EVALUATION OF UNDRAINED SHEAR STRENGTH UNDER EMBANKMENTS ON SOFT CLAYS

EVALUATION DES TENSIONS LATERALES NON DRAINEES SUR L'ARGILE MOLLE SOUS UNE LEVEE DE TERRE

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SYNOPSIS: This paper reviews the methods of stability analysis used for stage-constructed embankments. Effective stress analyses are not recommended. Undrained strength analyses based on an undrained shear strength computed as a function of the pre-shear vertical effective stress can be used, even if they can underestimate the real factor of safety in cases where there is a temporary decrease in vertical effective stress after the end of construction. Analyses based on the undrained shear strength measured in situ with the vane or with the piezocone can also be used.

INTRODUCTION

The methods for evaluating the undrained shear strength mobilized at failure under an embankment built in a single stage have been well calibrated against failures. However, for stage-constructed embankments, there is no well documented failure, and the methods of stability analysis used are some extensions of those used for single stage constructions. These methods are briefly described hereunder.

MAIN METHODS OF EVALUATION

Effective Stress Analysis

Researchers have examined effective stress analyses (ESA) and have raised some questions on several inherent difficulties. Their work is summarized by Ladd (1991). One of the difficulties is related to the significance of the calculated factor of safety in cases of stage-constructed embankments. In ESA, the factor of safety F_E is defined as the ratio of the shear stress at failure τ_f to the applied shear stress τ, both being at the same normal effective stress (path (0)-(1), Fig. 1). However, due to the pore pressures generated during loading, the stress path in the $\tau - \sigma'_n$ diagram corresponds to (0)-(2) rather than (0)-(1), and the actual factor of safety is $F_U = \tau_{fu}/\tau$. F_E thus overestimates the factor of safety corresponding to an undrained loading, except at failure where $\tau_f = \tau_{fu}$ and both F_E and F_U are equal to 1.0. This could be improved by introducing in the ESA a pore pressure increase associated to the next loading. However, the pore pressure generation is not easy to estimate in the entire clay foundation, especially when failure is approaching. Moreover, when the settlements become important, the stress redistribution in the fill and the foundation, the change in the specific weight of the clay due to settlement, and partial submergence of the embankment make the determination of the effective stresses difficult. Effective stress analyses are thus not recommended.

Undrained Strength Analysis

Ladd (1969) proposed a methodology in which the undrained shear strength is obtained from laboratory of CK_0U shear tests having different modes of failure: compression under the central part of the embankment, direct simple shear (DSS) where the failure surface is close to horizontal, and extension near the toe. The Norwegian Geotechnical Institute (Bjerrum, 1973) uses the Recompression technique in which the specimens are reconsolidated to in situ stresses prior to shearing. With the aim of minimizing the effects of disturbance on the measured shear strength, Ladd and Foott (1974) proposed the SHANSEP technique in which the specimens are first consolidated well beyond the in situ preconsolidation pressure of the intact clay and maintained in these

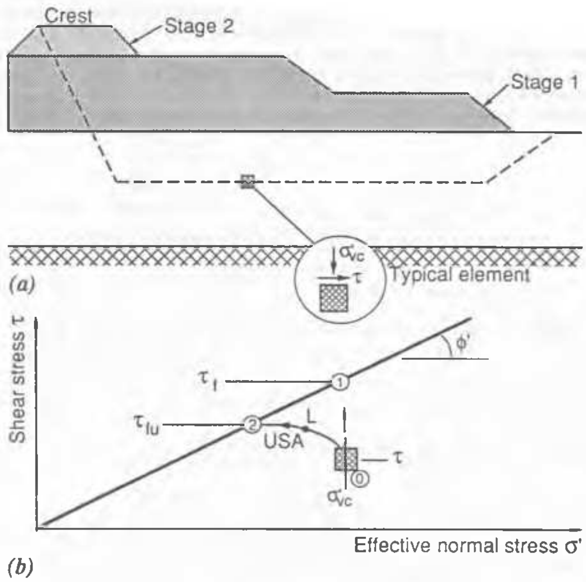


Fig.1. Comparison of effective stress and undrained strength analyses for evaluating stability analyses (after Ladd, 1991)

conditions or rebounded to varying OCR before being submitted to undrained shear tests. The undrained shear strength is then expressed as a function of the existing vertical effective stress σ'_v:

$$\tau_{fu} = (\tau_{fu} / \sigma'_{vc})_{SR}\, \sigma'_v \qquad (1)$$

in which $(\tau_{fu} / \sigma'_{vc})_{SR}$ is a strength ratio which depends on the type of test and the overconsolidation ratio.

Applying the strain compatibility technique to 10 normally consolidated soils, Ladd (1991) found an average strength ratio $(\tau_{fu} / \sigma'_{vc})_{SR}$ of 0.22, independent of the plasticity index. For DSS undrained tests, he found strength ratios of 0.225 ± 0.025 SD for 16 clays and 0.26 ± 0.035 SD for nine silts and organic soils.

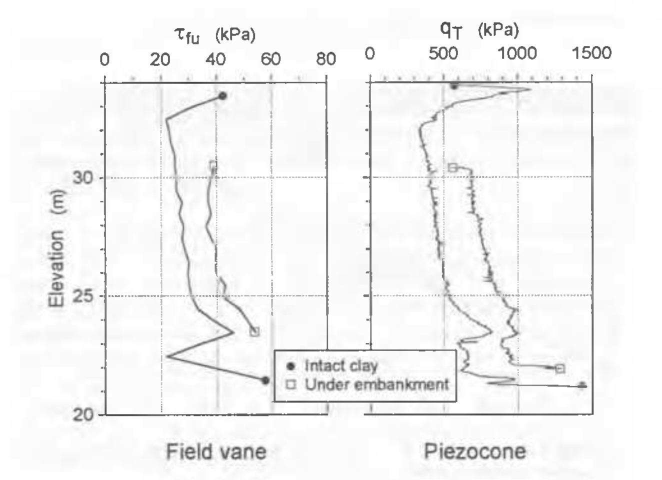
In situ, a soil element such as the one shown in Fig. 1 is subjected on a horizontal plane, to a vertical effective stress as well as to a shear stress. To simulate this situation, Ladd and co-workers, and more recently Girard (1992), performed DSS tests with a horizontal shear stress applied during consolidation. They observed an increase of the measured undrained shear strength typically of 20% when the shear stress applied during consolidation was 90% of the conventional DSS undrained shear strength (Girard, 1992). However, this strength is associated to a peak followed by a significant strain softening and it is not evident that this peak strength can be really mobilized in situ.

Although opened to criticisms for single stage constructions, the SHANSEP approach appears to be more appropriate for stage-constructed embankments. Leroueil et al. (1985) suggested an empirical approach also based on Eq. (1), with a strength ratio of 0.25. In both cases however, the difficulty in evaluating σ'_v previously mentioned for effective stress analyses also exists.

Analysis Based on Measured Undrained Shear Strength

The stability of stage-constructed embankments can also be established on the basis of directly measured undrained shear strength. Laboratory tests performed on samples taken under embankments are not recommended. Indeed, if the clay is reconsolidated to in situ stresses, the change of volume due to consolidation can be excessive and the measured strength too high; on the other hand, if the consolidation stresses is lower than the in situ stresses, then the measured strength, which depends on the consolidation stress, underestimates the real value.

Field tests, such as the vane and the piezocone, are more appropriate to evaluate strength increases under embankments. The field vane has often been used. However, analysing results obtained under 13 test embankments, Law (1985) observed strength ratios τ_{fuv}/σ'_v smaller than the values of (τ_{fuv}/σ'_p) observed on the intact clay. The average ratio is 0.23, thus very close to that obtained with DSS tests. He concluded, that the vane shear strength measured under an embankment should not be corrected for stability analyses. The piezocone has not been often used in the past. However, a study presently in progress on five different sites (Demers, 1994) shows that it gives interesting results, similar to the vane test but much more detailed. An example of strength increase evaluated with both the field vane and the piezocone, is shown in Fig. 2.



Comparaison of strength increase under St-Hyacinthe embankment

Fig.2. Strength increase under the St-Hyacinthe embankment (after Demers, 1994)

CASE WHERE THERE IS A DECREASE IN EFFECTIVE STRESS AFTER THE END OF CONSTRUCTION

It often happens in sensitive clays, and sometimes in non sensitive clays, that, during the days and weeks following the end of construction and the passage of the preconsolidation pressure, there is a continuous increase in pore pressure, and thus a decrease in vertical effective stress (Crooks et al., 1985; Kabbaj et al., 1988). The reasons for that are associated to creep and possible destructuration of the clay.

The question in term of stability is whether or not this decrease in effective stress is associated to a decrease in the real factor of safety. The evaluation by several methods (Leroueil et al., 1993) of the undrained shear strength of a clay foundation in these conditions indicates that there is no decrease. The fact that delayed failures of embankments have not been reported in the literature is also an indication that there is no significant decrease. In reference to Fig. 1, it can be said that, when the vertical effective stress decreases, the factor of safety F_E decreases and becomes closer to F_U, the factor of safety in terms of undrained shear strength, but there is no evidence that there is a decrease of this last value. In other words, when the vertical effective stress decreases during consolidation, the undrained shear strength of the intact clay remains valid and there is no decrease in strength as would be indicated by Eq. (1). This last equation would not be valid in these circumstances.

EVALUATION OF THE INCREASE IN UNDRAINED SHEAR STRENGTH AT THE STAGE OF DESIGN

As previously shown, the undrained shear strength under an embankment is estimated on the basis of the existing effective stresses or measured in situ strength values. For the design, only the first approach can be used, which is relatively easy to do at the end of consolidation, but not so easy during consolidation, particularly under the slopes and the berms of the embankment. An interesting complementary information is given by the variation of the undrained shear strength with the volumetric strain of the clay. The DSS test strength or the triaxial compression large strain strength (Trak et al., 1980; Leroueil et al., 1993) measured on specimens consolidated under various stresses or strains is quite appropriate to determine such a relationship. Applying this approach to the Olga test embankments built on a 12 m thick sensitive clay deposit, Leroueil et al. (1993) found that a settlement of about 1 m was necessary before the occurrence of some strength increase, value which has been confirmed by other estimations made from tests and field observations.

Conclusion

Two main approaches can be used to evaluate the stability of stage-constructed embankments:
- undrained strength analyses in which the undrained shear strength is computed as a function of the pre-shear vertical effective stress. However, this approach could underestimate the real factor of safety when used in cases where there is a decrease in vertical effective stress after the end of construction;
- analyses based on the undrained shear strength measured in situ with the vane test or with the piezocone.

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