

Relative modal displacements for the two masses in different mode of vibrations are obtained by substituting equations 3.8 in equation 3.9, given below

$$Z_i^{(r)} = \Phi_i^{(r)} \frac{\sum_{i=f}^{s} m_i \Phi_i^{(r)}}{\sum_{i=f}^{s} m_i \Phi_i^{2(r)}} \, Sd^{(r)} \qquad \text{... 3.9}$$

where $Z_i^{(r)}$ is the relative displacement to base of ith mass in rth mode.

$S_d^{(r)}$ is the spectral displacement for rth mode obtained from the Housner's velocity spectral equation quoted earlier.

From the above equation interstorey modal displacements for the super structures mass are computed as given by equation 3.10, given below

$$Z^{r} = Z_{s}^{(r)} - Z_{f}^{(r)}$$

The resultant of the inter-storey displacements in two modes is assumed to be given by the root mean square value as shown in equation 3.11, which is

$$Z = \sqrt{(Z^{(1)})^{2} + (Z^{(2)})^{2}} \qquad \ldots(3.11)$$

In figure 3.1(c), which represents the super-structure mounted on the rigid base, the maximum inter-storey shear (or displacement) Z has been computed directly from the Housner's spectra.

The response factor β is therefore given by equation 3.13, as below

$$\beta = Z/Z_{s} \qquad \ldots(3.12)$$

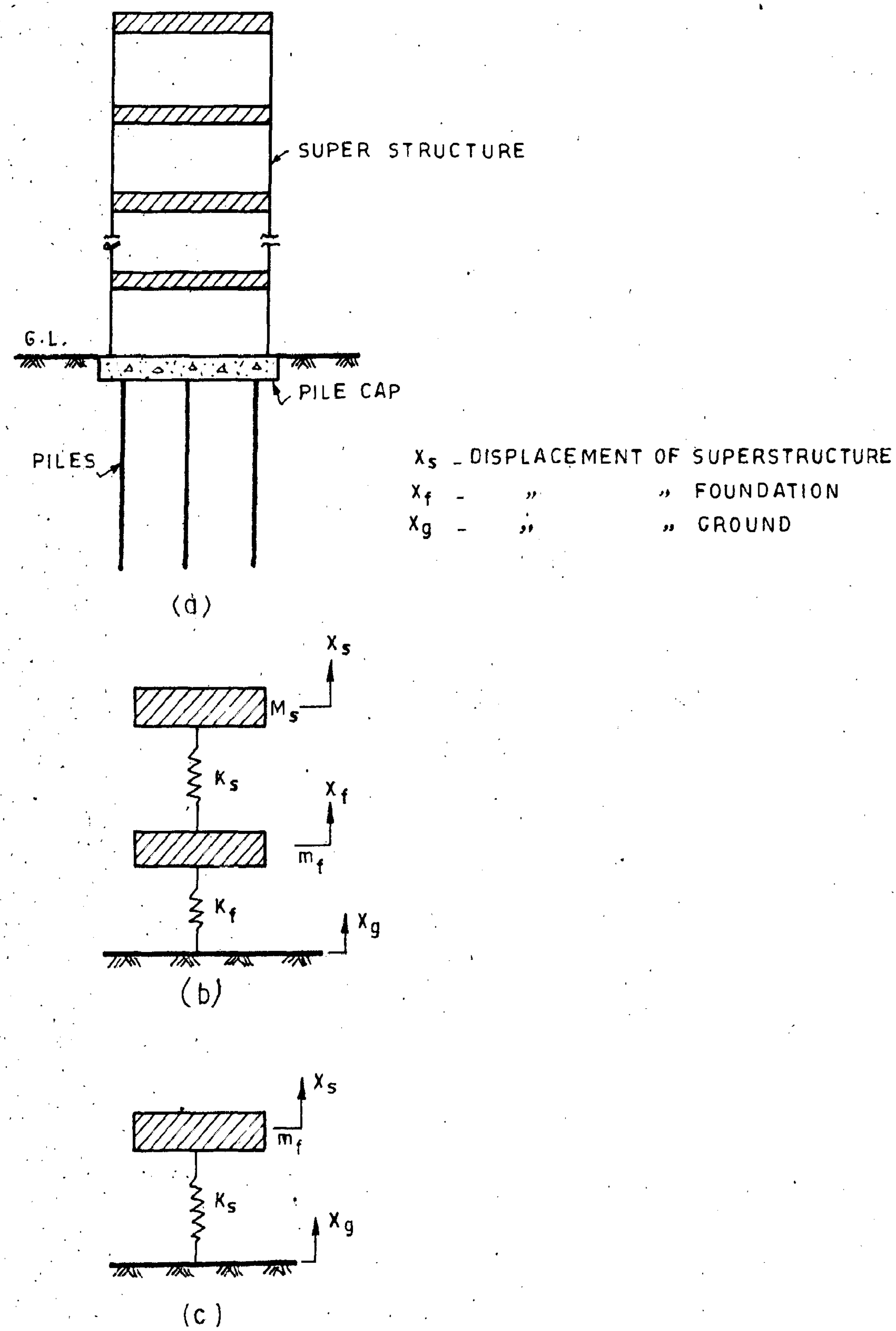


FIG. 3.1 - ASSUMED IDEALISED SYSTEM FOR THE STRUCTURE PILE SYSTEM

CHAPTER IV

THEORETICAL RESULTS

4.1' The effect of natural period of super-structure, natural period of foundation and mass ratio of super-structure to substructure on the response factor β, has been studied for a model structure as explained earlier, when it is subjected to ground motion. The results and their interpretation are presented here.

4.11 Effect of the natural foundation frequencies on the response factor

Figure 4.1 is a plot between natural frequency of foundation which is taken to vary from 2 cps to 12 cps and the response factor of the system for man ratio α equal to 0.8. Each curve in the figure represents the variation of response factor β for a particular natural period of the super-structure. Five curves have been drawn for the natural period of the super-structure varying from 0.1 sec to 0.5 sec.

These curves show that the response factor β initially increases as the natural frequency of foundation increases and then attains maximum value at some particular foundation frequency, and after it, it starts dimnishing. The natural frequency of foundation at which β attains maximum value is different for each natural period of super-structure and also the maximum value of β is different in each of the above mentioned case.

As the natural period of super-structure increases the maximum value of β decreases. The table shown below is reproduced from the figure 4.1, which illustrates how the natural period of super-structure, natural frequency of foundation and the maximum value of response factor are inter-related quantitatively.

Natural period of super-structure in seconds	Frequency at which optimum value of β occurs (in cps)	Optimum value of β
0.2	7.5	1.221
0.3	5.5	1.220
0.4	4.0	1.188
0.5	3.75	1.108

Similar plots are drawn from figure 4.2 to 4.5 for the mass ratio 1.10, 1.40, 1.70, 2.0. The nature of these curves is also in exact similarity to the curves in figure 4.1.

It is observed from these curves that with the decrease of the natural period of super-structure, the foundation frequency at which the maximum value of β occurs ~~de~~increases i.e. maximum response of the structure occurs in the lower ranges of the natural frequencies of the foundation system. In each case it can be noted that the natural frequency at which maximum response of super-structure occurs are comparable with the natural frequency of substructure.

In the lower ranges of mass ratio the curves exhibit a sharper increase or decrease in the response factor β. Smaller value of α means higher overall effective mass of the foundation participating in the vibrations. It indicates that the role of foundation in the dynamic case becomes significant with the increased foundation mass.

It may be observed from the curves, that for a structure foundation system, having the natural frequency very less than that of super-structure, if the natural frequency of foundation is increased, the response factor increases. However in case when the base frequency is very high as compared to that of super-structure an increase in foundation frequency shall cause a decrease in the response factor.

Hence it may be concluded from above that, when the foundation is very flexible (small natural frequency than super-structure) the stiffening of the foundation, in general shall cause an increase in the inter-storey shears in the super-structure. However for a system having foundation to be very rigid than the super-structure, the stiffening of the foundation shall decrease the inter-storey shears in the super-structure.

4.12 Effect of the Ratio of the Frequencies of Super-structure to that of Foundation on the response factor β

Curves shown in figure 4.6 illustrates the variation of response factor β with the ratio of natural frequencies of super-structure to sub-structure. Each curve is for different mass ratio α, i.e. for α equal to 0.8, 1.1, 1.4, 2.0 and 3.2. The natural period of super-structure has been kept as 0.3 sec. Similarly the curves in figure 4.7 are drawn for the natural period of super-structure as 0.4 seconds.

These curves also exhibit a well defined peak, similar to as in β frequency curves i.e. in each case maximum response of the structure occurs at a particular frequency ratio of super-structure to sub-structure.

The curves become steeper in the lower frequency and mass ratios. In the same frequency ratio ranges, curves tends to flatten for higher values of α. Therefore an intersection is obtained between the curves for different mass ratios.

It was found by Gupta Y.P.[20] that with the increase of mass ratio α, response factor β decreases. However present studies reveal that with the increase in α, a decrease in β is observed, only in the ranges of higher frequency ratios i.e. in the case when base is very flexible as compared to the super-structure. However in the range of lower frequency ratio, β increases with the increase α.

4.2 COMPARISON OF THE RESULTS WITH SIMILAR TYPE OF AVAILABLE WORK

A rigorous analysis of two degree freedom system has been performed by Gupta Y.P.[20] in the limited range of the periods of parent and vibration absorber systems. The analysis was carried out primarily to observe the effect of mounting parent system on the absorber. A single degree linearly elastic freedom system was considered to be mounted on a similar type of vibration absorber. Viscous type of damping was considered in both the systems. A modified fourth order Runge-Kutta's procedure was used for numerical solution of the equations. Shear response of the parent system was studied for the following two digitalised earthquake datas.

(a) El Centro, May 18, 1940, North South component

(b) Traft, July 21, 1953, S21W component.

Response factor β was defined as in the present studies. β was defined as the maximum shear in parent system when mounted on the absorber to the shear when without absorber.

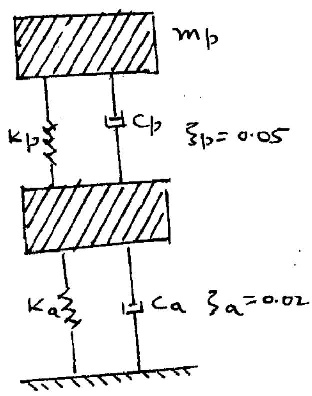


Fig-4.8

$\xi_p = 0.02$

Where suffix a is for absorber and p is for parent system. mass ratio α was varied from 4 to 20.

In figure 4.9 curve A illustrates the variation of β with $1/\alpha$ for ElCentro ground motion, while curve B is for traft earthquake.

In the present analysis, the damping has been considered same in the two modes of vibrations. Thus to compare the results from the presented theory, the system of similar characteristics as analysed by

Gupta, Y.P.[10] is considered and the damping is separated for the two modes by assuming the damping matrix of the coupled system as linear combination of the mass and spring coefficient matrix[21] as explained below.

In using this procedure, one implicitly assumes that the model columns for the undamped systems remain valid for the damped systems as well. This assumption leads to completely uncoupled equation for this case.

Expressing damping matrix as the linear combination of mass and spring matrix.

$$(B) = \alpha \ (A) + r \ (C)$$

Where (B) is the damping matrix

(A) is the mass matrix

(C) is the stiffness matrix

α is constant of dimension T^{-1} and r is a constant of dimension T.

Substituting this equation to the equation of motion, the dampings in the different modes are obtained as

$$\beta_k = \frac{\alpha + r \, w_k^2}{2 \, w_k}$$

Where β_k is the damping present to the critical damping in k^{th} mode and w_k be the modal frequency in k^{th} mode.

It is obvious from the study of figure 4.9 that the results obtained from the spectral analysis presented are comparable to that obtained from the rigorous analysis.

A significant conclusion is drawn from this comparison is that through the shear response factor β is effected by the characteristics of the dynamic load, but not significantly. The results obtained for the three different loading characteristics are comparable, for the practical purposes.

Figure (4.9) also varify the applicability of the presented theory for the analysis where, such systems are involved.

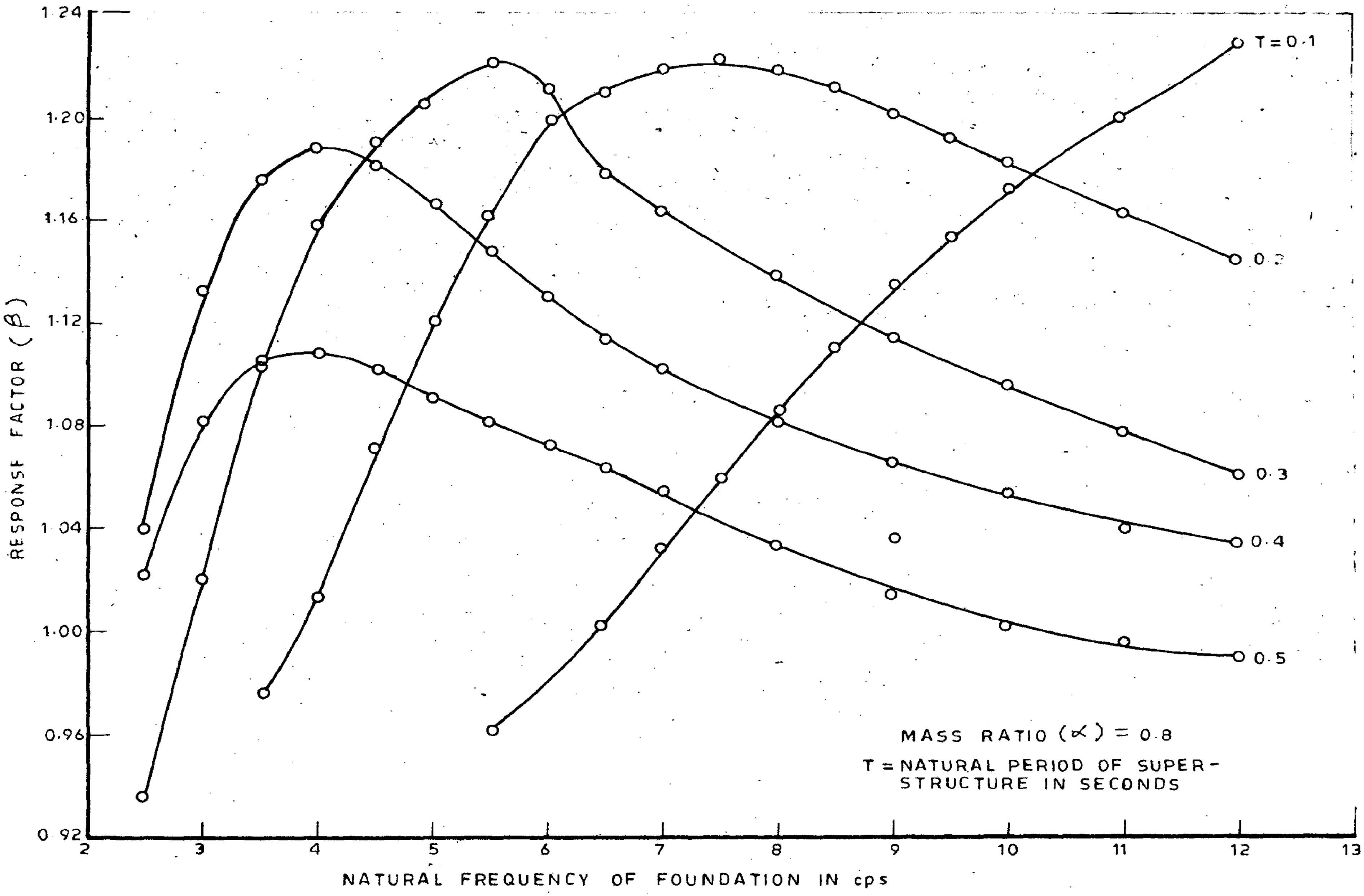


FIG. 4.1 _ NATURAL FREQUENCY OF FOUNDATION VS RESPONSE FACTOR

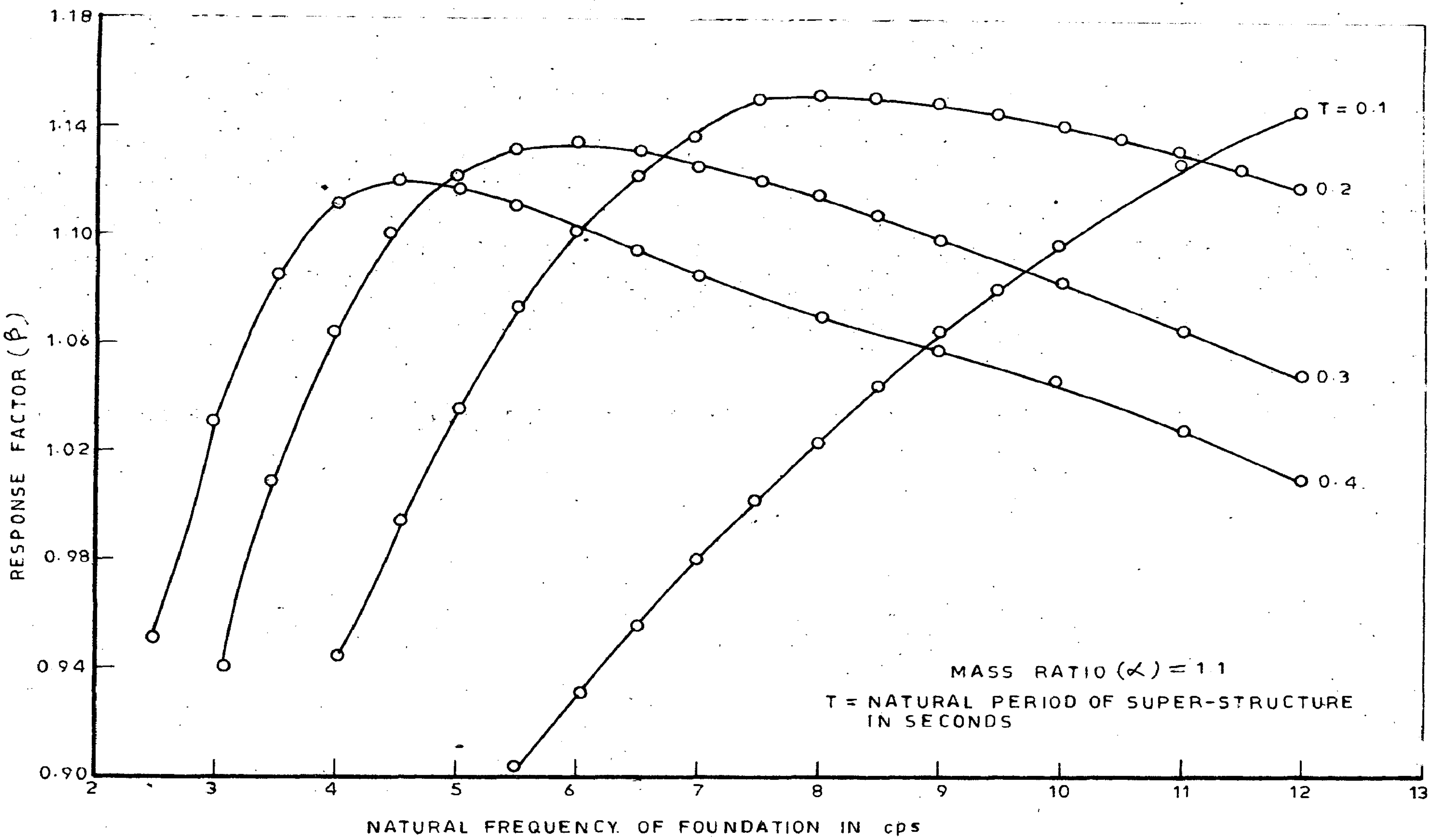


FIG. 4.2 _ NATURAL FREQUENCY OF FOUNDATION VS RESPONSE FACTOR

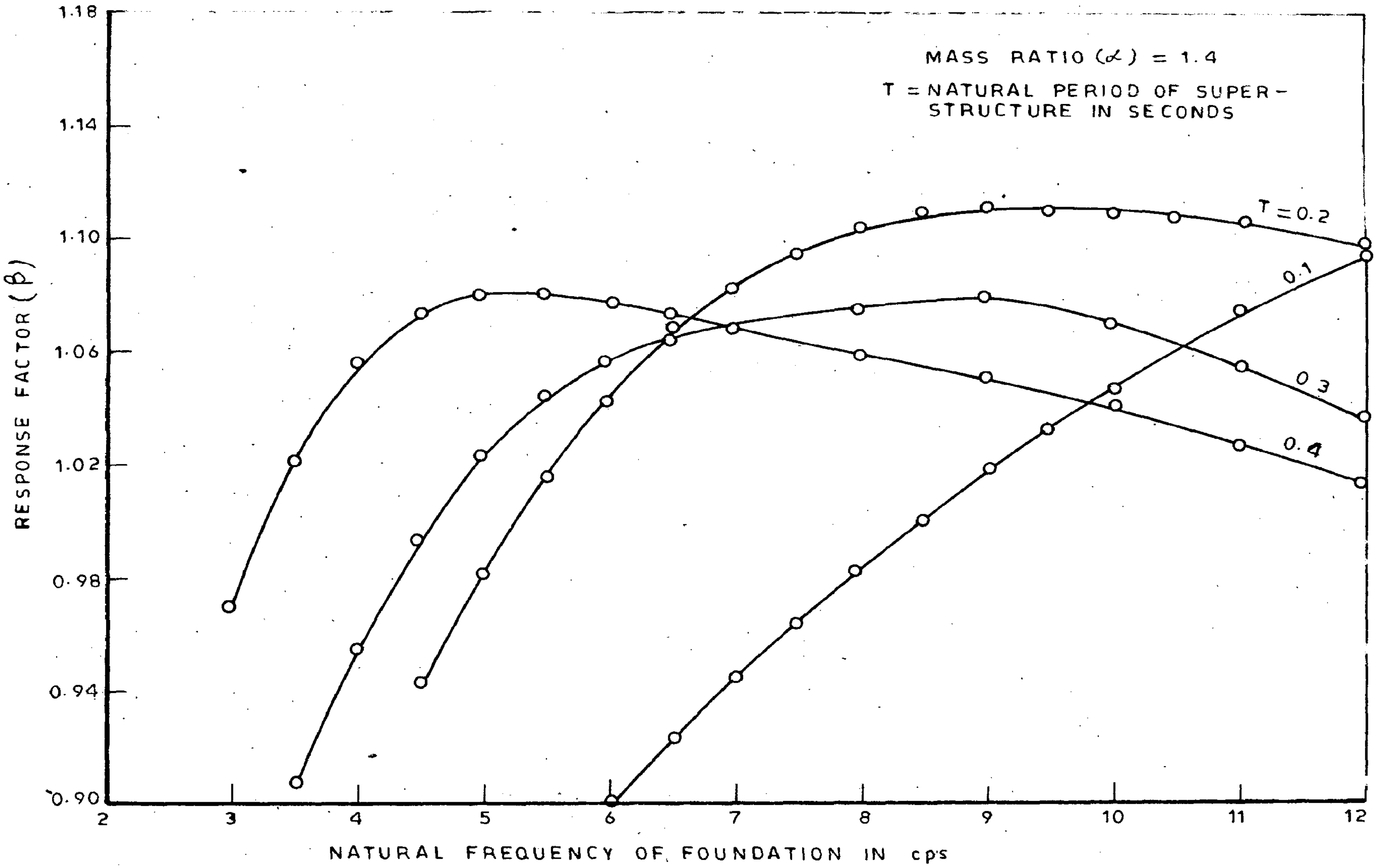


FIG. 4.3_NATURAL FREQUENCY OF FOUNDATION VS RESPONSE FACTOR

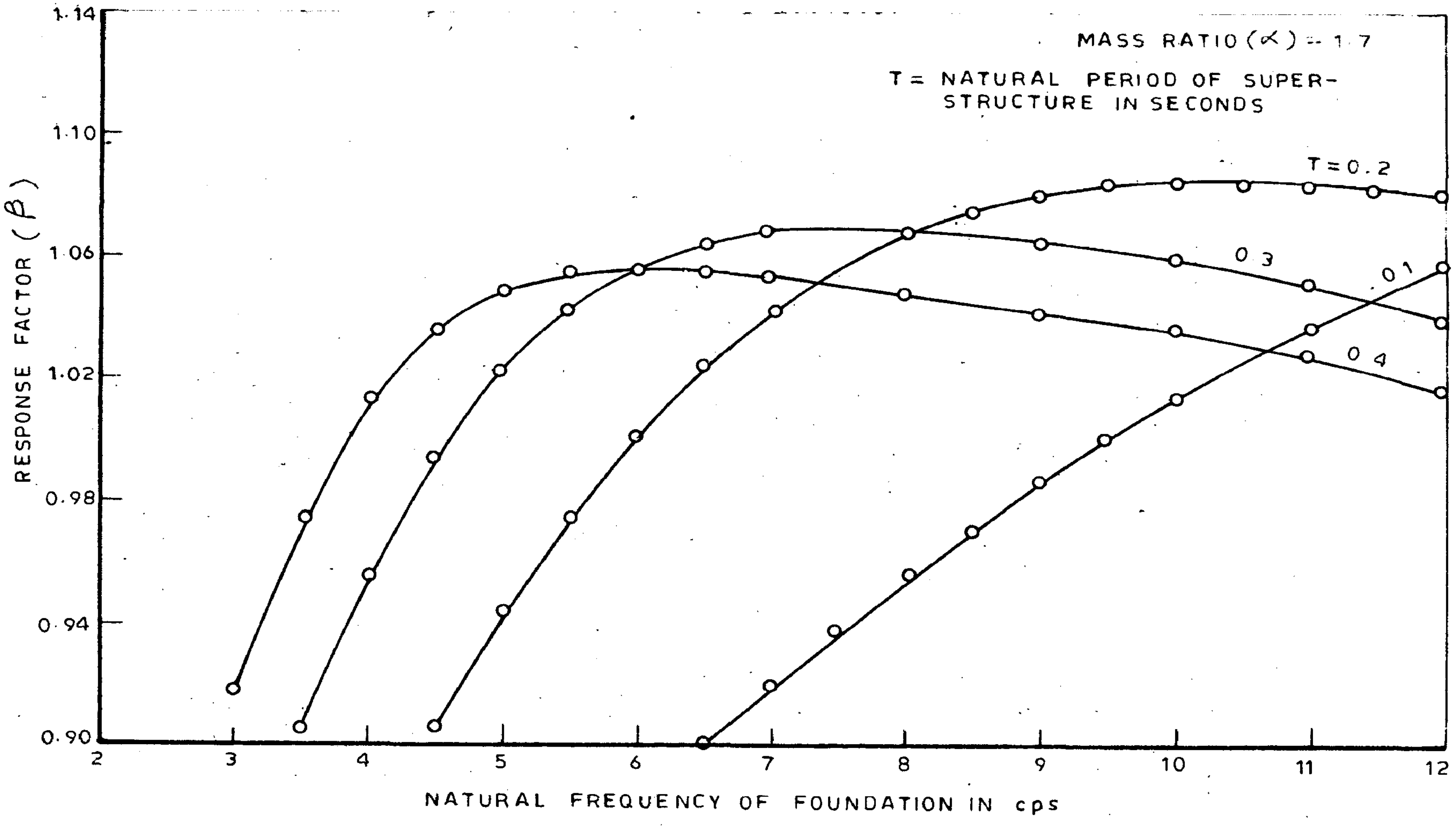


FIG.4.4_NATURAL FREQUENCY OF FOUNDATION VS RESPONSE FACTOR

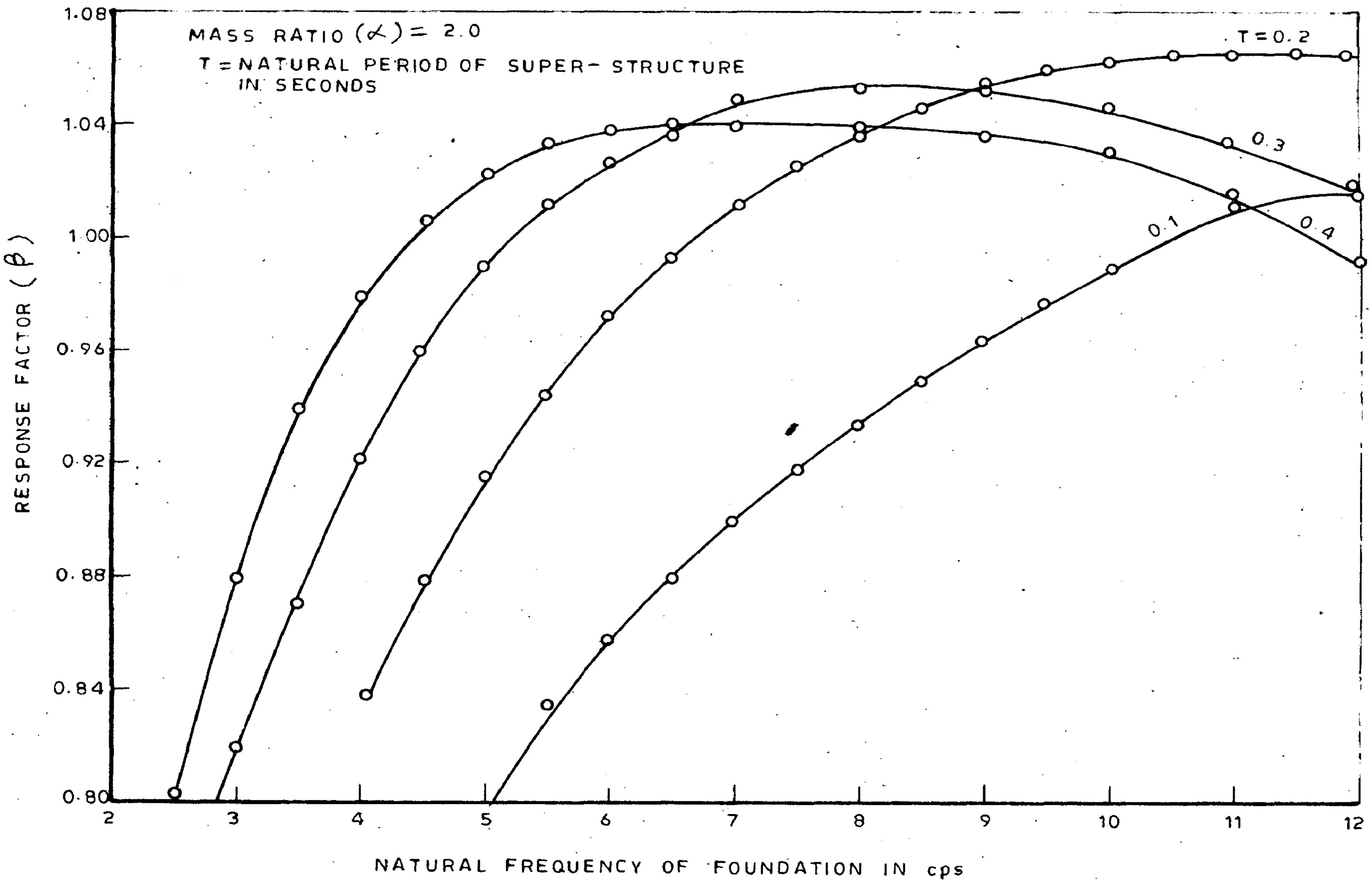


FIG. 4.5_NATURAL FREQUENCY OF FOUNDATION VS RESPONSE FACTOR

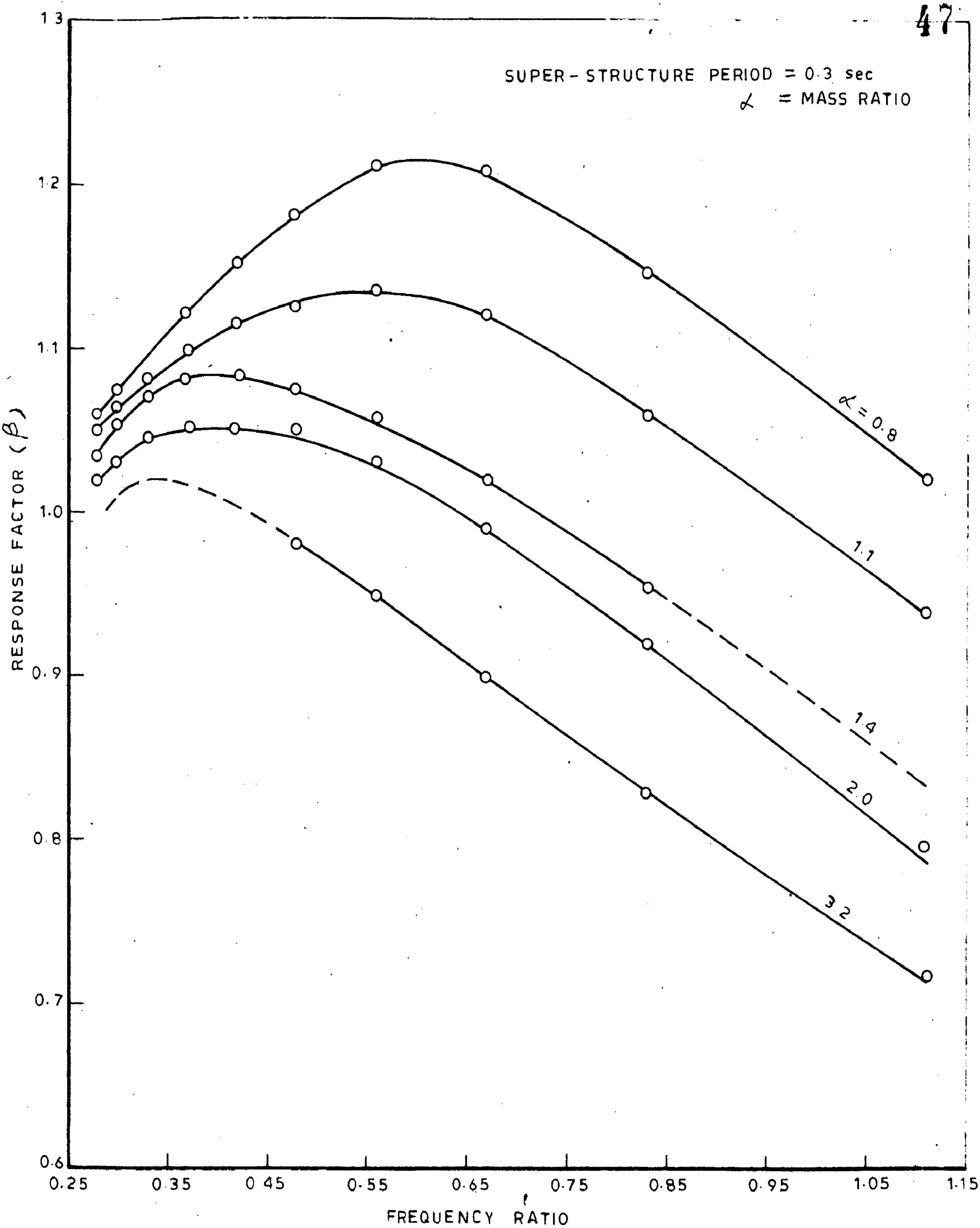


FIG.4.6_NATURAL FREQUENCY RATIO VS RESPONSE FACTOR

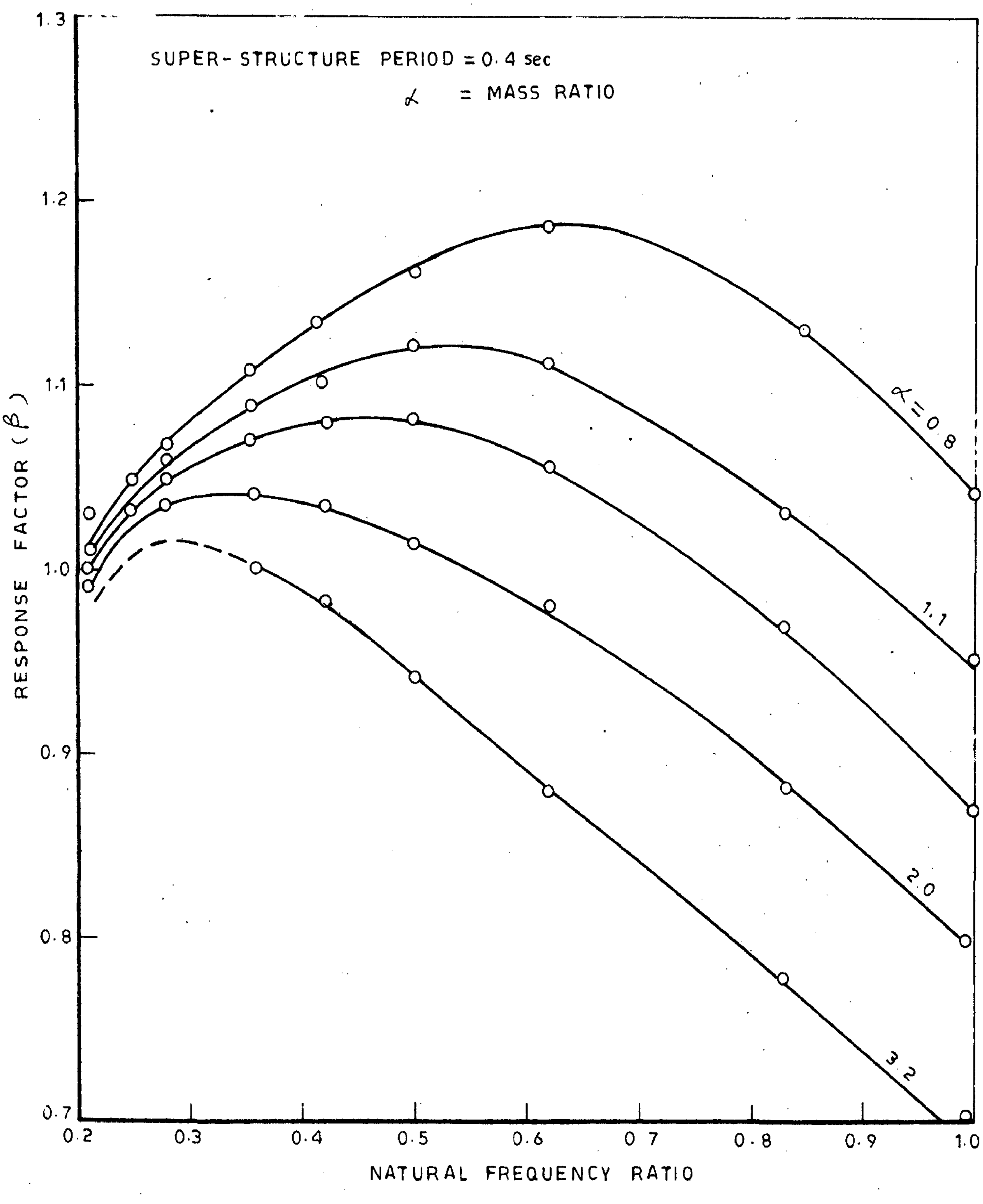


FIG. 4.7_ NATURAL FREQUENCY RATIO VS RESPONSE FACTOR

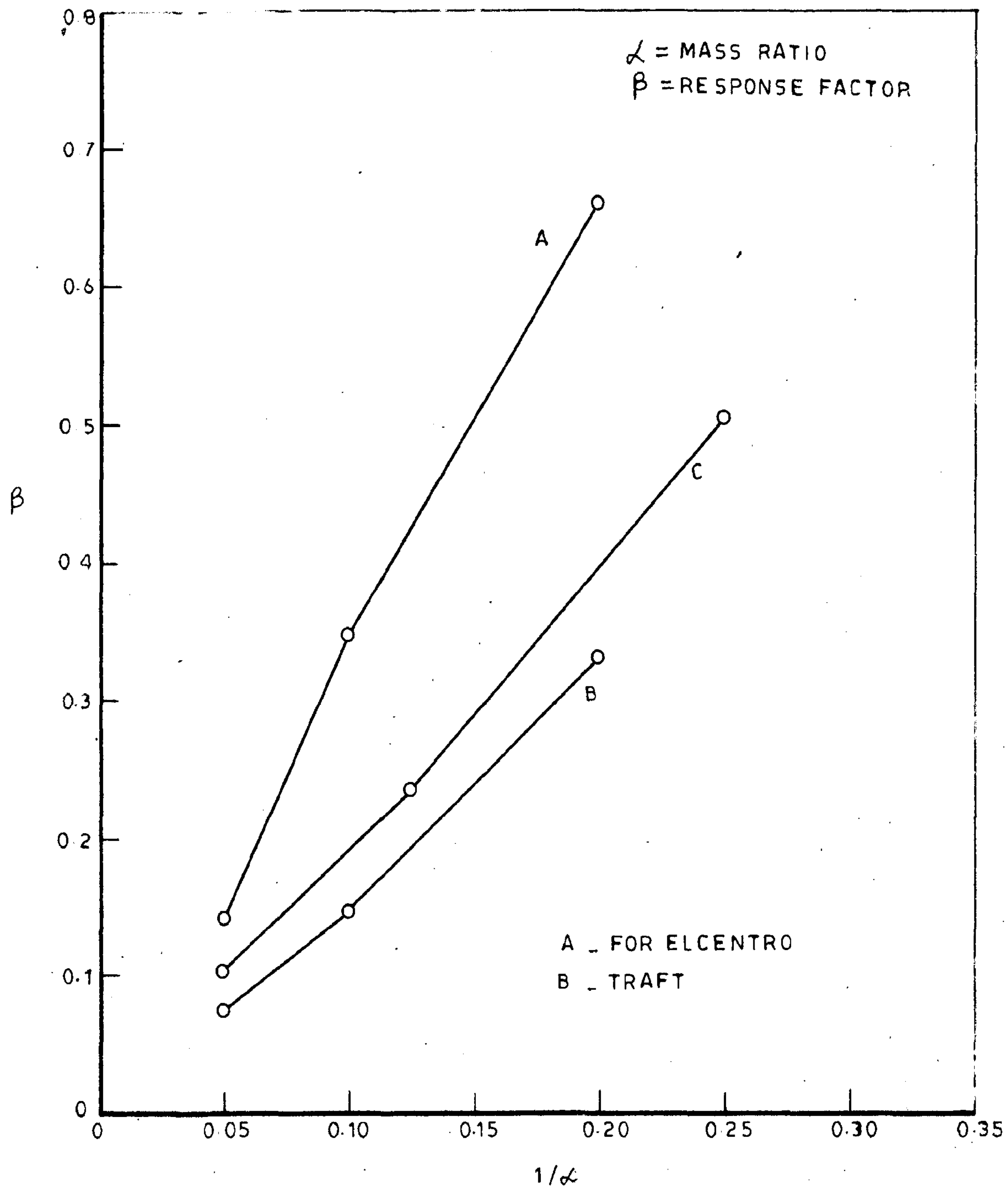


FIG. 4.9_ 1/α VS β CURVES

CHAPTER V

5.1 EXPERIMENTAL STUDIES

Experimental studies have been conducted to verify the theoretical approach developed in the previous chapters. A model was fabricated consisting of an aluminium pile supporting a mass spring system representing a model of the single degree superstructure system. The pile was embedded in a sand filled tank which was mounted on a shaking table. The response of the model structure was studied experimentally by giving horizontal vibrations to the table. Various constants for the system have been evaluated experimentally and then the system is analysed by a theoretical approach.

A detailed description of the experimental part of the work is presented in this chapter.

5.2 MODEL

The model consisted of a mass spring system mounted on a flexible aluminium pile. A sketch of the model fabricated is shown in figure 5.1. Figure 5.1(a) shows a cross section of the model which is shown to be embedded in the sand, which is filled in a tank. A wooden pile cap of size 2.5 x 6 x 1 cms is fixed on the treaded pile head with the help of two nuts. Two flexible steel strips are then screwed to the pile cap which supports a cast iron block as shown in figure. This steel block has a arrangement for varying the mass as shown in figure 5.1 (b). Specifications for the different elements of the model are given below.

Aluminium pile is 26" (66 cms) with following characteristics.

(a)	Outer diameter	15 mm
(b)	Inner diameter	10 mm
(c)	Modulus of elasticity	0.714×10^{6} kg/cm^2
(d)	Moment of inertia	180×10^{-3} cm^4
(e)	Stiffness (EI)	13.7×10^{4} kg/cm^2

The structure consists of a weight of cast iron connected to pile cap by two flexible steel strips. The weight of cast iron block is 0.480 kg.

5.3 TEST SET UP

As explained earlier the pile carrying the super-structure was embedded in the sand, which was filled in the tank. The tank was mounted on a shaking table fitted with an oscillator to give it horizontal vibration. The details of the various components of the test setup are given below

5.31 Steady State Horizontal Shaking Table

The vibration table is able to move horizontally giving sinusoidally varying oscillations. Table has platform of size 6.4'x4'. Steady state vibrations were given to table by Lazen type oscillator, which was centrally mounted on one of its edge. The oscillator was driven by a 3 H.P., D.C. motor. Speed of the motor was controlled by an independent speed control unit. This separate unit consists of rectifier and a D.C. Transformer i.e. potential divider, to change D.C. motor input power. Rectifier converts A.C. supply to direct current.

The oscillator motor assembly is capable of developing frequencies from 0 to 25 cps, and the amplitude upto 2 mm. Oscillator can develop different sinusoidally varying forces, with different eccentricity settings.

5.32 Vibration Recording Devices

Vibrations were recorded by mounting Miller acceleration pick ups to the vibrating object in the plane of vibrations. Varying signal response of the system was fed to the oscillograph, through universal amplifiers.

5.4 SAND USED

The sand used in the experiment was Ranipur sand. The grain size distribution of sand is shown in figure 5.2. This belongs to SP Group according to Indian Standard Specification.

The other properties of the sand are as follows.

(a) Specific gravity of sand grains	- 2.60
(b) Uniformity coefficient	- 2.28
(c) Grain size D_{10}	- 0.14
(d) Minimum void ratio e_{min}	- 0.54
(e) Maximum void ratio e_{max}	- 0.89

5.5 Experimental Response of the Model Under Steady State Vibrations

As explained earlier, the model was given steady state vibration, by mounting it on the shaking table. In order to study the affect of flexibility of foundation on the response of the structure, two type of tests were conducted. In one test the complete coupled system was given steady state vibrations by embedding the pile in the sand and in the other one the superstructure was removed from the pile and was mounted on the shaking table, giving it again the steady state vibrations. The two tests are explained below.

5.51 Experiments with the Coupled Pile-structure System

The model was placed vertically in the tank by clamping the pile head by a suitable arrangement mounted at the top of the tank. Sand was filled in the tank leaving 2" of free length of the pile above the surface of the sand.

One of the important requirements of the test was the control of the density of the sand. Therefore a similar method for the compaction was adopted in every test so as to get the same density of sand. It was observed that during vibrations when sand particles starts moving, maximum possible density on this table was obtained after stopping the vibrations. After placing the model in the empty tank, sand was poured in it in 6" layers. Each layer was compacted by the vibrations of the table. Speed of the motor was so adjusted, so as to get the resonence in the sand particles, and thus the maximum density was obtained. Density was obtained by dividing the weight of the sand by the volume measured.

After filling the tank upto 2" below the pile head, the clamp was removed. Three Miller acceleration pickups were mounted one each on the super structure mass, pile cap and on the shaking table.

In the first experimental set up the eccentricity of the oscillator was kept 35^{o}.

Speed of the motor (and hence the frequency of the oscillation table) was slowly increased by giving one round to the speed control whel each time. Records from the three acceleration pickups were recorded by the automatic pen recorders, connected to the pickup by amplifier channels.

Tests were repeated for the oscillator eccentricity settings of 70°, 105° and 140°.

Analysis of the records

The records obtained from the steady state vibration tests were analysed to obtain the response of the structure mass for the different forcing frequencies. The method of analysis is explained below.

A typical acceleration record obtained from the test is shown in the figure 5.3. The variation of acceleration has been recorded on a paper moving with a speed of 25 mm per second. In order to compute the displacements from this record, variation of the acceleration is assumed to be sinusoidal; and hence, represented by the following equation,

$$\ddot{x} = a \, \mathrm{Sin}\,(wt)$$

where $\ddot{x}$ is the second differential of displacement x at any instant t, with respect to time t, a is the maximum acceleration in a cycle and w be the frequency in radians per second.

Displacement x is obtained by intrigating acceleration twice with respect to time t, i.e.

$$x = -(a/w^2)\ \mathrm{Sin}\,(wt)$$

Thus the amplitude of oscillation is a/w^2

Response of the Structure with Different Forcing Frequencies

The computed results from the records of the above tests are shown in figures 5.4, 5.5, 5.6 and 5.7.

In figure 5.4, forcing frequencies are shown on the abscissa line while curve B is for the amplitudes of the structure mass, curve C, for amplitudes of table..

Curve A shows a definite peak at frequency 15.5 cycles per second. This is the resonence state of vibration, where the mass vibrates with the maximum amplitude.

Similarly in figures 5.5, 5.6, and 5.7, curves are drawn for the amplitudes of structure mass and shaking table for eccentricities of 70°, 105° and 140°.

A detailed discussion of the above curves shall be presented later.

5.52 Experiments with the Super-structure Mounted on the Shaking Tables

The superstructure system was removed from the pile along with the pile cap and was mounted on the shaking table by glueing the pile cap to table with Areldite Solution. Miller type acceleration pick-ups were fixed one to the mass of the model and one to the shaking ta table.

Speed of the motor was slowly increased by givimg one round to the speed control wheel each time, similar as in the earlier test with the pile-super structure model.

Tests were repeated for the eccentricity setting of oscillator at 70°, 105° and 140°.

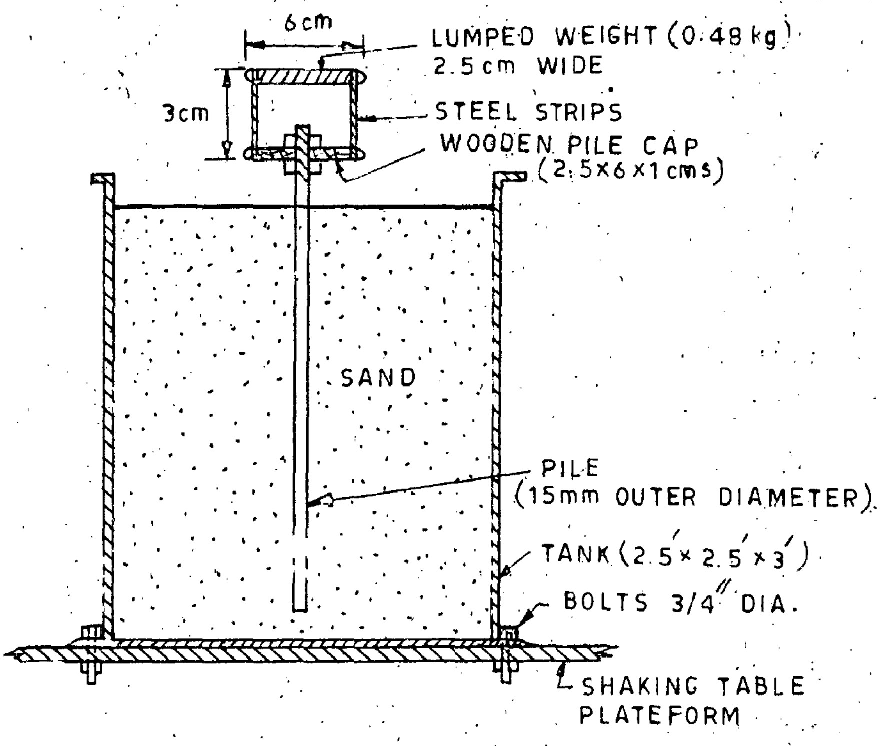
Records obtained from the tests are the steady state vibration records and thus the amplitude is computed in a similar way as described 5.51.

Curve A in figure 5.4 is the plot between the forcing frequency and the amplitude of the structure mass. Similar to the curve B in the figure, this has also a rising limb and a descending limb. Maximum amplitude of the mass is observed at the forcing frequency of 16 cycles per second. Similar curves are drawn for the eccentricity setting of 70°, 105° and 140° shown in figure 5.5, 5.6 and 5.7 respectively.

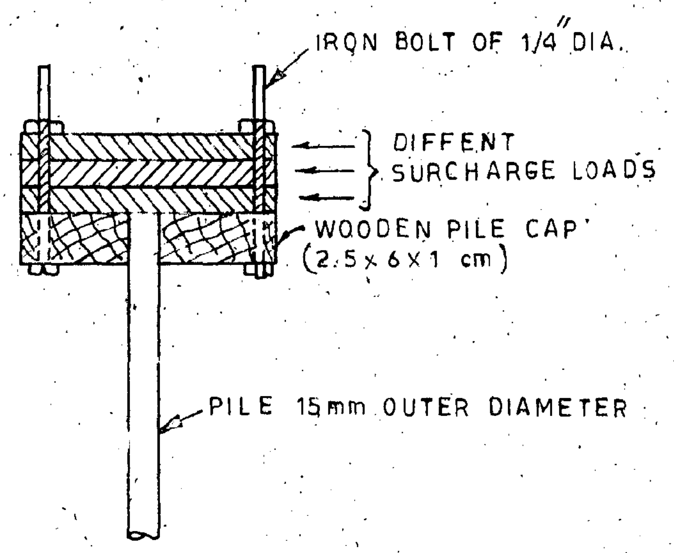
5.53 Experimental Results

As illustrated earlier, the experimentally recorded responses of the structure pile system with pile embedded in the sand and the structure alone mounted on the shaking table are shown in figures 5.4 to 5.7.

A study of these curves reveals that in general the absolute displacements of the superstructure mass when mounted on the pile are larger than the displacements when former has rigid base. A well established peak is the characteristic feature of the curves A and B. This peak represents the resonance stage of the systems. Resonance occurs in the coupled foundation structure system at an average frequency of 15.5 cycles per second, which represents the natural frequency of the system. Resonance in the super structure occurs at an average frequency of 16.02 cycles per second, which is the natural frequency of vibrations of the system.



a - SKETCH SHOWING THE MODEL FABRICATED



b - SECTION SHOWING THE ARRANGEMENT FOR LOADING DIFFERENT SURCHARGE LOADS

FIG. 5.1

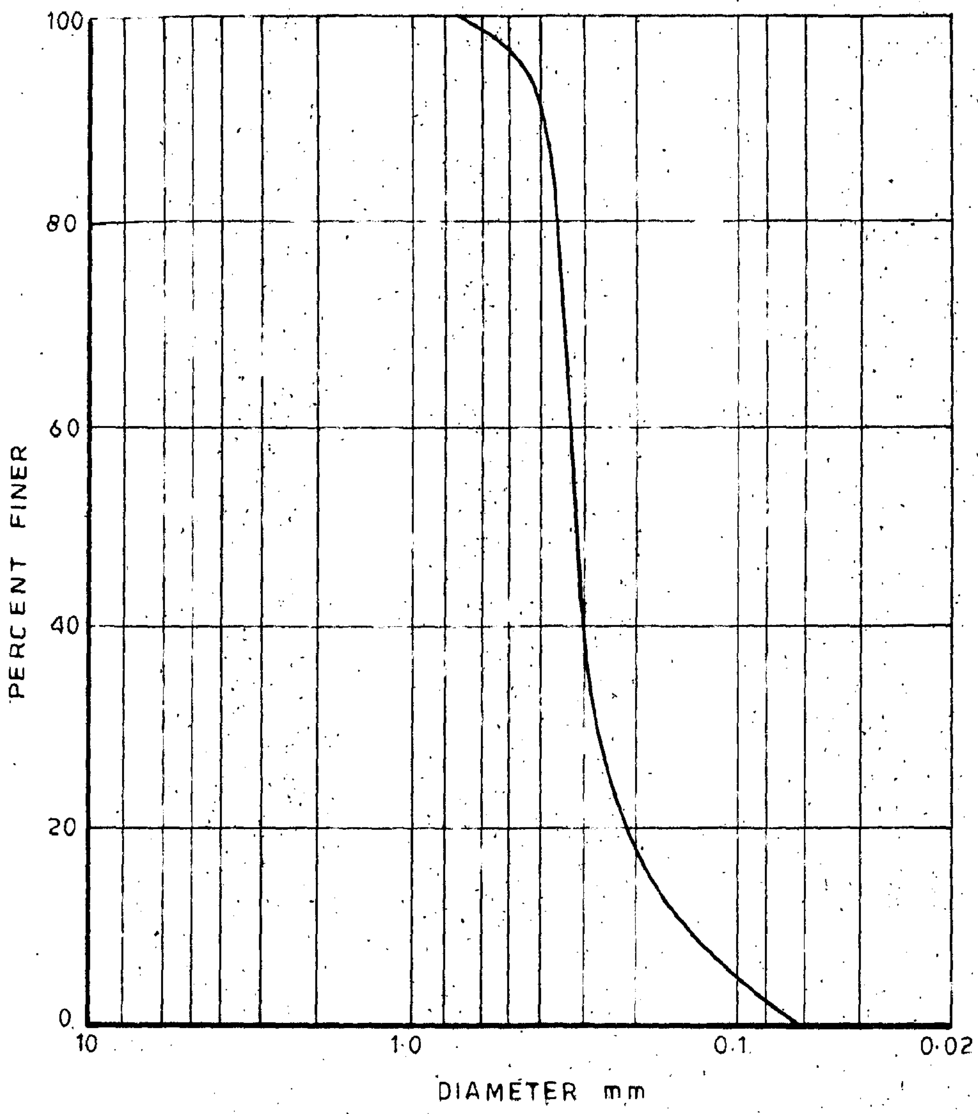


FIG. 5.2 _ GRAIN SIZE DISTRIBUTION OF SAND

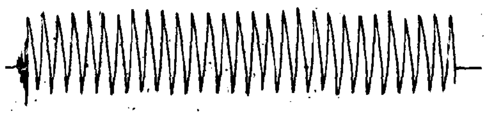


FIG. 5.3 _ A TYPICAL STEADY SLATE VIBRATION RECORD

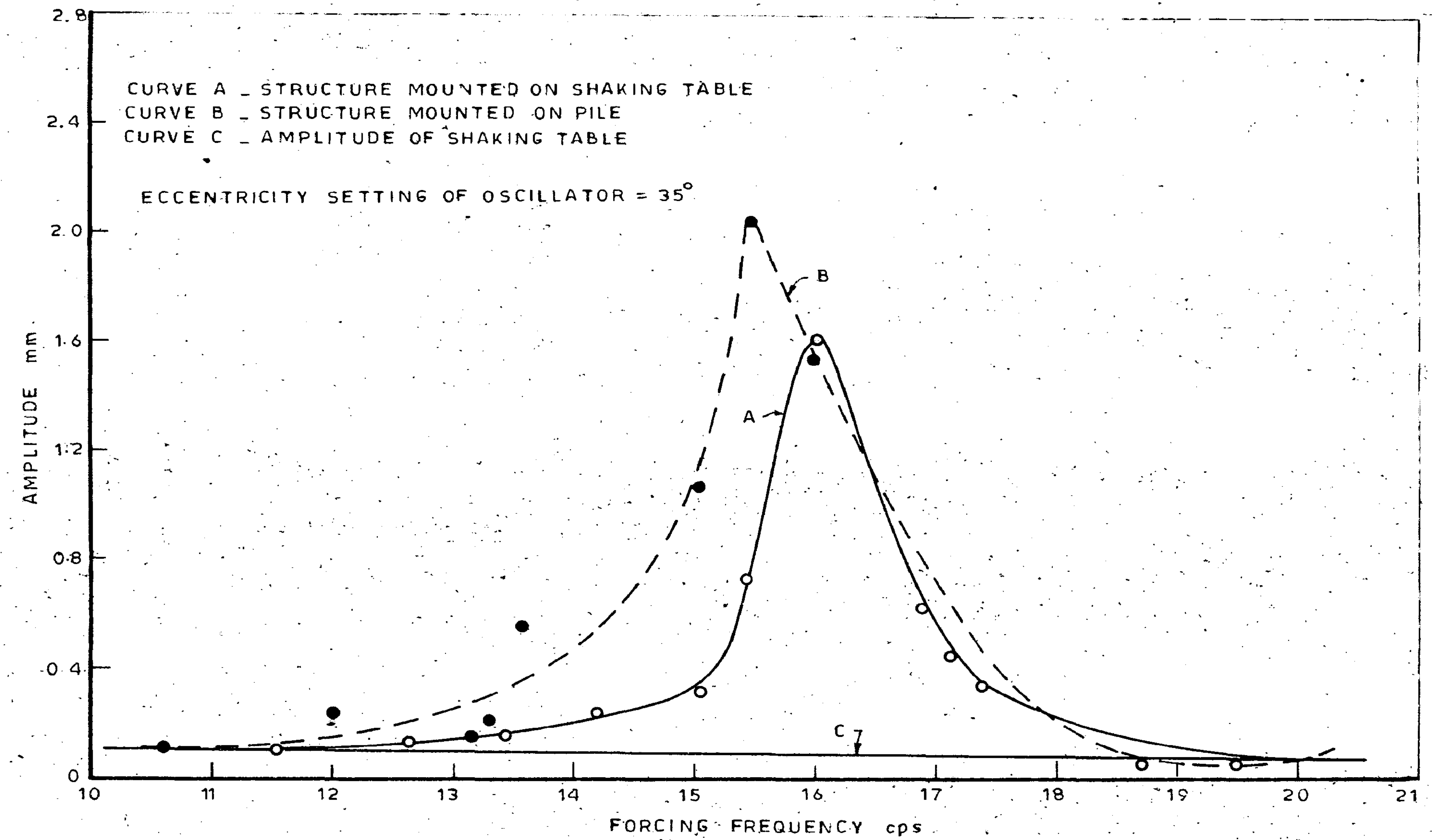


FIG. 5.4_ FORCING FREQUENCY VS AMPLITUDE CURVES

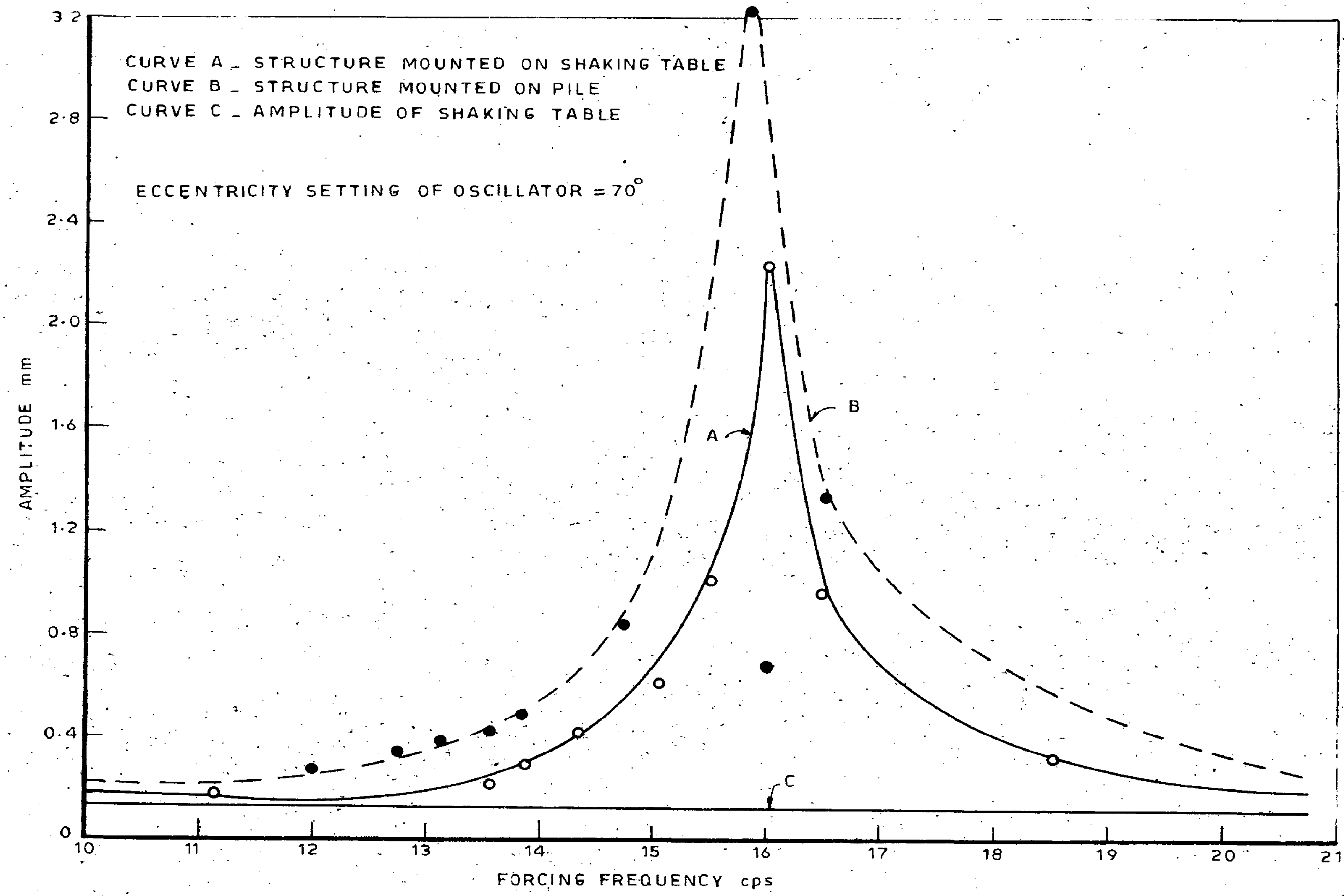


FIG. 5.5 _ FORCING FREQUENCY VS AMPLITUDE CURVES

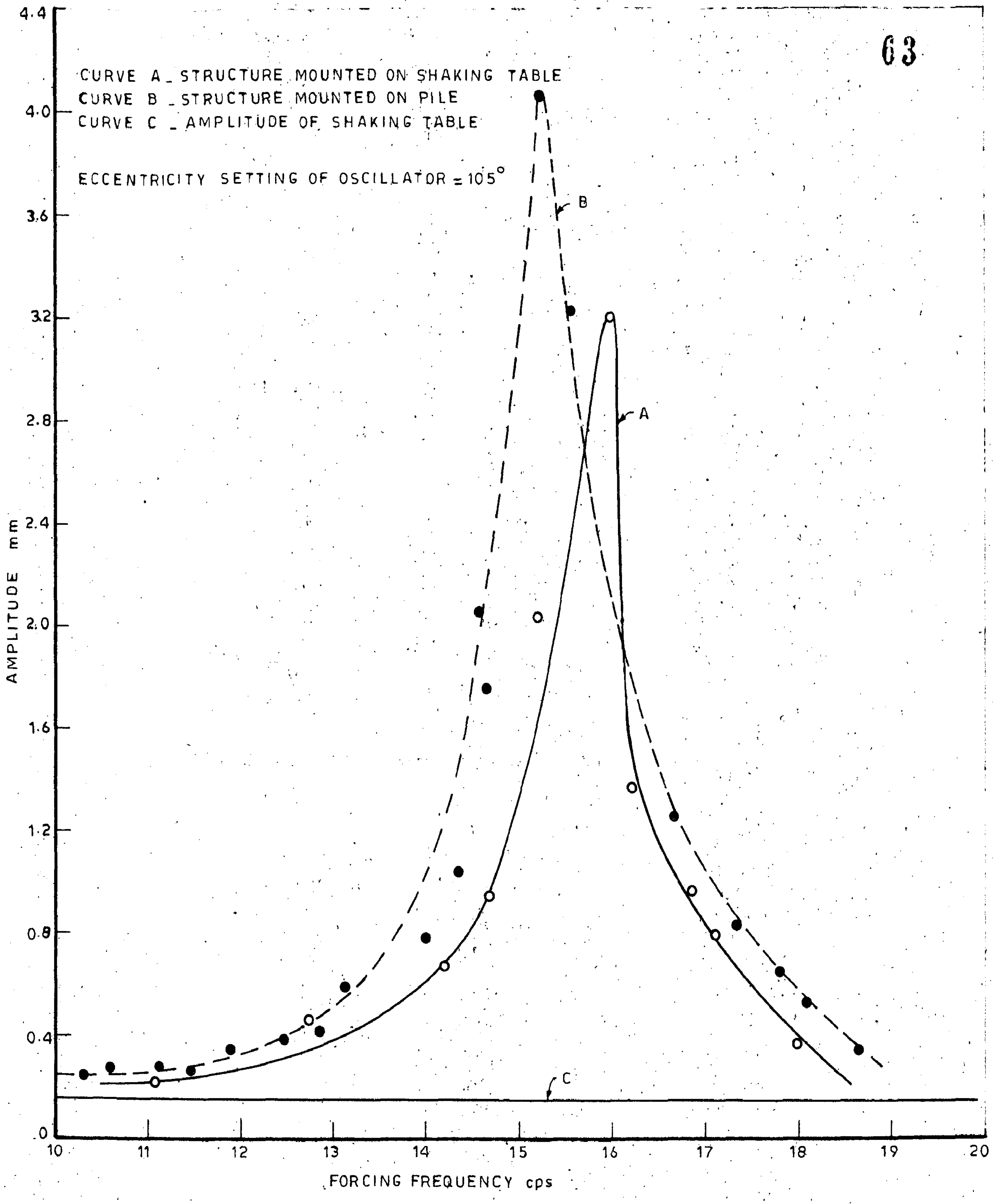


FIG. 5.6_ FORCING FREQUENCY VS AMPLITUDE CURVES

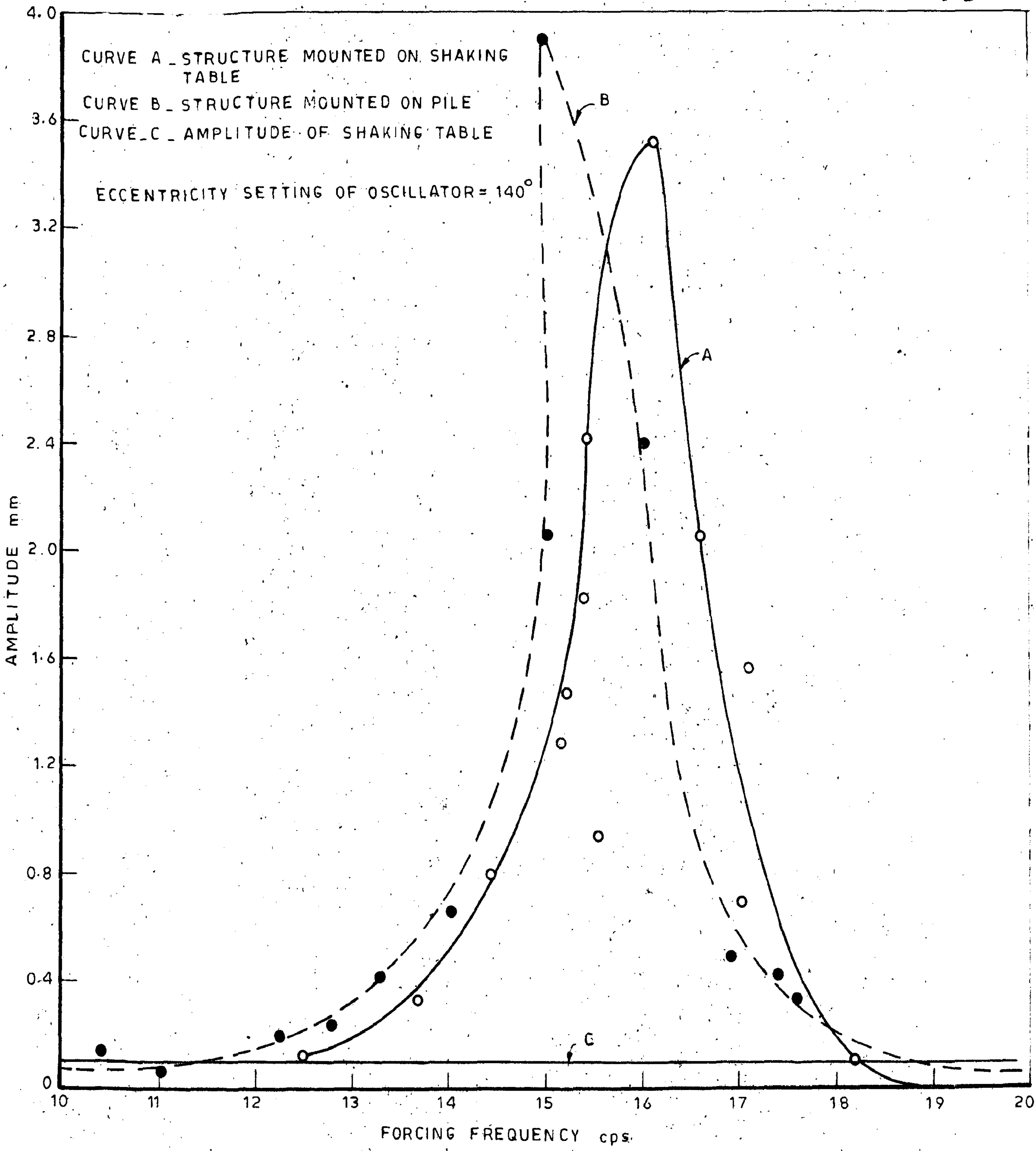


FIG. 5.7 _ FORCING FREQUENCY VS AMPLITUDE CURVES

CHAPTER VI

THEORETICAL ANALYSIS OF EXPERIMENTAL MODEL OF STRUCTURE PILE SYSTEM

6.1 General

The experimental model described in previous chapter and shown in figure 5.1 is analysed theoretically and thus the results obtained are compared with the actually observed response of the system on the shaking table. A detailed description of the theoretical approach and the experiments conducted is presented in this chapter.

6.2 Theoretical Analysis of the model

As explained in Chapter III, the analysis of the structure pile model system is done by assuming it to be reresented by a two degrees of freedom system. The super structure is a mass spring dashpot system of single degree of freedom. The soil pile system which has infinite degrees of freedom is converted into a single degree system by evaluating the various constants experimentally.

The theoretical approach is similar to as described in Chapter III except that in this case the damping effect has also been considered. Figure 6.1(a) shows idealised single degree mass-spring-dashpot. System for the soil -pile system.

In the figure, W_f is the net effective weight of the soil-pile system taking active part in the vibration. It includes the virtual mass of the soil, vibrating with the foundation.

K_f represents the net effective stiffness of the foundation for the lateral loads.

C_f represents the damping present in the foundation system.

Figure 6.1(b) is a idealised two degrees of freedom system for the structure-pile model. Substructure and the superstructure units are marked in the figure.

In the figure,

W_s represents the weight of the cast iron block of the superstructure.

K_s is the net stiffness of superstructure for the lateral loads.

C_s is the damping present in the system.

The motion of the shaking table was found to be sinusoidal to a fair degree of accuracy. It is therefore assumed that the system shown infigure 6.1(b) is excited by a sinusoidally varying motion at the base.

Assuming that;

X_f and X_s are the absolute displacements at any instant t, of the foundation and superstructure mass respectively y is the sinusoidally varying base motion and expressed as $y = y_o \sin wt$, y_o be the amplitude of motion and w be the radial frequency of vibrations.

The equations of motion for the system shown in figure 6.1 (b) are written as follows;

$$m_s \ddot{x}_s + c_s \dot{x}_s + k_s x_s - c_s \dot{x}_f - k_s x_f = 0 \qquad \ldots (a)$$

$$m_f \ddot{x}_f + (c_f + c_s) \dot{x}_f + (k_f + k_s) x_f - c_s \dot{x}_s - k_s x_s$$

$$= c_f \dot{y} + k_f y \qquad \ldots (b)$$

$$\ldots (6.1)$$

Non trivial solution of the equation (6.1) gives the amplitude x_s of the superstructure mass as follows

$$\frac{X_s}{y_o} = \sqrt{\frac{(K_f^2 + c_f^2)(K_s^2 + c_s w^2)}{(m_f m_s w^4 - m_f k_s w^2 - m_s k_f w^2 - c_s c_f w^2 + k_f k_s)^2 + (K_f c_s w + K_s c_f w - m_f c_s w^3 - m_s c_f w^3 + m_s c_s w^3)^2}}$$

$$\ldots (6.2)$$

By substituting

Mass ratio $\alpha = m_s/m_f$

Damping in foundation $C_f = m_f p_f \xi_f$

Damping in superstructure $C_s = m_s p_s \xi_s$

Where p_f and p_s are the natural frequencies of foundation system alone and superstructure system above respectively.

The equation 6.2 can be expressed as follows

$$\frac{x_s}{y_o} = \sqrt{\frac{(p_f^2+(2\xi_f p_f)^2)(p_s^4+4\xi_s^2 p_s^2 w^2)}{(w^4-p_s w^2-p_f w^2-4\xi_s\xi_f p_f p_s w^2+p_f^2 p_s^2)^2 + (p_f^2.2\xi_s p_s w+p_s^2 2\xi_f p_f w-2\xi_s p_s w^2 -2\xi_f p_s w^3+2\alpha\xi_s p_s w^3)^2}} \quad \ldots (6.3)$$

where ξ_f and ξ_s are the damping ratio to critical damping of the foundation and superstructures respectively.

The amplitudes for the different forcing frequencies w can therefore be obtained from equation 6.3, by obtaining the the value of different constant for the system, which are involved in the above mentioned equation. The various constants involved are as follows.

(a) p_s - natural frequency of free vibration for the superstructure system

(b) p_f - natural frequency of free vibration for the soil-pile system

(c) ξ_s - value of damping (as a percentage of critical damping) in super-structure system

(d) ξ_f - value of damping (as a percent of critical damping) in soil-pile system

(e) α - Ratio of the effective mass of super-structure to that of substructure.

The various properties of the two elements of the model were determined experimentally. The different tests conducted are described below.

6.21 Determination of the Natural Frequency of free vibration (p_s) for super-structure

Natural frequency of free vibration for the super structure system was computed by carrying out free vibration tests on the model. The superstructure system was removed from the pile and was mounted on a firm base. One Miller type acceleration pickup was mounted on the cast iron block of the model, which was connected to automatic pen recorder, connected through a universal amplifier. Acceleration records were taken by giving an impact to the superstructure mass, and thus allowing it to vibrate freely.

Analysis of the records

A typical record of the free vibration is shown in the figure (6.2). The frequency of free vibration is found by the following expression,

$$\text{Frequency} = NXS/_L$$

where L is the length of the record for N number of cycles and S be the speed of paper. Average frequency of free vibration of the superstructure was found as 16.2 cps.

6.22 Determination of Damping in Superstructure

Damping is determined as a prrcentage of the critical damping of the superstructure from the free vibration test records as described in 5.21.

ξ, the damping as a percentage of the critical damping is given by the following expression

$$\xi = \frac{100 \times 2.303}{2\pi} \log_{10} (A_2/A_1)$$

Where A_1 and A_2 are the amplitudes in the successive cycles of vibration.

For small dampings the above expression can be written as,

$$\xi = \frac{100 \times 2.303}{2\pi} \left(\frac{A_2}{A_1} \right)$$

The average value of the damping in superstructure model was found to be 1.58%.

6.23 Determination of the Natural Frequency of Free Vibrations (p_f) for the Soil-pile system

Natural frequency of free vibration for the soil-pile system was obtained by carrying out free vibration tests on the model. Superstructure model was removed from the wooden pile cap by unscrewing the steel strips from the wooden cap. The pile (fitted with the pile cap) was placed vertically in the center of the tank, and clamped at the top by a suitable device fitted at the top of the tank. Sand was poured in the tank in 6" layers and was compacted by vibrating it on the shaking table to get same density in each test as described in 5.51. A Miller type acceleration pickup was mounted on the wooden pile cap and connected to the automatic pen recorder, through a universal amplifier.

Free vibrations to the pile were given by displacing the pile from its mean position and then releasing it.

Frequency from the free vibration records is computed in a similar way as described in 5.21.

The average frequency of free vibration for the soil-pile system is found to be 60.2 cycles per second.

6.24 Determination of Damping in Soil-pile System

The damping in the soil-pile system was obtained from the free vibrations test record on the model, in a similar way as described in 5.22 for the super-structure case.

The average damping as a percentage of the critical damping is found to be 7%.

6.25 Determination of the Mass Ratio α

The mass ratio α is defined as the ratio of the effective mass of superstructure to that of sub-structure. Therefore the evaluation of mass ratio shall require the effective mass of the superstructure and substructure systems.

Superstructure is a single degree of freedom system and therefore the net effective mass participating in the vibration is directly obtained from the weight of the cast iron block. However in case of the soil-pile system it is a complex problem to evaluate the net effective mass of the foundation participating in the vibrations, since there is some virtual mass of soil which vibrates along with the pile. This aspect of the problem has been discussed in great detail in Chapter II.

In the present case, the net effective mass of the foundation is computed by a semi-experimental method. The characteristic ength of the pile was found experimentally and then the net effective mass of the foundation was obtained by a theoretical approach and with the help of the observed natural frequency of pile under free vibrations.

A detailed description of the experimental tests conducted and the computations done are given below.

Tests Conducted for the Evaluation of the Characteristics length of the pile

Characteristic length of the pile was obtained by carrying out free vibration tests on the different lengths of the pile. The pile was placed vertically in the tank and clamped at the top, by a suitable lamping device fitted at the top of it. Tank was mounted on the shaking table.

Sand was poured in the tank in 6" layers. Each layer was so compacted by the vibrations of the shaking table, so as to obtain the same density in each test, as described in detail under 5.51. Sand was poured up to the top of tank, leaving a free length of pile as 2" above the surface of the sand.

A Miller type acceleration pickup was mounted on the wooden pile cap, which was connected to the automatic pen recorder through a universal amplifier. Free vibration are given to the pile by displacing it from its mean position.

Feom the record natural frequency of free-vibration is found as described under 5.21.

Tests weree conducted for the different embedded length of the pile as 14", 17", 20", 23", 26", 28" and 29". Test with the each length of the pile was repeated with four different surcharge weiths as given below.

W_1 = 0.144 kg (only pile cap)

W_2 = 0.262 Kg

W_3 = 0.670 kg

W_4 = 0.914 kg

The results obtained from the above tests are shown in figure 6.3. Figure shows four different curves for different surcharge loads plotted between embedded length of pile and the natural frequency of the pile.

It is observed from the curves that as the surcharge load on the pile is increased, the natural frequency reduces. The natural frequency increases initially with the increase in the embedded length of the pile. For the each surcharge load, beyond the embedded length of 20 inches, no further increase in the natural frequency is observed. Therefore, on the basis of the discussions, in Chapter II, the embedded length of 20" is taken as the characteristic length of the pile, which is the minimum length of the pile when it starts behaving in a perfect flexible manner.

The length of the pile used in the model is 26" long against the minimum required length for its flexible behaviour, as 20". Therefore the theory of flexible piles applies to the experimental model.

Simulating the embedded pile in sand, to the beam on elastic foundation and assuming the subgrade modulus along the dephh, to be linearly varying i.e.

$$K(x) = \eta_H x$$

Where $K(x)$ is the subgrade modulus at depth x and η_H be the coefficient of subgrade modulus.

Following differential equation is written for the embedded pile

$$\frac{d^4 y}{dx^4} + \frac{k(x)y}{EI} = 0 \qquad \ldots(6.4)$$

where y be the lateral deflection of the pile at depth x.

EI be the flexural stiffness of the pile.

Solution of equation (6.4) are available in terms of non-diamensional parameters.

Using the notation

$T = 5\sqrt{EI/H}$

T be the relative stiffness factor for the pile and $Z = X/T$

$Z_{max} = L/T$

Where L is the embedded length of the pile.

Solution of the differential equation (6.4), for the deflection at ground level is obtained as

$$y_g = Q_{hg} T^3/EI \quad A_y \qquad \ldots (6.5)$$

where y_g is the deflection at ground level.

Q_{hg} is the lateral force at ground level.

A_y is the non-dimensional deflection coefficient. For long pile ($Z_{max} > 5$) embedded in cohesionless soils $A_y = 2.435$ (Rees and Matlock)

From equation 6.2 the overall effective stidfness of the soil pile system works out as

$$\frac{Q_{hg}}{y_g} = K = EI/T^3 A_y$$

The value of relative stiffness factor T, is worked out from the Matlock, criteria for the flexible length, which has been established by Prakash, S. and Aggarwal S.L.[15]. Therefore Z_{max} for 20" depth of embedment of pile is taken as 5.

Therefore

$$Z_{max} = L/_{T} = 5$$

Where L_s is the embedded length of the pile

Hence $T = L_s/5 = 4"$ (10.16 cms).

The overall stiffness of the soil pile system is obtained as

$$K = EI/_{T^3 A_y} = \frac{13.7 \times 10^4}{(10.16)^3 \times 2.435} = 53.6 \text{ kg/cm}$$

The overall effective mass of the pile soil system is worked out from the frequency, mass and stiffness relationship, as follows.

Frequency of vibrations (in radians) is given as

$$w_r = \sqrt{K/_M}$$

$$\text{or} \quad W = g.K/_{w_r{}^2} \qquad \ldots(6.6)$$

Where W is the weight participating in the vibrations. g be the acceleration due to gravity.

Substituting the values of g, K, and w_r in 6.6 effective mass of foundation (W) works out to be 0.368 kg.

The effective weight of the superstructure (weight of the cast iron block) = 0.480 kg.

Therefore the ratio of effective masses of superstructure to the substructure is evaluated as

$$\alpha = 0.480/_{0.368} = 1.3$$

6.26 Various Constants for the Experimental Model as Worked out above in this Chapter

(a) Superstructure system when mounted on rigid base

Natural frequency of free vibration

= 16.02 cps

Damping (percentage to critical damping)

= 1.5%

(b) Soil-pile System when Pile is Embedded in sand

Natural frequency of free vibration

= 60.2 cps

Damping (percentage to critical damping)

= 7%

(c) Effective mass Ratio of Superstructure to Substructure

$\alpha = 1.3$

6.3 Determination of Theoretical Response of the Experimental Model

The response of the structure-pile model is worked out from the equation 6.2, by substituting the various coefficients as given in 6.26.

A computer programme to suit IBM 1620 was developed for the solution of equation 6.2. Results of the analysis are shown in figures 6.4 to 6.7.

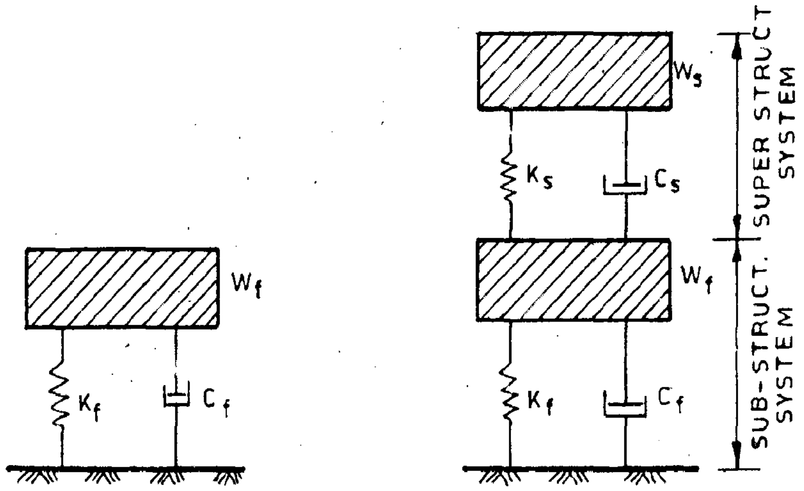
In figure 6.4, curve A is a plot between the forcing frequency and the actually observed response of the model, for eccentricity 25°. This is reproduced from the figure 5.4. Curve B is the plot between the theoretically computed amplitudes of the model from the equation 6.2 and the forcing frequencies.

It is observed from figure 6.4 that the experimental curves A and the theoretical curve B, in general show a striking similarity. The maximum amplitude observed in the experiment is 2.06 mm and occurs at a forcing frequency of 15.45 cps where as the computed maximum amplitude is 1.84 mm and occurs at a forcing frequency of 15 cps. The rising and the decending limbs of the two curves resembles well for the practical purposes.

A similar agreement between the two experimental and theoretical curves is exhibited in figures 6.5, 6.7 and 6.7.

It may be noted however that a slight disagreement is observed between the computed and observed curves for the model. In general the theoretically computed amplitudes are slightly smaller than the actual amplitudes of the superstructure mass. A probale cause of the above phenominon may be explained as below.

The natural frequencies of the soil pile system were obtained before and after the experiment and were found to be comparable. The density of the sand also remained same before and after the experiment. It may therefore be assumed that the lateral stiffness of the pile was not changed. Hence this disagreement between the two curves may be due to the change in the virtual mass of the soil during the vibration. This change in the virtual mass of the soil may have occured due to comparatively lower frequency of vibration during the experiment, than the natural frequency of the soil-pile system. The effective mass of the foundation system was calculated from the natural frequency of the vibration, which is 60.2 cps. However during the steady state vibrations, the model was vibrated in the frequency range of 10-20 cps, which is quite low as compared to the natural frequency of 60.2 cps. Hence it is possible, that there is some added mass of the soil, which becomes active, during the steady state vibrations, resulting in the reduced response of the structure.



a_FOUNDATION SYSTEM IDEALISED TO A SINGLE DEGREE FREEDOM SYSTEM

b_STRUCTURE-PILE SYSTEM IDEALISED IN A TWO DEGREE FREEDOM SYSTEM

FIG. 6.1

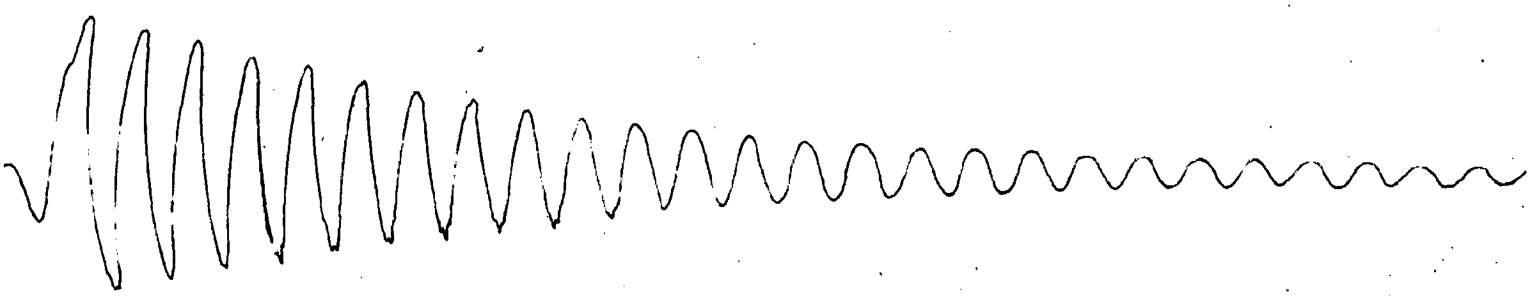


FIG.6.2_ A TYPICAL FREE VIBRATION RECORD

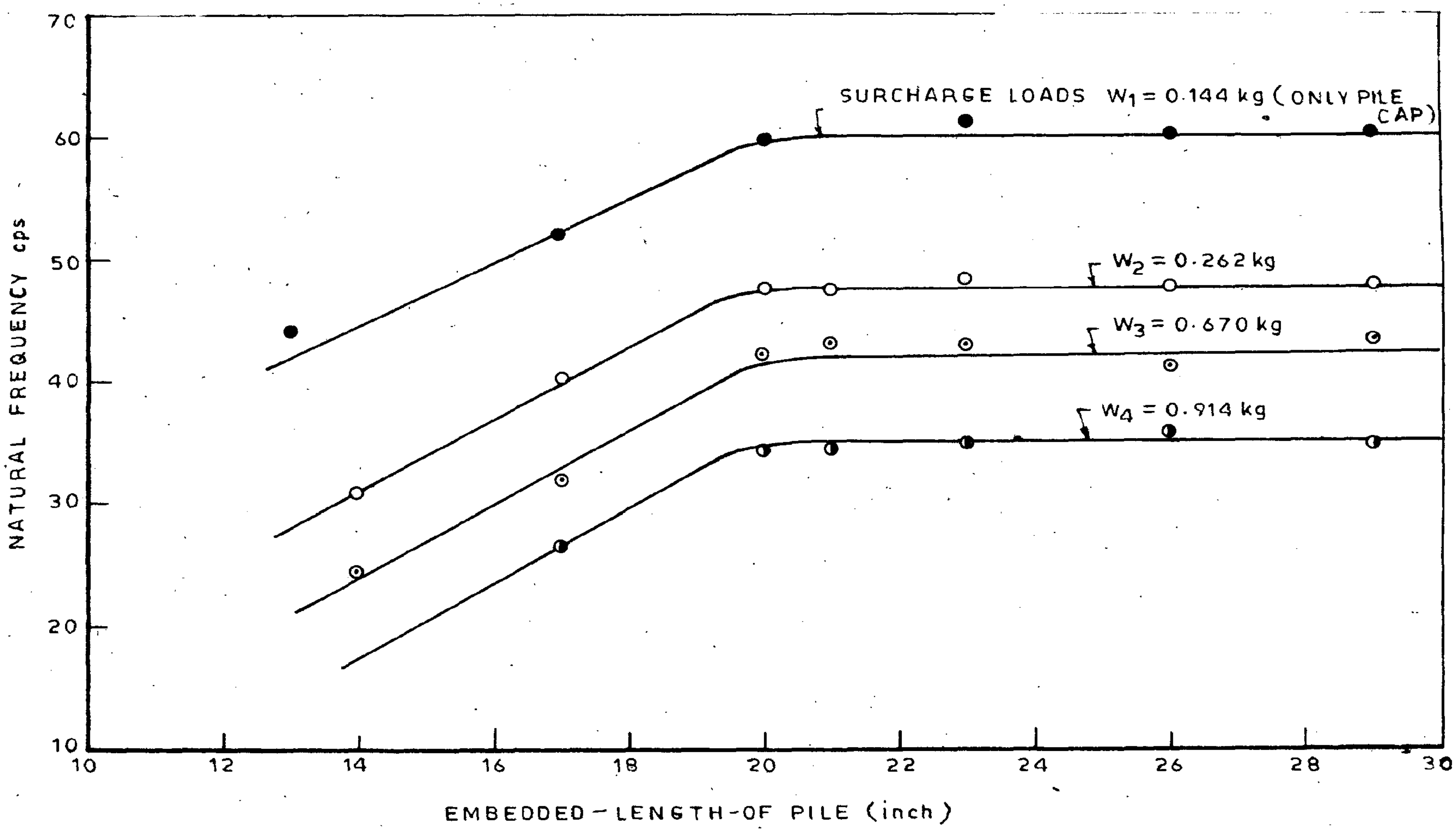


FIG. 6.3_ PILE LENGTH VS NATURAL FREQUENCY CURVES

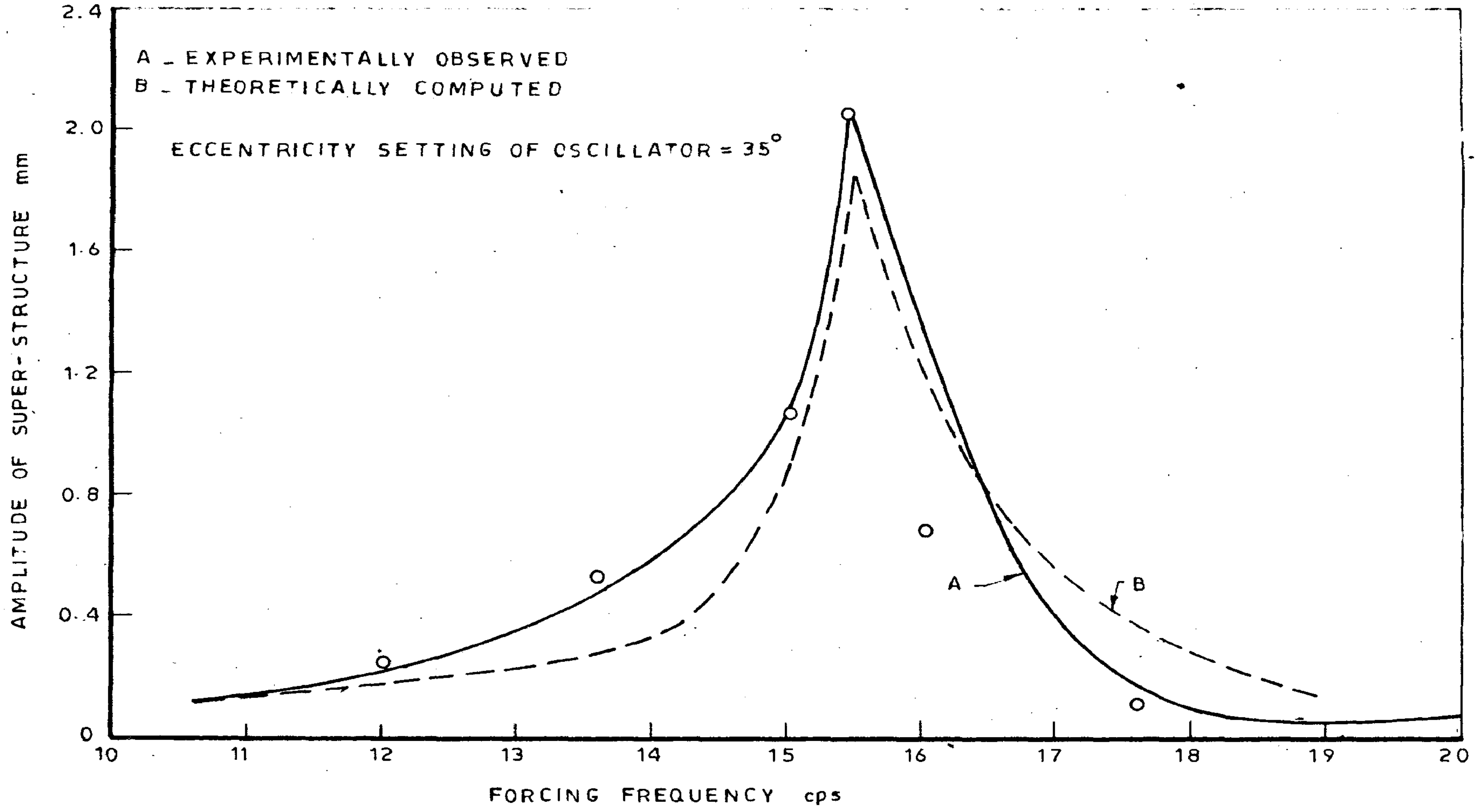


FIG. 6.4_ FORCING FREQUENCY VS AMPLITUDE OF SUPER STRUCTURE

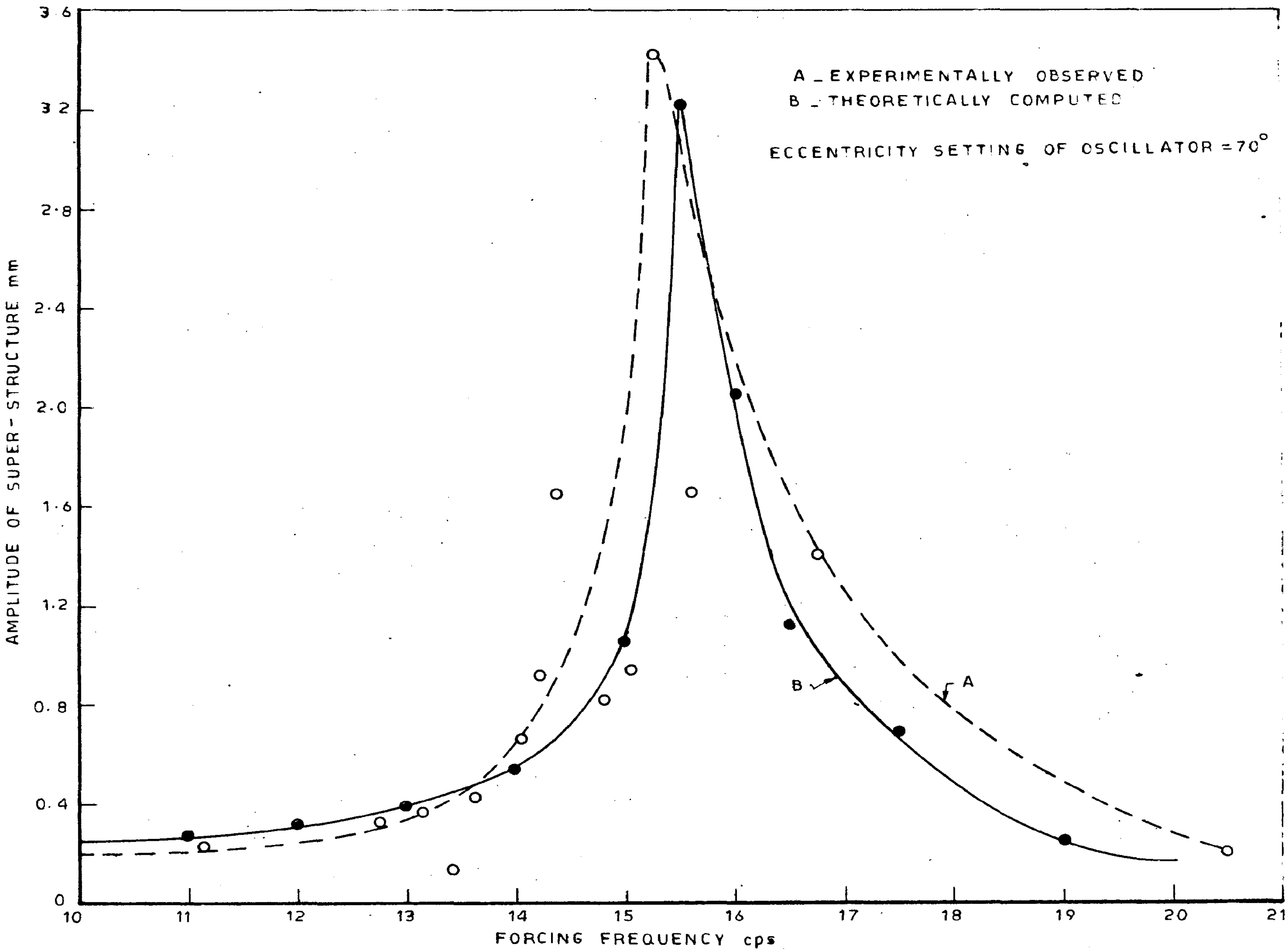


FIG. 6.5 _ FORCING FREQUENCY VS AMPLITUDE OF SUPERSTRUCTURE

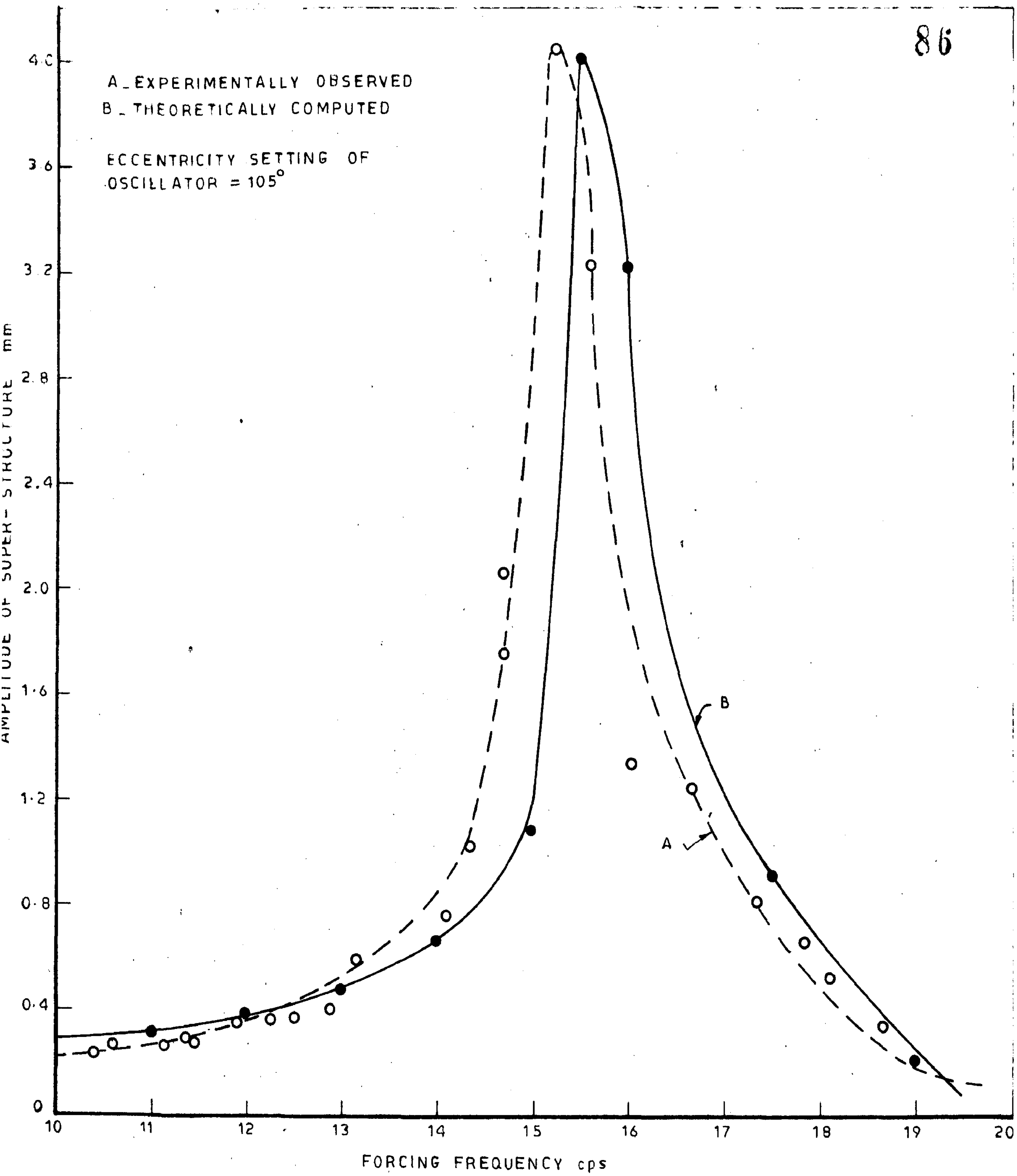


FIG.6.6_FORCING FREQUENCY VS AMPLITUDE OF SUPERSTRUCTURE

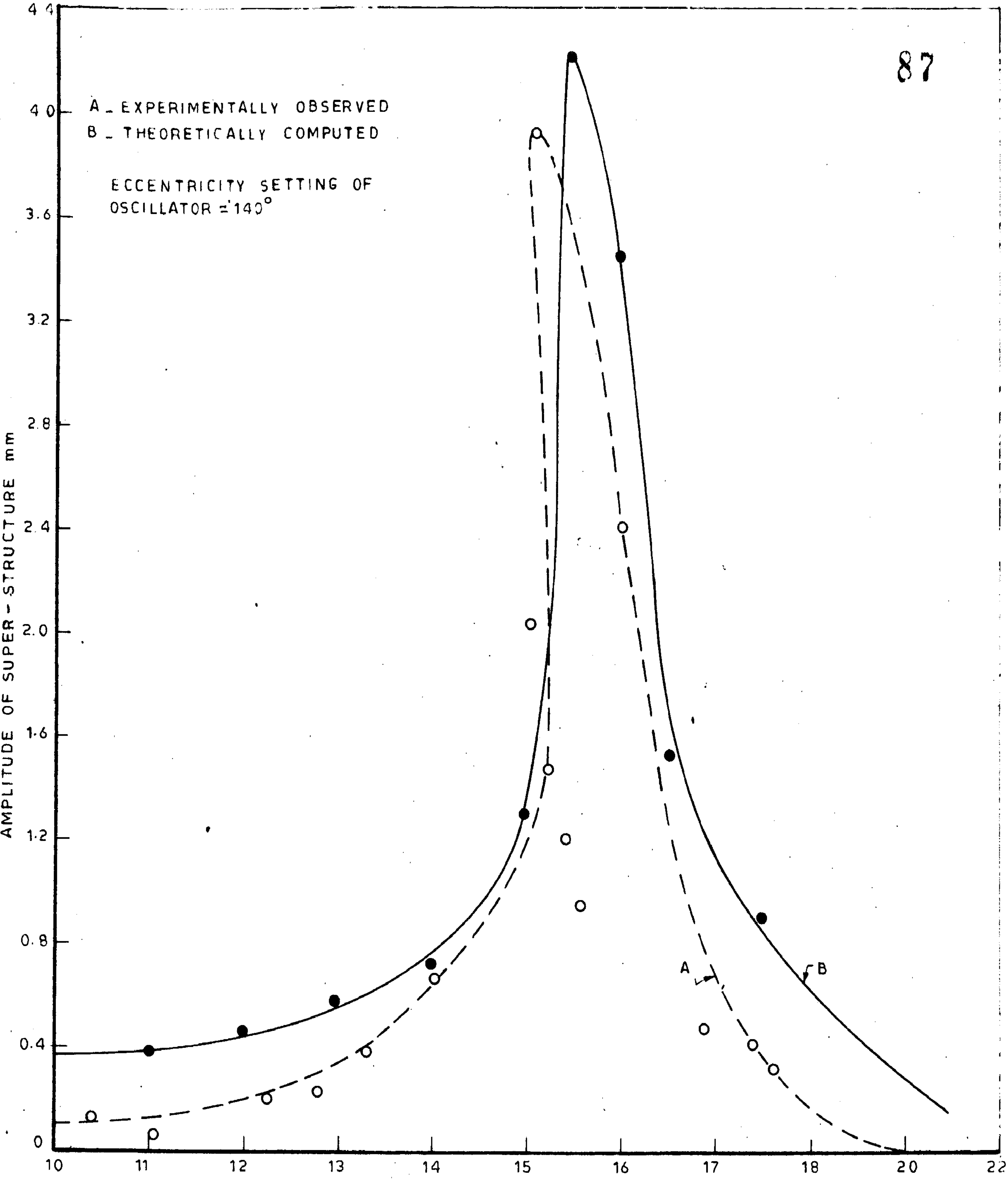


FIG. 6.7 _ FORCING FREQUENCY VS AMPLITUDE OF SUPERSTRUCTURE

CHAPTER VII

CONCLUSIONS

From the discussions presented in the previous Chapters, the following conclusions are drawn,

(1) The response of a structure supported on a flexible piles can be theoretically obtained, by representing it as an equivalent two degree freedom system i.e. a single degree freedom system representing the superstructure, coupled with another single degree system representing soil medium and foundation. The computed results compares well with the actual observed results, when the structure pile system is subjected to the lateral dynamic forces, varying sinusoidally.

(2) When the foundations are very flexible than the superstructure, the forces developed in the superstructure are small under earthquake type of ground motion. For a particular range of substructure natural frequencies, forces developed in the super-structure are large at a particular natural period of superstructure.

(3) When the foundations are very flexible than the super-structure, the forces in superstructure under earthquake type of ground motion, increases with the increase in stiffness of foundation system. However

if the super-structure is made sufficiently flexible as compared the foundation, an increase in foundation stiffness decreases the forces in super-structure.

(4) When the foundations are very flexible than the super-structure, the forces developed in super-structure, under earthquake type of ground motion increases with the increase of mass ratio of super-structure to sub-structure. However, if the super-structure is made sufficiently flexible as compared the foundation, an increase in mass ratio decrease the forces in super-structure.

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