

Numerical Modelling of Embankment on Soft Clay

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Abstract. This paper aims to predict deformation of embankment on soft clay of Muar. The prediction performance focusing on displacement at critical fill height of 5.5 m. The study was based on reported result in 1992. With the aid of computer intelligence, the advanced constitutive soil models could be adopted to analyze the soft clay behavior. The COMSOL Multiphysics (v4.4) has been used to simulate the problem with coupled physics available in the software. The vertical displacements are in good agreement close to published result.

Keywords: Embankment, soft clay, constitutive soil model, COMSOL multiphysics.

1. Introduction

Embankment has being used as a part of construction to build temporary structure on road, railway, dam and dike project in geotechnical engineering. Two components need to be taken seriously during initial stage of design are stability and deformation analysis. In present days, numerical tools provide solution in fast and accurate simulation of the geotechnical problems. The solution offers from closed form as simple to complex solution such as limit equilibrium method and finite element method. In theoretical solution four conditions should be satisfied in analysis and the basic input components are equilibrium, compatibility, material constitutive behaviour and boundary conditions [6].

Muar trial embankment on soft clay has become a subject since late 1980's to early 1990's to study the stability and deformation behaviour of the soft soil. The stability and deformation of the Muar soft clay of trial embankment were analysed and compared using Limit Equilibrium Method (total stress and effective stress analysis) and Finite Element Method (elastoplastic model) in computer programme [10].

The comparison also supported from previous data and presented by hand-calculation on similar problem and found some discrepancies had occurred. The discrepancies due to neglecting the fill strength, soil strength modulus and incomplete lateral movement data under the embankment. Due to neglecting fill strength during analysis, [2] took an effort to predict the fill strength and thickness of embankment at failure. It was found from back analysis at fill thickness of 5.4 m using uncorrected and corrected factor of 0.8 vane strength were 37 kPa and 51kPa respectively at internal friction angle, $\phi = 30^\circ$. The study predicted well the failure strength of fill embankment from back analysis were around 42 kPa and 56 kPa and found that corrected factor of 0.8 gave high strength.

The vertical displacement, horizontal displacement and consolidation effects on soft clay of Muar embankment behaviour also been carried out by [7]. The loading was placed in stages construction with soil deformation behaviour was observed in each stage (early, during and end of



construction). Undrained behaviour of soft clay developed in normal consolidated at early construction of embankment while the development of pore pressure in over consolidated clay which turn to normal consolidated clay during construction. The predicted embankment deformation failure also carried out in numerical simulation which effect of undrained, drained shear strength and effect of pore pressure incorporated in advanced constitutive soil models [8]. There are three ways to apply incremental embankment load in finite element analysis [3]: 1) surface loading, 2) increasing the gravity of embankment element and 3) placing a new layer of embankment element.

In present study, this paper is attempted to predict soft clay behaviour using different constitutive soil models on trial embankment in Muar. The deformation is focus on displacement of fill embankment and presented results in pattern. The behaviour factors influence the deformation of embankment on soft clay are identified and investigated.

Theoretical and Mathematical Model

The soil behaviour can be described using stress and strain quantities in p-q space. The mean and the deviatoric stresses are given by:

$$p' = \frac{1}{3}(\sigma'_{11} + \sigma'_{22} + \sigma'_{33}) \quad (1)$$

$$q = \frac{1}{\sqrt{2}} \sqrt{[(\sigma'_{11} - \sigma'_{22})^2 + (\sigma'_{22} - \sigma'_{33})^2 + (\sigma'_{33} - \sigma'_{11})^2 + 6(\sigma'^2_{12} + \sigma'^2_{23} + \sigma'^2_{31})]} \quad (2)$$

In which σ'_{ij} are the Cartesian components of effective stress. The stress ratio η is given by:

$$\eta = \frac{q}{p} \quad (3)$$

The corresponding incremental volumetric and deviatoric strains are defined as:

$$d\varepsilon_v = d\varepsilon_{11} + d\varepsilon_{22} + d\varepsilon_{33} \quad (4)$$

And

$$d\varepsilon_d = \frac{\sqrt{2}}{3} \sqrt{(d\varepsilon_{11} - d\varepsilon_{22})^2 + (d\varepsilon_{22} - d\varepsilon_{33})^2 + (d\varepsilon_{33} - d\varepsilon_{11})^2 + 6(d\varepsilon_{12}^2 + d\varepsilon_{23}^2 + d\varepsilon_{31}^2)} \quad (5)$$

2. Methodology

2.1 Constitutive Soil Model

The embankment problem is assumed in plane strain condition and behave as an elastic-perfectly plastic material. Before yielding within the elastic range, the stress-strain relationship of the soil is described by Hooke's law. Beyond the elastic range, the soil is assumed to behave as perfectly plastic in accordance with Mohr-Coulomb, Drucker-Prager and Modified Cam Clay. All material are assumed to fail in von Mises yield criterion. In COMSOL Multiphysics (v4.4) programme [5] these three models are selected to be used in analysis.

2.2 Modified Cam Clay Model

Modified Cam Clay is proposed and applied in most geotechnical problems associated with soft clay to account for flow rule which originally from Cam Clay model based on Critical State theory. Characteristics of linear isotropic Modified Cam Clay (MCC) are [1]:

- 1) Logarithmic relationship between the mean effective stress, p' and the void ratio, e
- 2) Linear stress dependency of the stiffness
- 3) Distinction between isotropic primary loading, unloading and reloading

The model best in performing geotechnical application such as embankment or foundation on soft soil which involve loading condition. The yield criteria of MCC model is an ellipse and smooth in p - q plane, with a cross section independent of Lode angle and defined as a function of the pre consolidated pressure (p_c) is described by:

$$f_y = q^2 + M^2(p - p_c)p = 0 \quad (6)$$

It is assumed that the soil obey the normality condition and plastic potentials (g) are the same as yield function (f) in the p - q plane:

$$g = f = q^2 + M^2(p - p_c)p = 0 \quad (7)$$

In the Cam Clay model, hardening is controlled by the consolidation pressure, p_c which depends exponentially on the volumetric plastic strain ε_{pvol} .

$$p_c = p_{co} e^{-B_p \varepsilon_{pvol}} \quad (8)$$

$$B_p = \frac{1 + e_o}{\lambda - \kappa} \quad (9)$$

Here the parameter p_{co} is the initial consolidation pressure, and the exponent B_p is a parameter which depends on the initial void ratio e_o , the swelling index κ and the compression index λ . The initial void ratio, the compression index and the swelling index are all positive parameters and must fulfil.

$$0 < \kappa < \lambda, B_p > 0$$

2.3 Numerical Modelling

2.3.1 Muar Embankment

A case study of Muar trial embankment [8] has been chosen based on the level of available information on the properties of the foundation and embankment filling, and the amount, type, location, and available published results of instrumentation.

The local highway authority had decided to construct fifteen trial embankment on the Muar plain to build Malaysian North-South expressway crosses 10-20 m thick soft clay deposits having low undrained shear strength with high water content threats to surface structure failure, low bearing capacity and excessive settlement. The site of trial embankment is located 20 km inland from Muar and 50 km due east of Malacca on the southwest coast of Malaysia.

Malaysian Highway Authority (MHA) [7] had provided an in situ data such as soil condition, water contents, Atterberg limit, vane shear strength, cone penetration resistance tests of the soft Muar clay to

a depth of about 17.5 m below ground level. Besides the in situ data, the laboratory testing were conducted such as direct shear test and triaxial test to obtain engineering properties of soft clay Muar. The water content is high as 100% (50-120%) and generally exceed the liquid limit (40-80%) and plasticity index in range 40-50% [2,8,10,7]. The vane shear strength has minimum value of 8kPa at 3 m depth and increase linearly with depth. The fill embankment properties were given as follow: $E=5100\text{kPa}$, $\nu=0.3$, $\gamma=20.5\text{kN/m}^3$, $c=19\text{kPa}$, and $\phi=26^\circ$). Table 1 summarizes information data from soil condition, index and engineering properties of soft clay **from MHA**.

2.3.2 COMSOL Multiphysics

The COMSOL Multiphysics [5] finite element software offers real simulation problems with partial differential equation applied in the software with elements of multiphysics such as heat, particle tracing and water flow. In this study there are three physic branches have been selected: 1) Solid Mechanics; Geomechanic, 2) Fluid and Flow; Darcy Law and 3) Mathematics; ODE and DAE. Normally the processes of numerical simulation involve with assigning geometry, material, physic branch, mesh, analyze and viewing result [11]. The deformation behavior and failure embankment are obtained from displacement curve. The analysis is done in function of undrained strength of fill embankment and different constitutive soil models.

Table 1 Soil Properties of Muar Embankment [8].

Depth	Soil type	Unit weight, γ	Grain size (%)				Coefficient of horizontal permeability, k_h	Compression ratio, C_r	Pre-consolidation pressure, p_c	Shear strength		Deformation
(m)	-	(kN/m ³)	Clay	Silt	Sand	Gravel	(m/sec)	$C_c/1+e_0$	(kPa)	c' (kPa)	ϕ' (°)	E_u (kPa)
0-2.0	Weathered crust	16.5	62	35	3	0	-	0.3	110	8	12.5	25500
2.0-8.5	Upper soft silty clay (very soft clay)	15.5	45	52	3	0	4×10^{-9}	0.5	40	14(2-5m) 22(5-8m)	14(2-5m) 7(5-8m)	6600(2-5m) 8933(5-8m)
8.5-18.5	Lower soft silty clay (soft clay)	15.5	50	47	3	0	10^{-9}	0.3	60	9(5-8m) 16(11-14m) 14(14-20m)	20(5-8m) 17(11-14m) 21.5(14-20m)	9120(8-11m) 6593(11-14m) 5884(14-20m)
18.5-19.0	Peat	-	-	-	-	-	-	-	-	-	-	-
19.0-19.9	Sandy clay	16.5	20	36	44	0	2×10^{-7}	0.1	60	-	-	-
19.0-22.5	Dense sand	-	4	20	71	5	-	-	-	-	-	-

2.3.3 Geometry of Embankment, Layered Soils, Soil Properties, Boundary Condition and Mesh Discretization

The embankment and soil conditions were model in two dimensional plane strain with stationary effect (Fig.1). The foundation depth of 22.5 m was considered enough in geometry model because the dense sand layer below the soft clay while the lateral boundary of geometry model was three times of vertical height below the toe of embankment [8]. The boundary conditions was assigned as a fixity at bottom of the soil geometry and a roller on right of the model. Due to assumption symmetrical geometry of the fill embankment and soil condition, only half side of the geometry was modelled and the symmetrical boundary condition has been selected on the left side of the geometry problem. The free triangular mesh and extra fine element size with number of five were selected to the soil body (Fig.2). The fill loading was acting as pressure beneath the embankment. All soil properties were undrained parameters and the initial horizontal, vertical stresses and pore pressure were given in Table 2.

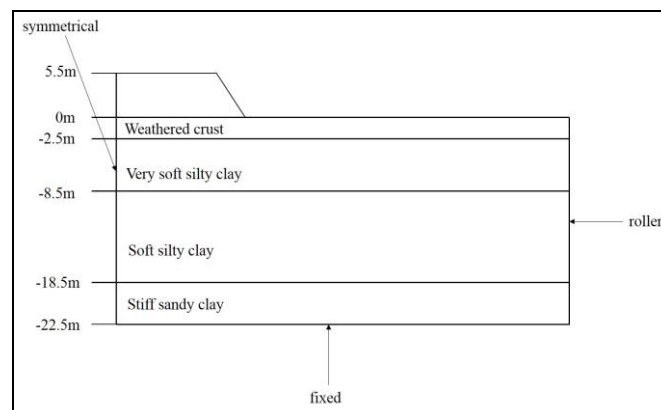


Figure 1. Geometry and boundary condition of embankment.

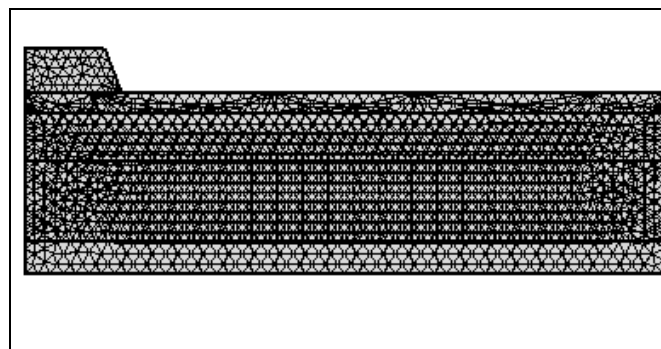


Figure 2. Mesh element assigned on the embankment model.

Table 2. In-situ stress condition and Modified Cam Clay (MCC) parameters at Muar Site.

Depth (m)	σ_{ho} (kPa)	σ_{vo} (kPa)	u (kPa)	κ	λ	e_{cs}	M	ν
0-2.5	13.2	22.0	16.7	0.05	0.13	3.07	1.19	0.3
2.5-8.5	33.7	56.1	75.5	0.05	0.13	3.07	1.19	0.3
8.5-18.5	67.9	113.1	173.6	0.08	0.11	1.61	1.07	0.3
18.5-22.5	81.5	135.9	212.9	0.10	0.10	1.55	1.04	0.3

3. Results and Discussions

For validity purpose, the deformation behaviour of embankment is conducted in three types of constitutive soil models from simple Mohr Coulomb to Drucker-Prager and finally Modified Cam Clay. The displacement results in Table 3 were obtained for fill height of 5.0 – 5.5 m. The Drucker-Prager model shows the highest total and vertical displacement of the embankment failure whereas the smallest horizontal displacement is Modified Cam Clay model. By referring to previous result the vertical displacement from Mohr-Coulomb and Modified Cam Clay models close to results presented in [8].

Table 3. Displacement result for embankment failure.

Displacement (m)	Mohr-Coulomb	Drucker-Prager	Modified Clay	Cam	Modified Cam Clay (Indraratna et al. 1992)
Vertical	0.25	0.02	0.22	0.20	

3.1 Deformation Pattern

Fig 3. shows displacement pattern for all constitutive model adopted in the study. The deformation pattern concentrate at 8.5-18.5 m depth below ground surface in soft clay layer. The Drucker-Prager model shows the highest difference compare to Mohr-Coulomb and Cam Clay which both the models have similarity with slightly difference. The deformation does not exceed beyond the soft clay layer where a state of K_0 condition is approached toward the sandy soil deposits [8].

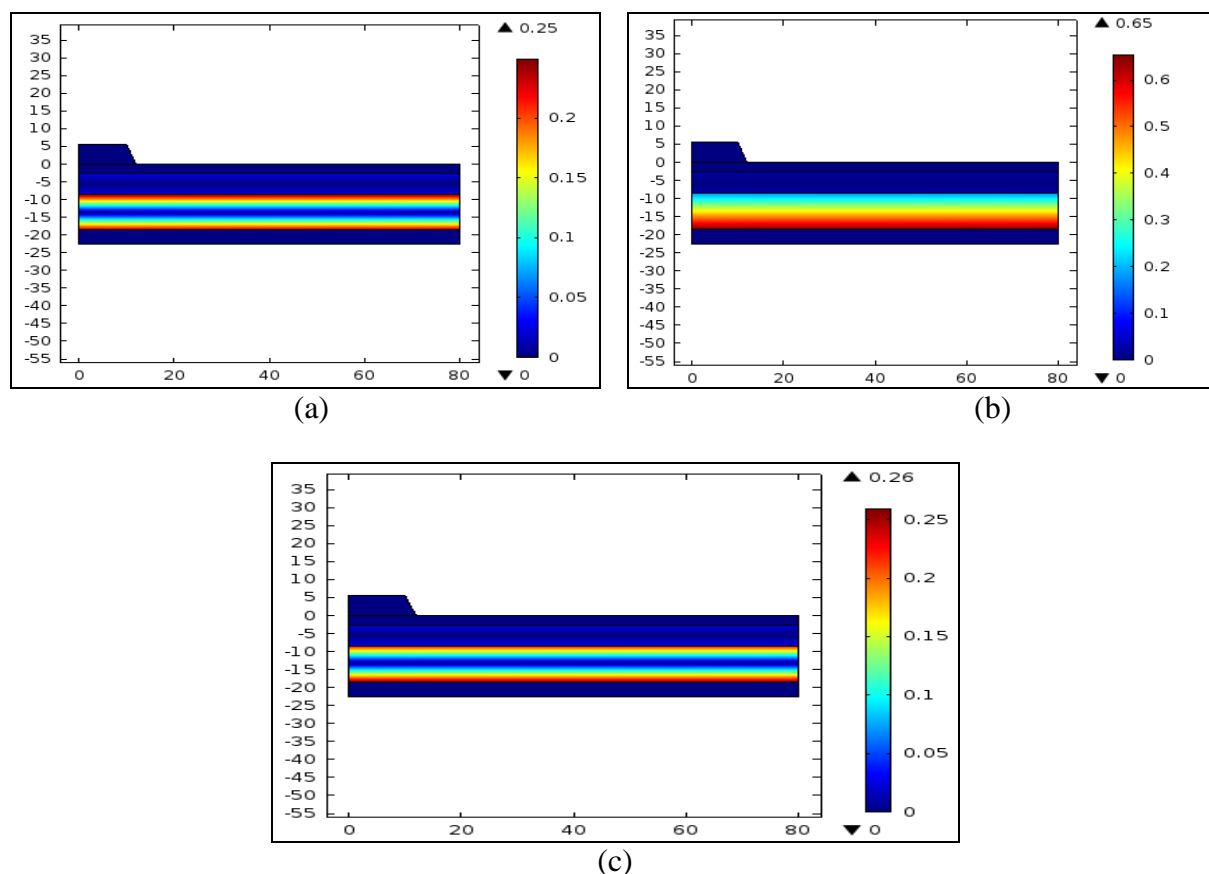


Figure 3. Displacement deformation pattern; (a) Mohr-Coulomb, (b) Drucker-Prager and (c) Modified Cam Clay.

3.2 Discussions

The deformation on displacement of fill embankment on soft clay has presented and several factors are identified due to discrepancy among the constitutive behaviour implemented in the analysis. Initially in numerical process input of material parameters play a significant role to the study. An uncorrected undrained vane shear strength of fill embankment has been assigned and basic properties were used accordingly to case study. A corrected factor of 0.8 on undrained vane shear strength increase the soil strength [4,2]. The significant difference is parameters for each soil models are not identical since the suitability in choosing the right soil model to be used in prediction the geotechnical problem behaviour accordingly to soil type. The Modified Cam Clay parameters on $e - \log p$ graph (virgin load/normal consolidated) give a parameter λ which change the displacement result influenced by initial stress condition and nature of stress path testing. Assumption in the associated flow rule overestimated the displacement and the MCC model most suitable for soft clay soil [8,1]. Assumption the absence of tension crack and shear band effect in the analysis could make some error in simulation and ignoring the importance of stages construction in embankment fieldwork and computer simulation [3] could lead to overestimated displacement to the deformation since the settlement occur in short term stability where undrained analysis take place in oppose to real in situ problem [9]. Besides this important assumptions on numerical analysis such as assigning boundary condition, loading and soil model could lead to differences in the result analysis. Even though Mohr-Coulomb and Drucker-Prager are elastic perfectly plastic, both soil models have different assumptions in their failure criterion in which Mohr-Coulomb defines an irregular hexagonal pyramid in the space of principal stresses, which generates singularities in the derivatives of the yield function and do not include the intermediate principal stress effect. The Drucker-Prager model neglects the influence of the invariant J_3 (introduced by the Lode angle) on the cross-sectional shape but give smooth of the yield surface.

4. Conclusion

The paper predicted and summarized influence factors on displacement of embankment study on soft clay soil with different constitutive soil models. The parameter estimation for displacement prediction of embankment on soft clay are should carefully determine in consideration of in situ stress condition and stress path before using in the analysis. The undrained analysis in not suitable for embankment deformation behaviour study for all types constitutive soil model which gave unreliable displacement at specific depth (8-18.5 m). The associated flow rule in general overestimates the displacement. The three types of soil model have different assumptions adopted in the analysis eventually turn to dissimilar results. It can be observed that the finite element solution offer a better result for complex problem in soil engineering.

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