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Excess Pore Water Pressure Response in Soft Clay under Embankment

성토하부 연약지반에서의 과잉간극수압 거동

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요 지

성토하부 연약지반에서 상재하중의 변화가 없는 상태에서 과잉간극수압의 증가가 Berthierville와 Olga 지역에서 보고되었다. 이와같은 비정상적인 현상들을 고전적인 압밀이론으로 설명하기는 어렵다. 본 논문은 비선형 점소성모델을 사용하여 자연점토지반에서 유발되는 간극수압 증가를 설명하였다. 제안된 모델은 Darcy 법칙에 의한 간극수압 소산과 점소성 변형에 의한 간극수압 생성에 대한 복합적인 거동을 모사할 뿐만 아니라 시간 의존적인 점소성 거동과 변형율 속도 의존적인 선행압밀하중 특성을 표현할 수 있다. 수치해석기법을 적용하여 Berthierville와 Olga 지역의 성토과정을 해석하였으며, 수치해석 결과와 계측된 값을 비교한 결과 제안된 비선형 점소성모델을 사용하여 성토가 끝난 직후 간극수압의 증가현상을 설명할 수 있다.

Abstract

Increases in excess pore water pressure without change of surcharge load were reported in clay underneath embankments at Berthierville and Olga sites after the end of construction. These abnormal phenomena could not be explained by classical consolidation theory. This paper presents a nonlinear viscoplastic model to interpret an increase in pore water pressure on natural clay. The proposed model can consider the combined processes of pore water pressure dissipation according to Darcy's law and pore water pressure generation due to viscoplastic strain, as well as time-dependent viscoplastic behaviour and strain rate dependency of preconsolidation pressure. The calculated results using numerical analysis are compared with measured ones under embankments built on soft clay at Berthierville and Olga in Québec, Canada. It may be possible to explain the phenomenon of excess pore water pressure increase after the end of construction using the proposed nonlinear viscoplastic model.

Keywords : Embankment, Excess pore water pressure, Viscoplastic behavior

1. Introduction

Consolidation is a time-dependent process associated with change of pore water pressure and deformation of soft clay. In classical consolidation theory, pore water pressure is only generated by external load and dissipation of pore water pressure is a process of consolidation.

Yoshikuni et al. (1995) performed special oedometer tests with repeated operations of opening and closing a drainage valve in order to demonstrate that "consolidation is a combined process of pore water pressure dissipation

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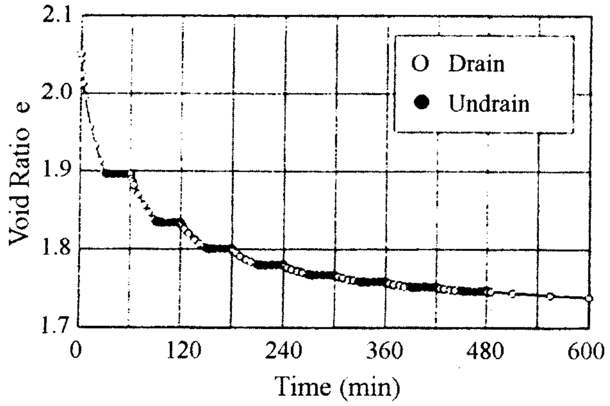


Fig. 1. Variation of void ratio with time (Yoshikuni et al., 1995)

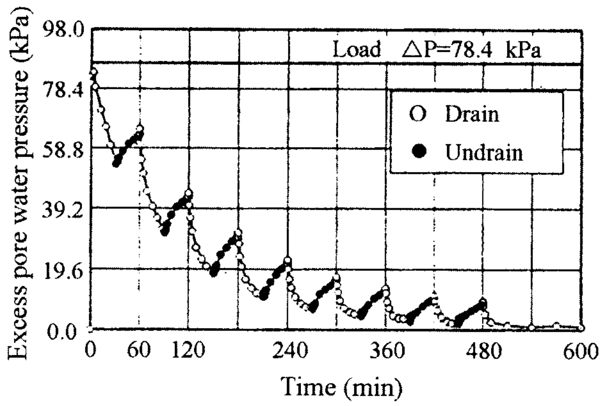


Fig. 2. Variation of excess pore water pressure with time (Yoshikuni et al., 1995)

due to drainage and pore water pressure generation due to stress relaxation, even under constant load". Figures 1 and 2 show the variation of void ratio and excess pore water pressure with time, respectively. It is obvious from Fig. 2 that pore water pressure is generated by stress relaxation throughout the consolidation process. In this particular test, 8 repetitions of drain and undrain cycles were carried out. Since the actual situation is regarded as an infinitive repetitions with an infinitesimal interval, it can be said that consolidation is a combined process of dissipation and generation of pore water pressure.

Chang (1981), Becker et al. (1985), and Kabbaj et al. (1988) observed abnormal phenomenon such as increase in pore water pressure for some time after loading of an embankment. Becker et al. (1985) reported that excess pore water pressure increased for approximately 4 months, even though the island construction was completed and

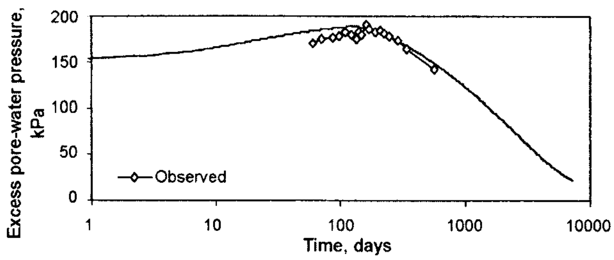
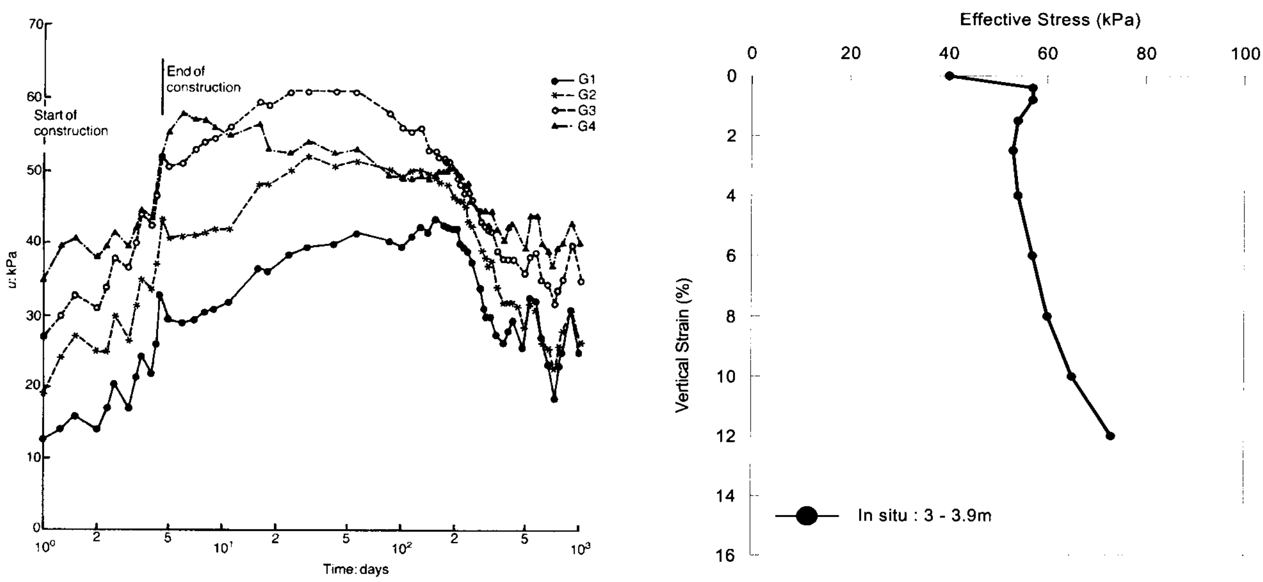


Fig. 3. Observed excess pore pressure in clay under Tarsiut Island



(a) Excess pore pressures with time

(b) Stress-strain curve

Fig. 4. Observed behaviors at Berthierville

the vertical load on the foundation was unchanged, as shown in Fig. 3. Kabbaj et al. (1988) also reported that pore water pressure increased after the end of construction, while the effective stress decreasing after in-situ stress approached to preconsolidation pressure (Fig. 4). G1, G2, G3 and G4 in Fig. 4 stand for piezometers installed in clay deposit to measure excess pore water pressure during and after construction of embankment.

Crawford (1986) suggested that excess pore water pressure was generated by the collapse of clay structure near preconsolidation pressure, especially in highly structured and sensitive soils. Chang (1981) and Kabbaj et al. (1988) suggested that increase in excess pore water pressure may result from viscous strain.

From the viewpoint of classical consolidation theory, it is hard to explain an increase in pore water pressure on natural clay due to viscoplastic strain. In order to simulate combined process of pore water pressure response during consolidation after loading of embankment and give an interpretation of increase in excess pore pressure, this paper uses a nonlinear viscoplastic model proposed by Kim et al. (2001).

2. Nonlinear Viscoplastic Model

The continuity equation for consolidation process can be obtained from the principle of continuity of mass and consideration of Darcy's law (Berry and Poskitt, 1972)

$$-\frac{\partial \varepsilon_v}{\partial t} = \frac{1+e_o}{\gamma_w}\frac{\partial}{\partial z}\left(\frac{k_v}{1+e}\frac{\partial u}{\partial z}\right) \qquad (1)$$

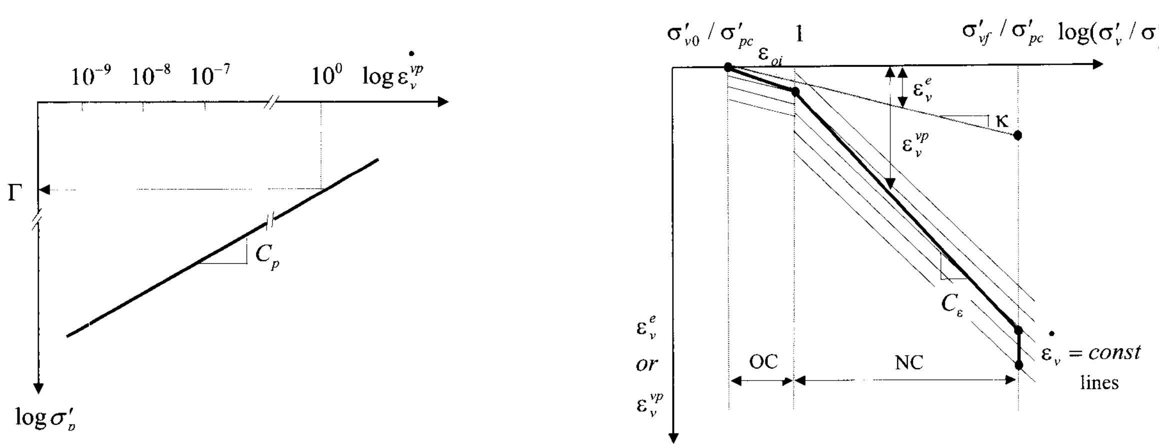
where t = time; z = depth; ε_v = vertical strain; u = excess pore water pressure; e_o and e = initial and current void ratios, respectively; k_v = hydraulic conductivity varying with void ratio; γ_w = unit weight of water.

A rate of total strain at any state point (σ_v', ε_v) can be expressed as the sum of an elastic strain rate component, $\dot{\varepsilon}_v^e$, and a time-dependent viscoplastic strain rate component, $\dot{\varepsilon}_v^{vp}$. The resulting constitutive equation for vertical strain rate is (for more detail, see Kim and Leroueil, 2001):

$$\dot{\varepsilon}_v = \dot{\varepsilon}_v^e + \dot{\varepsilon}_v^{vp} = \frac{\kappa}{1+e_o}\frac{\dot{\sigma}_v'}{\sigma_v'} + 10^{[(\log\sigma_v' - \Gamma - \varepsilon_{oi} - C_\varepsilon \varepsilon_v^{vp})/C_p]} \qquad (2)$$

where κ = recompression index, σ_v' = effective stress, Γ = the value of $\log\sigma_p'$ at $\dot{\varepsilon}_v^{vp} = 10^0$, C_p= preconsolidation index (the slope of $\log\sigma_p'$ against), C_ε = compression index (the slope of $\log(\sigma_v'/\sigma_p')$ against), and ε_{oi} = intercept. The values of Γ, C_p, C_ε, and ε_{oi} are constant for given ranges of strain rate and strain, respectively.

As schematically illustrated in Fig. 5, each preconsolidation pressure is dependent on the corresponding strain rate occurring during the consolidation process. Total strain during consolidation process is the sum of the



(a) $\dot{\varepsilon}_v^{vp} - \sigma_p'$ relation (b) $\sigma_v'/\sigma_p' - \varepsilon_v^{vp}$ relation

Fig. 5. Schematic diagram of nonlinear viscoplastic model

elastic and viscoplastic strain components. Viscoplastic strain takes place in both the overconsolidated and the normally consolidated regions (Leroueil et al., 1985), even though current excess pore water pressure is not fully dissipated. The viscoplastic component is however much smaller in the overconsolidated region than in the normally consolidated one.

The variation of excess pore water pressure with time can be induced from Eqs. (1) and (2) as follows:

$$\dot{u} = \dot{L} + \frac{(1+e_o)\sigma_v'}{\kappa}\left\{\frac{(1+e_o)}{\gamma_w}\frac{\partial}{\partial z}\left(\frac{k_v}{1+e}\frac{\partial u}{\partial z}\right) + 10^{[(\log \sigma_v' - \Gamma - \varepsilon_{oi} - C_\varepsilon \varepsilon_v^{vp})/C_p]}\right\} \tag{3a}$$

$$\frac{\partial u}{\partial t} = \left(\frac{\partial u}{\partial t}\right)_{Loading} + \left(\frac{\partial u}{\partial t}\right)_{Dissipation} + \left(\frac{\partial u}{\partial t}\right)_{Viscoplastic-strain} \tag{3b}$$

where $\dot{L}$ = rate of surcharge stress change, and $\log k_{vo} - (e_o - e)/C_k$ in which C_k is the permeability change index defined as the slope of the $e - \log k_v$ relationship.

As expressed in Eq. (3), variation of the excess pore water pressure with time can be produced by a change in pore water pressure due to surcharge loading, by pore water pressure dissipation due to vertical flow according to Darcy's law, or pore water pressure generation due to viscoplastic strain. The first and third terms of Eq. (3) show the rate of generation of pore water pressure and the second term governs the rate of dissipation of pore water pressure. The response of pore water pressure is influenced by the resultant of these three terms.

Relaxation effect on the effective stress is dominant when the amount of pore water pressure generation due to viscosity is larger than the amount of pore water pressure dissipation. It is worth noting from Eq. (3) that the smaller the values of C_ε and permeability, the more pore water pressure generation. Therefore, increase in excess pore water pressure might be produced in highly structured and sensitive clay with low permeability.

3. Comparison of Calculated and Measured Behaviors

3.1 Berthierville Test Embankment

A test embankment with diameter of 29m was built at Berthierville (Kabbaj et al. 1988). Below 10~20cm of top soil and about 2m of fine-to medium-sand layer is a soft silty clay of 3.2m thick, which is underlain by a fine sand layer. This clay is relatively homogeneous without a fissured crust, and is well confined between two layers of sand. The simulation of the behavior of the clay foundation at Berthierville embankment has been performed using the parameters defined in Table 1.

Fig. 6 shows the variation of strain rate occurring at

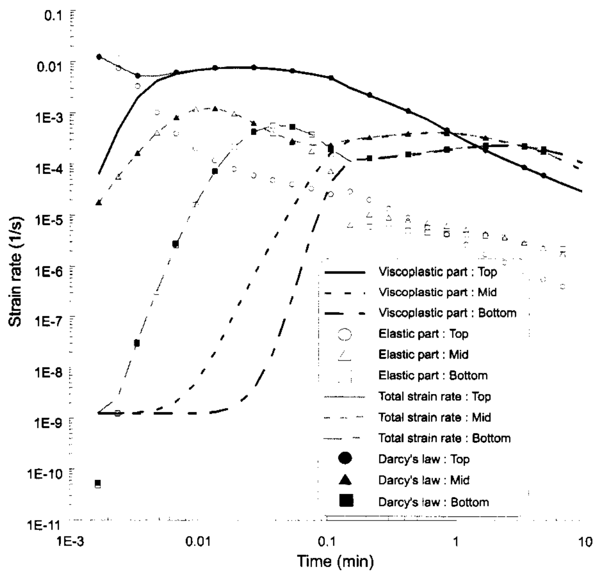


Fig. 6. Variation of strain rate occurred at several positions with time

Table 1. Material properties of Berthierville clay foundation

Layer (Depth, m)	e_o	C_s	k_{vo} (m/sec)	C_k	σ_o' (kPa)	σ_p' (kPa)*
# 1 (2.3~3.0)	1.73	0.01	2.5×10^{-9}	0.865	35.0	60.0
# 2 (3.0~3.9)	1.73	0.01	3.2×10^{-9}	0.865	39.0	64.0
# 3 (3.9~4.8)	1.57	0.01	4.3×10^{-9}	0.785	47.0	69.0
# 4 (4.8~5.5)	1.45	0.01	5.0×10^{-9}	0.725	52.0	73.0

* preconsolidation pressure at $\dot{\varepsilon}_v$ = 6.35×10^{-6} and T = 20℃

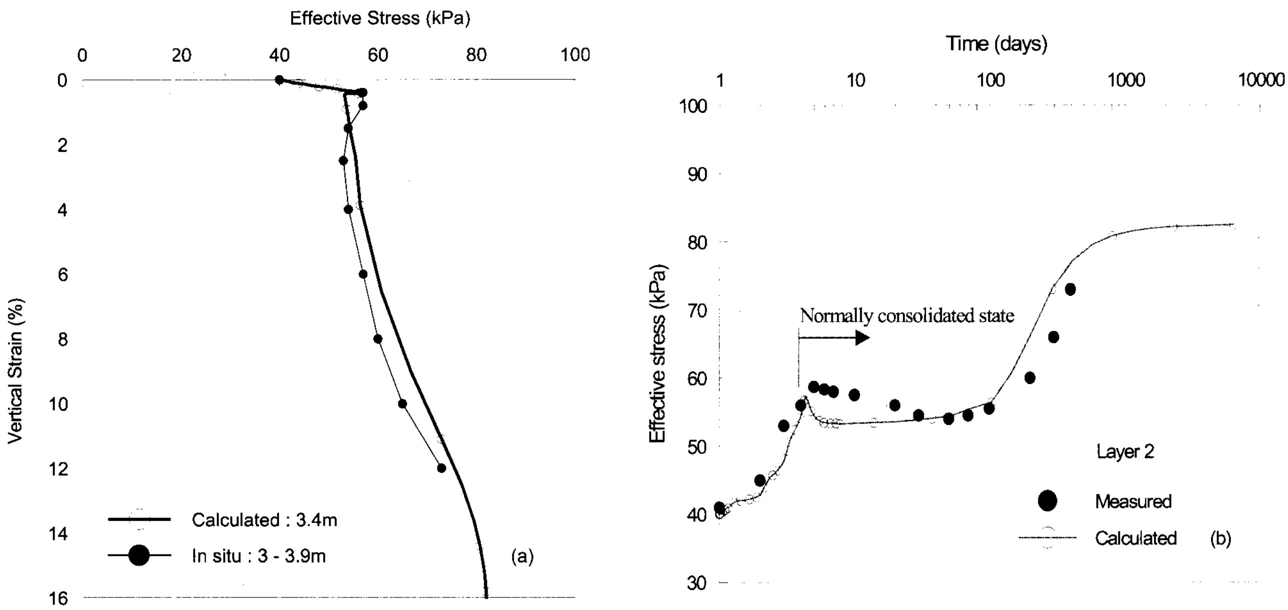


Fig. 7. Stress-strain relation and variation of effective stress with time

top, mid and bottom planes with time. Total strain rate is the sum of an elastic strain rate and viscoplastic strain rate. Also total strain rate is equal to the strain rate result from Darcy's law, as shown in Fig. 6. It is worth noting that the total strain rates are dominated by elastic strain rates in overconsolidated state, but are dominated by viscoplastic strain rates in normally consolidated state.

The calculated effective stress-strain curve is presented in Fig. 7a, together with the in-situ stress-strain curve. The calculated effective stress-strain relation for the considered layer is in good agreement with the measured one. It is noted from Fig. 7a that increase in excess pore water pressure is induced after effective stress approaches to preconsolidation pressure.

Variation of effective stress with time is also shown in Fig. 7b. Stress relaxation takes place after the end of construction in the case that pore water pressure generation due to viscoplastic strain is larger than what can be dissipated according to Darcy's Law. This generation of pore water pressure is affected by values of permeability and nonlinear viscoplastic model parameters.

3.2 Olga-C Test Embankment

The Olga-C test embankment was built on the deposit of soft, sensitive varved clay. The Olga-C test embankment with height of 6m is 50m in top length and 10m in top width. The observations made during construction and the following two years are described by St-Arnaud et al. (1992). Olga-C test embankment was built with four different zones in 1990 (Lavallee et al., 1990): Zone A had no drain; Amerdrain vertical drains were installed in zone B with a spacing of 1.5m and in zone C with a spacing of 1.0m; Alidrain vertical drains were used in zone D with a spacing of 1.0m. All vertical drains were installed in triangular pattern in Olga-C test site. In this study, zones A and B were chosen to compare field behaviors of soft clay with and without drains. The material parameters for numerical analysis are shown in Table 2.

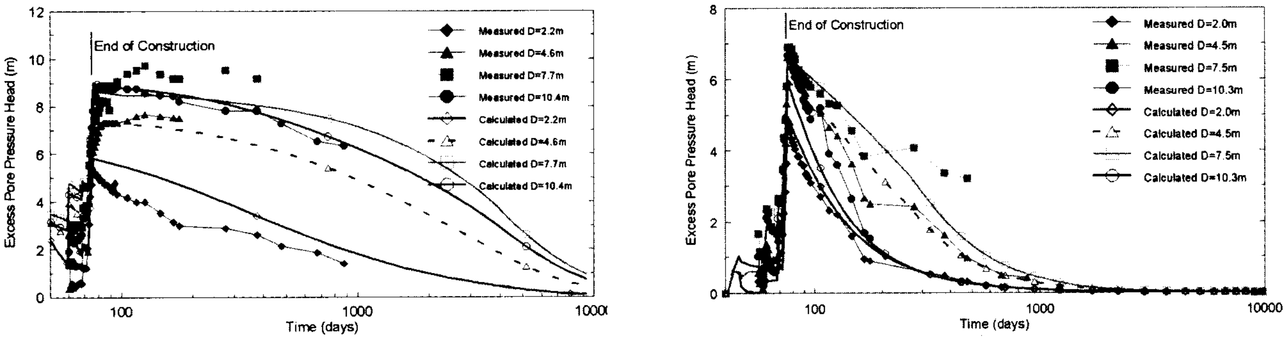
The calculated dissipation curves of excess pore water pressure in zones A and B are presented in Fig. 8, together with measured excess pore pressures. The calculated dissipation curves of excess pore water pressure are in good agreement with measured ones. The excess pore pressure in zone B was reduced to zero after about 1000days, but about 10000days in zone A. Excess pore pressures more rapidly decrease in zone B than in zone A. The excess pore water pressure in zone A increased after the end of construction, but not in zone B.

Table 2. Material properties of Olga-C embankment

Layer	Depth (m)	e_o	C_s	k_{vo} (m/sec)*	k_{ho}/k_{vo}	C_{kh}, C_{kv}	σ_p' (kPa)*
# 1	0 - 2	1.45	0.01	4.0×10^{-9}	1.0	0.60	120.0
# 2	2 - 4	2.31	0.01	4.0×10^{-9}	1.0	1.20	80.0
# 3	4 - 6.5	2.47	0.01	1.5×10^{-9}	1.25	1.35	78.0
# 4	6.5 - 9	2.32	0.01	2.0×10^{-9}	1.25	1.20	80.0
# 5	9 - 11	2.35	0.01	2.8×10^{-9}	1.25	1.25	85.0
# 6	11 - 13	1.15	0.01	3.0×10^{-9}	1.25	0.40	105.0
# 7	13 - 14	0.85	0.01	4.0×10^{-9}	1.25	0.20	115.0

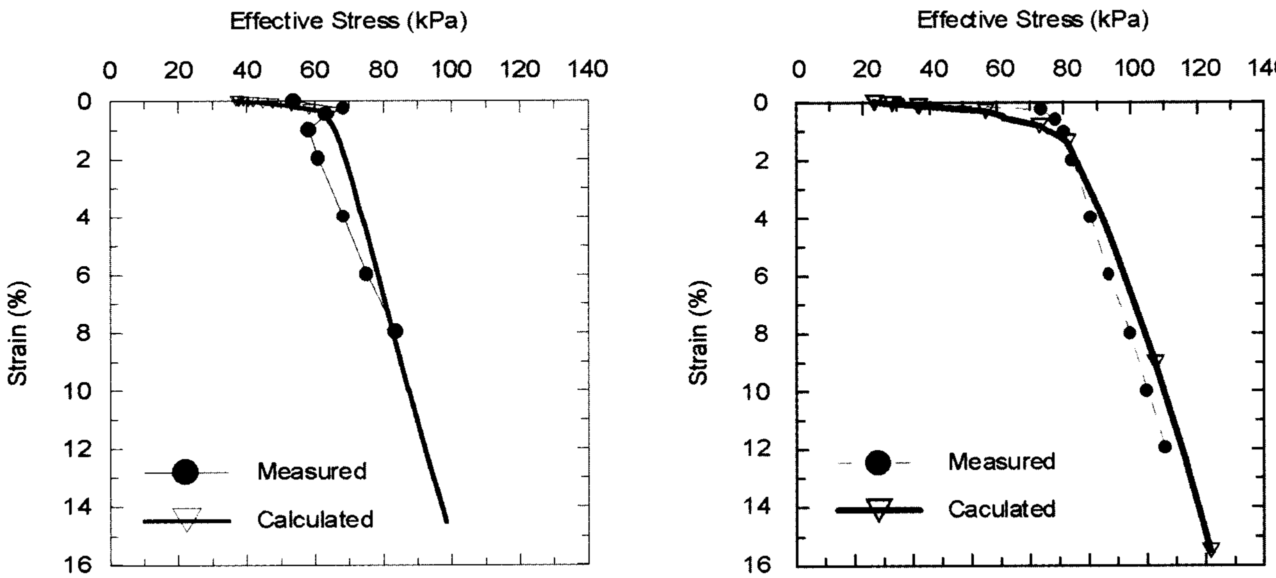
* Coefficient of permeability at the field temperature

** Preconsolidation pressure measured in 24 hrs conventional oedometer tests at 20℃



(a) Zone A without vertical drain (b) Zone B with vertical drain

Fig. 8. Dissipation of excess pore pressure with time



(a) Zone A without vertical drain (b) Zone B with vertical drain

Fig. 9. Stress-strain relations of layer 4 in Olga-C test embankment

Fig. 9 shows the effective stress-strain relations in zones A and B. There is effective stress relaxation in soft clay without vertical drain, but not in drainage-installed deposit of Olga-C site. For the case of zone A, the effective stress decreased (about 5 *kPa*) after in-situ effective stress approached to the preconsolidation pressure, with pore water pressure generation due to viscoplastic strain larger than what can be dissipated according to Darcy's law. For zone B, effective stress relaxation does not take place due to the dissipation of pore water pressure from combined vertical and radial flows are greater than the generation of pore pressure.

As shown in Fig. 9, in-situ preconsolidation pressures are about 70 kPa in zone A and 80 kPa in zone B, respectively. In-situ preconsolidation pressure is about 14% larger in zone B than in zone A. This means that strain rate occurring in drainage-installed deposit is larger than that of zone A without vertical drain.

3.3 Discussion of Increase in Excess Pore Water Pressure Caused by Viscoplastic Strain

The Mandel-Cryer effect is sometimes used to explain pore water pressure increase in linear elastic soil under plane strain condition and when vertical load was applied suddenly. The embankments of Tarsiut Island, Berthierville and Olga-C were not constructed suddenly and the clay behaviors were not linear elastic. It was also found that the finite element consolidation model using the modified Cam-clay constitutive relationship was unable to simulate the phenomenon of excess pore water pressure increase in the clay (Conlin et al. 1985). FE model could not consider the creep nature of soils. Therefore, the Mandel-Cryer effect could not explain increases in excess pore water pressure in these clays.

The excess pore water pressure dissipates much faster in the overconsolidated range than in the normally consolidated range. It indicates that k_v is not constant but dependent on void ratio or effective stress, and the value of permeability will be larger in the overconsolidated range than in the normally consolidated range. After effective stress approach to preconsolidation pressure, viscoplastic strain will have more tendencies to increase due to collapse of soil structure than in the overconsolidated range. However, the dissipation of excess pore water is slower due to low permeability, so the strain caused by dissipation is less than that caused by viscoplastic strain. Therefore, the generation of pore water pressure takes place to compensate the difference of volume changes between viscoplastic strain and pore water pressure dissipation.

Fig. 10 shows a schematic diagram for interpretation of the increase in pore water pressure due to viscoplastic strain and the associated decrease in effective stress. After the end of construction, total stress is not changed. Thus the first term of Eq. (3) is zero during consolidation. The rate of pore water pressure generation and corresponding strain are defined as follows:

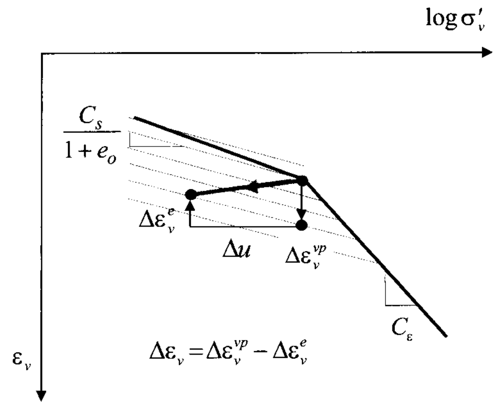


Fig. 10. Schematic diagram for interpretation of pore pressure increase due to viscoplastic strain

$$\frac{\partial u}{\partial t} = \left(\frac{\partial u}{\partial t}\right)_{Dissipation} + \left(\frac{\partial u}{\partial t}\right)_{Viscoplastic-strain} \quad (4)$$

$$\Delta\varepsilon_v = \Delta\varepsilon_v^{vp} - \Delta\varepsilon_v^{\varepsilon} \quad (5)$$

4. Conclusions

Increases in excess pore water pressure after the end of construction were observed under embankments, especially in highly structured and sensitive soils. A nonlinear viscoplastic model was used to explain the variation of pore water pressure during consolidation. This model was applied to simulate the behavior observed under test embankments at Berthierville and Olga-C, Quebec. Results show that the phenomenon of pore water pressure increase can be explained by viscoplastic model. From the results of analysis, the following conclusions could be drawn:

(1) Variation of the excess pore water pressure with time can be influenced by a change in pore water pressure due to surcharge load, by pore water pressure dissipation due to vertical flow according to Darcy's law, or pore water pressure generation due to viscoplastic

strain.

(2) As shown in Figs. 7(a) and 9(a), increase in excess pore water pressure was produced in clay after effective stress approaches to preconsolidation pressure at Berthierville and Olga-C embankments.

(3) Pore water pressure generation caused by viscoplastic strain occurs under the condition that amount of pore water pressure generation is larger than what can be dissipated according to Darcy's law.

(4) The generation of pore water pressure may take place to compensate the difference of volume changes between viscoplastic strain and pore water pressure dissipation.

References

1. Becker, D. E., Jefferies, M. G., Shinder, S. B., and Crooks, J. H. A. (1985), "Porewater pressure in clays below caisson island", *Proceedings of the American Society of Civil Engineers Arctic 85 Conference*, San Francisco, pp.75-83.
2. Berry, P. L., and Poskitt, T. J. (1972), "The Consolidation of Peat", *Géotechnique*, **22**(1): 27-52.
3. Chang, Y. C. E. (1981), "Long term consolidation beneath the test fill at Vasby, Sweden", *Swedish Geotechnical Institute*, Report 13, Linkoping, Sweden.
4. Conlin, B. H, Jefferies, M. G., and Maddock, W. P. (1985), "An assessment of the behaviour of foundation clay at Tarsiut N-44 caisson retained island", *In Proceeding of the Offshore Technology Conference, Houston*, Texas, pp.379-389.
5. Crawford, C. B. (1986), "State of the Art: Evaluation and Interpretation of Soil Consolidation Tests", *Consolidation of Soils: Testing and Evaluation*, ASTM STP 892, Philadelphia, pp.378-404.
6. Kabbaj, M., Tavenas, F. and Leroueil, S. (1988), "In situ and laboratory stress-strain relationships", *Geotechnique*, 38, No.1, pp. 83-100.
7. Kim, Y. T. and Leroueil, S. (2001), "Modeling the viscoplastic behavior of clays during consolidation: application to Berthierville clay in both laboratory and field conditions", *Canadian Geotechnical J.*, Vol.38, No.3, June 2001, pp.484-497.
8. Leroueil, S., and Kabbaj, M., and Tavenas, F., and Bouchard, R. (1985), "Stress-strain-strain rate relation for the compressibility of natural sensitive clays", *Geotechnique*, 35(2), 159-180.
9. Yoshikuni H., Kusakabe O., and Okada M. (1995), "Mechanism of one-dimensional consolidation", *Compression and Consolidation of Clayey Soils*, Balkema, pp.497-504.

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