

FINITE ELEMENT ANALYSIS FOR CONSOLIDATION IN INTERACTION BETWEEN STRUCTURE AND SATURATED SOIL FOUNDATION*

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Abstract

The consolidation analysis of interaction between structure and saturated soil foundation is discussed. With the use of substructure technique, the structure is condensed onto the interface of the soil, and then the consolidation governing equations to describe the interaction between soil and structure are derived. The solution with non-iterative algorithm is proposed in this paper. The pressure Master-Slave relation method is used to deal with the non-permeability conditions on soil boundaries. A numerical example is illustrated. Based on this paper, the interactive consolidation analysis between large structure and soil has been more practical.

Key words finite element method, consolidation/interaction

I. Introduction

Under the action of the external load, the ultimate settlement of the saturated soil foundation does not happen instantaneously, but with non-uniform rate. This phenomenon is called consolidation of soil. The rate of soil consolidation depends on the seepage rate of water. In this process, the porous water pressure varies with the soil skeleton stress. The phenomenon of consolidation in saturated sand foundation is mandatory, and if it could not be considered appropriately, damage of the building above the foundation might happen. So it is significant to do serious research work on this subject.

The theory of saturated soil consolidation was first developed by Terzaghi in 1925. Considering the interaction between the soil skeleton and porous water, Biot derived a general governing equation of three-dimensional consolidation problem, and it was used in dynamic analysis further. Biot theory combines the seepage and deformation of the saturated soil, but it is difficult to get the analytical solution.

With the development of finite element method, Zienkiewicz derived a new equation in which the unknown quantities are skeleton displacement and water pressure^[1]. This equation is appropriate for the analysis of consolidation with low seepage rate. Furthermore, Zienkiewicz and Prevost et al. derived the generalized Biot equation in which the physical and geometrical nonlinearities of soil are taken into consideration. In this paper, the combined analysis of structural deformation and soil consolidation is proposed. Based on the substructural analysis

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method, a new non-iterative algorithm for the saturated soil consolidation analysis method is proposed. The program system DIASS for finite element analysis and interaction between soil foundation and structure is implemented.

II. Generalized Biot Equation for Consolidation Analysis

In porous media mechanics, the total stress of soil is often divided into two parts, the effective stress σ'_{ij} and water pressure p ,

$$\sigma_{ij} = \sigma'_{ij} - \delta_{ij}p \quad (2.1)$$

Considering the compressibility of the soil pellet, equation (2.1) is expressed further

$$\sigma_{ij} = \sigma^*_{ij} - \alpha \delta_{ij}p \quad (2.2)$$

σ^*_{ij} is called the modified effective stress. The form of Biot constant α is

$$\alpha = 1 - \frac{\delta_{ij} D_{ijkl} \delta_{kl}}{9k_s} = 1 - \frac{k_t}{k_s} \quad (2.3)$$

The constitutive equation can be written as

$$\sigma^*_{ij} = D_{ijkl} e_{kl} = D_{ijkl} (u_{k,l} + u_{l,k})/2 \quad (2.4)$$

where k_s and k_t are the bulk moduli of the soil material respectively, and u_i is the displacement of soil skeleton.

Suppose all the inertia terms are neglected in saturated soil consolidation, so that the saturated soil equation of motion can be

$$\sigma^*_{ij,j} - \alpha \delta_{ij} p_{,i} = -\rho g_i \quad (2.5)$$

where ρ is the average density of saturated soil, g_i is the acceleration component. According to Darcy law, the differential equation for porous fluid is

$$-p_{,i} + \rho_f g_i = k^{-1} \gamma_f \psi_i \quad (2.6)$$

where ρ_f is the density of fluid phase; k is the Darcy permeability coefficient; γ_f is the specific gravity of fluid phase; and ψ_i is the rate at which the volume of fluid changes per unit.

The mass conservation equation applied to the fluid flow is

$$\alpha \dot{u}_{i,i} + \dot{p}/Q + \psi_{i,i} = 0 \quad (2.7)$$

In which $1/Q = n/k_f + (\alpha - n)/k_s$; n is the porosity of saturated soil, k_f is the bulk modulus of the fluid material.

After getting ψ_i from (2.6), then substituting it into (2.7), we have

$$[k\gamma_f^{-1}(-p_{,i} + \rho_f g_i)]_{,i} + \alpha \dot{u}_{i,i} + \dot{p}/Q = 0 \quad (2.8)$$

(2.5) and (2.8) form the governing equation for the consolidation analysis of saturated soil.

Now the governing equation can be discretized using the standard finite element procedure

$$u_i = N_i^u \bar{u}_{k,i}, \quad p = N_i^p \bar{p}_k \quad (2.9)$$

N_i^u , N_i^p represent the appropriate shape functions respectively. $\bar{u}_{k,i}$, \bar{p}_k are the corresponding element nodal values. Substituting (2.9) into (2.5) and (2.8), then using Galerkin process, we will get the discretization expression for Biot consolidation equation

$$\begin{bmatrix} 0 & 0 \\ Q_{i,u}^t & S_{i,u}^t \end{bmatrix} \begin{Bmatrix} \dot{u} \\ \dot{p} \end{Bmatrix} + \begin{bmatrix} K_{i,u}^t & -Q_{i,p}^t \\ 0 & H_{i,p}^t \end{bmatrix} \begin{Bmatrix} u \\ p \end{Bmatrix} = \begin{Bmatrix} f_i^t \\ f_p^t \end{Bmatrix} \quad (2.10)$$

where

$$\left. \begin{aligned} Q_{i,u}^t &= \int_{\Omega} N_L^t \alpha N_k^t d\Omega, \quad Q_{i,p}^t = Q_{p,i}^t \\ S_{i,u}^t &= \int_{\Omega} N_L^t (1/Q) N_k^t d\Omega, \quad K_{i,u}^t = \int_{\Omega} N_L^t D_{ijkl} N_m^t d\Omega \\ H_{i,p}^t &= \int_{\Omega} N_L^t k \gamma_i^{-1} N_k^t d\Omega \\ f_i^t &= \int_{\Omega} N_L^t \rho g_i d\Omega + \int_F N_L^t \alpha p n_i d\Gamma + \int_F N_L^t \sigma_{ij} n_j d\Gamma \\ f_p^t &= \int_{\Omega} N_L^t \rho_i g_i d\Omega - \int_F N_L^t k \gamma_i^{-1} n_i p_i d\Gamma \end{aligned} \right\} \quad (2.11)$$

III. The Substructure Technique for the Consolidation Analysis of Structure and Saturated Soil

The coupling effect from the interaction of structure and saturated soil at the interface can be handled with the interface conditions. In engineering approximate analysis, only the action from the structural load is considered for the consolidation subsidence computation, and the coupling effect from structure and soil is neglected, so that the final consolidation subsidence is found with this method, and then this subsidence is used as designated non-zero displacement to exert on the structure to find the internal force and deformation. It is easy to know that this approximate method can not reflect the real consolidation process without considering the deformation interaction of soil and structure. Besides, it is inconvenient to compute separately for the structure and the soil, because it needs two step computations.

One may find from (2.10) that the soil consolidation is actually a pseudo-static procedure, and that the skeleton displacement corresponding to soil is entirely analyzed according to the static case. So the equation for the analysis of structure can be expressed as

$$\begin{bmatrix} K_{II}^t & K_{IB}^t \\ K_{BI}^t & K_{BB}^t \end{bmatrix} \begin{Bmatrix} u_I^t \\ u_B^t \end{Bmatrix} = \begin{Bmatrix} f_I^t \\ f_B^t \end{Bmatrix} \quad (3.1)$$

where u_B^t , f_B^t are the displacement and external force of structure at the interface with soil respectively, and u_I^t , f_I^t are the internal vectors respectively. Evidently, (3.1) is the 'root-level expression' of multi-level substructure description, which can be described based on the general rule of the multi-level substructure method. It has at least twofold merit of introducing the multi-level substructure method. Firstly, the scale of the structure can be increased. Secondly, the efficiency of consolidation solution is also improved. The reason is that the multi-level substructuring eliminates the internal displacement of the structure from the consolidation analysis. Apparently, the soil consolidation analysis is in progress at the root-level substructure. Applying the static condensation for (3.1) gives

$$K_{BB}^{**} u_B^t = f_B^{**} \quad (3.2)$$

$$K_{BB}^{**} = K_{BB}^t - K_{BI}^t K_{II}^{-1} K_{IB}^t \quad (3.3)$$

$$f_{\theta}^{**} = f_{\theta}^* - K_{\theta\theta}^* K_{\theta u}^{*-1} f_u^* \quad (3.4)$$

Generally speaking, the degrees of freedom at the interface is far less than those of structure and soil, so the dimensions of equation (3.2) is greatly reduced. (3.2) can be further expressed as

$$\begin{bmatrix} K_{\theta\theta}^{**} & K_{\theta u}^{**} \\ K_{u\theta}^{**} & K_{uu}^{**} \end{bmatrix} \begin{Bmatrix} u_{\theta}^* \\ u_u^* \end{Bmatrix} = \begin{Bmatrix} f_{\theta}^{**} \\ f_u^{**} \end{Bmatrix} \quad (3.5)$$

where u_{θ}^* is the uncoupled displacement (e.g. rotation) at the interface of structure and soil, u_u^* is the coupling displacement (e.g. linear displacement).

Proceeding similarly to the condensation of (3.1), (3.5) can be further condensed to involve only u_u^* , the coupling displacement. Because equation (3.5) is more general, we will proceed based on it. Assembling (3.5) and (2.10) gives

$$\begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & Q_{\theta u}^* & Q_{\theta v}^* & S_{\theta\theta}^* \end{bmatrix} \begin{Bmatrix} \dot{u}_{\theta}^* \\ \dot{u}_u^* \\ \dot{u}_v^* \\ \dot{p} \end{Bmatrix} + \begin{bmatrix} K_{\theta\theta}^{**} & K_{\theta u}^{**} & 0 & 0 \\ K_{u\theta}^{**} & K_{uu}^{**} + K_{uu}^* & K_{uv}^* & -Q_{uv}^* \\ 0 & K_{vu}^* & K_{vv}^* & -Q_{vv}^* \\ 0 & 0 & 0 & H_{vv}^* \end{bmatrix} \begin{Bmatrix} u_{\theta}^* \\ u_u^* \\ u_v^* \\ p \end{Bmatrix} = \begin{Bmatrix} f_{\theta}^{**} \\ f_u^{**} + f_u^* \\ f_v^* \\ f_p^* \end{Bmatrix} \quad (3.6)$$

where u_v^* is the uncoupled displacement of soil; p is the pressure of porous fluid. For further derivation, (3.6) can be written in the simpler form by omitting all superscripts

$$\begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & Q_{\theta u} & Q_{\theta v} & S_{\theta\theta} \end{bmatrix} \begin{Bmatrix} \dot{u}_{\theta} \\ \dot{u}_u \\ \dot{u}_v \\ \dot{p} \end{Bmatrix} + \begin{bmatrix} K_{\theta\theta} & K_{\theta u} & 0 & 0 \\ K_{u\theta} & K_{uu} & K_{uv} & -Q_{uv} \\ 0 & K_{vu} & K_{vv} & -Q_{vv} \\ 0 & 0 & 0 & H_{vv} \end{bmatrix} \begin{Bmatrix} u_{\theta} \\ u_u \\ u_v \\ p \end{Bmatrix} = \begin{Bmatrix} f_{\theta} \\ f_u \\ f_v \\ f_p \end{Bmatrix} \quad (3.7)$$

All vectors of (3.7) correspond to those of (3.6) according to their position. The solution of (3.7) should satisfy the draining-water condition at the soil boundary and the interface of structure and soil. These treatments can be described as follows:

a. Porous water permeability at the interface of structure and soil is not good, and the far region soil boundary can be considered as non-draining, so $\partial p / \partial n|_B = 0$ along the normal direction n of these boundaries. To treat these boundary conditions, the pressure of finite element nodal points s_1, s_2, \dots, s_n on the boundary equals to the pressure of the points m_1, m_2, \dots, m_n , neighbour to the points s_1, s_2, \dots, s_n , along the boundary normal direction, namely.

$$p_{s_1} = p_{m_1}, \quad p_{s_2} = p_{m_2}, \quad \dots, \quad p_{s_n} = p_{m_n} \quad (3.8)$$

Obviously, the normal distance between these points should be small enough for precise result.

b. On the soil free surface f the free draining-water condition $p|_{f=0}$ must be satisfied, so that at the corresponding finite element points d_1, d_2, \dots, d_n , it must be

$$p_{d_1} = p_{d_2} = \dots = p_{d_n} = 0 \quad (3.9)$$

IV. The Integrated Algorithm for Consolidation Analysis of Structure and Saturated Soil

First, let us review the existing solution method for the consolidation equation (2.10).

Because the coefficient matrix is non-symmetrical, the recursive interaction algorithm is generally applied. In this method the porous water pressure is assumed at first for each time step, then the pressure is substituted into the first equation of (2.10) to solve u_i ; the next step is to get the new pressure value of porous water from the second equation of (2.10); by using these two equations alternatively the solution can be obtained for this time step. For nonlinear problem, more iterations will be necessary, which will cause numerical error. In view of this unfavourable situation, a one-step integrated algorithm is proposed in this paper. (3.7) can be discretized along the time domain, by using linear interpolation, for $t < \xi < t + \Delta t$, it gives

$$N_1^t = \frac{t + \Delta t - \xi}{\Delta t}, \quad N_2^t = \frac{\xi - t}{\Delta t}, \quad \dot{N}_1^t = -\frac{1}{\Delta t}, \quad \dot{N}_2^t = \frac{1}{\Delta t} \quad (4.1)$$

$$\left. \begin{aligned} u_\theta &= N_1^t u_{\theta t} + N_2^t u_{\theta, t+\Delta t}, & u_u &= N_1^t u_{ut} + N_2^t u_{u, t+\Delta t} \end{aligned} \right\} \quad (4.2)$$

$$\left. \begin{aligned} u_v &= N_1^t u_{vt} + N_2^t u_{v, t+\Delta t}, & p &= N_1^t p_t + N_2^t p_{t+\Delta t} \\ \dot{u}_\theta &= \dot{N}_1^t u_{\theta t} + \dot{N}_2^t u_{\theta, t+\Delta t}, & \dot{u}_u &= \dot{N}_1^t u_{ut} + \dot{N}_2^t u_{u, t+\Delta t} \end{aligned} \right\} \quad (4.3)$$

$$\left. \begin{aligned} \dot{u}_v &= \dot{N}_1^t u_{vt} + \dot{N}_2^t u_{v, t+\Delta t}, & \dot{p} &= \dot{N}_1^t p_t + \dot{N}_2^t p_{t+\Delta t} \\ f_\theta &= N_1^t f_{\theta t} + N_2^t f_{\theta, t+\Delta t}, & f_u &= N_1^t f_{ut} + N_2^t f_{u, t+\Delta t} \\ f_v &= N_1^t f_{vt} + N_2^t f_{v, t+\Delta t}, & f_p &= N_1^t f_{pt} + N_2^t f_{p, t+\Delta t} \end{aligned} \right\} \quad (4.4)$$

substituting (4.1)–(4.4) into the first equation of (3.7), then applying weighted integration in $(t, t + \Delta t)$ gives

$$\begin{aligned} & \int_t^{t+\Delta t} W K_{\theta\theta} (N_1^t u_{\theta t} + N_2^t u_{\theta, t+\Delta t}) d\xi + \int_t^{t+\Delta t} W K_{\theta u} (N_1^t u_{ut} \\ & + N_2^t u_{u, t+\Delta t}) d\xi = \int_t^{t+\Delta t} W (N_1^t f_{\theta t} + N_2^t f_{\theta, t+\Delta t}) d\xi \end{aligned} \quad (4.5)$$

where W is the weighted factor. Introducing $\eta = (\xi - t) / \Delta t$ and substituting the variables in (4.5), dividing both sides with $\int_0^1 W d\eta$ gives

$$\theta K_{\theta\theta} u_{\theta, t+\Delta t} + \theta K_{\theta u} u_{u, t+\Delta t} = (1 - \theta) f_{\theta t} + \theta f_{\theta, t+\Delta t} - (1 - \theta) K_{\theta\theta} u_{\theta t} - (1 - \theta) K_{\theta u} u_{ut} \quad (4.6)$$

$$\theta = \int_0^1 W \eta d\eta / \int_0^1 W d\eta \quad (4.7)$$

Next, the second, the third and fourth equations of (3.7) are treated by the same way

$$\begin{aligned} & \theta K_{u\theta} u_{\theta, t+\Delta t} + \theta K_{uu} u_{u, t+\Delta t} + \theta K_{uv} u_{v, t+\Delta t} - \theta Q_{u,p} p_{t+\Delta t} \\ & = (1 - \theta) f_{ut} + \theta f_{u, t+\Delta t} - (1 - \theta) K_{u\theta} u_{\theta t} - (1 - \theta) K_{uu} u_{ut} \\ & - (1 - \theta) K_{uv} u_{vt} + (1 - \theta) Q_{u,p} p_t \end{aligned} \quad (4.8)$$

$$\begin{aligned} & \theta K_{v\theta} u_{\theta, t+\Delta t} + \theta K_{vu} u_{u, t+\Delta t} - \theta Q_{v,p} p_{t+\Delta t} = (1 - \theta) f_{vt} \\ & + \theta f_{v, t+\Delta t} - (1 - \theta) K_{v\theta} u_{\theta t} - (1 - \theta) K_{vu} u_{ut} + (1 - \theta) Q_{v,p} p_t \end{aligned} \quad (4.9)$$

$$\frac{1}{\Delta t} Q_{p,u} u_{u, t+\Delta t} + \frac{1}{\Delta t} Q_{p,v} u_{v, t+\Delta t} + \left[\frac{1}{\Delta t} S_{p,p} + \theta H_{p,p} \right] p_{t+\Delta t}$$

$$= (1 - \theta) f_{pt} + \theta f_{p, t+\Delta t} + \frac{1}{\Delta t} Q_{p,u} u_{ut} + \frac{1}{\Delta t} Q_{p,v} u_{vt}$$

$$+\left[\frac{1}{\Delta t}S_{rr}-(1-\theta)H_{rr}\right]p_i \quad (4.10)$$

Combining (4.6), (4.8), (4.9), (4.10) gives

$$\begin{bmatrix} \theta K_{\theta\theta} & \theta K_{\theta u} & 0 & 0 \\ \theta K_{u\theta} & \theta K_{uu} & \theta K_{uv} & -\theta Q_u \\ 0 & \theta K_{vu} & \theta K_{vv} & -\theta Q_v \\ 0 & \frac{1}{\Delta t}Q_{ru} & \frac{1}{\Delta t}Q_{rv} & \left(\frac{1}{\Delta t}S_{rr}+\theta H_{rr}\right) \end{bmatrix} \begin{Bmatrix} u_{\theta i+\Delta t} \\ u_{ui+\Delta t} \\ u_{vi+\Delta t} \\ p_{i+\Delta t} \end{Bmatrix} = \begin{Bmatrix} (1-\theta)f_{\theta i}+\theta f_{\theta i+\Delta t} \\ (1-\theta)f_{ui}+\theta f_{ui+\Delta t} \\ (1-\theta)f_{vi}+\theta f_{vi} \\ (1-\theta)f_{ri}+\theta f_{ri} \end{Bmatrix} \\ - \begin{bmatrix} (1-\theta)K_{\theta\theta} & (1-\theta)K_{\theta u} & 0 & 0 \\ (1-\theta)K_{u\theta} & (1-\theta)K_{uu} & (1-\theta)K_{uv} & -(1-\theta)Q_u \\ 0 & (1-\theta)K_{vu} & (1-\theta)K_{vv} & -(1-\theta)Q_v \\ 0 & -\frac{1}{\Delta t}Q_{ru} & -\frac{1}{\Delta t}Q_{rv} & (1-\theta)H_{rr}-\frac{1}{\Delta t}S_{rr} \end{bmatrix} \begin{Bmatrix} u_{\theta i} \\ u_{ui} \\ u_{vi} \\ p_i \end{Bmatrix} \quad (4.11)$$

(4.11) can be further converted to the matrix form of symmetry coefficient

$$\begin{bmatrix} K_{\theta\theta} & K_{\theta u} & 0 & 0 \\ K_{u\theta} & K_{uu} & K_{uv} & -Q_u \\ 0 & K_{vu} & K_{vv} & -Q_v \\ 0 & -Q_{ru} & -Q_{rv} & -(S_{rr}+\theta\Delta t H_{rr}) \end{bmatrix} \begin{Bmatrix} u_{\theta i+\Delta t} \\ u_{ui+\Delta t} \\ u_{vi+\Delta t} \\ p_{i+\Delta t} \end{Bmatrix} = \begin{Bmatrix} \frac{1-\theta}{\theta}f_{\theta i}+f_{\theta i+\Delta t} \\ \frac{1-\theta}{\theta}f_{ui}+f_{ui+\Delta t} \\ \frac{1-\theta}{\theta}f_{vi}+f_{vi+\Delta t} \\ -\left(\frac{1-\theta}{\theta}f_{ri}+f_{ri+\Delta t}\right)\theta\Delta t \end{Bmatrix} \\ - \begin{bmatrix} \frac{1-\theta}{\theta}I_{\theta\theta} & 0 & 0 & 0 \\ 0 & \frac{1-\theta}{\theta}I_{uu} & 0 & 0 \\ 0 & 0 & \frac{1-\theta}{\theta}I_{vv} & 0 \\ 0 & 0 & 0 & -I_{rr} \end{bmatrix} \begin{bmatrix} K_{\theta\theta} & K_{\theta u} & 0 & 0 \\ K_{u\theta} & K_{uu} & K_{uv} & -Q_u \\ 0 & K_{vu} & K_{vv} & -Q_v \\ 0 & -Q_{ru} & -Q_{rv} & -(S_{rr}-(1-\theta)\Delta t H_{rr}) \end{bmatrix} \begin{Bmatrix} u_{\theta i} \\ u_{ui} \\ u_{vi} \\ p_i \end{Bmatrix} \quad (4.12)$$

where I_{uu} , $I_{\theta\theta}$, I_{vv} , I_{rr} are unit matrices. Up to now the coefficient matrices in (4.12) are all symmetric. It is good for numerical analysis. The non-iterative solution of the problem can be obtained by using (4.12), and the iteration within each time-step is avoided.

To select θ , an example of one degree of freedom is given here

$$ku+c\dot{u}=0 \quad (k>0, c>0) \quad (4.13)$$

Interpolating the two-point time interval gives

$$u_{i+\Delta t} = \lambda u_i \quad (4.14)$$

where

$$\lambda = [1 - (1 - \theta)\alpha] / (1 + \theta\alpha) \quad (4.15)$$

$$\alpha = k\Delta t / c \quad (4.16)$$

From stable solution, it must be $|\lambda| < 1$, namely,

$$-1 < [1 - (1 - \theta)\alpha] / (1 + \theta\alpha) < 1 \quad (4.17)$$

Therefore it gives

$$\theta > 1/2 - 1/\alpha \quad (4.18)$$

From the above relation, the problem is unconditionally stable while $\theta \geq 1/2$, and it is conditionally stable while $0 < \theta < 1/2$. Further analysis can show that the iterative procedure doesn't oscillate while $\theta \geq 1$, which oscillates while θ is between $1/2$ and 1 .

For time-step selection, the variable-step form can be used. When the following relation holds

$$\|p_{t_n+\Delta t_n} - p_{t_n}\| / \max \|p_{t_i+\Delta t_i} - p_{t_i}\| \leq \varepsilon, \quad (i=1, 2, \dots, n) \quad (4.19)$$

for the n -th time-step, adjusting can be carried out as

$$\Delta t_{n+1} = \mu \Delta t_n \quad (4.20)$$

where ε is the decision parameter of the variable-step (e.g.1); μ is the adjusting factor of the step, which can be between 1.0 and 2.0.

V. The Numerical Example

Here a simple example is given to show the effects of considering the coupling effect between structure and soil or not.

Fig. 1 shows a two-dimensional consolidation problem. A 3.5m length beam is put on the soil. As a structure, the beam is subdivided into three elements. But it can be treated as a super element with load exerted on the points of the beam. When the coupling effect is neglected the load acts on the soil points directly, and the structure stiffness will have no effect in the consolidation analysis.

The beam property is

$$E = 30 \text{ GPa}, F = 3.21 \text{ m}^2, J_z = 0.064 \text{ m}^4, G = 12.4 \text{ GPa}, F_y = 2.68 \text{ m}^2$$

The soil property is

$$E = 36 \text{ MPa}, \mu = 0.4, \delta = 1.0 \text{ m}, Q = 1 \text{ GPa}, k = 0.01 \text{ mm} \cdot \text{s}^{-1}, n = 0.53$$

Fig.2 is the time curve of the load.

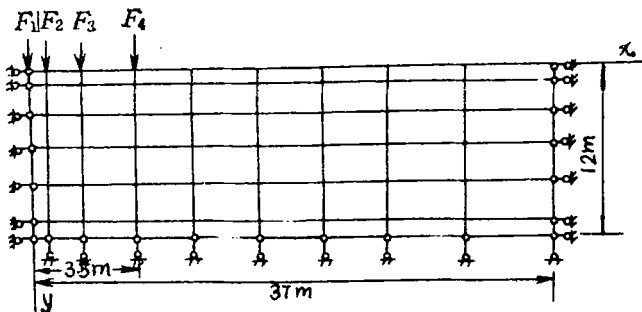


Fig. 1 The finite element model for two-dimensional soil

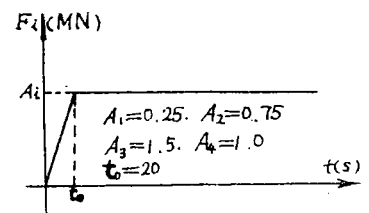


Fig. 2 The time curve of the load

Fig. 3 gives the porous water pressure versus time curve at the section $x=0$ with no structure stiffness, whereas Fig. 4 is the same curve with structure stiffness. Fig. 5 gives σ_y curve at the section $x=0$ while the structure stiffness doesn't exist. Fig. 6 is the same curve with structure stiffness. Fig. 7 gives the comparison of the porous water pressure at $x=0$ and $t=20s$ between the cases of the structure stiffness existing or not. Fig. 8 gives σ_y curve of $t=20s$ and $t=1648s$ while the structure exists or not at the section $x=0$. Fig. 9 gives the final subsidence computation between having structure or not. Fig. 10 is the subsidence curve at $y=0$ while the structure exists.

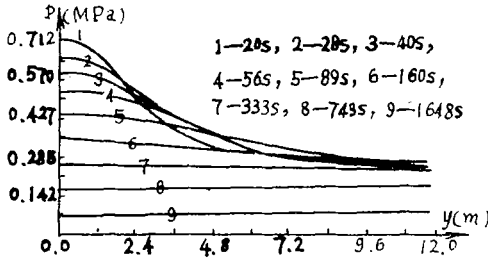


Fig. 3 The porous water pressure curve without structure stiffness at the section $x=0$

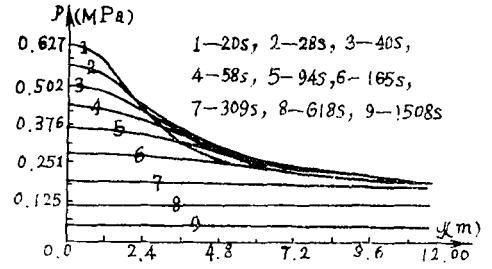


Fig. 4 The porous water pressure curve with structure stiffness at the section $x=0$

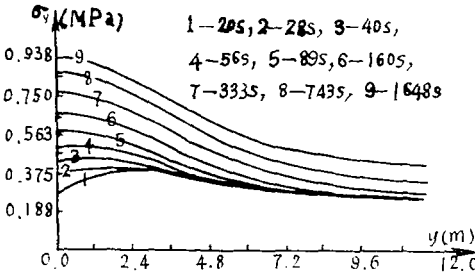


Fig. 5 σ_y curve without structure stiffness at the section $x=0$

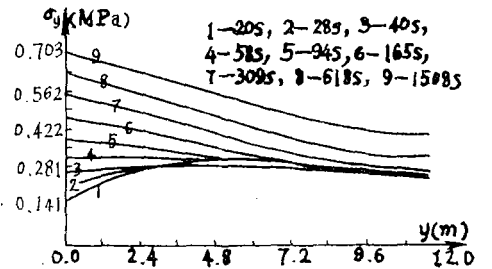


Fig. 6 σ_y curve with structure stiffness at the section $x=0$

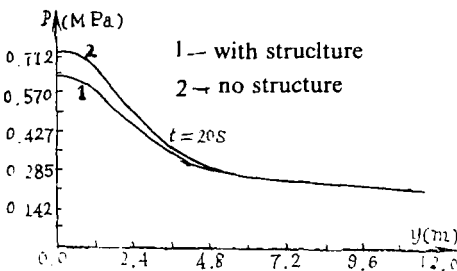


Fig. 7 The comparison of the porous water pressure between the cases of structure stiffness existing or not at the section $x=0$

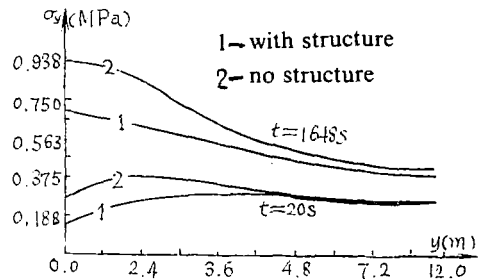


Fig. 8 The comparison of σ_y between the cases of structure stiffness exists or not at the section $x=0$

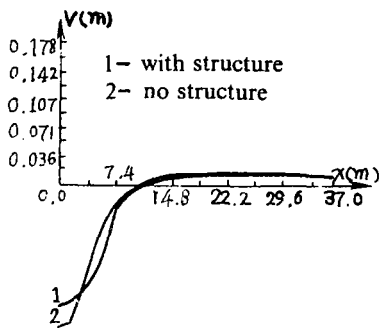


Fig. 9 The comparison of the final subsidence between having structure or not at the section $y=0$

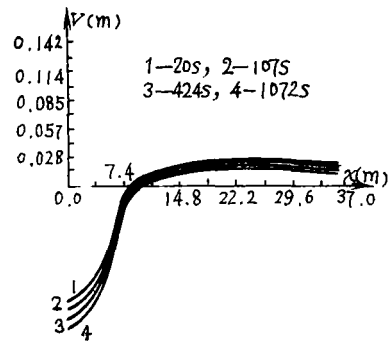


Fig. 10 The subsidence curve at $y=0$ while the structure stiffness exists

From the numerical result, it is obvious that the structure has important effect for consolidation computation of soil. In view of this example, the structure changes the distribution of porous water pressure in the soil. Generally speaking, the internal force distribution becomes more steady with the existence of the structure.

One must recognize that a lot of work need be done in soil consolidation. The present paper gives the method for solving consolidation problem of interaction between large-scale structure and soil, such as offshore platforms.

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