Computer Analysis of Steel Storage Tanks and Foundations in Soft Soils

by

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Introduction

THE soft *clayey soils are highly compressible and possess low strength,* as in the case of Bombay marine clays. The foundations on such soils cannot transmit high intensities of load, and they do undergo large settlements. In such cases, the costly methods for improvement of the bearing capacity, such as preloading with use of sand drains, or use of piles become inevitable. The steel storage tanks as used in petro-chemical industries are usually placed on such sites. In this paper, two aspects of the problem of steel storage tanks, and their foundations in such soils, are presented; which when used in appropriate situations, may eliminate the costly methods. These two aspects are (*a*) cost analysis of the structure, taking into account the foundation also, alongwith the supper structure, (*b*) step-loading method, during the first filling of the tank.

A generalized computer programme has been developed to predict the safe rate of loading of such tanks, resting on soft clayey soils.

Cost Analysis

Generally, the cost analysis available is pertinent to the superstructure only. The foundation costs are not usually included in this analysis [Aries and Newton, (1955); Brownell and Young (1959)]. Also it is reported that the foundation cost in such cases, may even exceed the cost of the superstructure [Aldrich (1957); Brownell and Young (1959); Roberts (1961)]. So it is thought, that if the diameter of the tank is increased somewhat more than that diameter, which is obtained by analysis for optimum tank dimensions from economic point of view (considering superstructure only), the reduction in intensity will occur. The reduction in intensity will decrease the foundation cost, and in some cases, it may completely eliminate the piles, in which case, the cost of the foundation will considerably decrease.

In many instances, it may not be possible to change the dimensions of the tank. In that case this approach will not help the designer to ease his problem. But in cases, where it is possible to modify the tank dimensions, the feasibility should be carefully considered, before going to tackle the actual foundation problem.

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This paper (modified) was received on 28 May 1973. It is open for discussion up to April 1974.

COST OF THE SUPERSTRUCTURE

In view of the fact that sufficient data regarding relative costs of different types of foundations are not available, the analysis is restricted to only in showing reduction in intensity corresponding to increase in cost of the superstructure only. The design of the tank is done in accordance with IS: 803-1962; with assuming different values of height/diameter ratio, ranging from 0·1 to 2·0 in steps of 0·1. While calculating the cost, the following assumptions are made :

(1) The cost is directly proportional to the weight of the tank in kg, the constant of proportionality being C_1 (at ground level).

(2) For every three metres rise after the first three metres, the cost increases by 10 percent of that cost for the earlier interval of 3 metres. So if F_1 is the height factor, which when multiplied to the cost of the roof takes care of the increase in cost due to height, then

$$F_1 = (1{\cdot}1)^{n-1} \qquad \ldots(1)$$

where n=height of the tank in metres/3. Similarly if F_2 is corresponding factor for the shell, then

$$F_2 = \sum_{n=1}^{n} (1{\cdot}1)^{n-1}/n \qquad \ldots(2)$$

(3) The weight of the annular ring, etc., is assumed to be 0·2 times the weight of the bottom plates.

(4) The weight of the columns, purlins, etc., is assumed to be 0·3 times the weight of the roof plates.

THE RELATION BETWEEN COST AND INTENSITY OF LOAD

The cost is calculated in terms of C_1, for different values of h/d. There is only one particular ratio of h/d at which the cost is minimum. This value of the cost is taken as the base cost $C(K)$MIN. The subsequent costs are calculated as the percentage of this base cost. The intensity corresponding to $C(K)$MIN is also taken as base, and other intensities are calculated as the percentage of this intensity.

A computer programme was prepared so that once the capacity is given, the above analysis will be carried out automatically. The block flow diagram of computer operations is given in Figure 1. The costs, calculated for some 30 capacities ranging from 10 m^3 up to 10,000 m^3, are shown graphically in Figure 2. The cost is represented in terms of C_1. The trend is similar to six-tenth rule [Aries and Newton (1955)].

Figure 3 shows for a tank of capacity of 600 m^3, the relation between increase in cost due to change in h/d ratio, and corresponding change in intensity. For example, it is seen that for 1 percent increase in cost, due to increased diameter, the intensity of load has decreased almost by 8 percent; for 10 percent increase in cost, the intensity has decreased by 37 percent, and for 30 percent increase in cost, the intensity has decreased almost by 50 percent.

This clearly shows that while designing the tank, the proportions of the tank must be fixed, not only by doing cost analysis for the superstructure

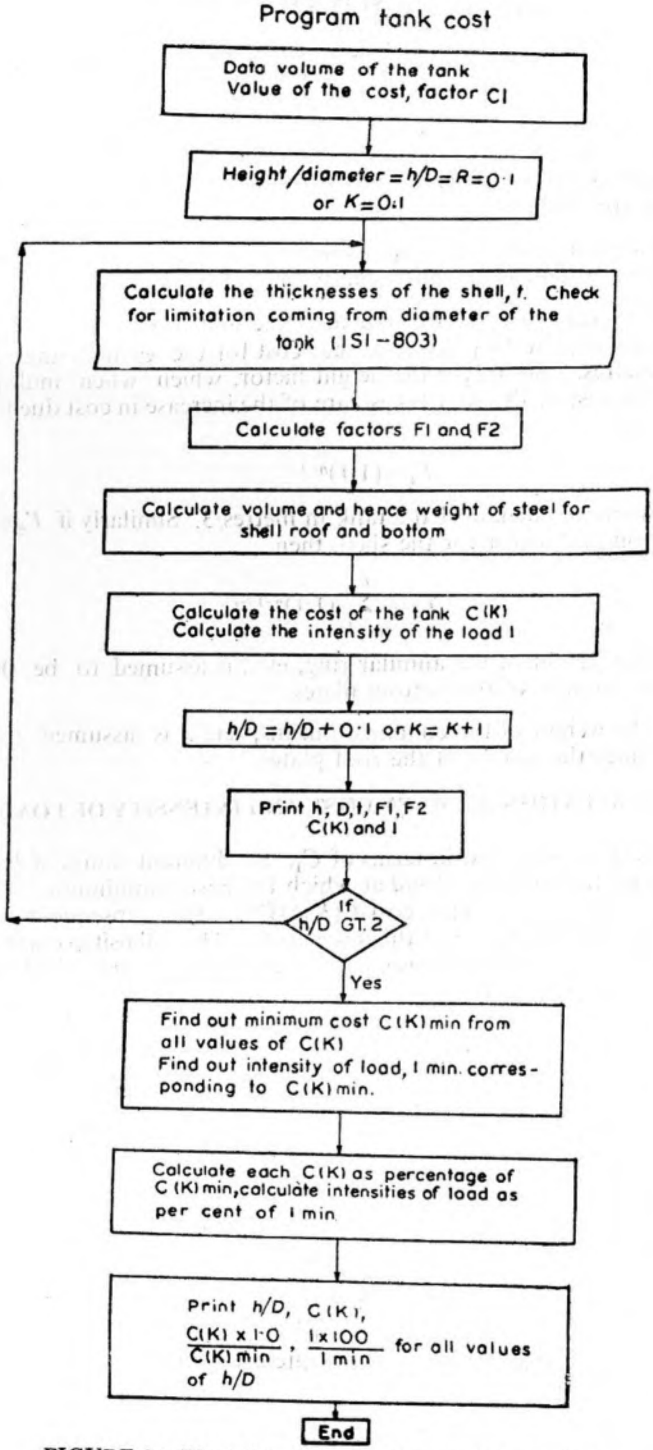

FIGURE 1 : Block flow diagram of computer operations.

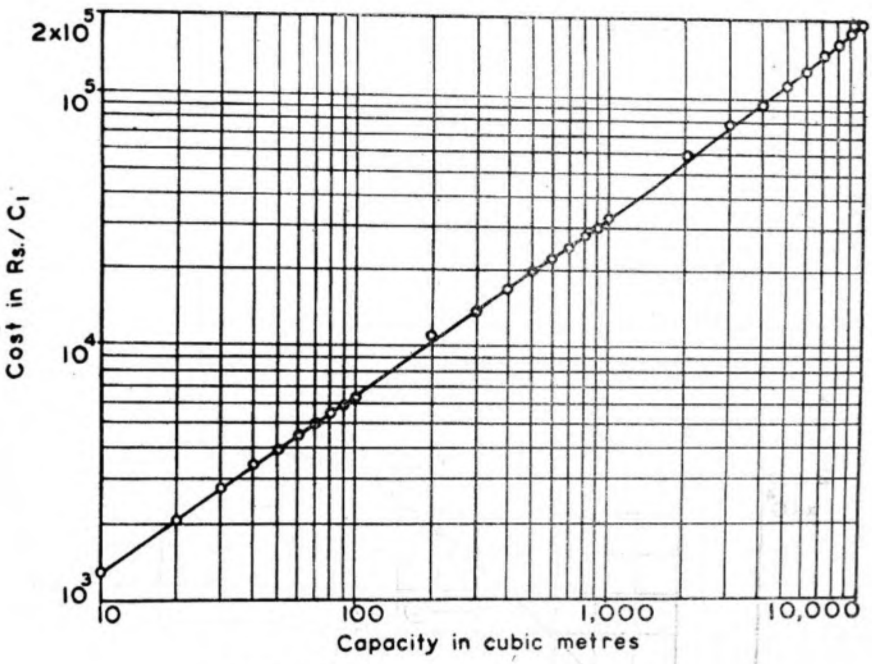

FIGURE 2 : Cost versus capacity of the tank.

alone, but by considering the foundations also. This is especially true, when the tanks are situated in deep soft soil deposits, where the cost of the foundation is equal to, or, some times greater than the cost of the tank.

The Step Loading Method

The improvement in the shear strength of the soil, due to consolidation is made use of, in the step loading method, of improving the bearing capacity of the soil. The method is exactly similar to the method of using pre-load. Since comparatively large settlements of the tank bottom are allowable [Carlson and Fricano (1961)] the tank itself is used here as the pre-loading device.

The maximum possible load, assuming a certain factor of safety against shear failure, is applied in the initial stage. Then the soil mass is allowed to consolidate, under the load. After some time, say a few weeks, the improved strength is calculated, and based on this strength, the maximum possible extra load that can be allowed, is applied. The procedure is repeated until the desired load comes on the foundation. The method is economical and can be employed whenever the corresponding time required to complete the step loading is available.

A systematic procedure to obtain the curve of load intensity versus time is given below. A computer programme was evolved, to carry out the numerous calculations. A block flow diagram of computer operations is given in Figure 4. The steps are as follows :

(1) The maximum load that can be applied initially is calculated, taking into account the desired factor of safety.

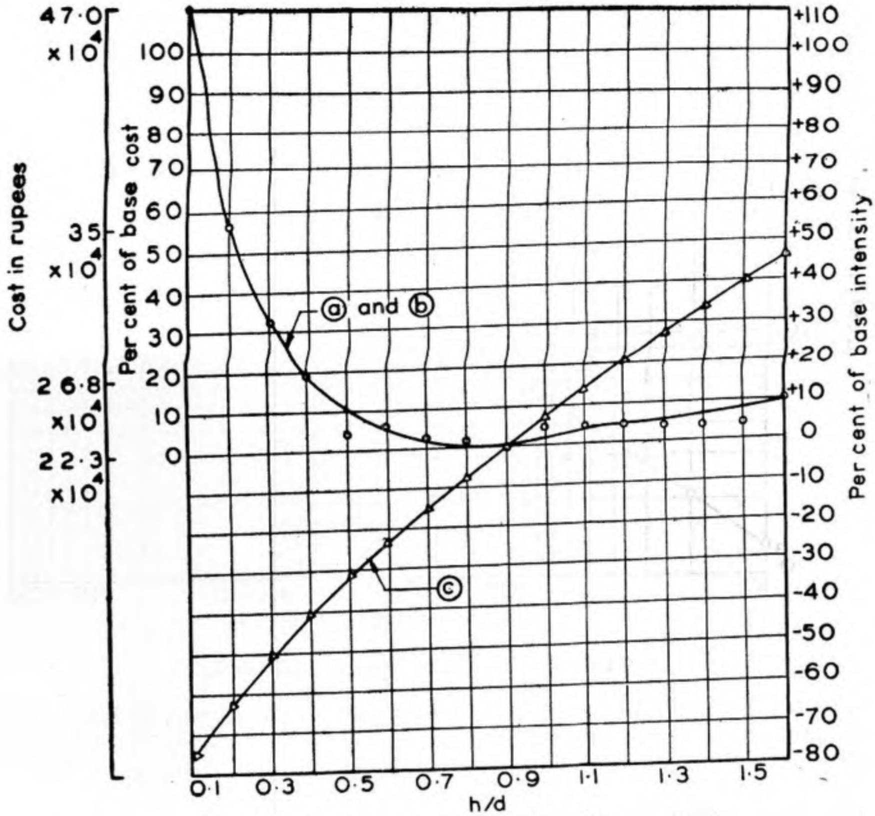

FIGURE 3.

(2) The consolidation process starts as soon as the load is applied. A proper time interval is chosen. The pore pressures are calculated at the end of every time interval; and the percent consolidation is also calculated.

(3) A certain value of percent consolidation is chosen, at which the improved strength is calculated.

(4) The new load is calculated, by first finding out the minimum factor of safety for unit intensity of load, and then back calculating the load for the desired factor of safety.

(5) It is checked, whether the desired load has come on the foundation or not. If not, then steps (2) to (5) are repeated until the desired load comes on the foundation.

EXPLANATION OF THE SYSTEMATIC PROCEDURE

Steps (1) and (4) : The $\Phi_u=0$ case is assumed. The failure surface is assumed to be circular arc. First unit intensity of load is assumed. From any point *P*, 100 trial failure circles are considered, such that their intercepts

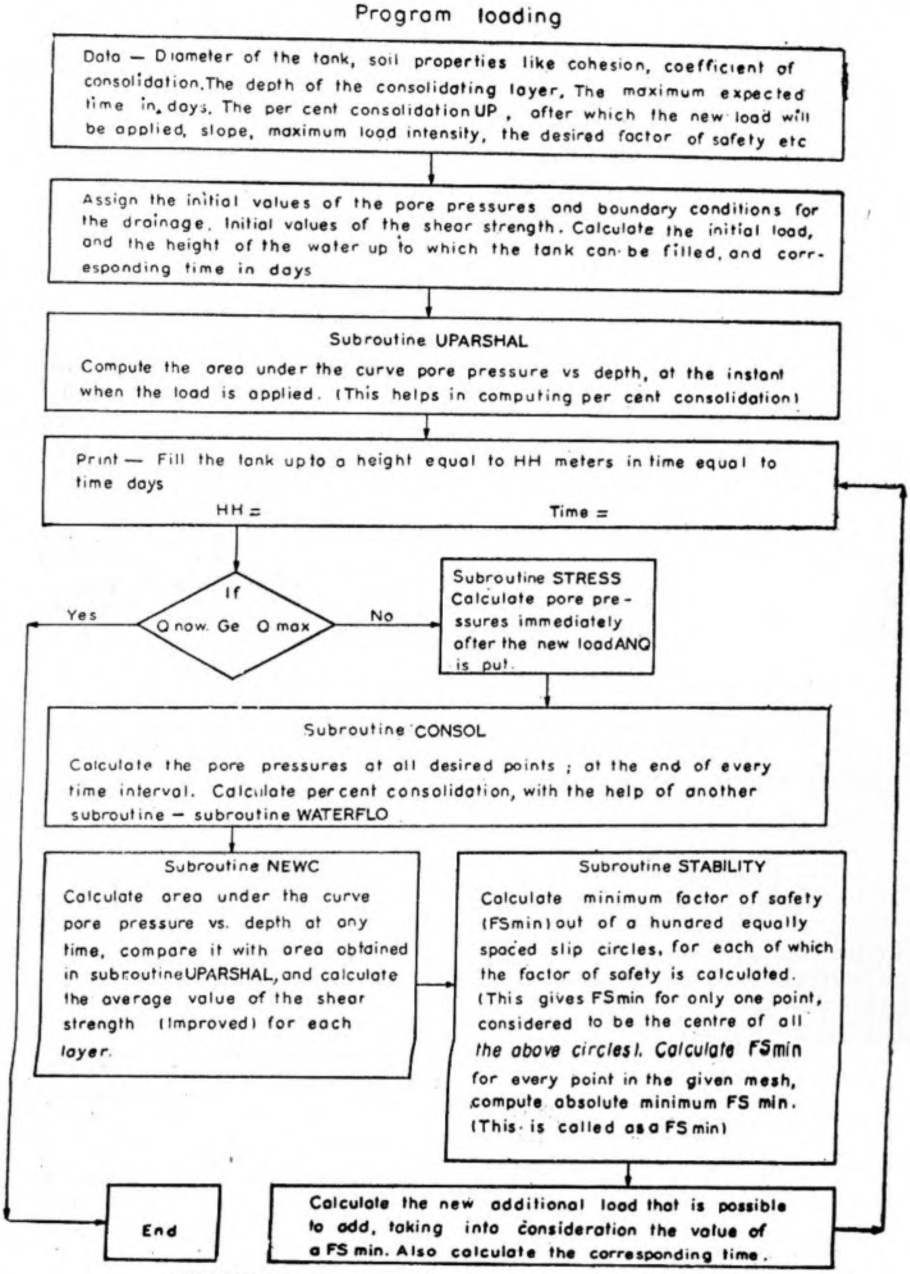

FIGURE 4 : Block flow diagram of computer operations.

on the foundation are equally spaced, from each other. A mesh is chosen for point P, such that the absolute factor minimum of safety falls within that mesh.

The analysis is done as for the strip footing. From the reported observations of the tank bottoms deformations [Carlson and Fricano (1961); Saurin (1958)] it is assumed that the tank fails by local shear failure, involving only a part of the loaded area. So the factor which takes into account the circular shape of the footing is reduced from 1·3 [Terzagi and Peck (1967)] to 1·0 only. This is an assumption on safer side. Actually it varies from slightly greater than 1 to 1·3, as the part of the loaded area causing failure increases.

From the obtained absolute minimum value of the factor of safety for unit load intensity, the actual load intensity is back calculated. When the strength is varying with depth, the depth is divided into 8 equal layers, and average strength for each layer is calculated. This average strength is considered for that part of the failure arc, passing through that particular layer.

Step (2) : The problem of consolidation is solved by numerical method, by writing down the one dimensional consolidation equation, in a finite difference form. The value of finite difference operator was fixed at 1/6 [Scott (1965)]. The numerical technique is very useful for problems with non-uniform initial pore pressure distribution, for load which is changing with time, and for complicated boundary conditions.

The depth of the consolidating clay layer, is assumed to be equal to or greater than the diameter of the tank. Only single vertical drainage is assumed. The C_v is assumed to be constant. The horizontal displacements are ignored.

INITIAL PORE PRESSURE DISTRIBUTION

The computer programme evolved, is very flexible and can take any initial pore pressure distribution. In the present case, the initial pore pressure distribution is assumed to be exactly similar to the Boussinesque's stress distribution, below the centre of the tank.

The finite difference equation is solved for the points below the centre of the tank. The consolidating depth is divided into 8 equal layers. The points which are below the centre of the tank and are also on the boundaries of these layers are chosen as nodal points. The pore pressures at these nodal points are calculated at the end of every time interval.

Step (3) : A certain average percent consolidation $\overline{U}$ (*UP*) is chosen for convenience. At the end of every time interval, from the values of the pore pressures at nodal points, the average percent consolidation is calculated. It is checked if this is equal to or greater than the chosen value of *UP*. If so, then the new values of shear strengths are calculated at all nodal points.

It is known that, in general, the clayey soils derive their strength due to the pressure to which they have been consolidated. The reported curves of effective stress versus shearing strength were used as a guide. [Figure 5 after Subrahamanya (1969)]. An ideal straight line relationship is assumed between the consolidating pressure, transferred to the soil mass versus undrained shear strength. The relationship is defined as

$$\text{SLOPE} = \frac{C_{u_2} - C_{u_1}}{CP_2 - CP_1}$$

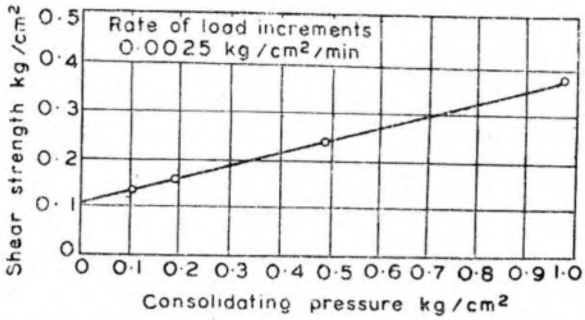

FIGURE 5 : Shear strength consolidating pressure relationship from squeeze test (After B. Subrahmanyam).

where CP_i = effective consolidating pressure in kg/cm²

C_{u_i} = values of undrained shear strength corresponding to CP_i in kg/cm².

The value of SLOPE is assumed to be 0·3. A uniform undrained shear strength is assumed initially. However, the computer programme is flexible enough to take any initial shear strength distribution along the depth.

The values of improved undrained shear strength are calculated as :

The new value of C_u for ith layer

= Initial value of C_u before the load is applied

+ SLOPE × New average effective stress.

The new average effective stress is calculated with the assumption that the stress transfer to the soil grains, in a particular layer during consolidation, is directly proportional to the average degree of consolidation for that layer.

Analysis

From the analysis of the failure circles, it was seen that the maximum depth up to which the most critical circle will reach is equal to the diameter of the tank. For the cases where the undrained shear strength is uniform, or is changing with depth due to improvement is shear strength due to consolidation, as in the present case. So the depth of the consolidating layer, which is of interest from stability point of view, in assumed to be equal to the diameter of the tank. Since the depth of the consolidating mass is connected with the diameter of the tank, the ultimate bearing capacity factors for unit load become independent of the diameter of the tank; at any stage of step loading.

So if the step loads are expressed against the time factor T, the solution is independent of the actual diameter of the tank, but depends only on the strength properties of the soil. This aspect was checked from the results of the analysis for 10, 20 and 30 m diameter tanks.

In Figure 6, the ultimate bearing capacity factor versus T is shown. As the time advances, the step loads become smaller and smaller. The

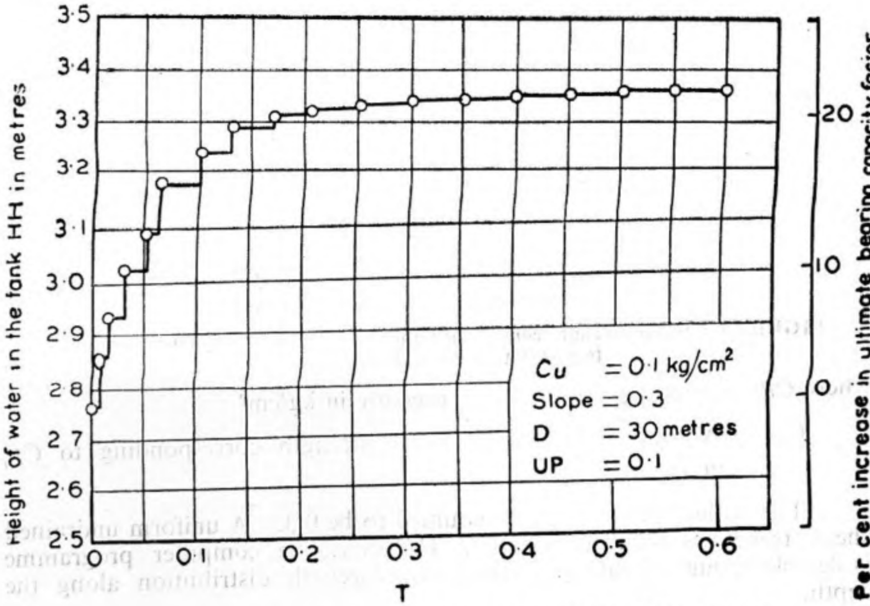

FIGURE 6 : Height of water versus time factor T (Single Drainage).

time, up to which the step loading method can be carried out, will depend on the available time, and the economic feasibility of affording the waiting time.

The Rate of Loading

A certain value of *UP* was chosen in the calculations; for convenience. Let the value of *UP* be 0·3 ; let it be that after the initial load is applied, the 30 percent consolidation is over after say 50 days. The shear strength of the soil is improving at every instant, after the initial load is applied, even though, for numerical analysis, it is considered that the strength improvement takes place only at the end of 50 days.

So if use is made of the improved strength, not after 50 days, but immediately as soon as it is gained, it is possible to fill the tank earlier. To analyse this aspect the value of *UP* is changed from 0·1 to 0·5 in steps of 0·2. The ultimate bearing capacity factor versus time factor *T* is plotted in Figure 7.

In the limit it can be said, that continuous loading of the tank at a rate, which may not be constant, is the best and most economic solution. The approximate continuous curves obtained from Figure 7 are given in Figure 8. This procedure of drawing continuous curves is very useful in the solution for the case, where the thickness of the clay layer is smaller than the diameter of the tank.

The Solution for the Case where the Thickness of the Clay Layer is smaller than the Diameter of the Tank

The solution is obtained only for the cases, where the clay layer is lying at the surface of the ground. Generally the thickness of the clay

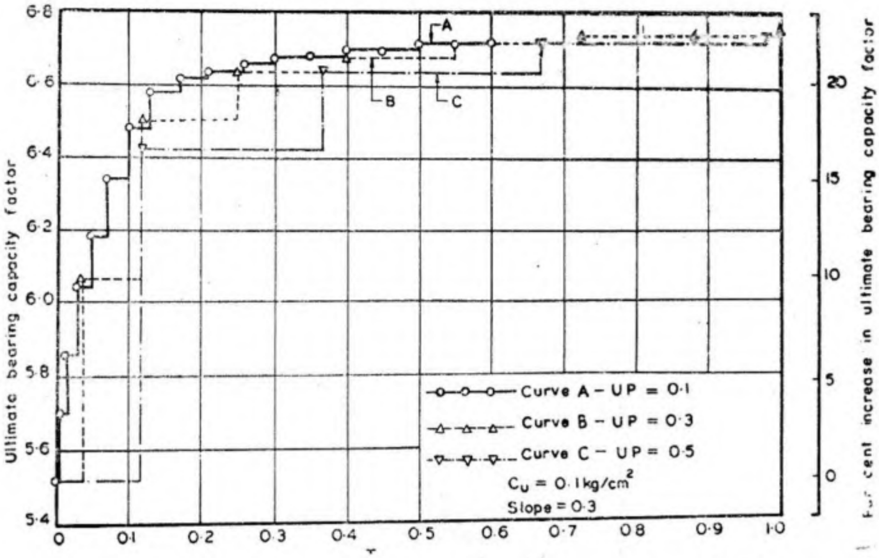

FIGURE 7 : Ultimate bearing capacity factor versus time factor T (Single Drainage).

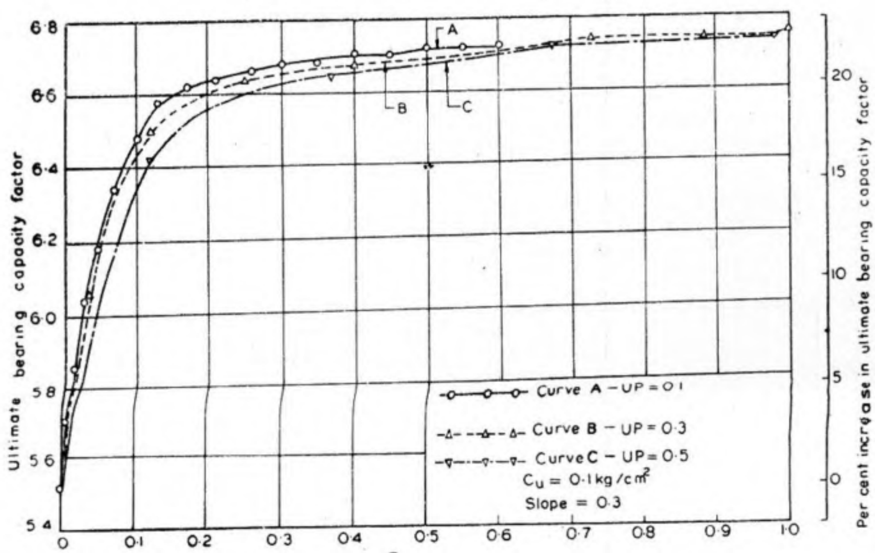

FIGURE 8 : Ultimate bearing capacity factor versus time factor T (Single Drainage).

layer is limited, and is smaller than the diameter of the tank. The problem can be solved with the available data, such as represented in Figure 6. It is clear from Figure 6, that, as the diameter of the tank increases, the time required for the completion of the step loads increases considerably, such that the solution becomes non-practicable. But in cases, where the thickness of the clay layer is smaller than the diameter of the tank, the step loading method can be satisfactorily applied, since the time required for completion of the step loads decreases considerably for thin clay layers, It is assumed that for thinner layers, the failure is a local failure, and it occurs in the zone of the thin clay layer. It is obvious, that analysis is done and step loads are calculated on the basis assuming $Z = d$ will prevent the local failure. The results are represented in Figures 9 and 10. The time factor T is calculated on the basis of length of the drainage path being equal to D, the actual diameter of the tank.

If the clay layer is thin, the initial pore pressure distribution is not as assumed in the solution, but tends to be near to the uniform initial pore pressure distribution. In that respect the analysis is approximate one only. But from the reported curve of $\bar{U}$ versus T for different initial pore pressure distribution [Taylor (1958)] it can be concluded that the approximation is quite reasonable one.

It should be remembered that all the discussion so far in this paper is for uniform initial strength distribution.

GENERAL CONCLUSIONS

(1) For cases such as non-uniform initial shear strength, which is also changing with time, and for non-uniform initial and intermediate pore pressure distribution, and where the failure slip circle is also obtained by trial and error method, the closed form or analytical solution is either difficult to arrive at, or if available, is cumbersome to use. In such cases a flexible computer programme can be prepared to solve such complicated problem. In the present case for each step loading, around 5000 trial failure circles were tried. The time required for the calculation of 18 load steps was approximately 5 minutes of computer CDC-3600.

(2) It is important to note that the solution obtained as given in Figure 6 is independent of the diameter of the tank. The solution only depends on

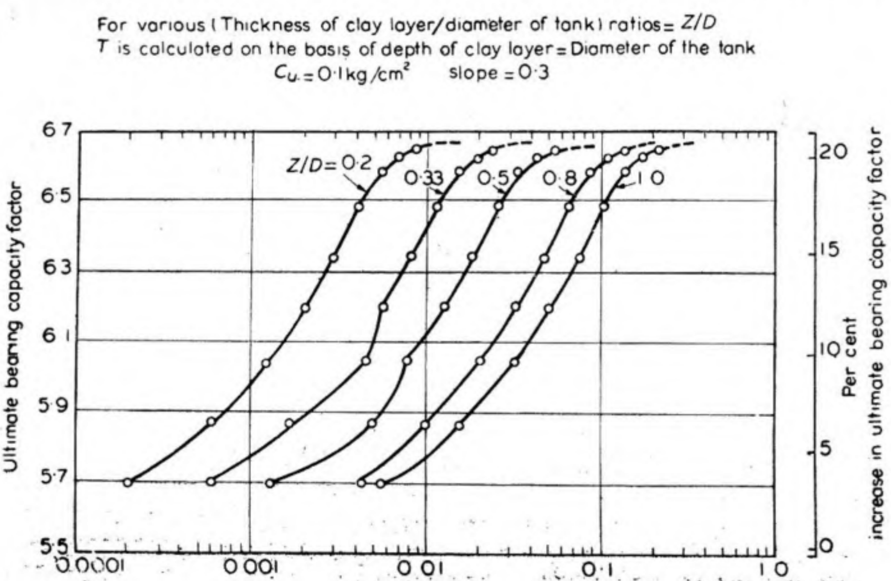

FIGURE 9 : Ultimate bearing capacity factor versus time factor T (Single Drainage).

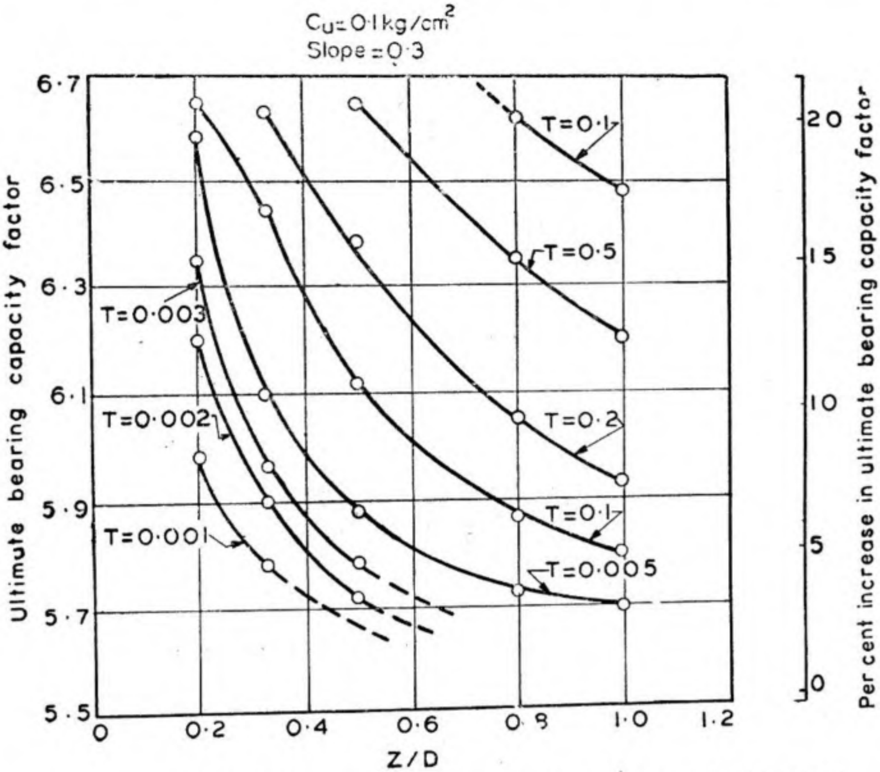

FIGURE 10 : Ultimate bearing capacity factor versus thickness of clay layer/ diameter (Z/D) for different values of T (Single Drainage).

the strength properties of the soil. The given solution is for single vertical drainage only.

(3) The solution for double drainage is expected to reduce the time for any step load, by more than one-fourth, than that required for single drainage. This is expected because of two reasons. One is well-known that the length of the drainage path is reduced to half, so that the time will *be reduced to* 0·25 *times* that for single drainage. Secondly, the position and radius of the critical circle is also changing. The critical circle, in case of double drainage will not go deeper and deeper as the consolidation proceeds, but will remain mainly around the centre of the consolidating mass. This will further increase the corresponding ultimate bearing capacity factor. In effect, the time required for a given step load will be further reduced.

(4) The solution gives the pore pressures at various points along the depth, below the centre of the tank, at the end of every time interval. By installing piezometers, the performance of the foundation, can be checked by comparing the measured pore pressure values with those obtained from the analysis.

(5) In the field, because of various reasons like higher permeability of the consolidating mass due to some large size particles or films of sand stratum acting as drains ; the pore pressures may get dissipated faster than

that given by solution, as obtained by running the computer programme for the problem under consideration. To account for such a possibility and to profit on it, the following method is suggested.

The solution from PROGRAM LOADING (Figure 4) gives a graph of step load versus T. The solution also gives the average pore pressure $\overline{U}$ against T. So a graph can be plotted, as step load versus $\overline{U}$. Based on the measured pore pressure, the step loads can be employed as soon as the corresponding $\overline{U}$ is attained.

Acknowledgements

The authors wish to thank Dr. R.K. Katti, Professor in Civil Engineering, Indian Institute of Technology, Bombay, for his keen interest and useful suggestions. Thanks are due to Dr. A.S. Patwardhan, U.S.A. for his pertinent suggestions, during the course of the analysis. Thanks are due the to Director, Indian Institute of Technology, Bombay for his encouragement in these investigations. Thanks are also due to the authorities of Tata Institute of Fundamental Research, Bombay for providing facilities to carry out the computer work.

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