CHAPTER 11

Analyses of Embankments with Partial Consolidation of Weak Foundations

Clay foundations are sometimes too weak to support the entire load of an embankment. Methods of improving stability in these cases include constructing the embankment in stages, allowing time for partial consolidation between stages, constructing the embankment continuously but slowly to allow time for partial consolidation, and using wick drains or sand drains to accelerate the rate of drainage and pore pressure dissipation. As the foundation clay consolidates, its strength increases and stability is improved.

Figure 11.1 shows a stress path (shear stress and effective normal stress on a potential failure plane) for an element of soil in the foundation of an embankment constructed in stages. The most critical periods (the periods when the factor of safety is lowest) are those corresponding to the end of placement of a new portion of the embankment, when the stresses are closest to the failure envelope. The factor of safety increases as consolidation occurs, and the stress point moves away from the failure envelope. After a period of consolidation, it is possible to increase the height of the embankment safely.

The benefits of consolidation during construction are illustrated clearly by studies of two test embankments constructed in Poland (Wolski et al., 1988, 1989). The engineers who performed the studies summarized their findings as follows (Wolski et al., 1989): ''[I]t was shown that by constructing the fill in stages, it was possible to safely construct a fill twice as thick as the original shear strength in the ground would have permitted. After another two years of consolidation even this load could be doubled to an 8 m thick fill before failure occurred.''

When it is impractical to construct an embankment slowly or to construct it in stages to achieve partial consolidation during construction, wick drains or sand drains can be used to accelerate the rate of consolidation. In any of these cases where consolidation during constuction is essential to the stability of the embankment, consolidation and stability analyses are needed to determine the amount of consolidation, the factor of safety, and the allowable rate of fill placement.

CONSOLIDATION DURING CONSTRUCTION

Although it is frequently assumed that there is no consolidation or dissipation of excess pore water pressure during construction of embankments on weak clays, this may not be a good approximation. The rate of consolidation of clays in the field is frequently fast enough so that a significant amount of dissipation of excess pore water pressure will occur during construction. As shown in Figure 11.2, undrained conditions would correspond to point 2a, whereas partial dissipation during construction would correspond to point 2b or point 2c, which would have higher factors of safety. If some dissipation does occur, stability analyses performed assuming completely undrained conditions at the end of construction (point 2a) will underestimate the factor of safety. This may lead to a perceived need for staged or slow construction, or for wick drains to accelerate the rate of consolidation, when in fact neither would be needed.

An example of such a case is the embankment of La Esperanza Dam in Ecuador. A portion of the dam was built on an ancient incised valley that contained recent alluvium, including as much as 15 m (50 ft) of weak, compressible clay. Stability analyses showed that if the clay was undrained, the embankment would be unstable. Settlement calculations indicated that the

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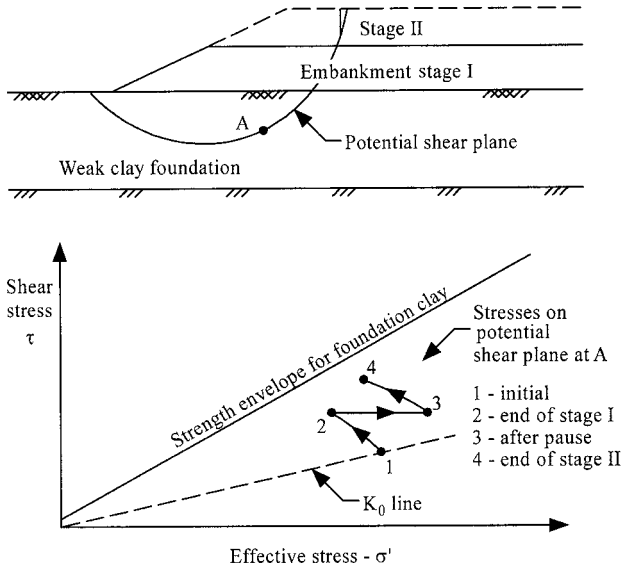


Figure 11.1 Stress changes during staged construction.

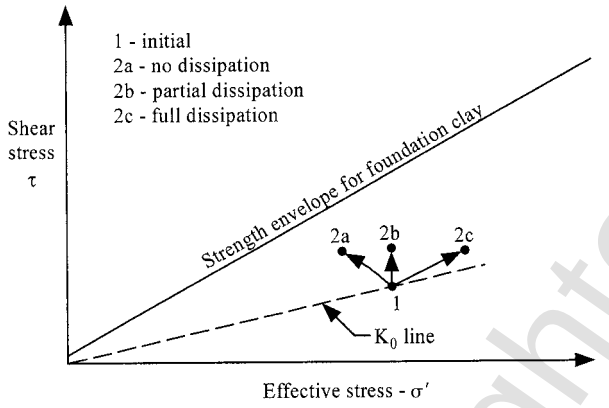


Figure 11.2 Effect of pore pressure dissipation during embankment construction.

settlement of the 45-m (150-ft)-high embankment would be approximately 1.5 m (5 ft). Measurements of the settlement of an instrumented test fill showed that the actual rate of settlement was much faster than the rate calculated using the results of laboratory consolidation tests. The actual rate of consolidation was about 12 times as fast as calculations had indicated they would be. Based on this finding it was concluded that it would be possible to construct the dam without excavating the foundation clay, without restricting the rate of construction of the embankment, and without installing sand drains to accelerate consolidation of the clay. As a result it was possible to reduce the time required for construction of the dam from four years to three. The dam was completed successfully in September 1995.

The lesson to be learned from this experience is that in circumstances where the factor of safety calculated assuming undrained conditions is lower than acceptable, the amount of consolidation and dissipation of excess pore water pressures during construction should be evaluated. It may be found that it is unnecessary to use slow construction, staged construction, or drains to accelerate consolidation, because sufficient dissipation of excess pore pressures will occur without these measures so that the factor of safety will be acceptable throughout and after construction.

ANALYSES OF STABILITY WITH PARTIAL CONSOLIDATION

Idealized variations of embankment height, excess pore water pressure, and factor of safety with time during staged construction are shown in Figure 11.3. The most critical times are those, like points 2 and 4, which correspond to the end of a period of rapid fill placement. These are the conditions for which stability must be evaluated to ensure an adequate factor of safety throughout and after construction. As shown in Fig. 11.3, the factor of safety increases with time after fill placement stops.

The mechanisms through which stability is improved by partial consolidation—dissipation of excess pore pressures and strength increasing as effective stress increases—are founded on well-established principles of soil mechanics. However, specific methods for applying these principles to evaluation of stability with partial consolidation have not been well established, and there is disagreement in the profession regarding whether it is preferable to use effective stress analyses or total stress analyses to evaluate stability for such cases (Ladd, 1991).

Effective Stress Approach

If stability is evaluated using the effective stress approach, the analyses are performed as follows:

1. Conduct drained strength tests, or consolidated–undrained tests with pore pressure measurements, to determine the effective stress shear strength parameters (c' and ϕ') for the foundation clay. Perform either drained or undrained tests to determine strength parameters for the fill. If sufficient experience already exists with the soils, it may be possible to estimate the values of the needed properties.
2. Estimate the variations of excess pore water pressures with depth and laterally in the foundation for a condition of no drainage. These pore water

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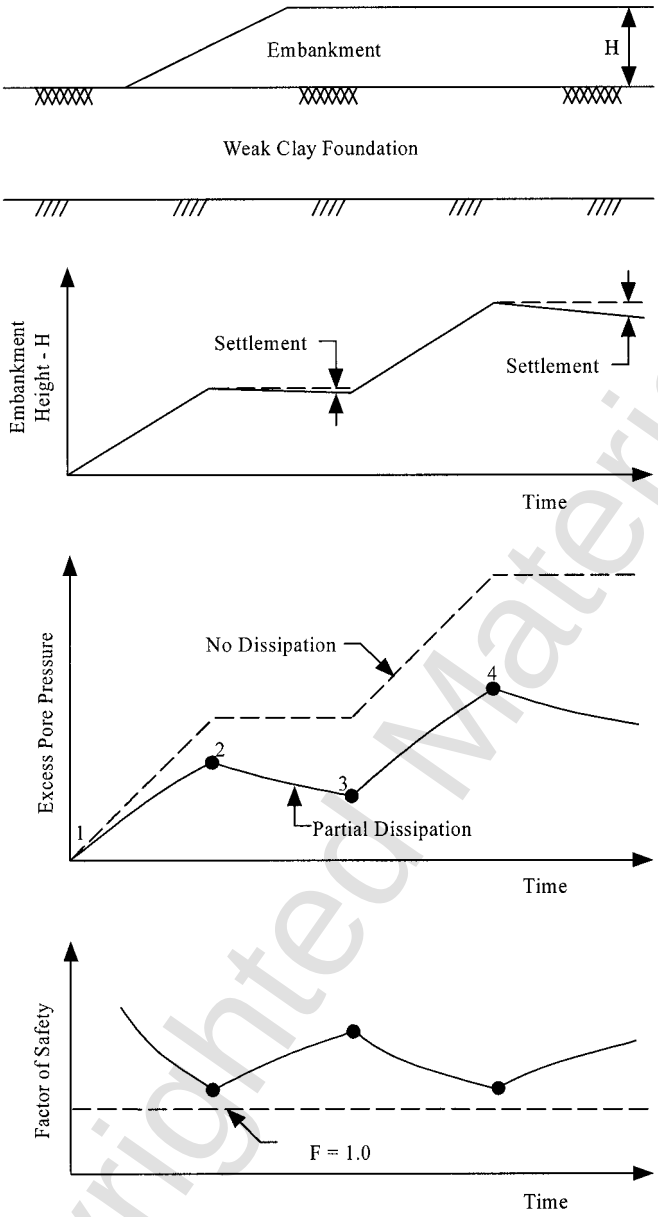


Figure 11.3 Variations of embankment height, excess pore pressure, and factor of safety with time.

pressures would correspond to the no dissipation line in Figure 11.3.

3. Perform consolidation analyses to determine how much excess pore water pressure dissipation will occur during construction. The results of these analyses would correspond to the partial dissipation line in Figure 11.3.
4. Perform stability analyses for stages such as points 2 and 4 in Figure 11.3, when the factor of safety would be lowest.

The primary advantage of the effective stress approach is that stability can be checked during construction by measuring the pore water pressures and performing additional stability analyses using the measured pore pressures.

Total Stress Approach

If stability is evaluated using the total stress (or undrained strength) approach, the analyses are performed using undrained shear strengths, expressed in terms of total stresses. For saturated soils the undrained shear strength is represented in the analyses by $c = s_u$, with $\phi = 0$. The analyses are performed as follows:

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1. Conduct laboratory tests to determine the variation of s_u with effective stress (σ') and *overconsolidation ratio* (OCR) for the foundation clay. Perform either drained or undrained tests to determine strength parameters for the fill. If sufficient experience already exists with the soils, it may be possible to estimate the values of the needed properties.
2. Determine the variation of *preconsolidation pressure,* p_p (also called *maximum past pressure,* $\sigma_{v\max}$) with depth in the foundation from the results of laboratory consolidation tests, in situ tests, or past experience.
3. Estimate the excess pore water pressures in the foundation for a condition of no drainage, and perform consolidation analyses to determine how much excess pore water pressure dissipation will occur during construction, as for effective stress analyses. Compute the variation of effective stress (σ') and OCR with depth and laterally for each stage at which stability will be evaluated. Use these to estimate the variation of undrained strength with depth and laterally beneath the embankment.
4. Perform stability analyses for stages such as points 2 and 4 in Figure 11.3, when the factor of safety would be lowest.

The primary advantage cited for the total stress approach is that if failure occurred, it would be undrained. Thus, using undrained strength is more appropriate because it corresponds more closely to the behavior being evaluated.

OBSERVED BEHAVIOR OF AN EMBANKMENT CONSTRUCTED IN STAGES

The studies performed by Wolski et al. (1988, 1989) are uniquely valuable because they are so well documented and because the embankment they studied was eventually loaded to failure. The reports of these studies do not provide information about the effectiveness of undrained pore pressure estimates or consolidation analyses, because the foundation pore pressures were measured rather than calculated. Even so, the studies are very instructive because of the wealth of detail they contain and the clarity with which the results are presented.

The earlier investigation (Wolski et al., 1988) included studies of two embankments constructed in stages, one with wick drains and the other without. The later investigation (Wolski et al., 1989) involved increasing the height of the embankment without wick drains until failure occurred.

The embankments were constructed at a site in Poland where the subsoil contained a layer of peat about 3 m (10 ft) thick, underlain by a layer of weak calcareous soil about 4.7 m (15 ft) thick. The calcareous layer was underlain by sand. Measurements included the consolidation characteristics of the peat and calcareous soil, drained and undrained shear strengths, pore water pressures in the foundation, and horizontal and vertical movements of the embankments.

Undrained strengths (vane shear strengths) in the foundation at three different times during construction are shown in Figure 11.4. It can be seen that the increase in strength with time was very significant, especially near the top of the peat layer and the bottom of the calcareous soil, where consolidation advanced most rapidly. In the center of the calcareous soil, where consolidation was slowest, the undrained strength was found to be relatively low. Although the vane shear strengths shown in Figure 11.4 required correction for use in evaluating stability, they provided a useful means for effective evaluation of strength increase due to consolidation.

Stage 1 of the embankment (1.2 m high) was built in November 1983. Stage 2 (which increased the height to 2.5 m) was added in April 1984. Stage 3 (which brought the height to 3.9 m) was completed in June 1985. In July 1987 the height of the embankment

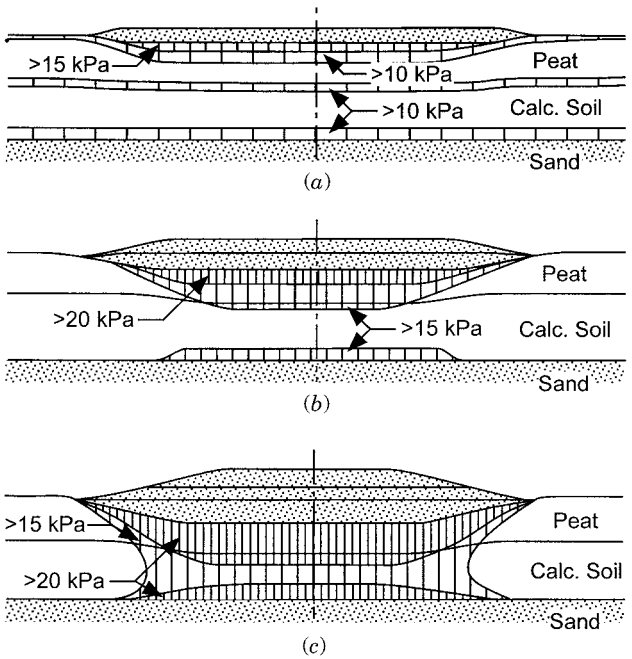


Figure 11.4 Corrected values of shear strengths in the foundation of a test embankment constructed in stages: (*a*) end of stage 1 (4.84); (*b*) end of stage 2 (5.85); (*c*) end of stage 3 (7.87). (From Wolski et al., 1989.)

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was increased to 7.95 m (26 ft) in a period of seven days, whereupon the embankment failed. Failure occurred in the middle of the night when there was no construction activity. The measurements made at the end of the previous day had given no sign that failure was imminent.

The shape of the failure zone was estimated based on surface observations and field vane shear tests to locate zones in the foundation where the undrained strength had been reduced by remolding that accompanied the large displacements at failure. The shape of the failure zone inferred from these measurements is shown in Figure 11.5. It can be noted that there is a steep ''active'' zone beneath the center of the embankment, a nearly horizontal section where failure followed the least consolidated and weakest part of the calcareous soil, and a gently inclined passive zone where the failure extended upward toward the ground surface. These three zones fit well into the active, direct simple shear and passive failure surface orientations described by Ladd (1991).

Factors of safety for the embankment were calculated for several stages during construction. Of greatest interest are those calculated for the conditions at the time of failure, which are shown in Table 11.1. Ideally, the factors of safety calculated would be 1.0 for conditions at failure. Within the range of accuracy of the calculations, this is true for both the total and effective stress analyses.

By the time the embankment reached its final height of 8 m, it was shaped like a truncated pyramid, much narrower at the top than at the base. As a result, the two-dimensional analyses represented the conditions at the maximum section but had to be adjusted to achieve a result that was representative of the average conditions for the entire embankment. These adjustments could be approximated for the undrained case only. Even so, two conclusions are clear from the results: (1) The effective stress and total stress factors of safety are very nearly equal at the failure condition. No significance can be attached to the small difference between the calculated values of F_t and F_e. (2) With a reasonable allowance for three-dimensional effects, the factor of safety calculated for the total stress analysis is unity. There is no reason to believe that the same would not be true for the effective stress analysis as well.

Table 11.1 Factors of Safety for an Embankment Constructed in Stages[a]

	F_t (total stress)	F_e (effective stress)
Two-dimensional factor of safety	0.85	0.89
Estimated increase due to three-dimensional effects	12 to 18%	N.A.
Estimated three-dimensional factor of safety	0.95 to 1.00	N.A.

Source: (Wolski et al. (1989).

[a]Both F_t and F_e were calculated using Janbu's (1973) Generalized Procedure of Slices. F_e was calculated using measured pore pressures.

DISCUSSION

As noted earlier, methods for analysis of staged construction have not been well established, and there is still disagreement concerning whether effective stress analyses or total stress analyses are preferable. The writers believe that this state of affairs is due to the

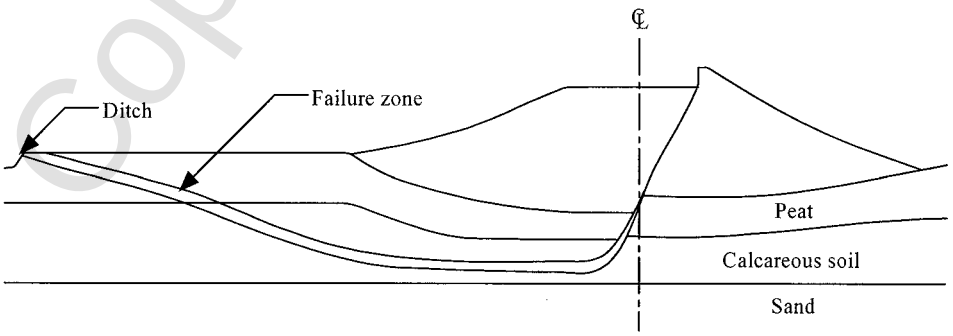


Figure 11.5 Estimated failure zone in a test embankment constructed in stages. (From Wolski et al., 1989.)

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fact that the there are not a sufficient number of well-documented case histories of staged construction failures against which to gage the effectiveness of the methods that have been proposed. Some excellent case studies have been performed, notably by Wolski et al. (1988, 1989) and Bromwell and Carrier (1983), but none of these provides information regarding the accuracy of stability evaluations based on consolidation analyses to estimate pore pressures for conditions of partial consolidation.

To the writers' knowledge, only the load test performed by Wolski et al. (1989) was continued to failure. The tailings dam studied by Bromwell and Carrier did not fail, and the one-dimensional consolidation analyses they performed (only vertical flow) did not match measured pore pressures, probably because there was significant horizontal flow during consolidation.

Ladd's exhaustive study (Ladd, 1991) shows how complex, how difficult, and how fraught with uncertainty analyses of staged construction can be. More case studies are needed to advance the state of the art in this area. Until the results of such studies are available, it seems prudent to use both total and effective stress analyses in tandem and to bear in mind the difficult aspects of this class of problems.

Difficulties in Estimating Pore Pressures

A considerable part of the uncertainty in both the effective stress approach and the total stress approach stems from the difficulties in estimating the excess pore pressures due to embankment loading and from the difficulties in estimating their rates of dissipation. The uncertainties in these processes can best be appreciated by considering some of the details of such analyses.

Estimating these pore water pressures requires performing three types of analyses: (1) a stress distribution analysis to calculate the increase in total stress in the clay due to construction of the embankment; (2) an analysis to estimate the values of excess pore water pressure that would result from these changes in total stress with no drainage (these pore water pressure changes should reflect the effects of changes in shear stress as well as changes in mean normal stress); and (3) a consolidation analysis to calculate the remaining excess pore water pressures after a period of dissipation. These remaining excess pore water pressures are added to the initial (before construction) pore pressures to determine the total pore water pressures remaining after dissipation.

Estimating the distribution of pore water pressure that results from undrained loading requires considerable effort and is difficult to do accurately. The most straightforward way of estimating these stress changes is by using elastic theory, but elastic theory may result in stresses at some locations that exceed the strength of the clay and would have to be adjusted to values that are consistent with the strength. Alternatively, it would be necessary to perform more sophisticated stress analyses that provide stresses compatible with the strength characteristics of the clay.

The pore pressures that result from the increases in total stress at each point in the foundation depend on (1) the properties of the clay, (2) the overconsolidation ratio (OCR), and (3) the magnitude of the stress increase, particularly how close to failure the clay is loaded. The value of the OCR and the magnitude of the changes in total stress vary from point to point through the foundation.

Skempton (1954) expressed the change in pore water pressure due to changes in total stress in the form

$$\Delta u = B\,\Delta\sigma_3 + \overline{A}(\Delta\sigma_1 - \Delta\sigma_3) \qquad (12.1)$$

where Δu is the change in pore water pressure caused by changes in total stress $\Delta\sigma_1$ and $\Delta\sigma_3$, and B and $\overline{A}$ are Skempton's pore pressure parameters.

If the clay is saturated, the value of B is equal to unity. The value of $\overline{A}$, however, depends on the properties of the clay, the OCR at the location where the pore water pressure is being calculated, and how close the stresses at the point are to the failure envelope. As a result, the value of $\overline{A}$ is difficult to estimate accurately.

Difficulties in Consolidation Analyses

To determine the distribution of pore water pressures after a period of consolidation, it is necessary to perform a consolidation analysis using the undrained condition as the initial condition. Variations in the values of coefficient of consolidation, compressibility, preconsolidation pressure, and change in stress with depth can have a significant effect on the rate of consolidation. In most cases consolidation occurs more rapidly in the field than would be expected based on conventional settlement calculations (Duncan, 1993). To take these factors into account, consolidation analyses should be performed using numerical techniques rather than conventional chart solutions.

Due to lateral flow, pore water pressures may increase in areas of initially low excess pore water pressures (beneath the toe of the embankment) while they are decreasing in other areas (beneath the center of the embankment). To include this effect, it would be necessary to perform two-dimensional consolidation anal-

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yses that take horizontal as well as vertical flow into account. Such analyses are possible, but difficult, and are not yet done routinely in practice.

If wick drains or sand drains are used to accelerate the rate of consolidation, suitable analyses are needed to estimate the rate of consolidation with radial flow to the drains. The book by Holtz et al. (1991) is a valuable resource that covers both the theoretical and practical aspects of designing wick drain systems. Hansbo (1981) has developed the most widely used theory for analysis of consolidation with wick drains. The theory includes the effects of smear due to disturbance when the wicks are installed, and the effects of the finite flow capacity of the wicks.

With or without drains to accelerate the rate of dissipation, predicting pore water pressures for staged construction analyses is a difficult task. It is clear from the preceding discussion that making these estimates requires extensive effort and is susceptible to considerable inaccuracy.

Difficulties in Estimating Undrained Shear Strengths

Ladd (1991) has shown that undrained shear strengths of clays depend on several factors:

- The magnitude of the effective consolidation pressure, p'
- The value of the overconsolidation ratio (OCR)
- The ratio of the effective principal stresses during consolidation, $K_c = \sigma'_1/\sigma'_3$
- The amount of reorientation of the principal stresses during loading
- The orientation of the failure plane

Accounting for each of these effects is difficult. Ladd recommends employing simplifying assumptions that result in conservative estimates of undrained shear strength. He assumes that p' is equal to the vertical effective stress, that the K_c ratio is equal to $1/K_0$, and that the amount of stress reorientation and the orientation of the failure plane are uniquely related. The amount of conservatism involved in these simplifications is difficult to estimate.

As discussed in Chapter 5, two methods of laboratory testing can be used to evaluate undrained strengths. Bjerrum (1973) recommended use of what is termed the *recompression procedure,* wherein test specimens are consolidated in the laboratory to their estimated in situ stresses, to overcome some of the effects of disturbance. Ladd and Foott (1974) and Ladd et al. (1977) advocate use of the SHANSEP procedure, wherein test specimens are consolidated to pressures several times higher than the in situ stresses and shear strengths are characterized in terms of ratios of undrained strength divided by effective vertical stress during consolidation, s_u/σ'_{vc}. There is fairly general agreement that the recompression procedure is preferable for sensitive and highly structured clays, and that SHANSEP is more suitable for young clays that are not very sensitive and which have no significant bonds or structures that are subject to damage by large strains during consolidation. In addition, if SHANSEP is used, it needs to be established (not just assumed) that s_u/σ'_{vc} is a suitable parameter for characterizing the strength of the clay in question (i.e., that undrained strength divided by consolidation pressure is a constant for the clay).

Intrinsic Difference in Effective Stress and Total Stress Factors of Safety

Effective stress and total stress factors of safety are intrinsically different because they use different measures of shear strength, as illustrated in Figure 3.4. The effective stress factor of safety (F_e) is equal to the shear stress required for equilibrium divided by the shear strength of the soil if the soil fails with no change in the effective stress on the failure plane. The total stress factor of safety (F_t) is equal to the shear stress required for equilibrium divided by the shear strength of the soil if the soil fails with no change in water content. For saturated soils this corresponds to failure with no change in void ratio. In general, the values of F_e and F_t are not the same. For clays that generate positive pore pressures due to changes in shear stress, as shown in Figure 3.4, F_e is greater than F_t. At failure, both F_e and F_t are equal to unity, but for stable conditions, F_e is not equal to F_t.

Instrumentation for Staged Construction

Because the results of analyses of stability during staged construction are so uncertain, it is appropriate to use the observational method (Peck, 1969) to supplement the results of analyses. Two types of instrumentation are especially useful for this purpose.

Piezometers can be used to measure pore water pressures at key points in the foundation, and comparisons of the measured and calculated pore water pressures provide an effective means of determining if the calculated values are high, low, or accurate. Effective stress stability analyses can be performed using the measured pore water pressures to check on stability during construction.

Inclinometers (slope indicators) and settlement plates can be used to measure horizontal movements in the foundation beneath the toe of the fill and settlements under the center of the embankment. Tavenas et al. (1979) have developed criteria that can be used to

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interpret whether the movements observed are due to consolidation of the foundation clay or whether they indicate impending instability.

Need for Additional Case Histories

As mentioned previously, because there are not enough published case histories of failures of embankments during staged construction, and none that include consolidation analyses, it is difficult to judge the accuracy of the methods of analysis that have been proposed. The studies conducted by Wolski et al. (1988, 1989) are extremely valuable. The instrumentation and testing that they used provide a model of what is desirable in such studies, and much can be learned from review of their results. However, more such studies, including both consolidation and stability analyses, will be needed to determine whether any of the analysis methods that have been proposed are accurate and reliable.

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