SECTION III Finite Strain FE Analysis of Sensitive Ground beneath Embankment with Elasto-Viscoplastic Model

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1.INTRODUCTION

It is known that small strain theory commonly used in FE analysis is of limits in the large strain field. It is known that clay is rate sensitive material. In this paper finite strain 2-D FE analysis is carried out for sensitive clay foundation beneath the embankment at St. Alban., Canada. Focus is put on the effect of strain softening factor in the constitutive laws.

2. CONSTITUTIVE LAWS

2.1 Finite Strain Theory

It is known that the material time derive of Cauchy stress is not objective. So Jaumann rate of Cauchy stress is used in the constitutive laws as well as deformation rate, which can be expressed as follows.

$$\hat{T}' = CD - Q \tag{1}$$

Where, T' is effective Cauchy stress tensor. C is tangent constitutive tensor of Elasto-Viscoplastic material. D is deformation rate and Q is residual stress tensor

2.2 Elasto-Viscoplastic Model

Based on the model firstly proposed by Adachi and Oka (1982, 1987),Oka et al (1991), viscoplastic deformation ratio in the field of finite strain field is expressed as the following.

$$D_{ij}^{vp} = \gamma \langle \phi_1(F) \rangle \phi_2(\xi) \frac{\partial f}{\partial T_{ij}'} \qquad \langle \phi_1(F) \rangle = \begin{Bmatrix} 0 & (F \le 0) \\ \phi_1(F) & (F > 0) \end{Bmatrix} \tag{2}$$

$$\gamma\phi_1(F) = M^* T_m' C_N \exp\left\{ m_N' \left(\ln\left(\frac{T_m'}{T_{m0}'} \right) + \frac{\eta^*}{M^*} - \frac{1+e}{\lambda - \kappa} v^p - \alpha_3 \Delta v^p \right) \right\}, \quad v^p = \int D_{ii}^{vp} dt \tag{3}$$

$$\phi_2 = 1 + \xi \quad , \quad \dot{\xi} = \frac{M^{*2}}{G_2^* \left(M^* - \eta^* \right)^2} \dot{\eta}^*, \quad \eta^* = \left\| \eta_{ij}^* \right\|^{\frac{1}{2}} \quad , \quad \eta_{ij}^* = \frac{S_{ij}}{T_m'} \tag{4}$$

where, D_{ij}^{vp}, T_m are viscoplastic deformation rate and mean effective Cauchy stress. λ, κ, M^*, e C_N, T_{m0}', m_N are compression index, swelling index, deviatoric stress ratio at failure, void ratio, initial viscoplastic parameter, and other two viscoplastic parameters respectively. S_{ij} is deviatoric stress. v is volumetric strain. t is the time. G_2^* is the model parameter to describe strain softening. α_3 is the model parameter related to second order gradient of viscoplastic strain. Elastic deformation ratio D_{ij}^e is based on Hooke's law as.

$$\dot{D}_{ij}^e = \frac{1}{2G} \dot{S}_{ij} + \frac{\kappa}{3(1+e)} \frac{\dot{\sigma}_m'}{\sigma_m'} \delta_{ij}, \quad D_{ij} = D_{ij}^e + D_{ij}^{vp} \tag{5}$$

Where, G is elastic shear modulus and δ_{ij} is Kronecker's delta.

3. FE ANALYSIS OF CLAY GROUND UNDER EMBANKMENT D

Based on the described above, a finite strain updated Lagrangian FEM has been coded to numerically simulate a embankment test at Saint Alban, west of Quebec, which has a height of 3.3 m and a width at the crest of 7.8 m. The slope of horizontal to vertical is 4:1. Density of the fill material of embankment is 1.857 tf/m^3. The width and depth of clay ground is 144 m and 13.8 m in computation. 245 8-noded quadrilateral elements and 920 nodes are used in the analysis. The method to determine material parameters is the same as that proposed by Oka et al (1991). M^* is 0.98.. $k = k_0 \exp((e - e_0)/C_k^*)$, $C_k^* = 0.5e$. $k_0 = 1.05 \times 10^{-8} m/\mathrm{sec}$. $K_0 = 0.8$. $\kappa = 0.0045(1+e)$.

$E_{oed} = 100\sigma'_p$. The other parameters necessary are shown in Table 1 and Table 2. Loading history is same as those in the reference (2). In this presentation, it is assumed that α_3=0.0, G_2^* is 1.00.
In Fig.1 is shown the comparison of calculated and observed relationships between settlement just below the center of the embankment and time. It can be found out that the computed analysis can reflect the abrupt change in settlement, although the computed result is still larger than the measured data. The computed settlement becomes greater due to existence of failure zone, which is related with strain softening factor. In Fig.2 are shown the developments of pore water pressure in the analysis and in the field. The computed analysis can also simulate the apparent change in pore water pressure. The computed value decreases a little with time after 70 days while the measured one does not. But the decreasing is smaller than the computed result reported before.

4. CONCLUSIONS

The numerical analysis can reflect the abrupt change in settlement and in pore water pressure, and can reflect the main behavior of the sensitive clay with time. There are still difference in the computed settlement, pore water pressure and the measured ones. The difference is dependent on the parameter G_2^* and the elastic parameters of the top layer. Because of the complexity of natural sensitive clay, further study is necessary to carry out in the future, including the comparison with small strain theory.

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Table.1 Material parameters

No. of layer	Depth (m)	λ_0	λ_1	λ_2	e_0	σ'_p (kgf/cm^2)	G (kgf/cm^2)
1	0.00 ~ 0.66	0.02	0.3	0.100	1.10	0.739	8.97
2	0.66 ~ 1.50	0.523	1.00	0.363	1.70	0.582	15.3
3	1.50 ~ 3.00	0.0719	1.14	0.495	2.30	0.469	21.2
4	3.00 ~ 4.80	0.0387	1.04	0.411	1.80	0.720	30.7
5	4.80 ~ 6.70	0.246	0.560	0.282	1.80	0.90	41.2
6	6.70 ~ 9.60	0.0104	0.409	0.175	1.40	1.40	57.1
7	9.60 ~ 13.5	0.008	0.409	0.100	1.40	1.80	112

Table.2 Dependency of parameters on volumetric plastic strain

Volumetric viscoplastic strain v^{vp}(%)	m'	λ	$C'_0(1/sec)$
$v^{vp} < 0.027$	17.8	λ_0	1.2×10^{-12}
$0.027 \leq v^{vp} < 4.2$	26.7	λ_1	5.9×10^{-11}
$4.2 < v^{vp}$	26.7	λ_2	5.9×10^{-11}

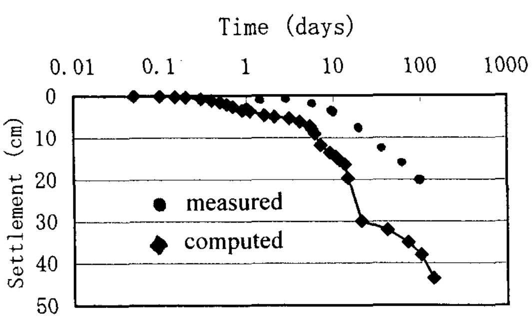

Fig. 1 Settlement below the centerline

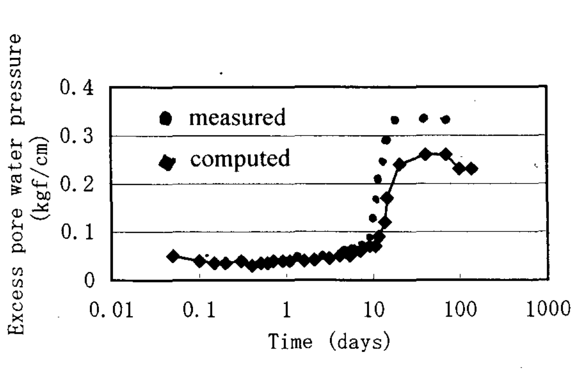

Fig. 2 Excess pore water pressure of element 36