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Performance of a large dam, field measurements and analytical approach

Comportement d'un grand barrage en terre, mesure dans la site et étude théorique

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ABSTRACT
Estimation of during construction settlement of embankment dams is highly important in design and safety control of embankment and appurtenant structures. In order to evaluate the ability of Mohr-Coulomb and Soft Soil Creep models in prediction of embankment settlement, Alavian dam was selected as a well constructed and monitored case. The features of mentioned models and the dam under study have been explained in brief. The parameters required for analyses were provided screening laboratory and field test results. Then the during construction settlement profiles were established using Mohr-Coulomb and Soft Soil Creep models and compared with those obtained by direct field measurements with settlement meters. It was disclosed that, in general, the predicted settlement profiles are in agreement with that of field measurements. With Mohr-Coulomb model, better agreement is observed only in short term than the long term. However, as time passes the field measured profiles precede the predicted values. With soft soil creep model, however, agreement between the predicted and field measured values is quite encouraging all through the construction period. The predicted maximum settlement using both models, occur at 1/3 of embankment height which is coincident with field measurements

RÉSUMÉ
Evaluation du tassement d'un grand barrage en terre en construction, a une grande importance sur le dessin et le control de stabilité du corps du barrage. La capabilité des models Mohr-Coulomb et Solifluxion pour la prédiction du tassement du barrage d'Alavian, un barrage étant bien réalise et ayant d'instrument du mesure, a été étudie. Les caractéristiques des models et le barrage mention a été brièvement étudie. L'analyse des paramètres nécessaires obtenus des mesures en laboratoire et dans le site a été faite. Compte tenu des résultats obtenus, on peut alors calculer les profile du tassement du corps du barrage en construction et comparer avec les profile obtenues de la mesure dans le site du barrage. On peut conclure que ces résultats se coïncident d'une façon satisfaisante, mais, sa coincidance avec le model Mohr-Coulomb en court temps est beaucoup plus satisfaisant qu'en longtemps. Les profiles du tassement mesure par rapport aux profiles calcules avec le temps sont plus progressives. Cependant, la compatibilité du model solifluxion avec les profiles calcules et les mesures obtenues pour tout la période du construction du corps du barrage est plus satisfaisante et en courageuse. Le maximum du tassement obtenue de ces deux model a été localise en 1/3 d'hauteur du barrage qui se coïncide bien avec les mesures dans la site.

1 INTRODUCTION

Soil and rock tend to behave in a highly non-linear way under load. This non-linear stress-strain behavior can be modeled at several levels of sophistication; the higher the level of sophistication the larger the number of model parameters. In the other hand, according to the available literature, the results obtained using simpler models may be considerably erratic and far from the measured values (Sadrekarimi et al., 2003). Accordingly, establish and selection of an appropriate model with a reasonable level of complication is one of the challenges for engineers. In the same time, it is very difficult in most cases to determine engineering properties of soil for design, and engineering judgment is mostly needed. Because, the field and laboratory tests only are carried out on selected representative samples. During construction period, provided the behavior of the constructed body is monitored, the validity of the selected data for design and precision of the applied model can be appraised. Then the required modifications may be applied (Dunnicliff, 1993). Accordingly, monitoring of during construction behavior may be considered as a complementary part of a geotechnical design.

The settlement of embankment may be separated into the during construction and post construction settlement components. During construction settlement is indeed a consequence of gradual loading of a soil layer by the upper layers that are constructed sequentially. Marsal (1958) states that for a given dam height h, the settlement profile Sz at the dam axis z assumes a parabolic shape with a maximum at mid-height Sz = a (h – z) z. However, according to the available literature the maximum end of construction settlement may occur at above or beneath the embankment mid-height, depending on the soil properties and geometrical zoning of embankment (e. g. Mahab-Ghods, 1996; Pagano et al., 1998). Moreover, it seems that the maximum settlement mostly occurs from 1/3 to 1/5 of the embankment height from the foundation level (Coumoulos & Koryalos, 1978; Fell et al., 1992). For settlement at the end of construction Speedie (1970) has given the following formula S = 0.035 (H – 13). In which S and H are settlement and height of the dam in meters. According to Sherard (1963) embankment dam settlement is in the order of 2 – 4 percent of the embankment height. Finally, the results of analytical solutions may be assessed comparing direct field measurements. However, the measurement results have less value in practice unless the results are properly interpreted (Sakurai, 1999).

In order to construct clay core, the moisture content of soil is kept near optimum and minimum relative compaction is limited to 98 % standard proctor. During compaction the degree of saturation of the soil layer rises up to 90 to 95 %. Then the soil becomes highly over consolidated and negative pore water pressure develops. However, loading due to the construction of next layers neutralize the negative pore pressure and even may develop positive pore pressure (Fell et al., 1992). Accordingly, as the embankment construction proceeds, pore pressure in soil layers develops and gradually dissipates in the same time and causes consolidation settlement.

In this paper, in order to evaluate validity of Mohr- Coulomb (MC) and soft soil creep (SSC) models in prediction of during construction settlements, Alavian dam is considered as a case study. The engineering features of the dam are introduced in brief, and then the validity of mentioned models is examined through comparing the predicted results with the field measurements. The safety of the dam also is appraised.

2 ALAVIAN DAM

Alavian dam has been constructed in the north-west of Iran. The reservoir impounding started March, 1995 while the embankment construction was completed in late 1996. The embankment height from bed rock, height from river bed, crest length and crest width are 76.8, 70.0, 935 and 10 meters, respectively. It is a zoned embankment dam with a central clay core and alluvial shell. The river bed overburden alluvial and the embankment shell material are of the same origin and engineering properties. Accordingly, only the alluvial beneath clay core foundation was totally removed.

Instrumentation system of Alavian dam consists of some considerable number of variety of instruments that have been installed along 6 cross sections since the foundation surface treatment was completed. The embankment cross section along Profile 3, together with the embedded instruments is depicted in Figure 1 as a representative section. Over 90% of instruments are in good working order and some reliable records have been achieved so far. The river bed overburden is underlain by a weathered to moderately weathered tuff and then clay stone with some conglomerate localities. Field records indicate that foundation ground maximum settlement was 27 cm a few months after the embankment was completed and no further settlement has bee observed so far (Mahab Ghods, 1997).

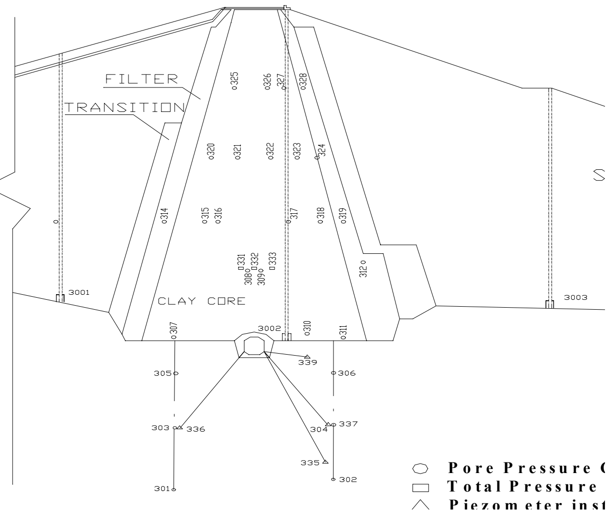

Figure 1. Cross section No. 3 and installed instruments.

3 SOFTWARE AND THEORETICAL CONCEPTS

3.1 *The software*

For computation purposes the PLAXIS package was employed. PLAXIS is Windows compatible finite element software that is suitable for analysis of mechanical behavior and flow problems in soil and rock masses (PLAXIS, 1998). Application of Linear Elastic, Mohr-Coulomb, Soft Soil Creep and Hardening Soil (HS) models are some eminent possibilities of this package. Besides, it is possible to conduct plastic, staged construction and consolidation with this software. For a 2D analysis either 6-node or 15 node triangular elements may be selected, and a second order interpolation or four order interpolation is provided for displacements, respectively. In PLAXIS, for generation of a finite element model the points and lines are generated by the user, whereas the clusters are generated by the program. Interfaces are composed of interface elements, the corresponding interface elements are defined by three pairs of nodes, whereas for 15-node soil elements the corresponding interface elements are defined by five pairs of nodes (PLAXIS, Version 7, 1998).

3.2 *Mohr-Coulomb model (perfect-plasticity)*

The Mohr-Coulomb model is a well known elastic perfectly plastic model in which the yield condition ensures that Coulomb's friction law is obeyed in any plane within a material element. The full Mohr-Coulomb yield condition can be defined in terms of principal stresses by three yield functions. These yield functions together represent a hexagonal cone in principal stress space. Three plastic potential functions also are defined for the Mohr-Coulomb model. Hence, the model involves five parameters, namely Young's modulus, E, Poisson's ratio, υ, the cohesion, C, the friction angle, φ, and the dilatancy angle, ψ. The parameter ψ is required to take into account the positive plastic volumetric strain increments if any. For stress states represented by points within the yield surface, the behavior is purely elastic and follows Hooke's law and all strains are reversible (Atkinson & Bransby, 1988; PLAXIS, 1998).

3.3 *Soft soil creep model*

As soft soils are considered normally consolidated clays, this term may be attributed to clayey silts and probably all recent fine grained non-cemented cohesive soils. The special feature of these materials is their high degree of compressibility. Another feature of the soft soil is the linear stress-dependency of soil stiffness. According to the Hardening-Soil model $E_{oed} = E_{oed}^{ref}(\sigma / P^{ref})^{m}$, at least for C = 0, and a linear relationship is obtained for m = 1. In this case this stiffness law reduces to $E_{oed} = \sigma / \lambda^{*}$,in which $\lambda^{*} = P^{ref} / E_{oed}^{ref}$. Accordingly, the Hardening-Soil model yields $\dot{\varepsilon} = \lambda^{*}\dot{\sigma} / \sigma$, which can be integrated to obtain the logarithmic compression law $\varepsilon = \lambda^{*} \ln \sigma$ for primary consolidation. In the mentioned equations C, E_{oed}, E_{oed}^{ref}, p^{ref}, σ, $\sigma^{\cdot}$, ε and $\varepsilon^{\cdot}$ are cohesion, oedometer young modulus, reference oedometer young modulus, reference pressure, stress, stress rate, strain and strain rate, respectively. From the above considerations it would seam that the HS-model is perfectly suitable for soft soils (PLAXIS, 1998). However, the HS-model is not suitable when considering creep, i.e., secondary consolidation. In practice, all soils exhibit some creep, and primary compression is thus always followed by a certain amount of secondary compression. For soils with considerable primary consolidation the secondary consolidation also will be considerable. Specially, in organic soils the major fraction of settlement may occur due to creep (PLAXIS, 1998 & Bowles, 1997). In earth dams the clay core soil is a selected soil with no/very low organic matter content and low to moderate plasticity index (ICOLD, 1988), that will be overconsolidated due to compaction (Fell et al., 1992). Accordingly, it may be inferred that the creep component of compacted clay core settlement, at least during construction and before reservoir impounding, is negligible. This, view is supported with the plate magnet displacement records when the earth fill work was halted for a few months no further appreciable displacements were recorded.

The soft soil models are basically Cam-Clay type models (Wood, 1990) which can be used to simulate the behavior of normally consolidated clays. Soft Soil Creep model, however, is a second order model formulated in the framework of visco-plasticity (Brinkgreve and Vermeer, 1998). The model can be used to simulate the time dependent behavior of soft soils. The parameters required to calculate primary consolidation with these models are failure parameters as in Mohr-Coulomb model as mentioned above, the basic stiffness parameters : κ^* ,modified swelling index ; λ^* , modified compression index; μ^* , modified creep index; and advanced parameters: υ_{ur} , Poisson's ratio for unloading-reloading; $k^{nc}_0 = \sigma'_{xx} / \sigma'_{yy}$ stress ratio in a state of normal consolidation and tends to be somewhat higher than $1 - \sin\varphi$; and M, related parameter which is calculated by default , ignoring soil cohesion, from equation $M = 6 \sin\varphi_{cv} / 3 - \sin\varphi_{cv}$ (Atkinson & Bransby, 1988)]. These parameters can be obtained both from an isotropic compression test and an oedometer test. Using oedometer test results these parameters are calculated as follows:

$$\lambda^* = \frac{C_c}{2.3(1+e)} \tag{1}$$

$$\kappa^* \approx \frac{3}{2.3}\,\frac{1-\upsilon_{ur}}{1+\upsilon_{ur}}\,\frac{C_r}{1+e} \tag{2}$$

$$\mu^* = \frac{C_\alpha}{2.3\ (1+e)} \tag{3}$$

where C_c, C_r, C_α and ν_{ur} are compression index, swell index, secondary consolidation index, and Poisson's ratio respectively (PLAXIS, 1998; Atkinson & Bransby, 1988).

4 REQUIRED DATA

4.1 *Laboratory data*

In order to achieve reliable data on embankment soil properties for design purposes some considerable number of physical and mechanical tests have been carried out in Iran and Romania. The results of these tests were collected and deliberately scrutinized, and then processed for required parameters. The final results are summarized in Table1. In order to calculate the modulus of elasticity E_s of compacted core material the following equations, together with the consolidation test results, were applied (Mesri & Terzaghi, 1999):

$$E_S = \frac{(1-2\nu)(1+\nu)}{(1-\nu)} E_{oed} \tag{4}$$

$$E_{oed} = \frac{1}{m_v} \tag{5}$$

in which:

$$m_v = \frac{a_v}{1+e_0} \quad , \qquad a_v = \frac{\Delta e}{\Delta\sigma'}$$

where e_0, a_v and m_v are initial voids ratio, coefficient of compressibility and coefficient of volume compressibility, respectively. Finally, the C_α value was calculated using equation $C_\alpha = 0.032C_c$ (Bowles, 1997).

4.2 *Field data*

With respect to the field data records, the diagrams of embankment construction progress and water level in the reservoir were established and shown in Figure 2. In order to plot the during construction settlement profiles of the embankment, settlement records of the plate magnets installed in each section were collected and plotted. Settlements corresponding to section No 3 are shown in Figures 3 and 4. These plates have been placed at 24 levels during construction of the embankment. According to the records of vibrating wire piezometers, the during construction pore water pressure values were zero or negligible and the average pore water pressure ratio r_u was 0.15.

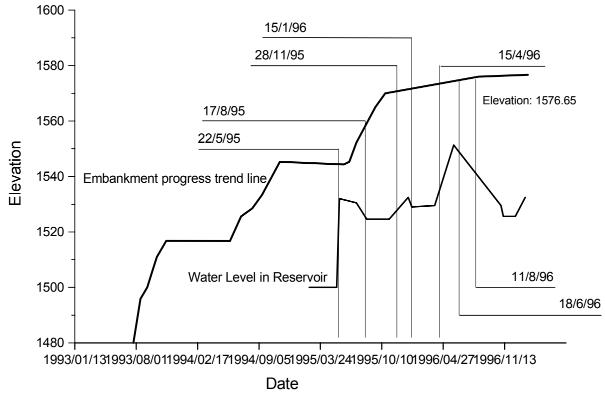

Figure 2. Embankment construction and reservoir impounding diagrams.

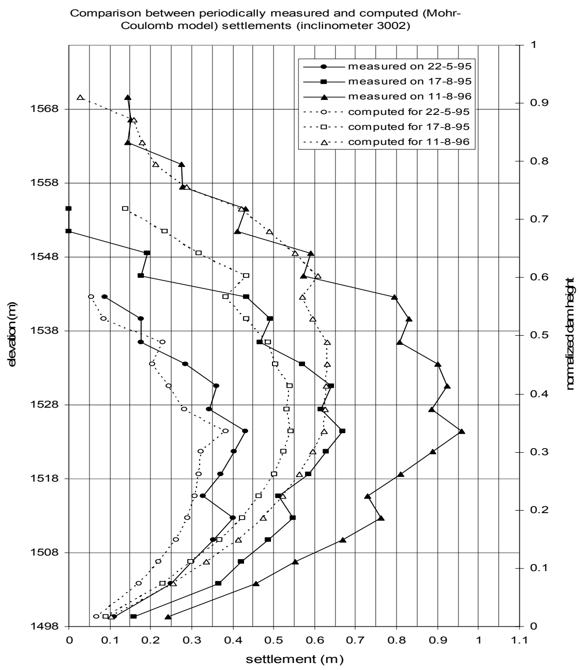

Figure 3. Comparison between field measured and computed settlement profiles (MC model).

5 ANALYSIS AND RESULTS

With a precise geometrical modeling using the practical trend of the embankment construction and reservoir impounding as shown in Figure 2, and applying both SSC and MC models, the during construction settlement profiles were established. In order to compute the values of settlement against time at each

Table 1. Engineering properties of the embankment materials.

material	φ' °	C' kPa	k cm/sec.	C_v cm^2/sec.	E_{oed} kPa	Es kPa	ν	C_c	C_s	C_α	λ^*	κ^*	κ^*
shell and drain	35	0.0	5E-2 ~ 5E-3	---	---	27E3 ~ 43E3	0.22	---	---	---			
clay core	20	50	5E-9 ~ 4E-8	3E-3 ~ 8E-3	1E4 ~ 3E4	7.5E3 ~ 22.5E3	0.28	0.1 ~ 0.2	0.01 ~ 0.02	0.003 ~ 0.005	33E-3	59 E-4	105 E-5

plate magnet level, the settlement magnitude due to the successive construction of embankment layers up to the specified level was calculated primarily and nullified geometrically while the stress conditions (stress levels and history) in the soil mass at the date of installation of the plate magnet was maintained. Then the settlement of the underlying embankment due to the weight of soil resting above the specified plate was computed at each specified time and assigned to the relevant plate. Accordingly, for each stage a new finite element mesh was required.

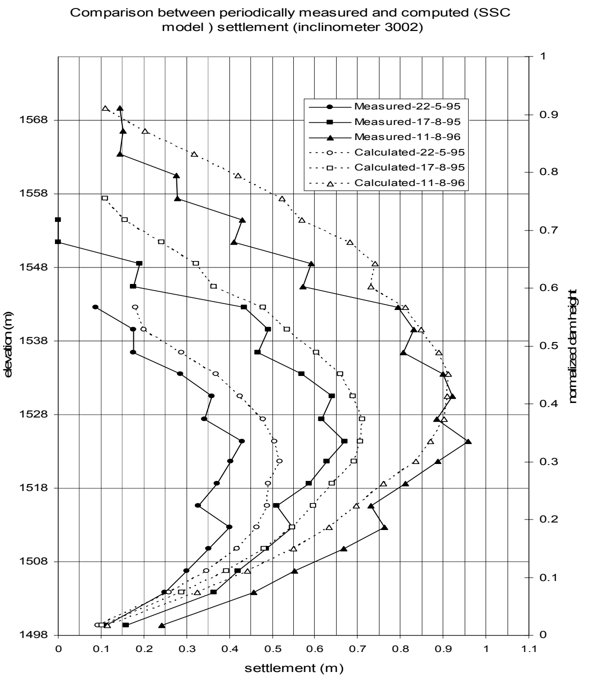

Figure 4. Comparison between field measured and computed settlement profiles (SSC model).

In Figures 3 and 4 the computed settlement results with Mohr-Coulomb (MC) and Soft Soil Creep (SSC) models are shown, respectively, and compared to the field measured values. It is valuable to note that neither computed nor field measured settlement profiles are smooth lines, these are due to the variations of materials properties , geometry of embankment cross section and also variations of free water level in reservoir and pore pressure pattern in the embankment.

6 SUMMARY AND CONCLUSIONS

1. Referring to Figures 3 and 4 it is observed that the practical maximum settlements and also the predicted values all have occurred at about 1/3 of the embankment height from the foundation level.
2. With both models, the general pattern of predicted settlement profiles is in acceptable level of agreement with the field measured profiles. However, some local discrepancies are observed. These are, of course due partially to the fact that the soil is produced by nature and so its engineering properties are not same all through the borrow area.
3. With SSC model much better and encouraging predictions than the MC model are possible and the results are quite acceptable for engineering purposes.
4. Predictions with MC model are quite acceptable in early stages of construction. However, in long term, interference of creep behavior of soil causes some erratic results with this model. While, this is not the case with SSC model.
5. The instrumentation system has been in a quite acceptable level of working order so far; and the records indicate that the dam is in a safe and stable condition.

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