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The Influence of Soil Non-Homogeneity Upon The Performance of Liquid Storage Tanks

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SYNOPSIS A method of analyzing the behaviour of a circular tank resting on a non-homogeneous crusted soil is described. This method of analysis allows full consideration of soil-structure interaction and is used to examine the effect of soil non-homogeneity upon the performance of a number of liquid storage tanks resting on both deep and shallow crusted soil profiles. This response is contrasted with that which would be predicted based upon an assumption of homogeneous soil. Particular attention is given to the differential settlement, the moments and thrusts developed within the tank for a number of realistic cases.

INTRODUCTION

Until recently, the design of cylindrical liquid storage tanks or reservoirs has been on the basis of an analysis (eg. see Timoshenko and Woinowski-Krieger (1959), Manning (1967) which neglects soil-structure interaction and essentially considers the tank to be a cylinder subjected to boundary conditions at the junction with the base plate. This approach neglects the effect of the weight of the tank and its contents upon the underlying soil. However settlement of the foundation material will occur and this may significantly affect the moments and, to a lesser extent, the thrusts developed in the walls and base of the tank.

A method of analysis which permits consideration of the interaction between wall stiffness, base stiffness and the underlying soil stiffness has been proposed by Booker and Small (1981). This approach is valid for a tank resting on a foundation which may be idealized as a deep, homogeneous elastic soil. However in many practical instances, the soil will not be homogeneous but, rather, may consist of a weathered crust overlying the main deposit. Typically, the soil stiffness will decrease with depth in the crust and then remain relatively constant or increase with depth below the crust. (eg. Soderman et al 1961; Lo and Becker, 1980).

A convenient finite layer approach suitable for determining the response of non-homogeneous soils whose elastic modulus varies with depth has been described by Rowe and Booker (1982) and has been used to examine the behaviour of flexible footings resting on a non-homogeneous soil mass with a crust (Rowe and Booker, 1981).

In this paper, the methods of analysis described by Booker and Small (1981) and Rowe and Booker (1982) are combined to give a method for determining the moments, thrust and settlements of circular storage tanks resting on a non-homogeneous elastic soil. The application of the theory is then illustrated by examining the effect of soil non-homogeneity upon two liquid storage tanks resting on both deep and shallow crusted soil profiles. This response is contrasted with that which would be predicted based on the assumption of a homogeneous soil. Particular attention is given to the differential settlement, the moments and thrusts developed within the tank for a number of realistic cases.

METHOD OF ANALYSIS

In deriving the governing equations it is convenient to develop the flexibility matrices for the tank base, the tank walls and roof, and the soil separately and to recombine these to obtain the complete flexibility of the soil-structure system.

Base Plate

Suppose that the tank base is an elastic disk of thickness t_p and radius a with a Youngs modulus E_p and Poissons ratio ν_p. The base plate will be subjected to an applied pressure (due to fluid and base weight) $q_f(r)$, a reaction $q_s(r)$ due to the presence of the soil, a vertical edge load W due to the weight of the wall, and an edge moment M_e and horizontal edge shear V_e, due to the presence of the wall. It will be assumed that the soil reaction may be approximated in the form:

$$q_s(r) = \sum_{n=0}^{N} F_n Q_n(r) \quad (0<r<a) \qquad \text{(1a)}$$
$$= 0 \quad (a<r<\infty)$$

The coefficients F_n may be thought of as generalized forces with corresponding generalized displacements δ_n defined by

$$\delta_n = \int_o^a r\, w(r)\, Q_n(r)\, dr \qquad (1b)$$

where w(r) denotes the vertical deflection of the base plate.

The net distributed load q_p acting on the plate is

$$q_p = q_f - q_s \qquad (2)$$

and so integrating the plate equation (Timoshenko and Woinowski-Kreiger (1959)) it is found that:

$$w(r) = w_o - \frac{M_e r^2}{2D_p(1+\nu_p)} + \int_o^a \frac{r_o G(r,r_o) q_p(r_o) dr_o}{D_p} \qquad (3)$$

where D_p is the flexural rigidity of the plate, w_o denotes an arbitrary rigid body translation and:

$$G(r,r_o) = \tfrac{1}{4}[r_o^2(1+\ln(\tfrac{r}{r_o})) + \tfrac{1}{2}\frac{(1-\nu_p)}{(1+\nu_p)} + \frac{r^2 r_o^2}{a^2}] \quad r_o \le r \le a$$

$$G(r,r_o) = G(r_o,r) \quad o \le r \le r_o \qquad (4)$$

Substitution of equations (3) and (4) into equation (1b) leads to the following expressions for the generalized deflections δ_n

$$\underset{\sim}{\delta} = w_o \underset{\sim}{\alpha} - M_e \underset{\sim}{\beta} + \underset{\sim}{\gamma} - P \underset{\sim}{F} \qquad (5)$$

where

$$\alpha_m = \int_o^a r\, Q_m(r)\, dr$$

$$\beta_m = \frac{1}{2D_p(1+\nu_p)} \int_o^a r^3\, Q_m(r) dr \qquad (6)$$

$$\gamma_m = \frac{1}{D_p} \int_o^a \int_o^a r\, r_o G(r,r_o) Q_m(r) q_f dr\, dr_o$$

$$P_{mn} = \frac{1}{D_p} \int_o^a \int_o^a r\, r_o G(r,r_o) Q_m(r) Q_n(r_o)\, dr\, dr_o$$

and for the edge rotation:

$$a\theta_e = k - \underset{\sim}{\beta}^T \underset{\sim}{F} - \frac{M_e a^2}{D_p(1+\nu_p)} \qquad (7)$$

where $k = \frac{1}{2D_p(1+\nu_p)} \int_o^a r_o^3\, q_f dr_o$

Clearly the base plate will be in a state of plane stress due to the horizontal edge shear V_e and so

$$u = u_e r/a \qquad (8)$$

where $\frac{u_e}{a} = \frac{V_e}{t_p} \frac{(1-\nu_p)}{E_p}$

E_p and t_p are Youngs modulus and the thickness of the plate respectively. Finally the condition for vertical equilibrium is:

$$2\pi \int_o^a q_s(r) r\, dr = 2\pi \int_o^a q_f(r) dr + W = P_A \qquad (9)$$

(the applied load)

so that

$$\underset{\sim}{\alpha}^T \underset{\sim}{F} = P_A/2\pi$$

Equations (6, 7, 8, 10) may now be written as a flexibility matrix, so that

$$\begin{bmatrix} \underset{\sim}{\delta} \\ a\theta_e \\ u_e \\ 0 \end{bmatrix} = \begin{bmatrix} \underset{\sim}{\gamma} \\ k \\ 0 \\ -P_A/2\pi \end{bmatrix} - \begin{bmatrix} P & \underset{\sim}{\beta} & 0 & -\underset{\sim}{\alpha} \\ \underset{\sim}{\beta}^T & X & 0 & 0 \\ 0 & 0 & Y & 0 \\ -\underset{\sim}{\alpha}^T & 0 & 0 & 0 \end{bmatrix} \begin{bmatrix} \underset{\sim}{F} \\ M_e \\ V_e \\ w_o \end{bmatrix} \qquad (11)$$

where $X = \frac{a^2}{D_p(1+\nu_p)}$

$Y = \frac{a}{t_p} \frac{(1-\nu_p)}{E_p}$

In this investigation the functions $Q_n(r)$ are assumed to be of the form

$$Q_n(r) = (1 - r^2/a^2)^{\lambda_n}$$

where $\{\lambda_n\} = \{-\tfrac{1}{2}, 0, 1, 2, \ldots, N-1\}$

For this case it is possible to evaluate the quantities α, β, γ, P exactly (Booker and Small (1981)).

Tank Walls and Top

Suppose that the tanks walls consist of a number of uniform sections of the type shown in Fig. (1b) subject to end moments and shears M_a, V_a, M_b, V_b.

The equation governing such a wall section is given by Timoshenko and Woinowski-Krieger (1959) as:

$$\frac{d^4u}{dz^4} + 4\zeta^4 u = \frac{l}{D_w} \qquad (12)$$

where u is the radial displacement of the wall, D_w is its flexural rigidity, l is the radial load intensity (assumed to be linear with z) $\zeta^4 = 3(1-\nu_w^2)/(a^2 t_w^2)$ and t_w, ν_w are the wall thickness and Poissons ratio respectively.

Equation (12) may be integrated to give the flexibility matrix of the wall and it is found that:

$$\begin{bmatrix} u_b \\ u_a \\ h\theta_b \\ h\theta_a \end{bmatrix} = \begin{bmatrix} \tilde{l}_b \\ \tilde{l}_a \\ \tilde{g} \\ \tilde{g} \end{bmatrix} - \frac{1}{2}\frac{h^3}{D_w} \begin{bmatrix} U & W & R & V \\ W & U & -V & -R \\ R & -V & Q & S \\ V & -R & S & Q \end{bmatrix} \begin{bmatrix} V_b \\ V_a \\ M_b/h \\ M_a/h \end{bmatrix} \qquad (13)$$

where the subscripts a, b denote values at these points, h is the half depth of the section and

$\tilde{l} = l/(4\zeta^4 D_w)$

$\tilde{g} = \frac{h}{2}(\tilde{l}_b - \tilde{l}_a)$

$$U = \frac{1}{2\xi^3}\left[\frac{\cosh 2\xi + \cos 2\xi}{\sinh 2\xi + \sin 2\xi} + \frac{\cosh 2\xi - \cos 2\xi}{\sinh 2\xi - \sin 2\xi}\right]$$

$$W = \frac{1}{2\xi^3}\left[\frac{\cosh 2\xi + \cos 2\xi}{\sinh 2\xi + \sin 2\xi} - \frac{\cosh 2\xi - \cos 2\xi}{\sinh 2\xi - \sin 2\xi}\right]$$

$$R = \frac{1}{2\xi^2}\left[\frac{\sinh 2\xi - \sin 2\xi}{\sinh 2\xi + \sin 2\xi} + \frac{\sinh 2\xi + \sin 2\xi}{\sinh 2\xi - \sin 2\xi}\right]$$

$$V = \frac{1}{2\xi^2}\left[\frac{\sinh 2\xi + \sin 2\xi}{\sinh 2\xi - \sin 2\xi} - \frac{\sinh 2\xi - \sin 2\xi}{\sinh 2\xi + \sin 2\xi}\right]$$

$$Q = \frac{1}{\xi}\left[\frac{\cosh 2\xi - \cos 2\xi}{\sinh 2\xi + \sin 2\xi} + \frac{\cosh 2\xi + \cos 2\xi}{\sinh 2\xi - \sin 2\xi}\right]$$

$$S = \frac{1}{\xi}\left[\frac{\cosh 2\xi + \cos 2\xi}{\sinh 2\xi - \sin 2\xi} - \frac{\cosh 2\xi - \cos 2\xi}{\sinh 2\xi + \sin 2\xi}\right]$$

and $\xi = \zeta h$

The flexibility of the top of the tank may be found in similar fashion to that of the base. If it is assumed that the top consists of a circular uniform elastic plate of radius a, thickness t_t Youngs modulus E_t, Poissons ratio ν_t and having a flexural rigidity D_t, which is subjected to zero applied loads, then the relation between the edge moment and shear (M_t, V_t) and the edge deflection and rotation (u_t, θ_t) is given by

$$\begin{bmatrix} a\theta_t \\ u_t \end{bmatrix} = \begin{bmatrix} \frac{a^2}{(1+\nu_t)D_t} & 0 \\ 0 & \frac{a}{t_t}\frac{(1-\nu_t)}{E_t} \end{bmatrix} \begin{bmatrix} M_t \\ V_t \end{bmatrix} \qquad (14)$$

The sections of the wall and the top of the tank form a simple "chain structure" and so the flexibility of the entire structure is easily assembled. For the purposes of this paper it is convenient to "condense out" all the internal nodes leaving a relation of the form:

$$\begin{bmatrix} a\theta_e \\ u_e \end{bmatrix} = \begin{bmatrix} c \\ d \end{bmatrix} + \begin{bmatrix} U^* & R^* \\ R^* & Q^* \end{bmatrix} \begin{bmatrix} M_e \\ V_e \end{bmatrix} \qquad (15)$$

Foundation

In analyzing the behaviour of an elastic layered soil it is often convenient to introduce a Hankel transform (Snedden (1951)). It is then found that the vertical deflection may be expressed in the form:

$$w(r,z) = \int_0^\infty \rho\chi(\rho,z) J_o(r\rho)\,\psi(\rho)\,d\rho \qquad (16)$$

where $\psi(\rho) = \int_0^\infty r q_s(r) J_o(r\rho)\,dr$ is the Hankel transform of the surface traction.

For homogeneous or discretely layered soils, the function $\chi(\rho,z)$ may be conveniently found using the finite layer approach Peutz, Van Kempen and Jones (1968), Wardle and Fraser (1975), Cheung (1976). However, in this paper a technique developed by Rowe and Booker (1982) in which the modulus in each layer may vary exponentially with depth is adopted because of its proven advantages in the analysis of nonhomogeneous soils.

It follows from equation (16) that the surface deflection is given by:

$$w(r) = \int_0^\infty \rho\chi(\rho,o)\; J_o(\rho r)\psi(\rho)\,d\rho \qquad (17a)$$

so that using equation (1a) it is found:

$$w(r) = \sum_{n=0}^{N} F_n \int^\infty \rho\chi(\rho,o) J_o(\rho r)\phi_n(\rho)\,d\rho \qquad (17b)$$

where

$$\phi_n(\rho) = \int_0^a r J_o(\rho r)\; Q_n(r)\,dr$$

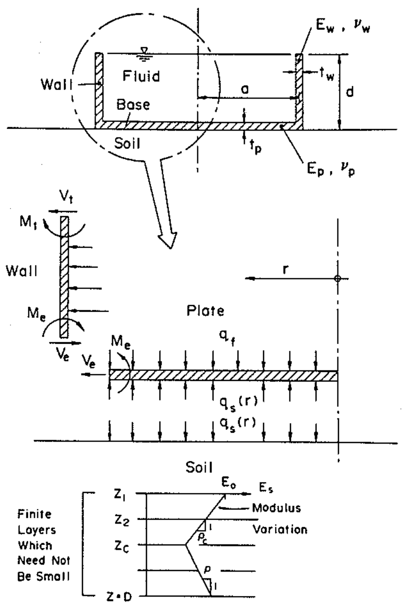

Fig. 1a Problem Idealization

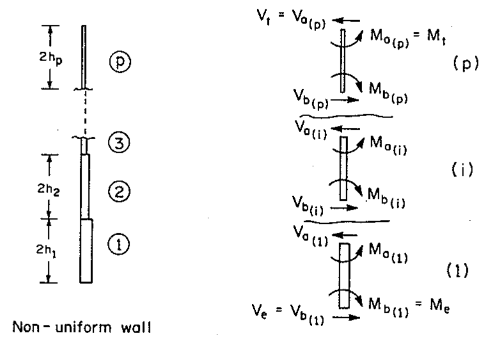

Fig. 1b Non-Uniform Wall

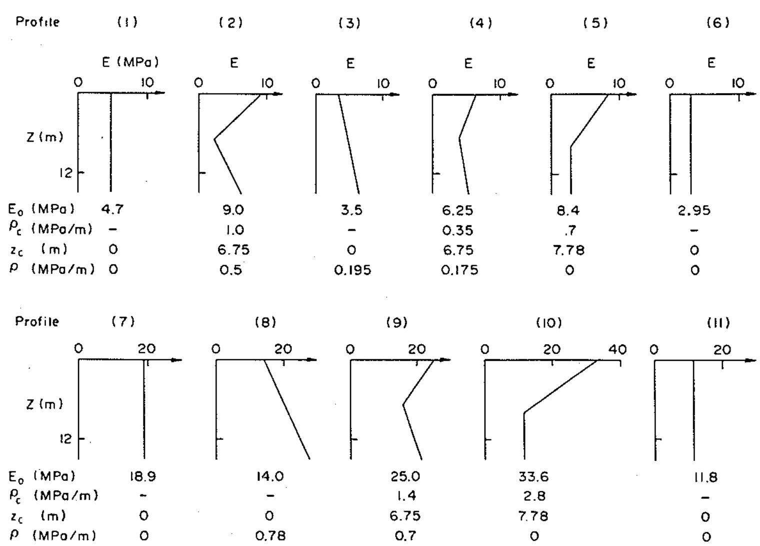

Fig. 2 Soil Profiles Examined

thus if $Q_n(r) = (1-r^2/a^2)^\lambda$

$$\Phi_n(\rho) = \frac{a^2}{2}\left(\frac{2}{a\rho}\right)^{\lambda+1}\Gamma(\lambda+1)J_{\lambda+1}(\rho a)$$

Thus the generalized deflections are given by:

$$\underset{\sim}{\delta} = S\underset{\sim}{F}$$

where $S_{mn} = \int_0^\infty \rho\chi(\rho,o)\ \Phi_m(\rho)\ \Phi_n(\rho)\ d\rho$

Complete Flexibility Matrix

The complete flexibility matrix may now be assembled and it is found that:

$$\begin{bmatrix} P+S & \underset{\sim}{\beta} & 0 & -\underset{\sim}{\alpha} \\ \underset{\sim}{\beta}^T & U^*+X & R^* & 0 \\ 0 & R^* & Q^*+Y & 0 \\ -\underset{\sim}{\alpha}^T & 0 & 0 & 0 \end{bmatrix} \begin{bmatrix} \underset{\sim}{F} \\ M_e \\ V_e \\ w_o \end{bmatrix} = \begin{bmatrix} \underset{\sim}{\gamma} \\ k-c \\ -d \\ -\frac{P_A}{2\pi} \end{bmatrix}$$

This equation may now be solved and hence the behaviour of both tank and soil determined.

PROBLEM DESCRIPTION

The application of the theory described in the previous section is illustrated by considering two tanks resting on eleven different soil profiles. The two reinforced concrete tanks are assumed to be open (see Fig. 1) with a radius a = 9m and a height of d = 7.5m. The reinforced concrete is considered to have an effective modulus ($E_p = E_w$) of 14 GPa, Poissons ratio $\nu_w = \nu_p = 0.15$ and a unit weight $\gamma = 24 kN/m^3$. The two tanks have the same wall thickness t_w = 360 mm but have different base thicknesses t_p of 360 mm and 150 mm for Tanks 1 and 2 respectively. The tanks are assumed to be full of water ($\gamma_w = 9.8\ kN/m^3$).

The soil is generally considered to have a weathered crust overlying the main deposit. It is assumed that Youngs modulus E(z) decreases linearly with depth z in the crust and is given by the expression (see Fig. 1):

$$E(z) = E_o - \rho_c z \quad \text{for } z < z_c$$

where E_o is Youngs modulus at the soil surface; ρ_c is the rate of decrease in modulus with depth (within the crust); and z_c is the depth of the crust. Below the crust, Youngs modulus is given by:

$$E(z) = E_o - \rho_c z_c + \rho(z - z_c) \quad \text{for } z > z_c$$

where ρ is the rate of increase in modulus with depth (in the main deposit) and the other terms are as defined above. The parameters E_o, ρ_c, z_c and ρ for the eleven soil profiles examined are given in Fig. 2 and the dimensionless parameters $\rho_c a/E_o$, $\rho a/E_o$ and z_c/a are given in Table 1.

Case	$\rho_c a/E_o$	z_c/a	$\rho a/E_o$	Case	$\rho_c a/E_o$	z_c/a	$\rho a/E_o$
(1)	0	0	0	(7)	0	0	0
(2)	1.	0.75	0.5				
(3)	0	0	0.5	(8)	0	0	0.5
(4)	0.5	0.75	0.25	(9)	0.5	0.75	0.25
(5)	0.75	0.86	0	(10)	0.75	0.86	0
(6)	0	0	0	(11)	0	0	0

Table 1: Summary of Dimensionless Soil Parameters

It is noted that the cases are "paired" so that in five instances the dimensionless soil parameters are repeated although there is a four fold difference in the magnitude of the modulus (eg. case (1) and (7); (3) and (8) etc). (A four fold increase in the moduli variation of case (2) is not considered realistic). In each case the soil is assumed to be isotropic with a Poissons ratio of 0.3. Two layer depths, D, were examined for each case, namely D = 2a = 18m and D = ∞.

RESULTS

The thickness of the tank base and walls governs both the stiffness and self weight of the tank and can significantly affect the moments and displacements. The effect of self weight and stiffness upon bending moment is illustrated in Fig. 3. For the tank with a uniform wall and base thickness of 360 mm, neglect of tank self-weight would lead to an underestimate of the maximum settlement by 25% and an overestimate of moment by 4% for soil profile (1). Reducing the base thickness to 150 mm reduces the stiffness and correspondingly reduces the moments developed within the tank while increasing the differential and maximum settlements. However it is noted that the effect upon settlement due to a reduction in stiffness is partially offset by the reduced self-weight of the tank with a thinner base. Clearly the effect of the tank self-weight is significant and will be considered in all the following analyses.

Effect of Non-Homogeneity Upon Settlement and Radial Moments

The results presented in Fig. 3 were obtained for a deep homogeneous layer with a modulus of 4.7 MPa. Even for a homogeneous soil, varying the magnitude of Youngs modulus will affect both the moments and displacements. Thus to obtain a meaningful comparison between the results for non-homogeneous soils it is necessary to compare soils with similar "representative" moduli. In the absence of solutions for a non-homogeneous soil, an approximate engineering approach might be to consider the soil to be homogeneous with a modulus $\bar{E}$ equal to the average modulus to a depth z_r, of two thirds the tank diameter

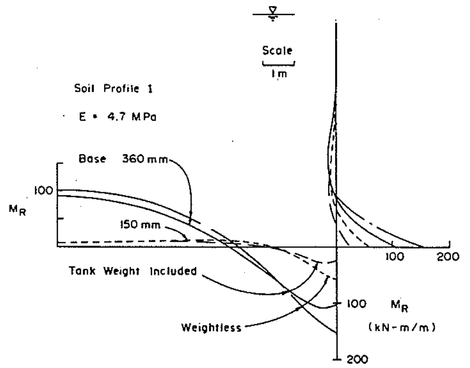

Fig. 3 Effect of Tank Weight Upon Radial Moments: Deep Layer: Profile 1

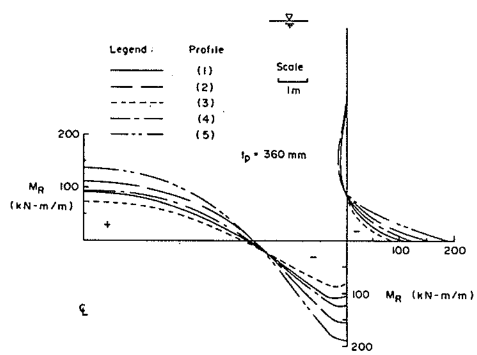

Fig. 4 Effect of Modulus Profile Upon Radial Moments: Deep Layer, t_p = 360 mm.

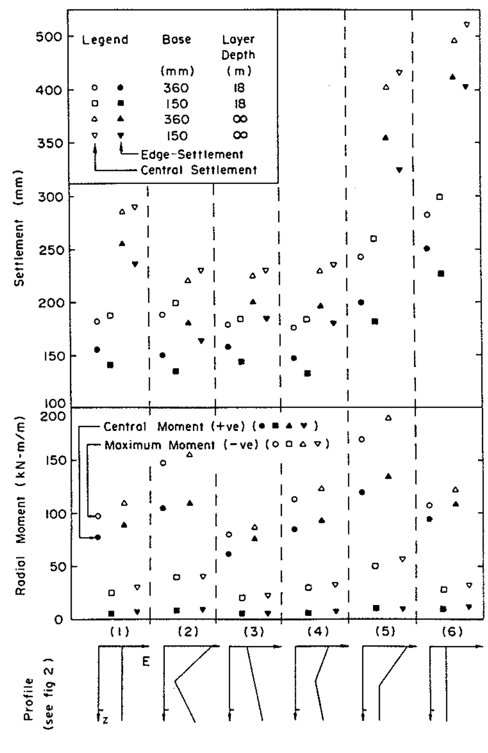

Fig. 5 Summary of Results for Profiles 1 - 6

(z_r = 1.333a) since experience has shown that this will often (but not always) give a "reasonable" estimate of the maximum settlement. To assess the validity of this approximate approach and to provide a common reference modulus, soil profiles 1 to 5 were all selected to have the same average modulus of approximately 4.7 MPa over a depth of 1.33a = 12m. Similarly profiles 7 to 10 were selected to have an average modulus of approximately 18.9 MP over a depth of 12m.

The effect of non-homogeneity upon bending moment distribution for Tank 1 (t_p = 360 mm) resting on a deep layer with profiles 1 to 5 is shown in Fig. 4. The central and edge moments and settlements are also summarized in Fig. 5. It is found that the bending moments are far more sensitive to non-homogeneity than are the displacements. Even though all the profiles considered in Fig. 4 have the same average modulus to a depth of 12 m, the maximum moments vary from 20% below (Profile 3) to 75% above (Profile 5) the value obtained for a homogeneous soil (Profile 1). Increasing soil modulus with depth (Profile 3) reduces the differential and maximum settlement as well as the maximum positive and negative moments and at least for the cases considered herein, the response of a tank on this type of soil profile may be conservatively predicted assuming the soil to be homogeneous with a representative modulus $\bar{E}$. This is not the case for crusted soil profiles.

The presence of the crust dominates the response of the tank. Decreasing modulus within the crust increases the maximum settlement, the differential settlement and the maximum bending moments. This may be partially offset by an increase in modulus with depth below the crust (compare the results for Profiles (2) and (5)), however, as previously noted by Rowe and Booker (1981) it is generally not possible to find a "representative" modulus which will represent the effect of a crusted soil profile upon the maximum and the differential settlement and, in this case, the bending moments.

The effect of non-homogeneity is greatest for a stiff crust overlying a relatively homogeneous main deposit as depicted by Profile 5. In the absence of non-homogeneous solutions, two homogeneous approximations might be attempted. The first corresponds to an average modulus $\bar{E}$ = 4.7 MPa (Profile 1) as previously described. The second would be to neglect the crust entirely and assume the entire soil is homogeneous with a modulus E = 2.95 MPa (Profile 6). The first approach underestimates the maximum and differential settlements by 117 mm (29%) and 19 mm (39%) respectively while the second approach overestimates the maximum settlement by 45 mm (11%) but still underestimates the differential settlement by 13 mm (27%). Of even greater interest are the moment distributions shown in Fig. 6. Both homogeneous approximations significantly underestimate the maximum positive and negative bending moment. The actual edge moment is 78% larger than that obtained for Profile 1 and even 55% greater than the apparently conservative (sic) case where the crust is completely neglected (Profile 6). This situation arises because the reaction distribution is quite sensitive to the soil non-homogeneity and this change in reaction distribution can not be simulated by altering the modulus of a homogeneous underlying soil.

Soil Profiles 1 - 6 correspond to a relatively soft to firm soil and the maximum settlement obtained for these cases is quite large. Large settlements of water tanks on soft foundations have been reported in the literature (eg. Kapp et al, 1966) however it is of some interest to examine the more common situation of a tank resting on firm to stiff soil as depicted by Profiles 7 - 11 in Fig. 2. The bending moment distributions obtained for Tank 1 (t_p = 360 mm) resting on deep soil layers with Profile 7 - 10 are shown in Fig. 7. The central and edge moments and settlements obtained for all cases with the stiffer profile are summarized in Fig.8.

A comparison of the results presented in Figs. 4 and 5 with the corresponding results of Figs. 7 and 8 indicate that the four fold increase in modulus significantly decreased the magnitude of the settlement (although not in direct proportion because of the change in the relative stiffness of the structure) and to a lesser extent decreased the moments. However it can be seen that the general trends regarding the effect of non-homogeneity discussed for soft soils are equally valid for these stiffer profiles.

The interaction between soil non-homogeneity and base thickness may be appreciated by comparing the results shown for Tank 2 (t_p = 150 mm) in Fig. 9 with the corresponding results for Tank 1 (t_p = 360 mm) in Fig. 4. Likewise the central and edge settlement and moment may be compared in Figs. 5 and 8. Decreasing the base stiffness slightly increases the maximum settlement (on average by 3%) and significantly increases the differential settlement (on average by 80%). However the maximum bending moments are greatly reduced (typically by 73%). Similar trends are observed when reducing the base stiffness for soil Profiles 7 - 11 although in this case the differential settlement is only increased by an average of 15%. It is of interest to note that the maximum positive moment developed within the wall of the tank is relatively insensitive to the base thickness and underlying soil profile for the range of cases considered. For Tank 1 (t_p = 360 mm) the maximum positive moment in the base is of the same order as the moment in the wall and with the stiffer soil profiles (Profiles 7 - 11) the positive moment within the wall may infact be the maximum moment in the structure as indicated in Fig. 8.

Effect of Layer Depth

The results discussed in the previous section were all obtained for a deep layer. However, in practice the soil will often be underlain by a stiff (rock) stratum at quite moderate depths. To illustrate the effect of layer depth, analyses were performed for both tanks and soil Profiles 1 - 11 for a soil layer of depth equal to the tank diameter (18 m) and these results are summarized in Figs. 5 and 8. As might be expected, increasing the layer depth gives rise to larger maximum and differential settlements. The relative magnitude of the effect is largely dependent upon the nature of the soil non-

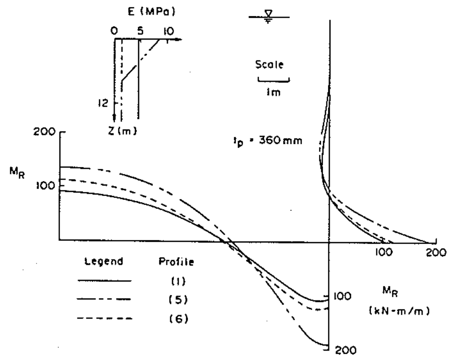

Fig. 6 Effect of Modulus Profile Upon Radial Moments: Deep Layer: t_p = 360 mm

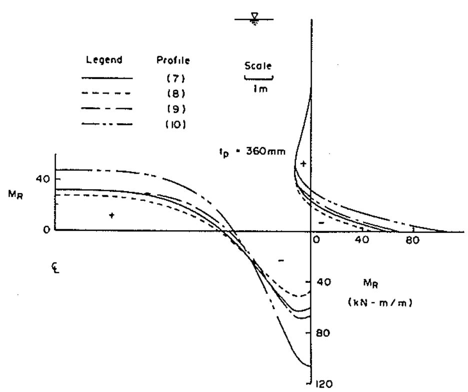

Fig. 7 Effect of Modulus Profile Upon Radial Moment: Deep Layer: t_p = 360 mm

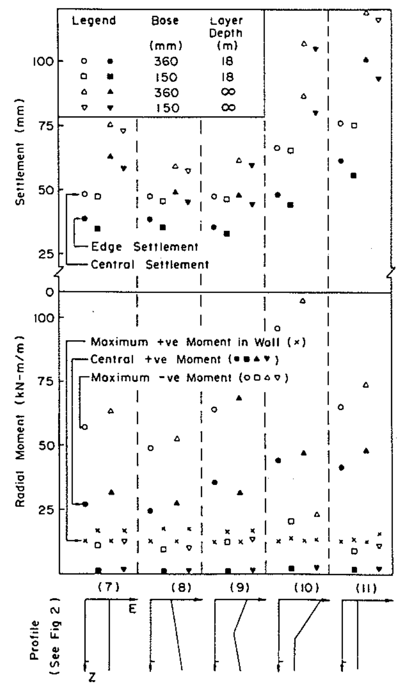

Fig. 8 Summary of Results for Profiles 7 - 11

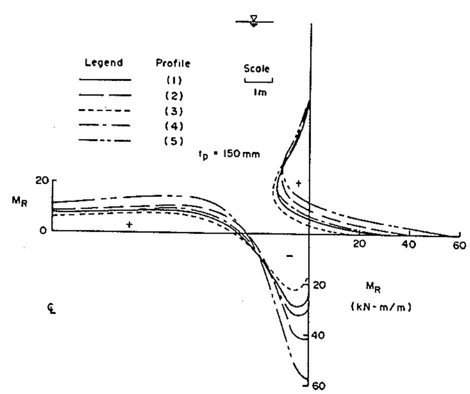

Fig. 9 Effect of Modulus Profile Upon Radial Moment: Deep Layer: t_p = 150 mm

homogeneity and is not greatly influenced by the actual stiffness of the soil or the tank. The effect of layer depth is greatest for soils with a homogeneous main deposit and for the cases considered herein, increasing the layer from 18 m to infinity typically increases the maximum settlement by 55 to 65%. Increasing modulus with depth reduced the influence of layer depth upon maximum settlement to between 15 and 30% for the cases examined (a more detailed discussion of this aspect is given by Rowe and Booker, 1981). The layer depth, D, has relatively little effect upon the bending moments (for D>2a). Increasing the depth from 18 m to infinity typically increased the bending moment by between 5 and 15%; again the greatest effect being observed for soil with a homogeneous main deposit.

Effect of Non-Homogeneity Upon the Thrust Resultants in the Wall

The thrust resultant, N_θ, in the wall of Tank 1 is shown in Fig. 10, for the six soft soil profiles (Nos. 1 - 6). The thrust distribution to a height of 3.5 m is quite sensitive to the soil non-homogeneity. In particular, the thrust just above the base of the tank may vary from large tension for a Gibson soil (Profile 3), to quite small thrusts for homogeneous and slightly non-homogeneous soils (Profiles 1, 2 and 6); to large compression for highly non-homogeneous crusted soils (Profiles 2 and 5). Again, the use of a homogeneous soil solution (irrespective of the choice of modulus used) will provide a poor prediction of the thrust distribution within the wall (and to a lesser extent the base) for truly non-homogeneous soils. However it is noted that the homogeneous solution does provide a reasonable estimate of the maximum thrust.

Increasing the soil modulus (eg. Profiles 7 - 11) tends to increase the thrust developed within the structure although the effects are quite modest (typically 8 - 15%) for Tank 1 (t_p = 360 mm). The effect of base thickness may be appreciated by comparison of Figs. 10b and 11a. The more flexible tank base gives rise to higher maximum thrust (up to 25%) but significantly reduces the effect of non-homogeneity. Indeed as the base stiffness approaches zero the thrust distribution in the wall should correspond to the free stretch line and should be independent of non-homogeneity. This trend can be appreciated by comparison of Figs. 11a and 11b. The stiffer soil profiles (No. 7, 10, 11) of Fig. 11b imply a lower relative stiffness of the tank base and give rise to a higher maximum thrust but a small variation in thrust distribution with non-homogeneity than was obtained in the softer soil (Fig. 11a). Clearly, soil-structure interaction has a significant effect upon the thrust distribution within the tank.

Layer depth (in the range from 18 m to infinity) had a negligible effect (less than 5%) upon the maximum thrust values for all cases considered.

CONCLUSIONS

A soil-structure interaction analysis for determining the bending moments, axial thrusts and settlements of a cylindrical liquid storage tank resting on a non-homogeneous elastic soil has been described. The application of this theory was then illustrated by examining the effect of soil non-homogeneity upon two storage tanks.

It was shown that soil-structure interaction had a significant effect upon the moments, thrusts and settlements of the tanks. In particular, the presence of a stiff crust overlying a relatively soft main deposit may give rise to much larger differential settlements and bending moments than would be predicted assuming that the soil is homogeneous. In fact the reaction distribution was found to be quite sensitive to soil non-homogeneity and this change in reaction distribution could not be simulated by adjusting the modulus in a homogeneous analysis.

Decreasing the tank base stiffness was found to increase differential settlement and reduce moments however the general effects of non-homogeneity upon moments and displacements were unaltered. This was not the case when considering thrust resultants. For soft soil, non-homogeneity had a significant effect upon thrust distribution. However increasing soil stiffness and decreasing base stiffness both considerably reduced the influence of non-homogeneity upon the wall thrusts.

Layer depth was found to have a significant effect upon the magnitude of settlements for relatively homogeneous soils. For non-homogeneous soils whose modulus increased with depth in the main deposits the effect of layer depth upon the settlements, moments and thrust was of secondary importance.

It is concluded that the interaction between soil and structure is quite complex and that in general, a full analysis which takes account of soil-structure interaction and soil non-homogeneity would be advisable if the design of liquid storage tanks is to be optimized. The proposed analysis is relatively straight forward, requires minimal data input and is computationally quite inexpensive.

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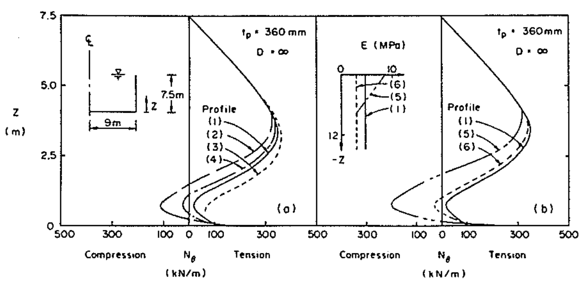

Fig. 10 Effect of Modulus Profile Upon Thrust Resultant N_θ: Deep Layer: t_p = 360 mm

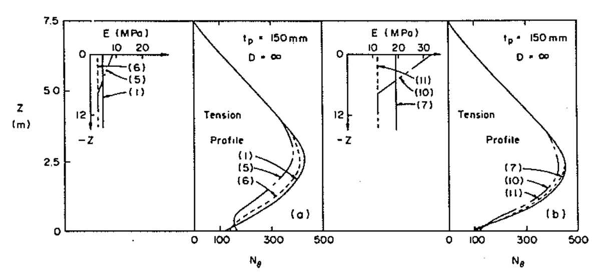

Fig. 11 Effect of Modulus Profile Upon Thrust Resultant N_θ: Deep Layer: t_p = 150 mm