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SYNOPSIS The investigation presented here utilises finite element technique coupled with Galerikins weighted residual process to predict the settlement of footing resting on the surface of clays and subjected to transient load. The predicted quantities have been compared with the experimental observations of model footing tests. The influence of fundamental natural period of soil and foundation system on the settlement and bearing capacity of footing has been brought out.

INTRODUCTION

The theoretical work on bearing capacity of soils under transient load can be grouped into (a) Single Degree Spring-Mass-Dashpot System (b) One Dimensional Wave Propagation (c) Equilibrium of failure wedge (d) Non-Dimensional Analysis and (e) Numerical Technique.

Description of above methods with all the pertinent available information has been critically and chronologically summarised by Basavanna et al (1974).

There are several important parameters which govern the settlement and bearing capacity of footings under transient load. The above theories based on several simplifying assumptions are inadequate to provide a guide line to the designer and therefore it is necessary to isolate and study the influence of each important parameters that govern the settlement of footing under transient loads. In this paper, finite element technique coupled with Galerikin's weighted residual process has been used to isolate and study the influence of fundamental natural period of soil foundation system on the behaviour of footings resting on the surface of clay.

MATRIX DIFFERENTIAL EQUATION

The difference in the deformational behaviour of soil under dynamic load from that of static load depends on type of soil, its density and strain rate. For linear visco-elastic behaviour of soil, Basavanna (1975) has shown that the damping matrix of an element consists of two parts, one is internal damping resistance due to rate of relative inter-particle displacements which is reflected as an effect of rate of strain in stress strain relations and the other is external damping due to air resistance. The method of determination of internal damping matrix of an element is the same as that for stiffness matrix except that the elasticity matrix [D] is replaced by viscous resistance matrix [H]. The method of determination of external damping matrix of an element is the same as that for the mass matrix of the element except that the mass density 'ρ' is replaced by coefficient of viscosity 'μ'. Accordingly the damping matrix of an element is :

$$[C]^e = \int_{vol} [B]^T [H][B]\, dv + \int_{vol} [N]^T \mu[N] dv \quad (1)$$

where $[C]^e$ is damping matrix of an element [D] is a matrix of 6x6 elastic constants, [H] is a matrix of 6x6 viscous resistance coefficients [B] is total strain matrix of an element, $[B]^T$ is transpose of strain matrix [B], [N] is shape function matrix of an element, $[N]^T$ is transpose of matrix [N].

Mass Matrix

Basavanna (1975) had shown that for axi-symmetric problems, the mass matrix of a constant stress triangular element depends on the global radial co-ordinate as given below and the same has been used in this analysis.

$$m_{ij} = \frac{m_e}{10}\left(1+\frac{R_i+R_j}{3R_c}\right) \quad \text{when } i = j \qquad (2a)$$

$$= \frac{m_e}{20}\left(1+\frac{R_i+R_j}{3R_c}\right) \quad \text{when } i \neq j \qquad (2b)$$

where m_e is mass of the element, m_{ij} is mass at node i corresponding to acceleration at node j of the element, R_i, R_j and R_c are global radial co-ordinates of nodes i, j and centroid of triangular element respectively.

Governing Matrix Differential Equation

Finally, on assembly of stiffness, damping and mass matrix of all the elements gives the following matrix differential equation of equilibrium at any instant of time 't'.

$$[K]\{\delta\} + [C]\frac{\partial}{\partial t}\{\delta\} + [M]\frac{\partial^2}{\partial t^2}\{\delta\} + \{F\} = \{O\} \quad (3)$$

where [K] is assembled stiffness matrix of the system, [C] is assembled damping matrix of the system, [M] is assembled mass matrix of the system, $\{\delta\}$ is assembled column vector of the system, {F} is assembled column vector of components of time dependent nodal external forces acting on the system.

RECURRENCE RELATION

The matrix differential equation (3) is of a type in which specified values of the functions and its first time derivative at the start, defines uniquely this function throughout the time interval. Such problem can be solved either by writing suitable recurrence relation within a small interval of time Δt or by normal mode proceedure. If the stress strain relation of the material is non-linear, it is advantageous to obtain solution by writing recurrence relation, within a small interval of time (Δt) and applying it step-by-step from the start with modified stiffness in each step depending on the stress prevailing at the begining of each step until the full solution is obtained. The required recurrence relation can be established in different ways, out of which Galerikin's weighted residual process allows a more comprehensive treatment. Hence the recurrence relation given below is obtained by this process and is used in this analysis.

$$[A]\{X\}_{\Delta t} = [B]\{X\}_o + \{C\} \quad (4)$$

where [A] and [B] are square matrices of 2n x 2n, $\{X\}_{\Delta t}$ and $\{X\}_o$ are column vector containing components of nodal displacements and velocities at time 't + Δt' and 't' respectively. {C} is column vector, whose elements depends on the external nodal force vector between 't' and 't +Δt'.

The elements of both [A] and [B] matrix are dependent on elements of stiffness, mass and damping matrices and time interval 'Δt' are unsymmetric. However, for undamped system the unsymmetric matrix [A] in equation (6) can be transformed into a symmetric matrix.

ANALYSIS OF FOOTING BEHAVIOUR

In order to verify the suitability of the proposed method, two square footings resting on the surface of partially saturated buckshot clay and subjected to transient and static load are taken as examples for analysis and comparison. Experimental data for stress strain relationship and load settlement characteristics are taken from the report of carrol (1963).

Properties of the Clay Used in the Analysis

Out of many transient load test reported by carrol the load test on 0.254 m x 0.254 m and 0.458 m x 0.458 m aluminium square plates are chosen for this analysis. The properties of the clay reported by him and used in this analysis are as follows :

Initial tangent modulus $E_i = 2445\ t/m^2$
Poisson's ratio $\nu = 0.375$
Density of Soil $\gamma = 1.81\ t/m^3$
Angle of internal friction $\phi = 0$

Cohesion 'C' is 6.986 t/sq m for the clay below 0.254 m x 0.254 m footing and is 6.518 t/sq m for the clay below 0.458 m x 0.458 m footing.

According to Carrol, the equation of hyperbola proposed by Kondner (1963) was found to represent closely the stress strain relationship of Buckshot clay under transient loading condition also. Hence, hyperbolic representation of deviator stress axial strain relation is used in this analysis. The effect of confining pressure on initial tangent modulus and shear strength is neglected, due to negligible influence indicated by the test results reported by him. The ratio of ultimate strength of hyperbolic representation and actual strength of clay reported and used in the analysis is 1.07.

Transient Load Considered in the Analysis

Carrol (1963) reported that the reaction load measured between loading column and the test plate indicated smooth parabolic or sinusoidal variation of load with time upto 1.5 t_o, where 't_o' is the time at which peak load occurs. For this analysis only the sinusoidal variation of load is considered. The time (t_o) at which the peak load acted on 0.254 m x 0.254 m and 0.458 m x 0.458 m square plates are 32 m sec and 19 m sec respectively.

Properties of Aluminium Plate

Since, no information is reported by Carrol on deformational properties of aluminium, the following deformational properties of 2014-T6 aluminium alloy as reported by Arya and Ajmani (1965) is used in this analysis.

Modulus of elasticity $E = 7.45 \times 10^6\ t/m^2$
Poisson's ratio $\nu = 0.32$
Density $\gamma_a = 2.8\ t/m^3$

Finite Element Mesh and Time Interval

Due to tremendous difference in computational effort involved in application of finite element method to a three dimensional and an axi-symmetric problem, the two square test plates are analysed by considering their equivalent circular footings. The equivalent diameter of the circular plates for the 0.254 m and 0.458 m square plates considered in this analysis are 0.287 m and 0.518 m respectively.

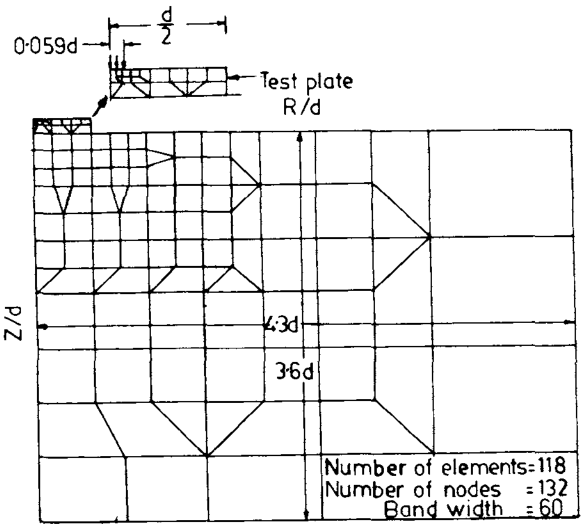

Fig 1 - Finite Element Mesh.

The division of test plate and the clay medium into a finite element mesh is shown in Fig.1 The time interval (Δt) used in the analysis is 0.25 m sec and 1.5 m sec respectively for 0.287 m and 0.518 m diameter footings.

Load Settlement Curves

The load settlement curves obtained from the analysis along with experimental curves are shown in Fig 2. The ultimate bearing capacity of 0.287 m and 0.518 m diameter footings as obtained by finite element method are equal to 18 t/sq m and 12.2 t/sq m respectively against the respective experimental values of 22.2 t/sq m and 12.4 t/sq m. The values obtained by analysis are less by about 19 and 1.62 percent of experimental values for 0.287 m and 0.518 m diameter plates. In Fig 2, for 0.518 m diameter footing, below 9.6 t/sq m average bearing pressure, the computed settlement is less than that observed in plate load test. Above 9.6 t/sq m, the computed settlement is greater than that observed in the test. For 0.287 m diameter footing the computed settlement is greater than that observed in the test at all intensities of applied load. Considering the approximation used in the analysis for accounting the strain rate effect on deformational behaviour of soil and numerous sources of errors in the determination of stress-strain relationship of Buckshot clay under transient loading, the computed load settlement characteristics of footing considered appears to agree reasonably well with experimental observation. The effect of approximation used in the analysis for accounting strain rate effect is discussed below.

The total resistance offered by the soil under transient load consists of two parts, out of which one is dependent on magnitude of strain 'ϵ' and the other is dependent on magnitude of strain rate '$\dot{\epsilon}$'. This behaviour of soil can be accurately represented by considering non-linear visco-elastic model. In the analysis, this representation, results in stiffness

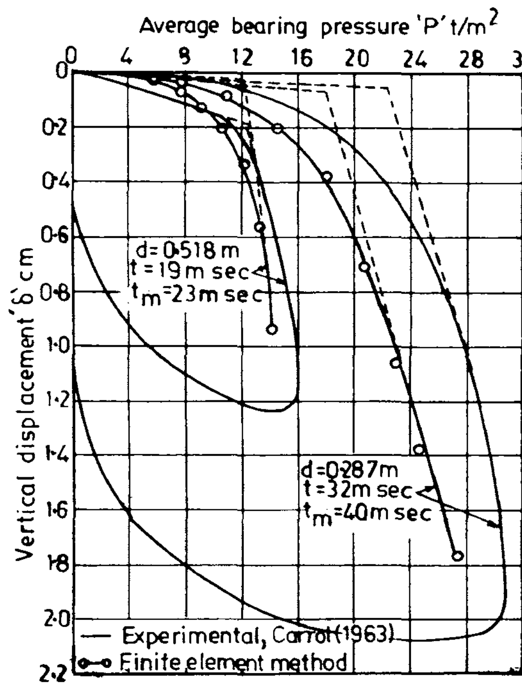

Fig 2 - Load Settlement Curves-Transient Load.

matrix equal to the stiffness matrix under static load and damping matrix which depends on effect of rate of strain.

In absence of non-linear visco-elastic model for representation of stress strain relationship of Buckshot clay under transient load, the effect of rate of strain is approximately accounted by representing the stress-strain relationship under transient load by equation of hyperbola with an increase in values of initial tangent modulus and cohesion. In the analysis, this approximation results in higher coefficients of stiffness matrix and zero internal damping. In other words, it results in (i) lower natural periods of the clay foundation system than the actual and (ii) smaller damping. It is well known that the natural periods and damping of the system affects the response of footing under transient loading depending on duration of applied load. Hence it is possible that the computed settlement would have been more close to the experimental data, if the non-linear visco-elastic properties of the clay had been used in the analysis.

Factor of Safety Contours

The mechanism of failure or development of plastic zone in the soil below footing, can be represented by drawing the contours of factor of safety.

Using the stresses computed and strength of clays, factor of safety are obtained at various points at a given time and contours of equal factor of safety are drawn as shown in Fig. 3 for 0.281 m diameter footing. In this figure, the least factor of safety contour lies always

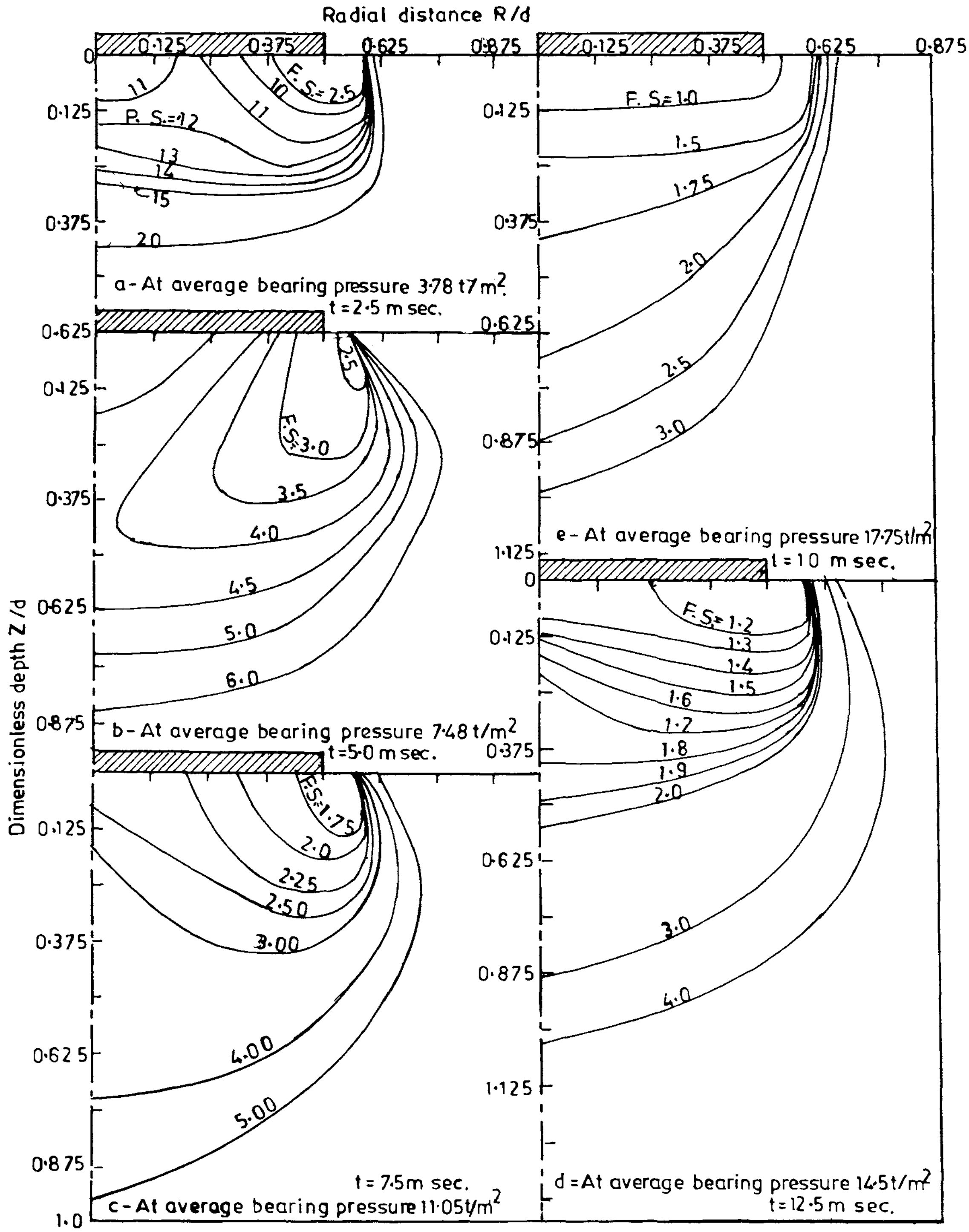

Fig 3 - Factor of Safety Contours-Transient Load.

towards the edge of footing until the failure is reached. However, at 17.75 t/sq m average bearing pressure, the clay medium lying between the footing and upto depth equal to 0.1 times the diameter (bounded by the contour of F.S. = 1) is in complete plastic state.

Contact Pressure Distribution

The contact pressure distribution below 0.287 m diameter footing at 2.5, 5, 7.5, 10 and 12.5 m sec are shown Fig 4. Examination of this figure reveals that the contact pressure at the edge of footing is greater than that at the centre of footing at the begining of loading. This is so because at the begining of load, the factor safety in the clay below footing is same at all places and greater stresses are required at the edge compared to that at the centre to get almost uniform settlement of footing.

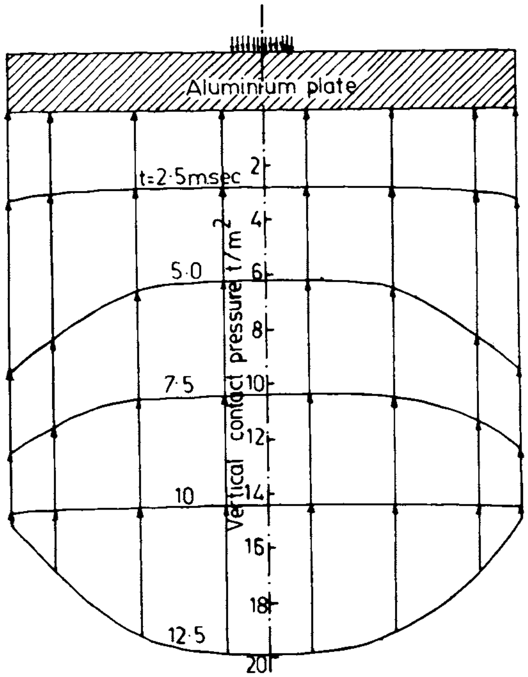

Fig 4 - Contact Pressure Distribution-Transient Load.

As the load acting on the footing increases the factor of safety of the clay close to the edge decreases at faster rate as such, stresses being higher towards the edge, than those close to centre of footing. This state of stress prevails up to 7.5 m sec. At this stage the average bearing pressure acting on the footing is equal to 11.05 t/sq m and the factor of safety of the clay close to edge is 1.75 and that close to centre lies between 2.25 to 2.5 (See Fig 3c). For further increase in load acting on the footing greater increase in stress at the centre than that at the edge is required to get uniform settlemt below the footing. Thus the total contact pressure below footing becomes almost constant at 10 m sec. At this stage the average bearing pressure on the footing is equal to 14.5 t/sq m and the factor of safety of the clay close to the centre lies between 1.2 to 1.3 (See Fig 3d). This indicates that still greater stresses at the centre are required, if the clay below footing has to reach its plastic state. Finally at 12.5 m sec, the total contact stress at the centre is higher than that at the edge. At this stage the clay lying between footing and upto depth equal to 0.1 d is in state of plastic equilibrium (See Fig 3e). Where 'd' is the diameter of footing.

Influence of Fundamental Natural Period of Soil Foundation System

At the start of the loading, the fundamental natural period 'T' of 0.287 m and 0.518 m diameter footings are 0.0113 sec and 0.0204 sec. Since, the stiffness of soil decreases with increase in magnitude of load, these periods increase with increase in magnitude of applied load.

Transient loads applied on 0.287 m and 0.518 m diameter footings attained their peak value at time 't' equal to 0.032 sec and 0.019 sec respectively. Since, the applied transient loads are considered to vary sinusoidally with time up to its peak value, the equivalent periods of applied transient load are equal to four times the value stated above.

The ratio of the cohesion value of the clay under very fast rate of loading to that under static loading is equal to 1.9.

Comprehensive details of bearing capacities and other properties of both the footing are listed in Table - I.

Inspite of the fact, that the strength of the clay under transient loading is much higher than that under static loading, the ultimate bearing capacity of 0.518 m diameter footing under transient load is less than the static value. However, for 0.218 m diameter footing, the value of bearing capacity under transient load is higher than the static value. It appears that the lower value of ultimate bearing capacity of 0.518 m diameter footing under transient load may be due to the fact that the period of applied transient load of this footing is more close to its fundamental natural period whereas, the period of applied load on 0.287 m diameter footing is far away from its natural period. Thus it can be concluded that, even if there is an increase in shear strength of soil due to strain rate effect, the ultimate bearing capacity of the footings under transient load may increase or decrease from the static value depending on how close the period of applied transient load is to the fundamental natural period of the soil and structure as a whole. However, this needs further detailed parametric study.

Moreover, the duration of applied transient load considered above are far less than the duration of earthquake forces acting on the footing during an earthquake. Once, the ultimate bearing capacity of soil under transient load with period close to fundamental natural period and of short duration is less than the static value, the same may be the case under earthquake loading, if the predominant period of ground motion is close to the fundamental natural period of soil foundation system. Hence, the increase in permissible bearing capacity of soil under earthquake loding as recommended by I.S. code (1970) seems to be unrealistic, if the predominant period of ground motion is close to the fundamental natural period of soil foundation system. This fact also needs detailed investigation for making definite and quantitative recommendation.

CONCLUSION

The analysis presented here seems to give reasonably agreable behaviour of footings with that observed in experiments. This study shows that

1. The non-linear visco-elastic properties of soil need be evaluated for better evaluation

TABLE I. Comprehensive Details of Clay Footing System

Diameter (m)	Equivalent period of applied transient load, (sec)	Natural period (sec)	Ratio of applied load period to natural* period.	Ratio of strength of clay under Transient load to that under static load	Ratio of Bearing capacity under Transient load to that under static load	
					F.E.M.	Load Test
0.287	0.128	0.0113	11.35	1.9	1.27	1.58
0.518	0.056	0.0204	3.725	1.9	0.915	0.90

* These ratios are at the start of loading. Since, the natural periods increase with increase in magnitude of load, these ratios decrease with increase in magnitude of load.

of footing behaviour under transient load.

2. Even if there is an increase in shear strength of clays, due to strain rate, the bearing capacity of clays under transient load may be higher or lower than that under static load depending upon how close is the period of applied transient load to the fundamental natural period. This aspect needs a detailed parametric analysis.

3. The increase in permissible bearing capacity of soil under earthquake loading as recommended by I.S. code (1970) seems to be unrealistic, if the predominent period of ground motion is close to the fundamental natural period of soil foundation system. This also needs detailed investigation for making definite and quantitative recommendation.

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