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DYNAMIC BEARING CAPACITY OF SOILS UNDER TRANSIENT LOADING

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INTRODUCTION

With modern developments in science and technology, many Civil, industrial, and defence oriented structures have come up which are subjected to time dependant external forces. After second world war, development in nuclear appliances have led to the necessity of a more precise study of foundations under transient loads. As a consequence, the design engineers are confronted with the problem of large dynamic loads acting for a very small duration of time. While design of superstructure against such loads has received considerable attention from many investigators with some degree of refinement, the design of substructure lagged behind because of erratic, heterogeneous and intricate characteristics of the supporting soil medium.

Due to a large number of parameters influencing dynamic bearing capacity and a limited number of investigations performed till to-date (1974), very little information is available on the subject. Only a few theories based on several simplifying assumptions have been advanced. Experimental data in their support is lacking.

In this paper all the pertinent available information has been critically and chronologically examined, Areas requiring further research have been clearly brought out. A comprehensive bibliography has also been included.

DESIGN REQUIREMENT OF FOUNDATIONS

In case of static loads, allowable pressure on footing depends on :

(i) ultimate bearing capacity of soil.

(ii) settlement or deformation consideration.

In constant to static loads, in case of dynamic load acting for short duration the allowable pressure on any footing mainly depends on the deformations caused by the applied load. The elastic, plastic or elastoplastic deformation caused by the applied load should in any case be less than the tolerable deformation. The topic of bearing capacity under dynamic loads of short duration is therefore directed towards the estimation of these deformations of the soil caused by the dynamic load.

The first work on dynamic bearing capacity of soils dates back to 1954, when Landale reported his work at M. I. T. By about this time, several other investigators got interested in blast resistant design of structures. Since 1958, a number of agencies like Armour Research Foundation at M. I. T., Naval Civil Engineering Laboratory at California, University of Illinois, W.E.S., Mississippi, and School of Research and Training in Earthquake Engineering at Roorkee started theoretical and experimental investigation in this field.

There are six methods of tackling this problem, which are—

(1) Pseudo static method

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(2) Method based on Single Degree Freedom System
(3) Method based on Wave Propagation
(4) Method based on equilibrium of failure wedge
(5) Non-dimensional analysis
(6) Numerical technique

A brief description of these methods is as follows:

PSEUDOSTATIC METHOD

In this method a static failure wedge is assumed to be acted upon by a pseudo-static force equal to the peak dynamic force and the problem is analysed like a static case. Clearly it is an inadequate procedure because it does not essentially consider the dynamic nature of the force and because, static failure wedge need not necessarily be valid for dynamic case too. So it is not discussed with greater details overhere.

METHOD BASED ON SINGLE DEGREE FREEDOM SYSTEM

In this method, the soil medium is replaced by an equivalent spring and deshpot (Fig. 1). The mass of the vibrating system represented by a concentrated mass including

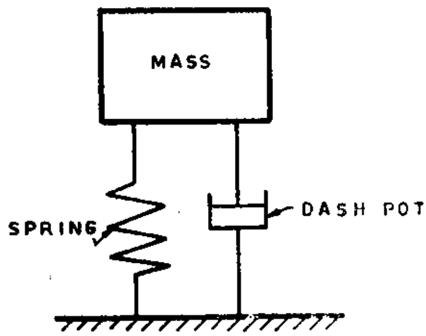

Fig. 1. Spring-Mass-Dash Pot System

the mass of the footing and an equivalent soil mass associated displacement. This is a very simple and convenient tool for analysis of footing-soil system. The predicted displacement is elastic if the displacement is less than or equal to yield displacement and elastoplastic if it is greater than yeild displacement. If the predicted maximum displacement is equal to allowable displacement, the average maximum pressure acting on the footing may be defined as the dynamic bearing capacity of soils.

Landale (1954) proposed a solution of this problem utilizing elasto-plastic soil spring obtained by approximating static load settlement curve in his analysis. He considered the mass of the vibrating system to increase linearly with footing displacement from an initial mass equal to the soil mass contained in a hemisphere of diameter equal to 1.5 times the footing width to a final mass of soil contained within the failure surfaces comparable to those assumed by Terzaghi for long footings. The equation also included a term accounting for increase in strength due to strain rate effects. The displacements predicted compared poorly with his test results.

Fisher (1962) employed an analogue computer set to behave as an elastic system and fed it with step—pulse voltage similar to load pulse. The assumed values of equivalent spring stiffness, mass and damping constants of the system were so adjusted by hit and trial method that the displacement-time graph obtained by the analog computer matched well with that obtained experimentally. Then by retaining or disconnecting from the circuit the element introducing damping, both, damped and undamped maximum deflections were obtained by similar procedures. From these deflections critical damping can be obtained by:—

$$\text{Log}_e\left\{\frac{2\Delta d}{\Delta u}-1\right\}=-\frac{\rho\pi}{\{1-\rho^2\}^{1/2}} \qquad \ldots(1)$$

where Δd=damped maximum displacement

Δu=undamped maximum displacement

ρ=percent critical damping

It is to be noted that the circuitry of the analog computer used was such that the test results could be matched only upto their maximum settlement. Upon unloading the settlement of analog S. D. F. System returns to zero whereas in experiments only a small portion of settlement was recoverable. Finally, period of cohensionless model proposed by him using static test results were approximately one fifth that computed when the dynamic test results were matched with analog computer.

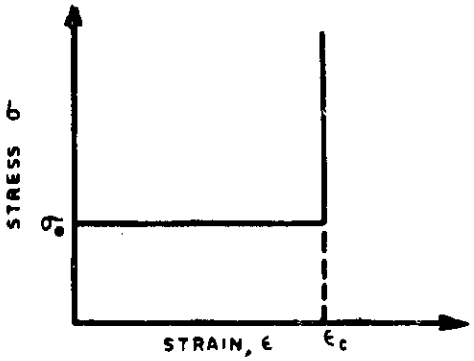

Fig. 2A. Rigid-Plastic Locking Media

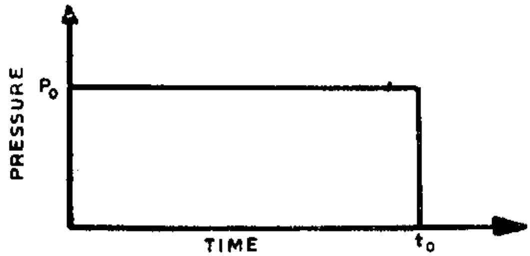

Fig. 2B. Step Pulse

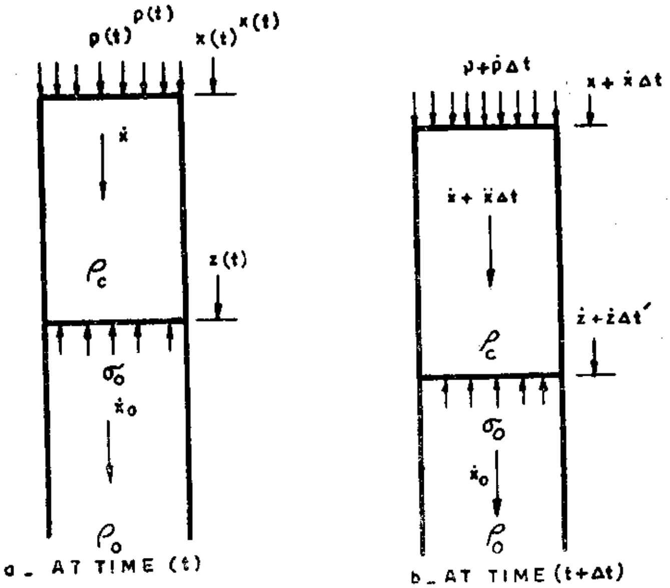

Fig. 2C. Soil Column Below Footing

METHOD BASED ON WAVE PROPAGATION

In this method, the propagation of stress waves into the soil medium when foundation is dynamically loaded is made use of. The stress strain relationships used are linear elastic or rigid plastic locking media (Fig. 2a). The equation of motion is developed using laws of conservation of mass and Newton's second law of motion. The solution of equation gives the resulting displacement under the applied dynamic load, from which the dynamic bearing capacity of soils is determined as per definition.

Selig (1960) proposed a model where in the soil is represented by a truneated pyramid of infinite depth with its top having the same size and shape as that of the base of footing. The sides of the pyramid slope outward and make an angle ψ with the vertical. The vertical stresses acting on any horizontal section within the pyramid are considered to be linearly elastic. Using Newton's Second Law of Motion, and equilibrium of an element "dz" at a depth "z" below the footing base, the equation of motion for square and circular footings is obtained as—

$$\frac{\partial^2 U(z,t)}{\partial z^2}+\frac{4\tan\psi}{B+2z\tan\psi}\cdot\frac{\partial U(z,t)}{\partial z}=\frac{1}{C_0^2}\frac{\partial^2 U(z,t)}{\partial t^2} \quad \ldots(2)$$

where $U(z, t)$ = Displacement at depth 'z' and time 't'

C_0 = wave velocity

$\frac{\partial U(z,t)}{\partial z}$ = First order derivative of $U(z, t)$ with respect to z

$\frac{\partial^2 U(z,t)}{\partial^2 z}$ = Second order partial derivative of $U(z, t)$ with respect to 'z'

$\frac{\partial^2 U(z,t)}{\partial^2 t}$ = Second order partial derivation of $U(z, t)$ with respect to 't'

E = Slope of the stress-strain relation

ρ = Mass density of the soil contained in the pyramid

B = Width or diameter of footing

For a very deep pyramid, the solution of the equation gives footing displacement as—

$$\delta=U(o,t)=\frac{C_0}{E}\int_0^t P(\tau).e^{-\left\{\frac{2C_0.\tan(t-\tau)}{B}\right\}}.d\tau \quad \ldots(3)$$

where $P(\tau)$ = applied dynamic bearing pressure

δ = footing displacement

The predicted displacements compared poorly with the experimental results from Armour Research Foundation.

McKee (1962) proposed a model of footing soil system with an assumed column of soil below the footing symmetrical above its vertical axis (Fig. 2c). In his first attempt he assumed a rigid plastic locking medium whose stress strain characteristics are independent of depth (Fig. 2a). Fig. 2c shows soil column at time 't' and a short time later at $t+t'$ based on material property shown in Fig. 2a. By using Newton's second law of motion, conservation of mass and initial velocity 'x_0' as zero the resulting equation of motion is :

$$x(t)\ddot{x}(t)+\{\dot{x}(t)\}^2=\{P(t)-\sigma_0\}\left\{\frac{1}{\rho_0}-\frac{1}{\rho_c}\right\} \quad \ldots(4)$$

where $P(t)$ = Dynamic pressure acting on soil column below footing

$x(t); \dot{x}(t); \ddot{x}(t)$ = Dynamic diplacement, velocity and acceleration of the footing respectively

$z(t); \dot{z}(t)$ = Vertical location and velocity of compaction front respectively

σ_0=Plastic stress
ρ_c=Mass density associated with locking
ρ_0—Initial mass density
ε_c=Strain associated with locking

The solution of equation (4) which satisfies the initial condition $x(0)=0$, $\dot{x}(0)=0$ is—

$$x(t)=\sqrt{2\left\{\frac{1}{\rho_0}-\frac{1}{\rho_c}\right\}.\int_0^t\int_0^\tau [P(\gamma)-\sigma_0].d\gamma.d\tau.} \quad \ldots(5)$$

Since the stress-strain relationship allows no recovery, this solution is meaningful only so long as:

$$\{P(\tau)-\sigma_0\}\, d\tau \geqslant 0. \quad \ldots(6)$$

If a rigid mass at top of the soil column is considered, the solution is modified as:

$$x(t)=\left\{\left[h.\rho_r\left(\frac{1}{\rho_c}-\frac{1}{\rho_0}\right)\right]^2+2\left\{\frac{1}{\rho_c}-\frac{1}{\rho_c}\right\}.\int_0^t\int_0^\tau [P(\gamma)-\sigma_0].d\gamma.d\tau\right\}^{1/2}$$
$$-h.\rho_r\left(\frac{1}{\rho_0}-\frac{1}{\rho_c}\right) \quad \ldots(7)$$

where h=height of rigid mass resting on soil column
ρ_r=mass density of rigid mass

The Predicted displacements were only in qualitative agreement with his experimental results. In the second attempt, with Runge-Kutta numerical technique he predicted dynamic displacement for cases in which the yield stress or area of soil column linearly varies with depth. Again the predicted results were in qualitative agreement with the experimental values. For better agreement, a better understanding of the stress strain behaviour of soil and its variation with depth are required.

Carrol (1963) proposed an elementry one dimensional wave propagation theory, in which he considered column of soil extending from the bottom of the footings to infinite depth. The area of soil column considered at any depth is equal to the area of footing. Using Newton's second law of motion and conservation of mass for monotonically increasing stress wave, he obtained an equation for predicting footing displacement, which for linear elastic stress strain relationships $\left(\sigma=\frac{\varepsilon}{a}\right)$ is given by:

$$\frac{\delta}{C_0 t_0}\left(\frac{b}{a}\right)=b\int_0^{t/t_0} P\left(\frac{t}{t_0}\right).d\left(\frac{t}{t_0}\right) \quad \ldots(8)$$

For hyperbolic stress strain relationship $\left(\sigma=\frac{E}{a+b\varepsilon}\right)$, the expression for displacement is—

$$\frac{\delta}{C_0 t_0}\left(\frac{b}{a}\right)=-\int_0^{t/t_0} L_n\left[1+b.\rho\left(\frac{t}{t_0}\right)\right].d\left(\frac{t}{t_0}\right) \quad \ldots(9)$$

where, δ=surface displacement

C_0=wave velocity=$\sqrt{\frac{1}{a\rho}}$

t_0=time at which maximum P is reached

$\frac{1}{a}$=Initial tangent modulus of stress strain curve

$\frac{1}{b}$=horizontal asymptote of stress-strain curve which is proportional to maximum deviatric stress.

From the predicted time dependent displacement using linear and transient pulse $\left\{P=P_0\left(\frac{t}{t_0}\right);\ \text{and}\ P=P_0\left(\frac{t}{t_0}\right)\left(2-\frac{t}{t_0}\right)\ \text{respectively}\right\}$ where P is maximum intensity of pulse, he concludes (i) for times considerably less than t_0 and for both linear and hyperbolic stress strain relationship the linear transient pulse produces greater displacements than does parabolic pulse. At times approaching 't_0' this phenomenon reverses itself. (ii) both pulses produce greater displacements with hyperbolic stress strain relationship than with the linear one.

Fig. 3 shows nondimensional displacement $\left\{\frac{\delta}{C_0 t_0}\cdot\frac{E}{P_0}\right\}$ versus nondimensional parameter N given by $\left(2\frac{C_0 t_0}{B}.\tan\psi\right)$ which is obtained by using Selig model for following conditions:

(i) Static loading	$P=P_0$ at any time t	
(ii) Step pulse	$P=P_0$ for $t \leqslant t_0$	
	$P=0$ for $t > t_0$	
(iii) Parabolic	$P=P_0\frac{t}{t_0}\left(2-\frac{t}{t_0}\right)$	for $t \leqslant t_0$

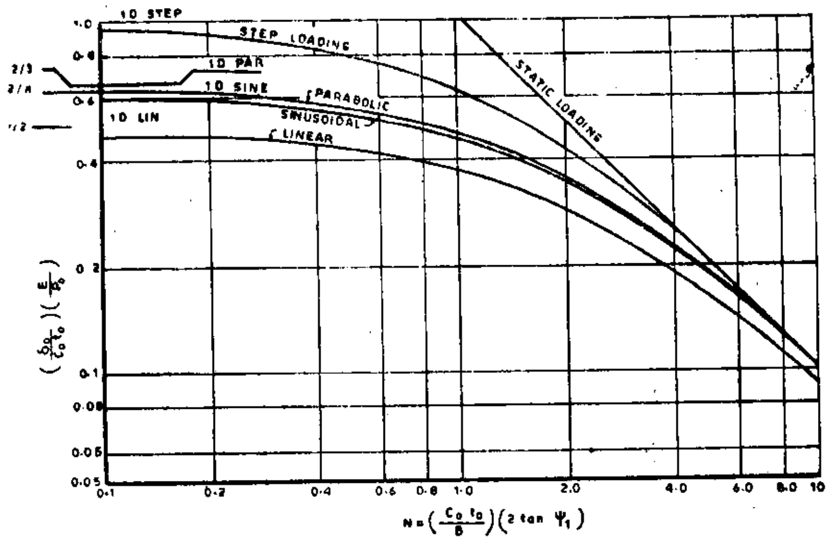

Fig. 3. Dynamic Non-Dimensional Displacement vs Non-Dimensional Factor, N

	$P=0$ for	$t > t_0$
(iv) Sinusoidal	$P=P_0 \sin\frac{\pi t}{2t_0}$	for $t \leqslant t_0$
	$P=P_0$	for $t > t_0$
(v) Linear	$P=P_0\frac{t}{t_0}$	for $t \leqslant t_0$
	$P=0$	for $t \geqslant t_0$

From this plot and using ψ equal to 35.5°, he concludes that (i) if $\frac{C_0 t_0}{D} > 3.5$, the effect of inertial stresses can be neglected and static and dynamic load-Displacement time relationship are one and the same, (ii) if $\frac{C_d t_0}{D} \leqslant 1$ the displacement pressure relationship can be predicted using simple one dimensional wave propagation theory proposed by him, (iii) if $\frac{C_d t_0}{D} > 1$ and $\frac{C_0 t_0}{D} < 3, 5$, the displacement pressure relationship should be obtained from three dimensional wave propagation theory,

where C_d=dilational wave velocity $= \sqrt{\frac{\text{Constrained Modulus}}{\text{Mass density}}}$

C_0=rod wave velocity $= \sqrt{\frac{\text{Young's Modulus}}{\text{Mass density}}}$

The value of $\psi=35.5°$ which he used in the above conclusions was obtained by him by fitting the static displacement of the pyramid by rigorous elastic theory and the experimental results.

After modification of Sung's theory to predict dynamic displacement produced by sinusoidal transient pulse, it was again concluded that, if $\frac{C_0 t_0}{D} > 3.5$, effect of inertial stresses can be neglected and static and dynamic load-displacement functions are one and the same. However, since, the initial condition for transient pulse and steady state condition are different, use of Sung steady state solution for transient pulse may not be justified.

METHOD BASED ON EQUILIBRIUM OF FAILURE WEDGE

This method assumes that the symmetrical or unsymmetrical failure wedge under static load conditions to hold good under dynamic loads as well, and the shear strength mobilized along that surface is given by :

$$\tau = C + \sigma \tan \phi$$

where τ=Shear Stress

σ=Normal Stress

C=Cohesion of tensoil

ϕ=Angle of shearing resistance

The soil mass contained in the rupture wedge together with the mass of footing is assumed to cut like a rigid body. Using priniciples of Mechanics, equation of motion is developed; solution of which gives the time dependent plastic displacement of the footing.

Wallace (1961) considers Terzaghi's symmetrical static failure wedge to be true for dynamic case (Fig. 4). The centre of logspiral lies on the line "a-d" or its extension. The

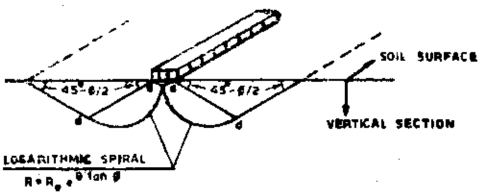

Fig. 4. Failure Surface

rupture surface leading to minimum bearing capacity under static loads is obtained by trial and error procedure. A triangular loading function constituted the disturbing force. Restoring forces acting on the failure wedge due to static resistance of soil, inertia force due to acceleration of failure wedge, displacement of centre of gravity of failure wedge and surcharge were considered. Equilibrium consideration of failure wedge yields a second order, nonhomogeneous differential equation, the solution of which gives time dependent plastic displacement. The results are plotted as a set of curve with non dimensional displacement versus nondimensional load duration. By comparing the results of his analysis with symmetric failure surface for cohesive soil with those Triandafilidis analysis using one sided semi-circular and Fellanious of failure surfaces, for the same loading conditions, he concludes that the symmetrical failure is more critical compared to one sided failure.

Triandafilidis (1961 and 1965) considered only cohesive soils because, the frictional component of the resisting force due to applied dyamic load becomes time dependent and hence difficult to get accommodated in the analysis. He considered semi-circular failure surfaces, with rectangular and triangular loading functions and a Fellanious failure surface with exponentially decaying loading function. Equilibrium consideration of failure wedge yields a second order, nonhomogeneous differential equation the solution of which gives time dependent plastic displacement. The results are plotted in the form of dynamic load factor *vs.* dimensionless time curves. This method yields some interesting results, *i. e.* displacement of footing under a rectangular loading function is greater than displacement of same footing under triangular loading function of same peak load and time duration, which is expected. The results have not been compared with any experimental results.

McKee (1962) assumes a failure surface (Fig. 5) similar to that proposed by Anderson (1956), wherein 'B' is footing width, 'D' is depth of burial of footing, P (*t*) is time dependent force per unit length acting on the footing, '*r*' is radius of failure circle and 'θ' is

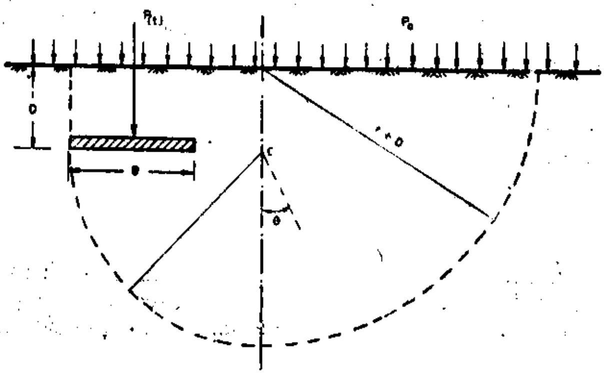

Fig. 5. Model for Dynamic Analysis

rotation of soil mass. Assuming that the radius of failure circle is known and that the shear strength along the failure surface can be incorporated in terms of an equivalent resisting force acting at the centre of footing, the equation of motion can be written as :

$$J\ddot{\theta} + R(\theta) = M(t) \qquad \ldots(10)$$

where J = Rotational inertia

$\ddot{\theta}$ = Angular acceleration of soil mass

$M(t)$ = time dependent actuating moments

The loading function is defined as :

$$P(t) = P_o \cdot e^{-\alpha t} \qquad \ldots(11)$$

where α = decay factor

From his analysis he concludes that increase in surcharge load (P_o) results in decreases of inertial effect, and increase in depth of burial result in increase of inertial effect. From experimental load time and displacement time record and using linear equation of motion

$$M\ddot{x} + R(x) = P(t) \quad \ldots(12)$$

where M = Equivalent mass

x; $\ddot{x}$ = Vertical displacement and acceleration functions of time ruptively.

$R(x)$ = Resistance function of vertical displacement

$P(t)$ = Applied vertical force as function of time.

He predicted $R(x)$ as a function of x. From these resistance displacement curves he concludes that the use of static resistance, directly or with simple modifications for a general dynamic load is inappropriate.

Chummar (1965), Prakash and Chummar (1967) proposed a method for predicting plastic rotational displacement (θ) when a horizontal dynamic load acting at a height 'h' from the base of the footing. In addition, the footing carries a vertical static load also. The failure surface assumed in this analysis is logspiral with centre of rotations at the corner of the strip footing. Using the principles of mechanics, a second order non-homogeneous differential equation is obtained the solution of which gives time dependent plastic rotational displacement (θ). The results obtained by using triangular loading function are plotted in the form of θ_{max} versus ϕ, C, B and t_d, and also T versus ϕ for various values of B, where θ_{max} is maximum rotational displacement, ϕ is angle of shearing resistance, C is cohesion, B is width of footing, t_d is the duration of transient load and T is natural period of the system. No comparison with experimental data has been attempted.

METHOD OF NON-DIMENSIONAL ANALYSIS

The main principle of dimensionless analysis is that a problem with 'n' variables defined in terms of three dimensional units (force, length, and time) can be stated in terms of $(n-3)$ nondimensional terms. However it is a difficult task in practical problem to identify the significant variables affecting the problem.

Jackson and Hadla (1964) considered following significant variables affecting the dynamic bearing capacity.

Symbol	Quantity	Dimensions
m	Mass of the load column of the dynamic machine	FT^2L^{-1}
P	Applied dynamic column load	F
t	Any time variable	T
t_0	Total pulse time	T
ρ	Mass Density of soil	FT^2L^{-4}
τ_f	Shear Strength (unconfined compression)	FL^{-2}
z	Displacement of the footing	L
b	Footing width	L

The non dimensional equation obtained by them is :

$$\frac{Z}{b} = \left(\frac{t}{t_o}, \frac{P}{b^2}, \frac{M}{b^3}, \frac{Pt^2}{bm}\right) \quad \ldots(13)$$

They carried out static and dynamic tests on plates of sizes 11.4 cm, 15.24 cm and 20.32 cms (4.5", 6" and 8") resting on three types of clays with un-confined strengths of 0.488 kg cm², 0.685 kg cm², 8.325 kg cm² (1.0, 1.4 and 1.7 kips/sq. ft). On analyzing the test results they obtained three relationships between non-dimensional terms. The first is between resistance parameter and the displacement parameter (Fig. 6a), the second is between strength parameter and the displacement parameter (Fig. 6b), and last one is between the strength parameter and the inertia parameter (Fig. 6c). These parameters are explained in the figure itself.

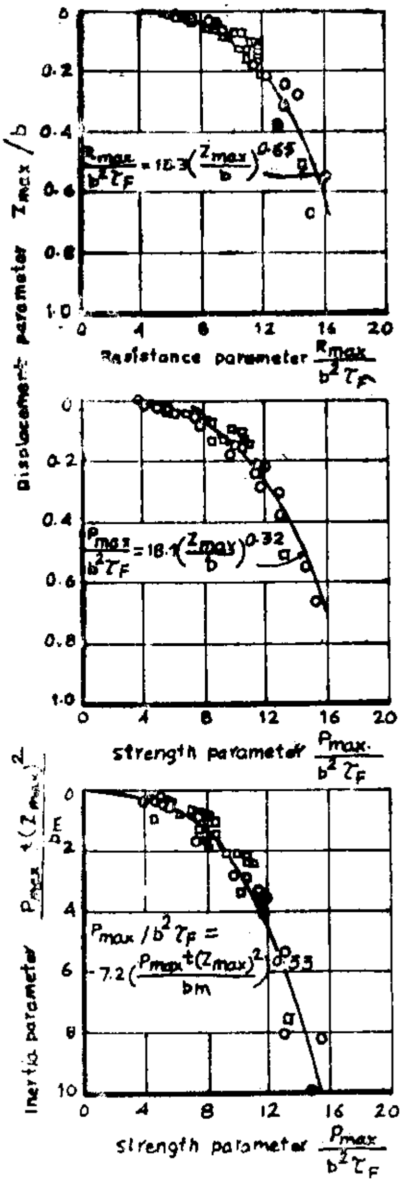

Fig. 6. Plots of Nondimensional Quantities

Hadla (1965) conducted additional tests using plates of sizes ranging from 5 to 16 inch, to verify these non-dimensional relationship when the independent parameter were not related by scaling rules. His results and results obtained previously, are shown in Fig. 7. Fig. 7 shows results plotted on log scale using his and others results. He obtained empirical relations between non dimensional parameter given by :

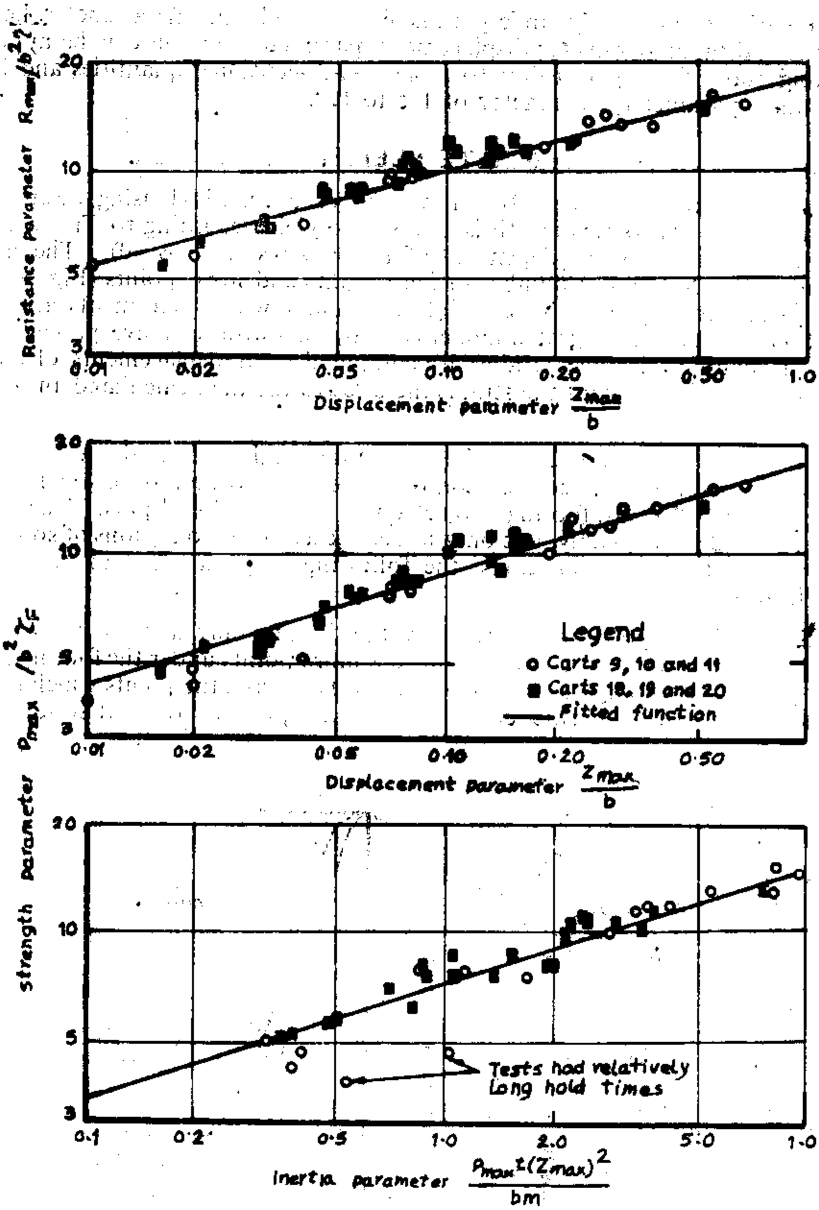

Fig. 7. Logarithmic Plots of Nondimension Quantities

$$\frac{R_{max}}{b^2\tau_f}=18.3\left(\frac{Z_{max}}{b}\right)^{0.26} \qquad \ldots (14)$$

$$\frac{P_{max}}{b^2\tau_f}=18.1\left(\frac{Z_{max}}{b}\right)^{0.32} \qquad \ldots (15)$$

$$\frac{P_{max}}{b^2\tau_f}=7.2\left[\frac{P_{max}\, t\, (Z_{max})^2}{b}\right]^{0.32} \qquad \ldots (16)$$

By comparing static and dynamic parameter and displacement parameter, he observed that dynamic resistance parameter-displacement parameter relation can be approximated by converting static footing test data to same non-dimensional quantities and increasing static resistance parameter, by a factor of 1.5 to 1.9.

METHOD BASED UPON DISCRETE ELEMENT ANALYSIS

Hoeg and Balakrishna Rao (1970) have proposed a method using discrete element analysis. Here, the elasto plastic soil is assumed to yield according to Tresea's criterion. Soil strength is assumed to be independent of normal stress (*i. e.* $\phi=0$). The mechanical model divides the soil into a number of mass points and stress points (Fig. 8) All the prescribed forces are applied at mass points and unknown displacements are calculated from equations of motion integrated numerically by stepwise recursive process. For any increment of loading, average strains are computed from displacements of mass points around the stress points. From the strains the stresses are then calculated into forces on mass points.

They compared the computed results with those obtained from the tests conducted at W.E.S. (13). Fig. 10 shows the comparison of the computed and observed response of a 15.25 cm × 30.5 cm (6" × 30") footing subjected to a triangular load-pulse. The results may be considered to tally reasonably, but only under the assumed conditions of soil properties. Based upon their analytical solutions, the following observations can be made for a surface strip-footing.

(i) The first yielding of soil occurs below the footing only after a certain minimum stress is reached. With continued stress, more and more points under the footing attain yield stress. As stress wave propagates and as load intensity decays, the points which had reached yield stress will return to elastic condition. In this process, the soil suffers permanent deformations.

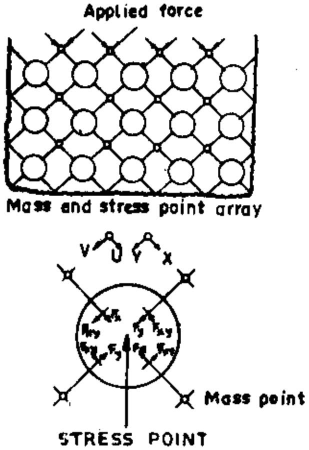

Fig. 8. Mathematical Model

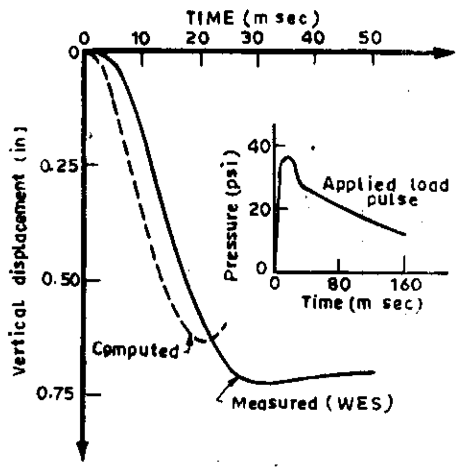

Fig. 9. Comparision of Computed and Experimental Results

(ii) If the soil is not sensitive to strain rates, the strip footing cannot carry appreciable transient over load compared to static bearing capacity, unless the load pulse has a very short duration, or large plastic displacements are acceptable.

(iii) Peak displacements decrease appreciably with increasing footing width, upto a critical width for a given load-time function per unit length of the footing. For any further increase in the width of the footing, there will not be a corresponding appreciable decrease in the peak displacements (Fig. 10).

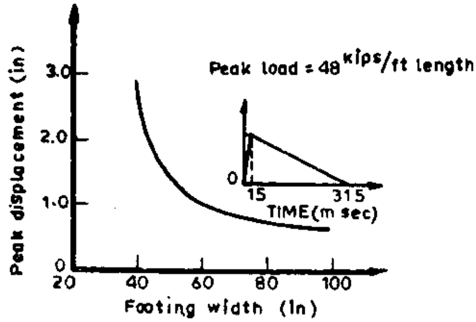

Fig. 10. Influence of Footing Width

EXPERIMENTAL WORK

Experimental work is required to compare the predicted results by proposed theories with the test results. They not only help in assessing the validity of a theory but also in formulating better theories. The set up for dynamic bearing capacity testing generally consists of a tank to contain soil under consideration compacted to desired density and concrete or metal blocks with roughened bases form the footings. Transient loads are produced either by dropping weights or through electrically and pneumatically controlled systems. The latter is having better control over loading. Applied loads and resultant deformations are measured with suitable transducers. In studying the failure surface profile in dynamic tests, two dimentional tests are also done with box having transparent sides.

Selig and McKee (1961) conducted tests on Ottawa sand. Annular drop-weights of 5·08 kg. (11.2 lbs.) falling through various heights were used to produce transient loads. Aluminium blocks represented footings. Both two and three dimensional tests were tested. Their two dimensional dynamic test results indicated that chances of symmetrical and one sided failures were equal with central loading. For symmetrical failure, the shapes were similar to Terzagi's static failure. For one sided failures, it was roughly log spiral. In the three dimension tests observed settlement of footing was linearly proportional to energy of impact for any footing. But for a given energy of impact and shape of footing, settlement decreased rapidly with increasing footing size.

Cunny and Sloan (1961) have reported an equipment for transient loading with rise time of 3 to 150 milli seconds, dwell time from 0 to 1 sec. and decay time from 23 m. s. to 10 seconds. Nitrogen gas under pressure controlled by solenoid valves was used to produce desired transient load. They have reported that it has worked satisfactorily during the dynamic loading tests they conducted to study its performance.

Fisher (1962) conducted tests on lake sand, using a 1·56m^3 (55 cft.) capacity plywood box whose sides were reinforced with wooden planks. A pneumatically electrically controlled loading equipment designed by Illinois University was adopted. During testing, a rise times form 26 to 64m. sec. and duration of loading from few m. sec. to many hours were used. Test plates from 15 25 cm to 22·87 cms (6" to 9") were used as footings.

The test results indicated that dynamic failure surface encloses larger volume compared to static one and this varies with the type of soil and magnitude and duration of loading. Density of soil after testing will be smaller in upper layers compared to that of lower layers. The settlement velocity is constant for the first 10 to 15 m. sec. The recovery of settlement is 6.5 to 20% for square footing compared to negligible quantity with long footings for tests with short duration of loading. Restraints on the movements of footing plate due to the types of connection between plate and loading piston, and short size of box compared to sizes of footings used resulted in over estimation of bearing capacity. As such good comparison between theory and test results could not be made.

Shenkman and McKee (1962) used a pressure vessel of 122·0 cms (4′) diameter with an air tight cap. Suitable openings in cap accommodated the loading and measuring devices. There was provision to apply surcharge load around the footings by pressuring a water bag on soil surface. Maximum surcharge (design) pressure was 21·1 kg/cm² (300 psi.) But due to leaks, this could not function properly. Instead of controlling applied load, they subjected the footing to uniform rate of settlement, by a constant drive motor and gear arrangement and corresponding loads were measured by suitable transducers. They conducted three dimensional static tests on 10·16 × 10·16 cms (4″ × 4″) and 5·08 × 5·08 cms (2″ × 2″) plate with and without over pressure, three dimensional dynamic tests on 10.16 × 10.16 cm (4″ × 4″) plate with and without over pressure and two dimensional static and dynamic tests on 7.62 × 10.16 cms (3″ × 4″) plate

Their test results indicated that displacements were linear with loading time for short duration of time at the starting. Also, amount of displacement decreases with increasing surcharge pressure at footing level as can be expected. The behaviour of inclined footings was also studied. But since they all invariably tilted resulting into further eccentricity of loading, improvements in loading arrangements were warranted. Due to leaks, high surcharge pressure were not practicable.

An important finding was that static resistance was entirely different from the dynamic one. Comparison of experimental results with those predicted with theory, indicate that the treatment of dynamic bearing capacity analysis at par with the static analysis is advisable only far dynamic loads with sufficiently larger rise time compared to peak load reached. For the more rapid rates of loading a new approach considering compressibility of soils need be designed.

Vesic, Banks and Woodward (1965) used dense sand in a steel box of 127 × 127 × 177.8cm deep (50″ × 50″ × 70″ deep). Rates of displacements from 2·54 × 10^{-6} cm/sec. (10^{-6} inch/sec.) to 0.25 cm/sec. (0.1″/sec.) were used with a maximum loading capacity of 204,120 kg. (450,000 lbs). Rigid circular footings of 10.16 cms. (4″) diameter were used. As per test results, ultimate loads are reached approximately at displacements 8% of the width or diameter of the footing. With increased loading velocity, soil resistance decreases in general and more so for saturated sands than for dry sands. (This contradicts findings in triaxial tests, where resistance of soil increases with increased rate of strains.) Starting from static case, the bearing capacity at first decreases with loading velocity till

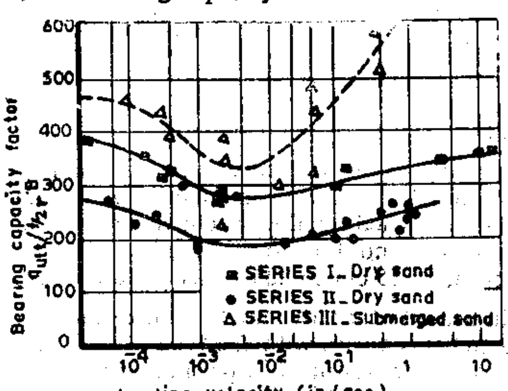

Fig. 11. Bearing Capacity Factor VS Loading Velocity

the velocity is equal to 0.00508 cm/sec (0.002"/sec.) and then starts increasing, more rapidly for saturated sand due to negative pore pressures resulting from dilation (Fig 11). This, they explain, is because there is enough time for soil particles to move in the direction of least resistance under slow loads leading to definite bearing capacity, and as the rate of loading increases due to shortage of time line of least resistance can not be followed, and as such any other type of movements must lead to larger bearing capacity. Another interesting observation was that for slow rates of loading velocities, one sided failures were common, for medium rates general shear failure and for very high rates local punching shear failure was observed. The reason for the punching shear they say is that under very high rates of loading there is no time for the soil out-side the plan area of the footing to share the load coming from footing to soil below footing and as such severe compression of soil occurs.

COMPARISON OF THEORIES

Most of the initial theories were based on equilibrium of failure wedge. This was natural because investigators to extend principles of analysis for bearing capacity in static conditions to dynamic case. The basic assumption of this theory is that a definite rupture surface of known shape developes at failure. They were encouraged to see from experiments the clear one sided and two sided failure surfaces under dynamic loads also. The method is very simple and clear in giving a mechanistic picture of the soil foundation system behaviour.

But it has disadvantages also. Firstly, there is no definite size of failure surface even if its spiral shape is accepted. Fisher has clearly shown that the failure surface under dynamic loads increases in size with rate of loading. Moreover, with increasing footing displacements, successively new failure surfaces are developed. This proves convincingly that static failure surface do not hold good for dynamic analysis. Even otherwise till a method is evolved to correlate final size of failure surface to rate of loading, this approach cannot be recommended at all. Moreover all the dynamic loads and types of soil do not lead to rupture surfaces. An additional and challanging difficulty associated with this theory is the angle of shearing resistance ϕ with reference to dynamic loads, which is expected to be greater than that in static case. There is no realiable method for its estimation. Since the applied load is time dependent, the frictional component of soil resistance is also is likely to be time dependent, making its consideration all the more complex. Still another draw back with them is that they neglect the compressibility of the soil which is very important under dynamic loads. So it is no wonder that the predicted results compared poorly with the test results. It may be reasonable only for slower rates of loading near to static loadings, but not for rapid rates of loading.

Wave propagation theory ideally suits to consider the compressibility of soil. By use of well known Donnel's principle, soils can be brought within the reach of such analysis. For adopting this method elastic properties (i. e. resistance function) of the soil which is likely to vary with rate of loading must be known. But replacement of real soil by an elastic half space model which is nonlinear and inelastic in nature and strain rate dependent is not justified. Moreover, the method considers only geometric damping and neglects the material damping, leading to conservative estimates of dynamic bearing capacity.

Carrol is the only person who reports that the predicted results agree with test results with usual engineering approximations. But even here the scatter was quite appreciable.

Spring mass dashpot systems have grown to be more popular approach than the other two in recent developments even though no successful reports have come forward. This ideally suits to represent soil-footing system. The spring represents elastic property dashpot, the damping property and the mass of footing and soil vibrating with footing as the lumped mass. The finding that the soil mass vibrating with the foundations is small and its effect on the dynamic bearing capacity is negligible has made an easy choice of mass 'm' to be simply the mass of foundation. Linear spring with single degree freedom were tried by Landale and Fisher. Although Fisher's experimental curves could be approximately matched by the response of the S. D. F. Model, the parameters of the model were found to be functions of the details of transient loading as well as of soil properties and footing characteristics. Unfortunately the nature of these functions has not been obtained.

It is an advantage that both geometric and material damping could be accommodated by suitable dashpot. The Nonlinear elastic behaviour can be represented by a nonlinear spring. Work in this direction are well in progress though in connection with some other investigations like response of rigid footing under steady state hormonic vibrations conveniently extended to this field also. But here again one must know resistance functions of soil under design dynamic loading and its damping characteristics.

A common feature of the wave propagation method, Spring-mass Dashpot method, and the method based upon the discrete element analysis is that they do not define failure. The dynamic bearing capacity is to be determined from the settlement considerations only.

The discrete element method is a more powerful tool for analysis of such a complex problem. Hoeg and Balakrishna Rao have used it with certain degree of success. The theory predicts that there will be no clear cut failure surface developed at any stage for transient loads. On the other hand certain points under footing attain yield stress for certain duration and spring back to elastic state as the stress wave propagates and the loading function alternates. During this process, yielding stress travels to different points from time to time. Hence the assumption of a defined failure surface of Engineering approach is not justified. The theory also points out the inadequacy of elastic wave propagation theory, which does not take into account the plastic deformation caused by yield stresses.

CONCLUSIONS

A critical review of the published literature on the dynamic bearing Capacity of Soils under transient loads has been presented in this paper.

With the present day knowledge, there is no single analytical or experimental approach, which can give even a reasonable estimate of bearing capacity of soils under transient loads. The engineering approach method is suitable, perhaps for very slow rates of loading, nearing the static loading, which are likely to yield rupture surfaces and where the compressibility of the Soil is likely to be less important. For higher rates of loading they are not suitable with the present day knowledge of soil mechanics and soil dynamics.

The wave propagation theory and the spring-mass-dashpot method appear quite promising, particularly the latter, if it is possible arrive at the necessary soil constants. An extensive study on at least some simple soils and loading conditions is required even for conditionally agreeable theory.

Analytical methods based upon more powerful numerical techniques are surely the most promising for such complicated problems. Further improvements on these lines are likely to be much more useful in solution of this problem.

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