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Finite Element Analysis of an Embankment over soft ground: A case of a Trial Embankment

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Abstract

This paper summarizes the performance of numerically models using four constitutive laws as Mohr-Coulomb (MC), Modified Cam-Clay (MCC), Hardening Soil (HS), and Soft Soil (SS) with comparing the numerical results with field measurement of the Saga trial embankment in Japan. The input parameters for these constitutive models are derived from the laboratory test results. The embankment is equipped with settlement gauges, piezometers, and inclinometer. PLAXIS 2D software is used for the numerical analysis. The study reveals that the SS model captures satisfactory the embankment behavior on the soft soil in terms of settlement, excess pore water pressure distribution and lateral displacement.

Keywords: *Soft soil; consolidation; embankment; plaxis; finite element modelling*.

1 Introduction

The use of preload or surcharge design techniques, regardless of the incorporation of prefabricated vertical drains (PVDs), necessitates predicting the period-dependent deformations of embankments built on very soft ground. For results to be cost-effective, precise prediction is crucial. Underestimating settlements can result in lower-than-expected embankments necessitating more earthwork during construction or post-construction performance that falls short of expectations (Kelly et al., 2018). If settlements are overestimated, additional fill will need to be removed and ruined once settlement is complete than was originally anticipated. If either of these things happens, it will waste money. Verifying that the constitutive model and the implemented values of the computational parameters can capture the strength and deformation behaviors of the subsoil effectively is crucial for predicting the mechanical behavior of a clay-rich deposit beneath embankment loading employing the finite element (FE) approach (Huang Wenxiong et al., 2006). However, can be no tried-and-true way to do such a check.
Based on this statement, a series of studies have been performed. (Rajesh Bande et al., 2018) Performed a study on embankment resting on soft clay foundation at KUMPP in India numerically. They employed MC, HS and SSC materials models to check the suitability of capturing the consolidation behavior of the soft clay. Their study revealed that the MC model had been shown the well in agreement with the field measurements. (Nasvi & Krishnya, 2019) performed the stability analysis of CKE embankment over the organic peat soil. They used PLAXIS 2D software in which MC, SS and SSC had been applied to model the organic peat and the other subsoil soil parameters. They concluded that the SS and SSC models agree well with the field time-settlement measurements. The MC model yielded poor output due to the simple linear model.
On September 12th and 13th, 2016, in Newcastle, Australia, a symposium was conducted to predict the Ballina Bypass's embankment (Kelly et al., 2018). The seminar drew 28 predictions, 12 from Australian professionals and 16 between Australian and foreign academics. The MC, HCM (Hunter Clay Model), HS, SS, MCC, SSC (Soft Soil Creep) models were employed to analyze the Ballina Bypass embankment behavior over the soft soil. Finally, the study showed that the SS and SSC models agreed well with the field measurement.
In this study, the Saga trial embankment case study (Chai et al., 2013) has been chosen to validate different constitutive models such as MC, MCC, HS and SS. The input parameters of these constitutive models are derived based on the laboratory tests and the site investigation data.

1.1 Saga Trial Embankment

In 2013, construction began on a roadway around the Ariake Sea. The entire length of this highway is built on top of pliable Ariake clay. A test embankment was constructed on the naturally occurring deposit. Its performance concerning settlements, lateral displacements, and excess pore water pressures was monitored for over three years to confirm the accuracy of the assumed strength of design and deformation characteristics of the soft deposit, as calculated from laboratory tests. The relevant dimension of the embankment has been depicted in Figure 1. The groundwater table was arrested at 1m below the surface level. A total of six boreholes were drilled up to 20m deep from the surface to collect the undisturbed sample for laboratory testing. The laboratory tests include 1D consolidation test, index properties, unconfined compressive strength tests, consolidated undrained triaxial compression test, etc. The summary of the laboratory tests is given in Table 1. To observe the design assumption in the field, the monitoring instruments as settlement gauge, piezometer, and inclinometer were installed, as seen in Figure 1 .

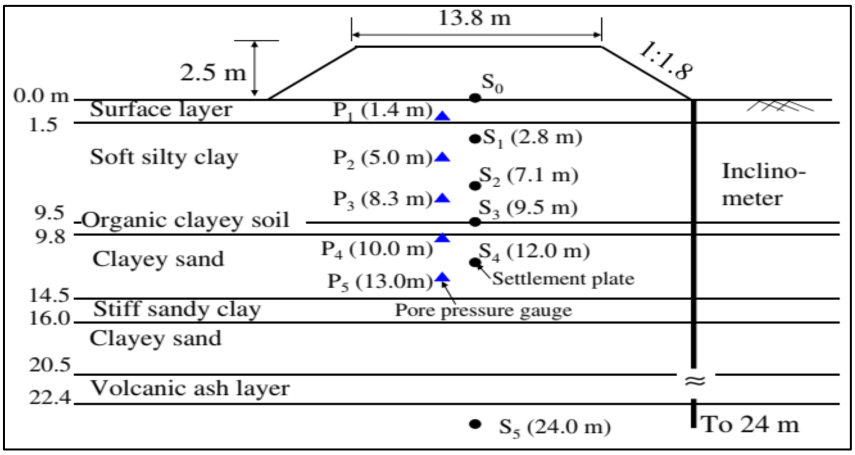

Figure 1. Typical cross section of the Saga trial embankment (Chai et al., 2013)

Table 1. In situ properties of the sub-soil (After Chai et al., 2013)

Depth	Soil Strata	γ (kN/m³)	C_c	C_r	e_0	OCR	k_v (m/day) 10^{-4}	k_h (m/day) 10^{-4}	Su (kPa) / [SPT-N]
0-1.5	Surface	16	0.58	0.03	1.5	2.05	6	9.1	13.9
1.5-4	Soft silty Clay-1	13.1	2.46	0.12	3.71	1.4	10.4	15.6	12
4-6m	Soft silty Clay-2	13.9	2.74	0.14	2.88	1.2	17.3	25.9	14
6-8m	Soft silty Clay-3	14.1	1.93	0.10	2.72	1.1	16.5	24.7	16
8-9.8m	Soft silty Clay-4	15.7	1.52	0.10	1.91	1.05	7	10.6	15.8
9.8-12m	Clayey Sand-1	18	-	-	-	-	2500	2500	[3]
12-15m	Clayey Sand-2	18	-	-	-	-	2500	2500	[8]
15-20m	Clayey Sand-3	19	-	-	-	-	2500	2500	[15]

γ =Unit weight; C_c=compression index; C_r=recompression index; e_0=initial void ratio; k_x and k_y= Horizontal and vertical permeability; OCR=over consolidation ratio; Su=undrained shear strength.

2 Numerical Modelling

To formulate this problem in PLAXIS 2D v2021, 15 node triangular element and plane strain conditions have been selected. As the embankment is symmetric, only half section has been considered in the analysis. The groundwater flow is permitted in the top vertical boundary only. The standard fixities have been applied as displacement boundaries (Chai et al., 2013). Due to computing efficiency, a very fine mesh has been selected, as shown Figure 2 . The large deformation has been employed in this modeling to capture the real field behavior.

Total 5 Nos phases have been defined as follows:
(i) construction of 2.5m embankment in 50 days,
(ii) observation for 600 days consolidation,
(iii) observation for 730 days consolidation,
(iv) observation for 1000 days consolidation,
(v) observation for 1200 days consolidation.

To capture the consolidation behavior of the soft ground in the field, four constitutive models as Mohr-Coulomb (MC), Modified Cam-Clay (MCC), Hardening Soil (HS), and Soft Soil (SS) have been used. The input parameters of these constitutive models have been derived from laboratory tests and available field measurements. The input parameters are depicted in Table 2 to

Table 5. The change of permeability index c_K=0.4e_0 and Young's modulus E'=2500N have been adopted.

Table 2. Input parameters of the MC model

Depth (m)	γ (kN/m³)	e_0	c′ (kPa)	φ′ (°)	E′ (kN/m²)	ν
0-1.5	16	1.5	1	35	5328.3	0.15
1.5-4	13.1	3.71	1	35	4600.0	0.15
4-6m	13.9	2.88	1	35	5366.7	0.15
6-8m	14.1	2.72	1	35	6133.3	0.15
8-9m	15.7	1.91	1	35	6056.7	0.15
9.8-12m	18	-	20	35	5328.3	0.15
12-15m	18	-	20	35	20000	0.15
15-20m	19	-	20	35	37500	0.15

Table 3. Input parameters of the MCC model

Depth (m)	γ (kN/m³)	e_0	E (kN/m²)	ν	λ	κ	M
0-1.5	16	1.5	-	0.15	0.25	0.025	1.6
1.5-4	13.1	3.71	-	0.15	1.07	0.107	1.6
4-6m	13.9	2.88	-	0.15	1.19	0.119	1.6
6-8m	14.1	2.72	-	0.15	0.84	0.084	1.6
8-9m	15.7	1.91	-	0.15	0.66	0.066	1.6
9.8-12m	18	-	7500	0.15	-	-	-
12-15m	18	-	20000	0.15	-	-	-
15-20m	19	-	37500	0.15	-	-	-

γ =Unit weight; E=Young's modulus; ν=Poisson's ratio; **κ** *= slope of rebound line in e-lnp_0 plot;* **λ** *= slope of consolidation line in e-lnp_0 plot (e is voids ratio and p_0 is effective mean stress); M= strength parameter for Cam-clay model, stress ratio at failure, q_f/p_0 (q is deviator stress).*

Table 4. Input parameters of the SS model

Depth (m)	γ (kN/m³)	e_0	E (kN/m²)	ν	λ*	κ*
0-1.5	16	1.5	-	0.15	0.100	0.010
1.5-4	13.1	3.71	-	0.15	0.227	0.023
4-6m	13.9	2.88	-	0.15	0.307	0.031
6-8m	14.1	2.72	-	0.15	0.226	0.023
8-9m	15.7	1.91	-	0.15	0.227	0.023
9.8-12m	18	-	7500	0.15	-	-
12-15m	18	-	20000	0.15	-	-

Depth (m)	**γ (kN/m³)**	**e₀**	**E (kN/m²)**	**ν**	**λ***	**κ***
15-20m	19	-	37500	0.15	-	-

γ =Unit weight; E=Young's modulus; ν=Poisson's ratio; λ=C_c/2.303(1+e_o), Modified compression index; κ*= 2C_r/2.303(1+e_o), Modified swelling index;*

Table 5. Input parameters of the HS model

Depth (m)	**γ (kN/m³)**	**e₀**	**E (kN/m²)**	**ν**	**E_{50} ref (kN/m²)**	**E_{oed} ref (kN/m²)**	**E_{ur} ref (kN/m²)**	**m**
0-1.5	16	1.5	-	0.15	1250	1000	18000	1
1.5-4	13.1	3.71	-	0.15	550.7	440.5	7930	1
4-6m	13.9	2.88	-	0.15	407.1	325.7	5862	1
6-8m	14.1	2.72	-	0.15	553	442.4	7963	1
8-9m	15.7	1.91	-	0.15	550.8	440.6	7931	1
9.8-12m	18	-	7500	0.15	-	-	-	-
12-15m	18	-	20000	0.15	-	-	-	-
15-20m	19	-	37500	0.15	-	-	-	-

Stiffness modulus for primary loading in the drained triaxial shear test E_{50}^{ref} =1.25 E_{oed}^{ref} ; stiffness modulus for primary loading in the oedometer test E_{oed}^{ref} =2.3× (1+e_0) ×$\frac{P_{ref}}{C_c}$; stiffness modulus for unloading and reloading in drained triaxial shear test E_{ur}^{ref} =2.3(1+e_0) × $\frac{P_{ref}}{C_c}$ × $\frac{(1+\gamma)(1-2\gamma)}{(1-\gamma)}$; m=power

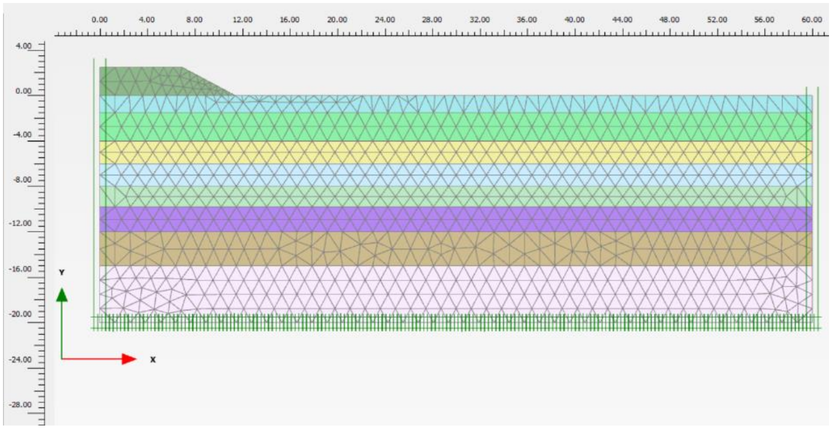

Figure 2. Finite element mesh discretization of the Saga trial embankment

3 Results and Discussion

Contrasting the predicted settlement from the PLAXIS 2D (MC, MCC, HS and SS) models with the actual settlement seen in the field was used to assess the surface settling. Figure 3 displays the projected values of surface settlement along the centerline for the four constitutive models. The estimated settlement by HS, SS, and MCC values agrees alongside the monitoring data within a respectable range. After 500 days, a fluctuating consolidation coefficient (C_v) during loading may account for the remaining minor variance in settlement (SS). The analyses used a fixed value of C_v. (Manh et al., 2020) reported that the C_v values were lower for subsoil layers in the elastic state (when loading first begins) than they were for subsoil layers in the plastic state (after loading ends). The Mohr-Coulomb model is simple and linear, predicting lower settlement magnitude (Sadiq et al., 2022) .

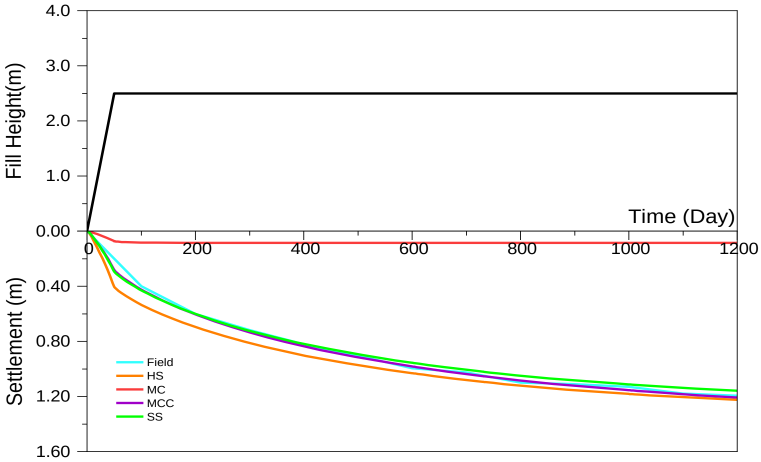

Figure 3. Time-settlement curve with filling height

The Figure 4 shows the excess pore water pressure distribution over the time for the four constitutive model. Except for the Mohr-Coulomb model, the soft soil, modified cam-clay, and hardening soil indicate better agreement with the field piezometer reading. Beyond the 1100 days, there exists a small discrepancy with field measurement. The excess pore water is dissipating lately, while the field piezometer has already shown the dissipation. It is called Mandel Cryer effect as reported by (Parsa-Pajouh et al., 2014).

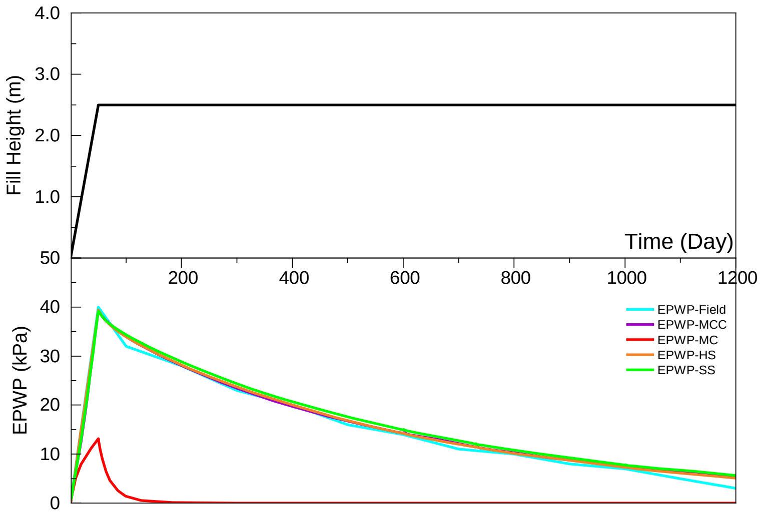

Figure 4. Excess pore water pressure distribution with time

The lateral displacements of the subsoil at 730 days have been plotted in Figure 5. As seen from the Figure 5, the soft soil model accurately captures the field measurement than other models. But the HS model produced a large displacement than the field and other constitutive models. Poulos can explain these phenomena 1972 findings: wrong assumption, non-linear stress-strain behavior of the soil, anisotropy, Poisson's ratio, and variation of permeabilities.

4 Conclusion

Based on the studies, the following conclusion can be drawn:

1) For the prediction of the soft ground consolidation behavior, it is essential to check the suitability of the constitutive model

2) The Soft Soil and Modified Cam-Clay model agreed better with the field measurement as settlement, excess pore water pressure distribution and lateral displacement. But the better accurate result was revealed by the soft soil model.

3) The simple linear Mohr-Coulomb model yielded very poor output compared to the field.

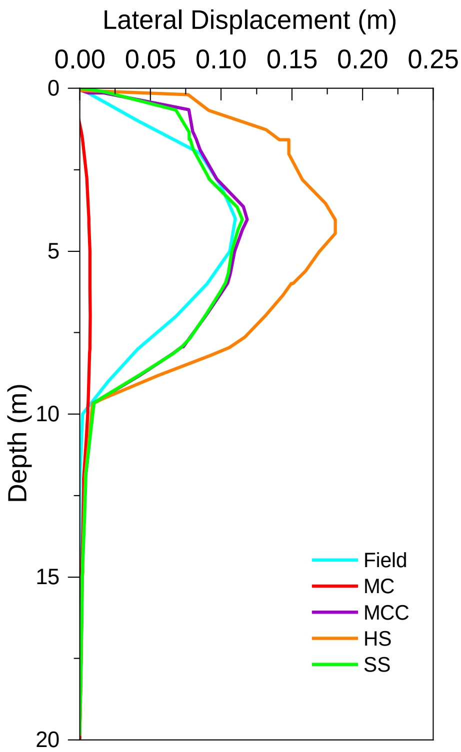

Figure 5. Lateral displacement at the toe of the embankment

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