Calculation and Forecast of Settlement of widened embankment

Ma Songlin
Sch. of Commun. Sci. and Eng. Harbin Inst. of Technol., Harbin 150090, China

Wang Pengfei
School of Transportation Engineering of Tongji University Shanghai, China

Wang Caixia
Sch. of Commun. Sci. and Eng. Harbin Inst. of Technol., Harbin 150090, China

ABSTRACT: The 2-D FEM has been employed to analyze the settlement of widened embankment and the interactional behavior of new and old embankments. In the calculation the elastic-plastic constitutive model (Drucker-Prager model) and constitutive model of interface with no thickness have been applied to consider the plasticity of soils and the nonlinear behavior of Interface element. This text gives the result of settlement according to the modeling method. The comparison between the predicted and measured values proves that the constitution and the models are reliable.

KEY WORDS: Widened embankment, nonlinear analysis, interface elements, FEM, two-dimension, elastic-plastic constitution.

1 INTRODUCTION

Highway is widened because of the increase of amount of traffic. When the embankment is built beside the old ones, some problems will appear in joint of embankment such as different settlement and displacement because of loading time, permanent displacement and bearing capacity. So the important work which should be fulfilled at present is to calculate and analyze the different settlement of embankments and forecast the displacement current.

In settlement calculation the layerwise summation method and Terzaghi 1-D theory of consolidation are applied at present. The two theories assume that the deformation and seepage flow is vertical. But in fact the horizontal deformation and seepage exist. Strictly 2-D method should be applied to embankment settlement calculation, so it is necessary to adopt nonlinear FEM analysis and Biot theory of consolidation in which the horizontal deformation and seepage, nonlinear characteristic of soil and stage embankment are considered. Moreover the present work calculates the widened embankment as a new one and the effect of old embankment to it is ignored. This work is not in accordance with the fact.

2 NONLINEAR ANALYSIS

There are two kinds of nonlinearities in the text: the first one is material nonlinearity because of inelasticity of soil; the second one is state nonlinearity because of part slide and abruption between embankments.

2.2 Constitutive Model of Soil and Material Nonlinearity

The selection of constitutive model not only depends on whether the model can represent the mechanics quality of soil in real condition but also the reliability and obtainment of parameters used in this model. The other factors are capability and speed of computer, expense of calculation, etc. In this text the model is built in 2-D plane and the Drucker-Prager(D-P) model is applied. This model belongs to elastic-ideal plastic ones. It is very convenient and definite to use extensively.

2.3 State Nonlinearity in Contact Interface between Embankments

Because the quality of soil in two parts of embankment differs greatly, the continuity of displacement will be broken when the stress comes to certain moment. The contact elements will be set up in two parts of soil of embankment. The contact couple is composed of two surfaces. One is interface; the other is target surface. The relationship between these surfaces is built with a same real constant. To simulate the bond, slide, separation, etc. between embankments successfully, the parameters in contact couple should be set reasonably.

3 CALCULATION METHOD

3.1 Constitutive Model

The Drucker-Prager(D-P) model is applied in the text(Zhang,1995). The yield surface F is here:

$$F = \sqrt{J_2} - \alpha I_1 - K = 0 \tag{1}$$

$$J_2 = \frac{1}{6}\left[(\sigma_X - \sigma_Y)^2 + (\sigma_Y - \sigma_Z)^2 + (\sigma_Z - \sigma_X)^2\right] + \tau_{XY}^2 + \tau_{YZ}^2 + \tau_{ZX}^2 \tag{2}$$

Where:

$$I_1 = \sigma_X + \sigma_Y + \sigma_Z \tag{3}$$

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3 - \sin\phi)} \tag{4}$$

$$K = \frac{6C\cos\phi}{\sqrt{3}(3 - \sin\phi)} \tag{5}$$

When the material is in elastic state ($F < 0$) or unloading state ($F = 0$ and $dF < 0$)
The constitutive relation is here according to yield condition and flow rule.

$$d\varepsilon_{ij} = \frac{1}{9K} dI_1 \delta_{ij} + \frac{1}{2G} dS_{ij} \tag{6}$$

When the material is in yield state ($F = 0$ and $dF = 0$), the constitutive relation is here.

$$d\varepsilon_{ij} = \frac{1}{9K} dI_1 \delta_{ij} + \frac{1}{2G} dS_{ij} + d\lambda\left(-\alpha\delta_{ij} + \frac{S_{ij}}{2\sqrt{J_2}}\right) \tag{7}$$

Where:

$$d\lambda = \frac{-3\alpha K d\varepsilon_{kk} + \left(G/\sqrt{J_2}\right)S_{mn} de_{mn}}{9\alpha^2 K + G} \tag{8}$$

The parameters "α"and "K" in formula one are represented by cohesion value"C"and angle of internal friction "ϕ" obtained from triaxial compression test.

3.2 Theory of Consolidation

The Biot theory of consolidation is applied to the present work (Qian, 1996). The network in groundwork and embankment is produced as a whole. To calculate the neutral pressure and node displacement in plane the balance equation and continuity equation of theory of consolidation is applied here.

$$\begin{cases} -G\nabla^2 w_x + \dfrac{G}{1-2\nu}\cdot\dfrac{\partial}{\partial x}\varepsilon_V + \dfrac{\partial u}{\partial x} = 0 \\ -G\nabla^2 w_z + \dfrac{G}{1-2\nu}\cdot\dfrac{\partial}{\partial z}\varepsilon_V + \dfrac{\partial u}{\partial z} = -\gamma \end{cases} \tag{9}$$

$$\frac{\partial \varepsilon_V}{\partial t} + \frac{K}{\gamma_w}\nabla^2 u = 0 \tag{10}$$

Increment method is applied to nonlinear analysis. This method is dividing the whole loading into some stages according to the construction process. Each stage has a stiffness degradation to calculate the displacement increment and stress increment.

3. 3 Building Model and Calculation

The model should reflect the real condition of embankment and some other effect. At first, the plane element having four nodes is applied to simulate the soil according to embankment model and calculation precision. The material model adopts D-P model. To reduce the calculation work, the right side is used to build real model by taking advantage of symmetry of embankment. The network is produced according to the quality of each layer of soil. The quadrangle element having four nodes is used based on figure of real model. The network is made closer in boundary between embankments. The filling load is a typical uniform load. It is the main load to conduce to the settlement. So the filling is transformed into uniform load and linear load.

The boundary condition is here (Xie, 2002): there is no level displacement in each node of the left side of groundwork. The earth's surface is freedom. The bottom and right side is fixed. The surface of groundwork is perviousness. The surface and side face of embankment are freedom. There is no level displacement in the left boundary.

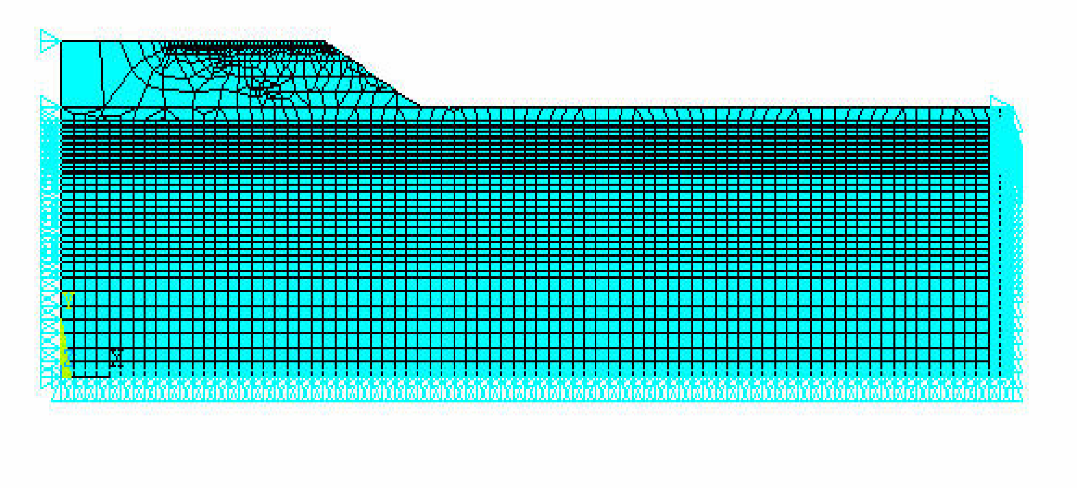

Figure 1: network and boundary condition

4 EXAMPLE AND ANALYSIS

Shen Da freeway rebuilding engineering is the first great freeway rebuilding project in our country, so its experience is very important to other similar engineering. The project has three experimental sections. The embankment is widened on both sides. The groundwork is made up of clayey loam, clay and silt. The compressibility of clay is high. And the carrying capacity is low.

Calculation section: there are four sections in this work. These are k160+660， k135+520,k171+520，k169+920.

Parameter: in order to compare the result with layerwise summation method, the routine analysis is made to the sections.

Table 1: parameter of k160+660

number	γ (kN·m^{-3})	φ (°)	C(kPa)	E_s(Mpa)	ρ (g·cm^{-3})	μ
1	17.9	9	49.4	3.4	1.83	0.3
2	18.0	12.7	28.5	2.9	1.84	0.3
3	18.1	9.1	53.2	3.9	1.85	0.3
4	18.6	15.2	95.1	4.8	1.90	0.3
5	19.1	13.9	84.3	6.5	1.95	0.35
6	18.7	17	70.5	5.7	1.91	0.3
7	19.2	14.6	76.9	8.1	1.96	0.3
8	19.2	19.6	92.7	9.2	1.96	0.3
9	22.0	15.1	120		2.24	
10	22.4	15.1	150		2.29	

Note: the ninth soil is new embankment. The tenth is old one.

4.1 Final Settlements Comparison

The results of FEM are compared with some routine method. The coefficient of compressibility method and e-p curve method are applied.

Table 2: comparison of some methods

section	coefficient of compressibility method	e-p curve method	FEM	FEM/ e-p curve method
k135+520	8.4	7.9	8.3	1.05
k160+660	18.7	20.0	21.3	1.07
k169+920	8.1	8.9	9.9	1.11
k171+520	15.8	16.1	16.9	1.05

From the result it can be seen that the calculated coefficient in compressibility method and e-p curve method are smaller than those of FEM. Because the side deformation and stress history are not considered in routine method, and the routine methods are based on 1-D compression experiment (Krize, 1977). Therefore, the FEM consider these factors. Its results fit the real instance.

4.2 Comparison between Forecast and Experiment

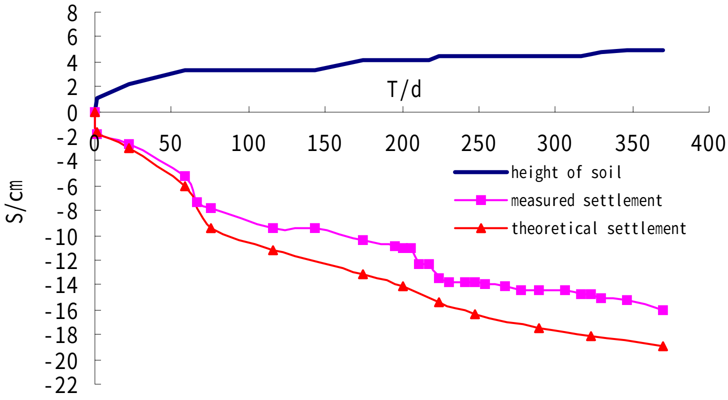

Figure 2: predicted (measured) settlement curve of section k160+660

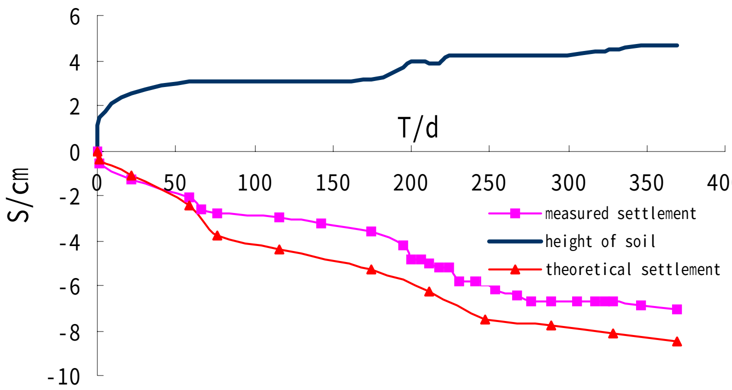

Figure 3: predicted (measured) settlement curve of section k169+920

From the figure the calculation curve and experiment curve are close. But the calculation results are greater than experiment ones as a whole. There are two inflexions in curves according to settlement process (Woboda, 1987). The first one shows that the soil is in elastic state before the load comes to yield value. When the curve comes to the second inflexion, plasticity shear deformation happens in soil. When the soil is between two inflexions stages, settlement increases steadily. In this stage the soil is in elastic-plastic state. The subsequent yield surface changes according to load.

5 CONCLUSION

FEM and contact element are applied to calculate the settlement of widened embankment successfully. In this work the real status of soil is reflected. The nonlinearity of soil and interface is stimulated by D-P model and contact element faithfully. Through FEM we can get acquainted with the rule of deformation and stress of embankment.

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