GÉORISQUES IV GEOHAZARDS

LONG TERM STABILITY OF SLOPES AND EMBANKMENTS ON SOFT SOILS

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RÉSUMÉ
Dans cet article, un modèle conceptuel pour la stabilité à long terme des talus et remblais sur sols mous basé sur la théorie de l'Etat Critique des Sols est présenté. En utilisant une expérience acquise principalement sur les sols argileux, les effets de viscosité, vitesse de déformation et structure sur le comportement du sol sont discutés et une attention particulière est portée à la résistance à long terme. L'approche de dimensionnement qui en découle suggère l'utilisation du concept des lignes d'instabilité comme surface limite pour l'état limite en compression ou cisaillement et l'augmentation ou la réduction de résistance qui en suit.
L'applicabilité de cette approche conceptuelle pour la garantie de la stabilité à long terme des talus et remblais construits sur une grande variété de sols est discutée et des recommandations pour de nouvelles recherches sont présentées.

ABSTRACT
In this article a conceptual model for long term stability of slopes and embankments on soft soils is presented, which is based on the Critical State Soil Mechanics framework. Using experience gained from predominantly clayey soils, the effects of viscosity, strain rate and structure on the material behaviour are discussed while particular attention is paid to the long term strength. The resulting design approach suggests the use of the concept of instability lines as limiting boundary for compression or shearing yield and subsequent strength increase or decrease respectively.
The applicability of this conceptual approach for the safeguarding of the long term stability of slopes and embankments on a wide range of soils is discussed and recommendations for further research and developments are presented.

1. GENERAL

Both the climate change and related rising sea level result in changing boundary conditions for sea and river dikes. Furthermore an increased level of development takes place in areas in Western Europe with very soft silty clay and organic soil deposits. This urges for a better understanding of the material behaviour resulting in more reliable geotechnical designs and a reduction of the risks involved. While classical slope and embankment design treats stability and settlement issues separately, Critical State Soil Mechanics (CSSM) presents a framework that enables an interrelated approach.

This paper is aimed at discussing the long term stability of slopes and embankments using the CSSM framework, while taking into account relevant effects like viscosity, strain rate, structure and partial saturation to provide insight in their influence, following the work of Skempton (1964, 1970), Bjerrum (1967a,b), Burland (1990), Leroueil & Vaughan (1990), Leroueil (2001, 2006) and Leroueil & Hight (2003).

First the relevant material behaviour for long term slope stability is discussed using experience from laboratory testing and field case histories. A conceptual design approach is presented using the concept of instability lines introduced by Lade (1992) and the effects of viscosity, strain rate, structure and partial saturation on this instability line framework are discussed in a qualitative manner. Both a conservative and more detailed implementation is possible, which depends on the design implications of the project under consideration. The applicability of the presented framework for a wide range of soils is discussed and recommendations are presented.

2. RELEVANT MATERIAL BEHAVIOUR

The Critical State Soil Mechanics (CSSM) framework originates from the work by Casagrande (1936) and Taylor (1948) and is based on the work presented by Roscoe *et al.* (1958) and Schofield & Wroth (1968). This fundamental framework provides a clear interrelationship between effective stresses, void ratio and critical void ratio in which the concepts of limit and critical states are incorporated.

The existence of a unique relationship between void ratio, isotropic stress and mobilised shear strength combined with interrelationships between drained and undrained strengths was discussed by Taylor (1948), while Bjerrum (1967b) clearly showed the relation between the vertical yield stress for oedometer compression and the undrained shear strength in an isotache framework.

2.1 Model concept for associative elasto-plasticity and critical state

A schematic presentation of imposed undrained behaviour in a triaxial test for a material on the wet and dry side of the

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critical state line (CSL) is presented in Figure 1. Undrained triaxial compression of a sample on the dry side of the CSL results in dilation, where excess suction and resulting increase in effective stresses occur until the CSL is reached. Undrained compression of a sample on the wet side of the CSL results in excess pore water pressure and a decrease of mean effective stress until the CSL is reached.
The relationship between the undrained shear strength ratio for normal and overconsolidated soils after Wroth (1984) are briefly presented in Figure 1. For this idealised relationship associative flow behaviour, with a coinciding elasto-plastic yield surface and plastic potential, is assumed. In the model this surface is indicated as the limit state surface.

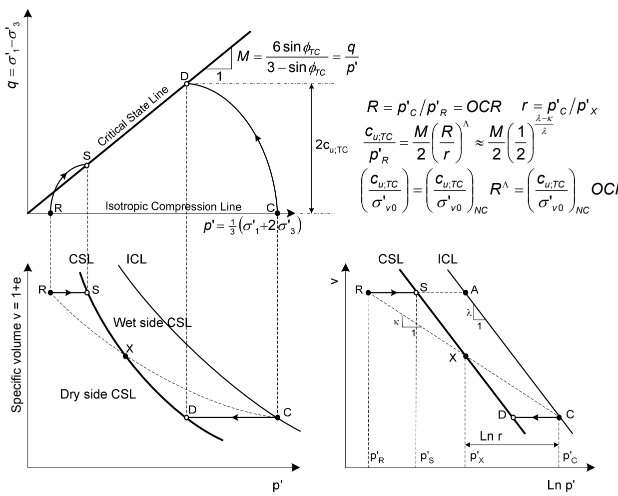

Figure 1. Idealised undrained triaxial test results in CSSM using associative flow rule (after Wroth 1984).

2.2 Viscous behaviour of soft soils

The material behaviour of predominantly lightly overconsolidated clayey soils during long duration creep tests has been investigated by many researchers, like for instance Murayama & Shibata (1961), Singh & Mitchell (1969), Bishop & Lovenbury (1969) and Tavenas *et al.* (1978). Murayama & Shibata (1961) reported the failure of clay samples in long term creep tests, which was explained by the breaking of activated bonds between clay particles.
Singh & Mitchell (1969) extended this work and presented an empirical relationship for the slope of the axial creep strain rate, shear stresses and the logarithm of time with an indication of resulting soil behaviour.

A marked instability of strain rate, observed during creep tests under triaxial compression conditions, is reported by Bishop & Lovenbury (1969) for overconsolidated London clay and for normal consolidated Pancone Clay from Pisa. The observed instabilities were also apparent in long duration oedometer tests on undisturbed samples on Pancone Clay but were absent in remoulded samples.
The creep behaviour of undisturbed lightly overconsolidated Saint-Alban clay, sampled with the Laval sampler (La Rochelle *et al.* 1981), has been thoroughly investigated by Tavenas *et al.* (1978). A selection of the numerous stress points tested is presented in Figure 2a combined with the 1 day limit state surface, which will be defined after Figure 3.

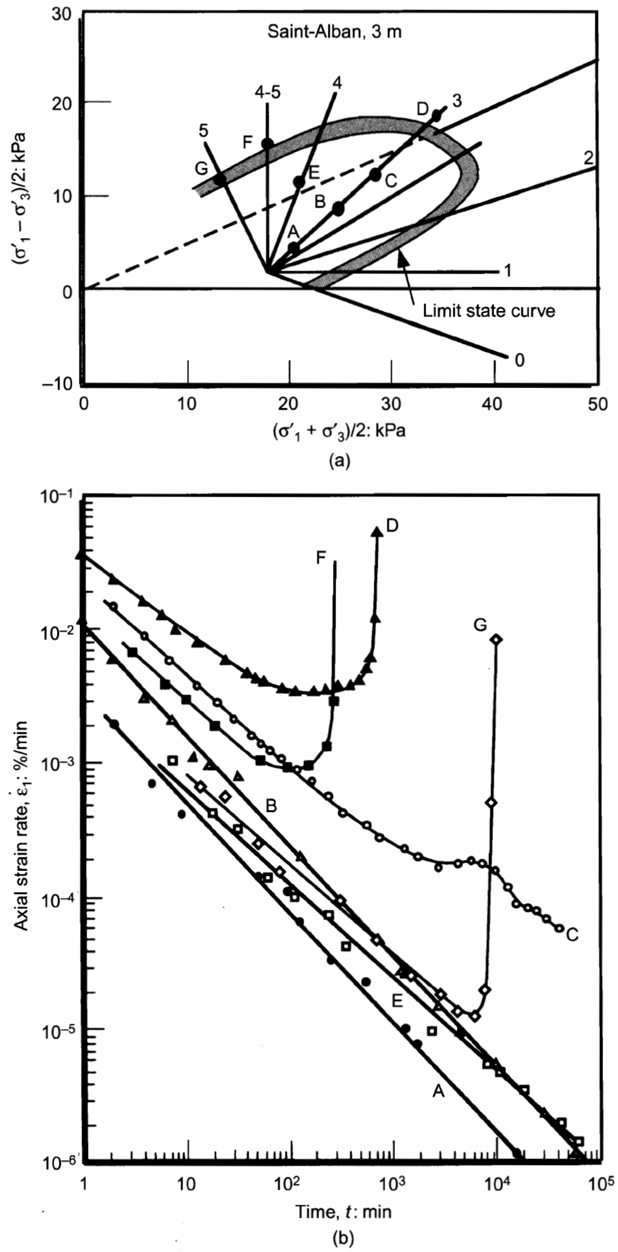

Figure 2. Axial strain rate versus log time for creep tests on Saint-Alban clay (Fig. 4 – Leroueil 2001. Courtesy of Thomas Telford).

The resulting creep behaviour versus the logarithm of time is presented in Figure 2b. This figure is of significant importance and gives a perfect overview of the creep behaviour at different positions from the limit state surface. Stress points D, F and G lie outside or on the 1 day limit state surface (Fig. 2a) and the creep rate results in instability (Fig. 2b). All other presented stress points, except for stress point C, show a linear relationship of the strain rate with the logarithm of time, without the occurrence of a discontinuity or instability. Point C lies within the limit state surface and below the critical state line, but does show a significant discontinuity followed by a marginally increased creep rate. Both Bishop & Lovenbury (1969) and Tavenas *et al.* (1978) associate this with a particular behaviour of the clay structure under constant effective stress. According to Tavenas *et al.* (1977, 1978) both triaxial and oedometer results indicate that the isotache strain rate surface and the limit state curve have identical shapes, resulting in a homothetic relationship.

The experimental evidence of both triaxial and oedometer data on Saint-Alban Clay from Tavenas *et al.* (1977, 1978) is presented in Figure 3. Volume changes measured from triaxial stress states are combined with elapsed time from the constructed isotache model resulting from oedometer test data. The resulting linear relation between the principal effective stress σ'_1 and log time is presented in Figure 3b. In Figure 3a the combined results are presented in their stress space indicating a diminishing limit state surface, or equal volumetric strain rate surface, in time.

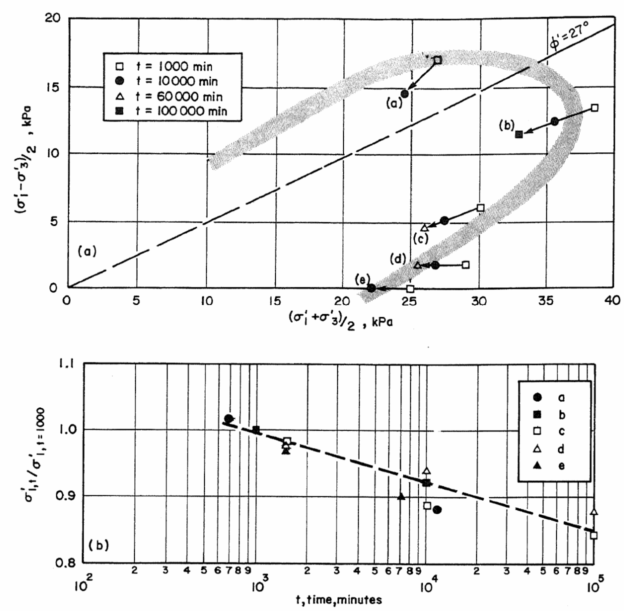

Figure 3. Location limit state surface as a function of time for creep tests on Saint-Alban Clay (Fig. 20 – Tavenas *et al.* 1978. Courtesy of NRC Research Press).

They observed that the spacing of the constant strain rate lines is larger for over-consolidated soil than for normally consolidated soil. The creep parameter C_α in eq. 3 is therefore not constant and appears to be time dependent. Both axial and volumetric strains can be compared using eq. 1, while using the stress function f(p',q) for volumetric strains, which is derived from the formulation of the limit state surface. The stress function f(p',q) defines the distance to the limit state surface of a material under a stress condition at a certain void ratio. It can be concluded that according to Tavenas *et al.* (1978), the implications of these observations are that the limit state surface is a surface of equal volumetric strain rate where the creep parameter C_α is only constant on the limit state surface. The equal volumetric strain rate at the 1 day limit state surface in Figure 2a and 3a is ~ 2x10^{-3} %/min. For clarity the limit state surface is from this point on referred to as an isotache surface with a corresponding strain rate.

$$\dot{v} = Bf(p',q)(t_1/t)^m \qquad [1]$$

Temperature effects

The temperature effects on the viscosity and resulting strength and compressibility behaviour of OC soils has been investigated by many researchers. Boudali (1995) investigated the effects of temperature on the isotache surface of Berthierville clay from Quebec using both oedometer and triaxial testing. A relationship between the strain rate and temperature on the vertical yield stress σ'_p was found, which is presented in Figure 4. For an increasing strain rate (Figure 4a) an expanding isotache surface is found, while for an increasing temperature (Figure 4b) a shrinking isotache surface is found. Marques *et al.* (2004) found similar effects on St-Roch-de-l'Achigan clay. The cited research also showed that the compression line λ, the swelling line κ and the critical state line in q-p' plane are independent of the temperature history.

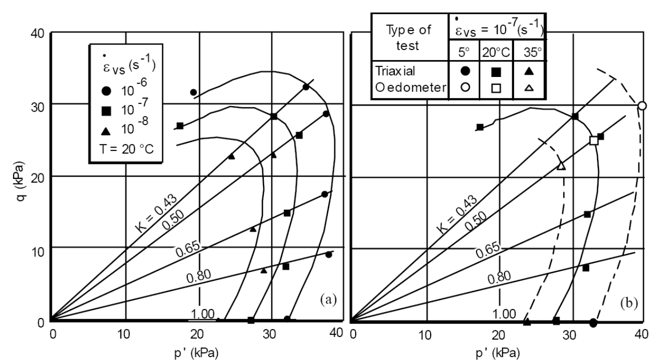

Figure 4. Strain rate (a) and temperature dependent behaviour of isotache surface of Berthierville clay (Fig. 32 – Leroueil 2006, after Boudali 1995).

In Figure 5 the normalised viscosity and vertical yield stress for oedometer testing is presented as a function of temperature for clays of different origin. Normalisation at a temperature of 20ºC automatically implies normalisation at a strain rate depending on the viscosity of the material. The curves show relatively good agreement with the relation, presented in eq. 2, proposed by Moritz (1995), and allows for a prediction of the temperature dependency of the vertical yield stress and thus the position of the isotache surface with the corresponding strain rate. The observed behaviour can be approximated by extending the well known one-dimensional drained compression expression of Bjerrum (1967b) by adding a temperature term, presented in eq. 3.

$$\sigma'_{pT} = \sigma'_{pT_0}\left(T_0/T\right)^{\alpha} \qquad [2]$$

$$e_v = e_{v0} - C_C\left(\log\left(\frac{\sigma'_{v;T_0}}{\sigma'_{vc;T_0}}\right) - \alpha\log\left(\frac{T_0}{T}\right)\right) - C_\alpha\log\left(\frac{t}{t_{ref}}\right) \qquad [3]$$

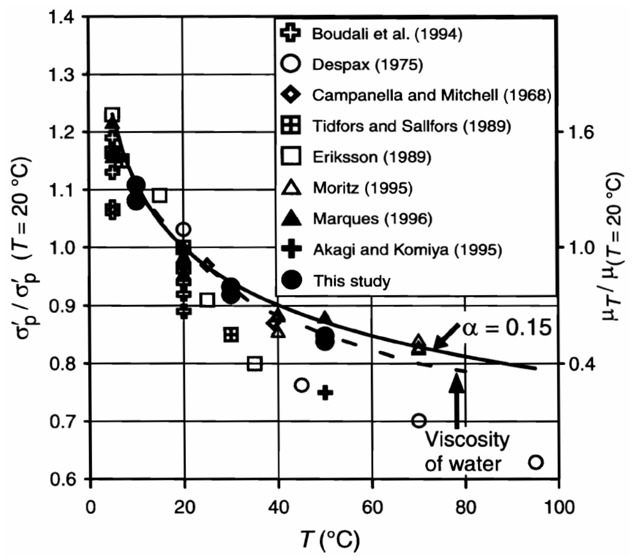

Figure 5. Normalised viscosity and vertical yield stress for clays of different origin as a function of temperature from oedometer testing (Fig. 6 - Marques *et al.* 2004. Courtesy of NRC Research Press).

2.3 Structure & destructuration

In this article the definitions of structure and destructuration according to Leroueil & Vaughan (1990) are adopted. Structure is the cementation or bonding of particles resulting in a strength and stiffness behaviour which cannot be explained from void ratio and stress-history alone. Fabric is also regarded as structure according to Thornton (2000) and is the influence in behaviour due to the relative position of (spherical) particles. Destructuration is defined as the process in which cementation and bonding diminish due to straining or remoulding.
According to Taylor (1948) and Andressen (1981), loss of structure is attributed to changes in stress history, water content and void ratio, chemical changes and mixing and segregation of constituents.

The definitions of the intrinsic and sedimentation compression lines by Burland (1990) give indications of the properties of soils, which have been reconstituted at a water content of 1-1.5 the liquid limit, and the compressibility and shear strength of the same soil in natural conditions respectively. In Figure 6 a comparison of structured and destructured soil in the oedometer test is illustrated. According to Leroueil & Vaughan (1990) it can be concluded that no large plastic strains will occur to the left of the destructured or intrinsic compression line (ICL). It can furthermore be concluded that only structure will allow the soil to exist to the right of the ICL and this structure will diminish to the right of the yield point resulting in large compressive strains. Finally, undrained shearing in the structure permitted space will result in loss of undrained shear strength and sensitivity.

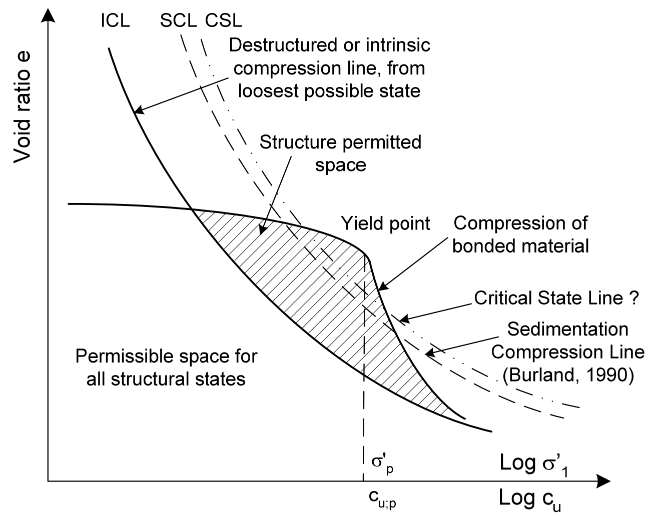

Figure 6. Schematic comparison of structured and destructured compression in oedometer test (after Leroueil & Vaughan 1990).

According to Burland (1990), the prediction of the peak value of the undrained shear strength c_u using the CSSM framework is not possible for normally consolidated clays, since this predominantly depends on structure. An interesting example is the effects of sampling induced disturbance for three sensitive Norwegian Clays, presented by Lacasse *et al.* (1985) (see Figure 7). Burland (1990) showed that the results from oedometer tests on high quality Sherbrooke block samples (Lefebvre *et al.* 1979) plotted above the SCL, while 95mm tube samples plot slightly below the SCL. All three clays had higher c_u values than expected, where the clay with the lowest liquidity index had the highest value due to fabric and bonding. In Figure 7 the results from anisotropic consolidated triaxial compression (CAU) tests are presented. The peak values of the tube samples are significant lower, but are still much higher than values at large deformation.

Destructuration changes the soil response in p'-q space and many geotechnical parameters such as the yield stress, undrained shear strength and compressibility are affected (Lunne *et al.* 1997). Investigation of the rate effects of residual strengths of clays of varying plasticity by Lemos & Vaughan (2000) indicated that the presented strain rate and destructuration behaviour are also valid for residual soils. There is no clear relation between the ICL, SCL and CSL but there is some evidence that destructured soil doesn't necessarily fall back to the ICL. Furthermore soils with oedometric compression behaviour plotting above the SCL tend to fall back on the CSL.

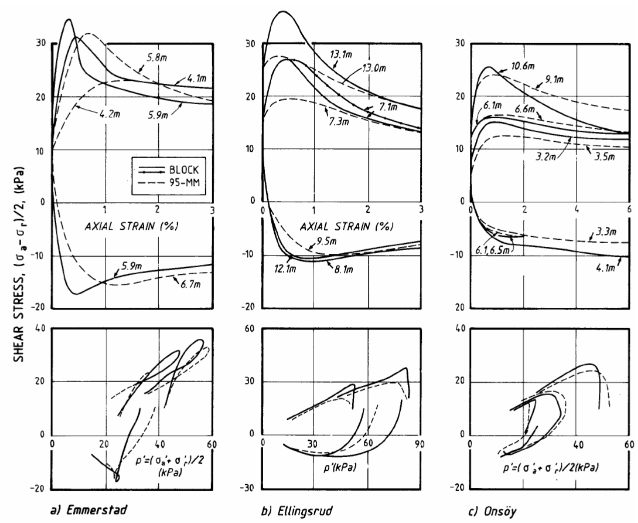

Figure 7. Results of undrained triaxial compression and extension tests on block and 95mm samples of sensitive clays (Fig. 8 – Lacasse *et al.* 1985. Courtesy of Taylor & Francis).

2.4 Static liquefaction & undrained instability lines

Background information on static liquefaction of sand under undrained conditions is provided by Castro (1969), Molenkamp (1983, 1989), Chu *et al.* (2003), Jefferies & Been (2006) and De Jager *et al.* (2008).

The principle of the undrained instability line was introduced by Lade (1992) for liquefaction analysis of sand, who defined the instability point at the top of the undrained effective stress path and the linear representation in q-p' plane as instability line (see Figure 8). In natural conditions the soil can be at its instability point without resulting in instability. It should be noted that a non-associated flow rule is necessary for liquefaction to occur since the instability point otherwise coincides with the top of the yield surface preventing instability.

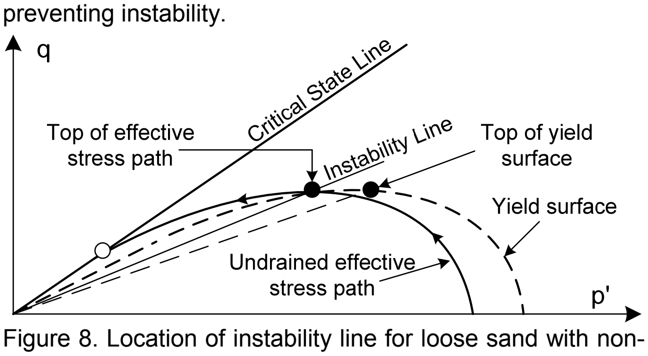

Figure 8. Location of instability line for loose sand with non-associative flow rule (after Lade 1992)

The undrained instability points are positioned below the critical state line. Taking the undrained shear strength at the CSL would clearly underestimate the true effective strength of the soil. Instability can be prevented by staying within the stress gradient (q/p') boundaries of the undrained instability line and even at the boundary, instability doesn't occur under drained conditions. Chu *et al.* (2003) formulated a modified state parameter framework for the analysis of loose and dense sand slopes. The modified state parameter ψ_{mod} (which is a modification of the state parameter concept of Been & Jefferies, 1985), is defined as the difference in void ratio at the instability point e_{IL} and the void ratio at the CSL e_{cr} (see eq. 5).

$$\Psi_{mod} = e_{IL} - e_{cr} = e_{IL} - \lambda \log p' \qquad [5]$$

The transition between contractive and dilative soil behaviour is defined by Chu *et al.* (2003) at the CSL, while Castro (1969), Molenkamp (1983) and Been & Jefferies (2006) indicated a transition for sand at a negative Ψ_{mod} ($\geq$ -0.10). This is explained by a small amount of contraction before dilation occurs, which can already result in instability. An example of undrained instability lines below the CSL for clayey soils, indicating a non-associative flow rule, is presented in Figure 9. In these tests reconstituted Magnus Clay is aged during one-dimensional compression followed by undrained triaxial compression testing resulting in an increase of the vertical yield stress. The authors added instability lines in Figure 9, and for these tests and increase of the slope of the instability line with reduction in void ratio due to ageing is visible. The concept of instability lines, introduced for static liquefaction of sandy slopes, thus appears to be applicable for a wider range of soils.

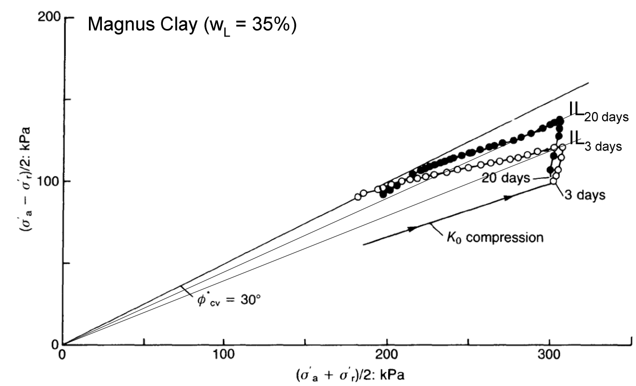

Figure 9. Influence of ageing on undrained effective stress paths and instability lines (after Fig. 26a – Burland 1990. Courtesy of Thomas Telford).

3. CASE HISTORIES LONG TERM SLOPE STABILITY

3.1 Overconsolidated and stiff fissured clay

According to Skempton (1970) a principal distinction can be made for the first time slides in overconsolidated clays in peak strength values, the fully softened strength (approximation of critical state strength) and the residual strength values of soil. Based on various field evidence Skempton (1970) proposes the use of the fully softened strength (or critical state strength) as the limit of strength reduction in first-time slides for London Clay and probably in many other stiff fissured clays. The residual strength is only reached after instability already has occurred.

This view is supported by Lefebvre (1981) who reported 14 slope stability case histories on both excavated and natural slopes where 3 of 7 excavated slopes with inclinations around the post-peak strength (~ CSL) failed within 2 years after construction. This can be explained by destructuration of the structured sensitive clay. Of the 7 reported natural slopes, 5 failed due to too steep inclinations compared to the post-peak strength and one shallow instability occurred due to weathering.

3.2 Seasonal effects

Results of physical model tests on Kaolin clay have been reported by Take & Bolton (2004), where the effects of seasonal moisture cycles on the creep rate of the slope has been investigated. The progressive slope movement in the tests, combined with tension cracks in the slope surface and irrecoverable shear ruptures at the toe, are strong indicators of soil destructuration. First analysis of the results support Skempton's (1970) recommendation to design slopes in heavily over-consolidated clay using the critical state friction angle ϕ_{cv} to prevent first time slides.

Experimental evidence of rainfall infiltration in a partly saturated loose slope is reported by Take *et al.* (2004). The matric suction of the unsaturated soil shifts the isotache surfaces in the q-p' plane mainly along the isotropic effective stress axis due to the suction-induced inter-granular stress increase (Molenkamp & Nazemi 2003).

In the physical model tests with 45° slope the lack of groundwater pounding and thus the remaining presence of matric suction has prevented the development of static liquefaction.

3.3 Destructuration

The progressive Sandnes slope failure in 1963 in Norway in unweathered clay with weak bonds, described by Bjerrum (1967a), was initiated by a relatively shallow excavation at the toe of the slope. Initially a high peak shear strength was present in the clay layer with large horizontal stresses, until local passive shear failure in the plastic clay at the bottom of the excavation caused the gradual development of a horizontal slip plane. The brittle overlying sandy clay layer followed the underlying sensitive clay layer of which the shear strength was reduced to its residual value.

The mobilised shear strength in reactivated landslides in which there is movement along one or more existing shear surfaces is usually assumed to correspond to residual conditions. Skempton (1964) found for various instabilities from back-calculations both residual and intermediate values between peak and residual.

The destructuration of the in-situ undrained shear strength for the Murro test embankment in Finland is described by Karstunen *et al.* (2005). During the consolidation period a reduction in both the undrained shear strength resulting from field vanes and the sensitivity was measured, which could only be explained after performing numerical simulations incorporating anisotropy and destructuration. The high strain rate during the consolidation stage was dominant while after consolidation ageing and structuring could overtake.

4. CONCEPTUAL DESIGN APPROACH

4.1 General outline

As a result of the presented experimental field and laboratory evidence in the previous sections, a generalised consistent material behaviour can be recognised applicable for a wide range of soils. For the long term stability the position and the type of yielding along the isotache surfaces, presented in Figure 10, is of crucial importance.

For light and heavily overconsolidated (OC) soils, the position of the yield surface can change due to variation in strain rate, temperature and structuring, but no instabilities were reported for stress points below the CSL.

It is assumed by the authors that at a point in time for OC soils, instability may occur outside the CSL and compressive yielding can occur within the CSL, which is in compliance with Figure 10.

These phenomena can be explained by the diminishing isotache surface due to either temperature increase or decrease in strain rate due to creep effects. During the lifetime of the construction the stress point with diminishing strain rate may be passed by an isotache surface with increasing strain rate. The relative position of the fixed stress point in time with respect to the CSL will determine the type of soil response including potential instabiliity. According to this philosophy yielding above the CSL results in instability, possibly with localization, and compression yielding at or below the CSL will results in strength increase. This is in agreement with discussions by Skempton (1970) and Bolton (2006).

There is however some reason for caution given the evidence in section 2 on saturated sand that a static liquefaction boundary is present at a negative modified state parameter.

Furthermore, caution is required when the material is sensitive to destructuration of saturated soils, since undrained instability lines are likely to occur below the CSL as destructuration implies also increase in void ratio. Severe destructuration is expected in highly fissured OC soils with weak bonds, sensitive soils in general and at reactivated landslides.

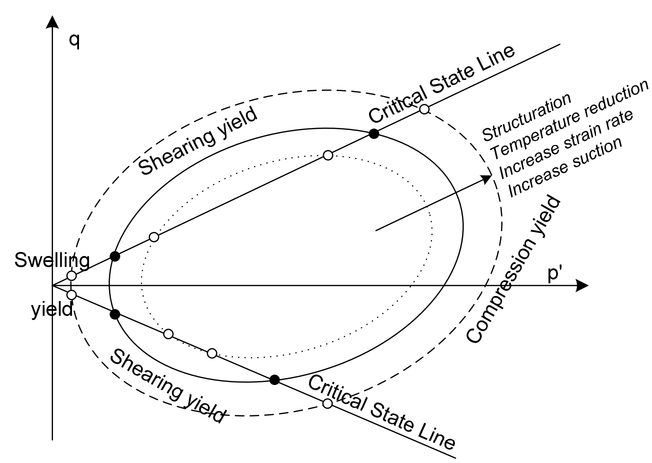

Figure 10. Different types of yield along the isotache surface (after Leroueil & Vaughan 1990)

The schematic presentation of the instability line framework is presented in Figure 11 combined with effect of strain rate, temperature, structure and suction on the isotache surfaces. Increasing isotache surfaces result in an increase of the slope of the instability line and vice versa under the strict condition that the stress state in the slope stays on the right hand side and below the undrained instability line (see Figure 9).

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The concept of the undrained instability lines, proposed by Lade (1992) for static liquefaction analysis of saturated sand, can be used for assessing the drained and undrained stability for a wide range of soils. For saturated soils the lower stability limit will be governing the occurrence of instability in engineering practice, where for the loose soils the undrained instability will be dominant.

The schematic presentation of the undrained effective stress path in Figure 8 for soils exhibiting non-associative flow behaviour clearly indicates the large strength difference between the top of the effective stress path and the strength at the critical state line. In the latter case the effective strength is not fully utilised. However, the consequence of fully utilising the strength requires proper risk assessment given the catastrophic nature of an eventual failure.

For soils exhibiting an associative flow rule, the top of the undrained effective stress path approximates the critical state strength on the yield curve. In this case the most right stress point is selected (see Figure 11). The consequence of an eventual instability is however in this case less severe.

The dashed line and dotted curves in Figure 11 are schematic presentations of the instability lines for sand and clay respectively. The lines are not on scale and the magnitude of both materials is expected to vary. When no or quantifiable destructuration, temperature or strain rate increase occurs, a design may be based on the corresponding void ratio and slope of the instability line.

For lightly overconsolidated soils Tavenas *et al.* (1977, 1978) have shown that creep effects result in drained instability outside the CSL and in yielding in time within the CSL. For lightly overconsolidated soils the CSL would be a safe choice when no or quantifiable destructuration occurs, since the temperature increase doesn't affect the CSL.

The instability line of saturated remoulded soils is likely to be positioned below the CSL. The type of yielding outside and below this particular instability line is compressive and shearing in nature, in analogy with Figure 10.

Destructuration, which may also be caused by temperature increase, creep or straining, may reduce the IL requiring a careful quantification of the safety margin.

4.2 Practical implications

Depending on the type of project on hand a rather straightforward or more advanced approach may be adopted. Testing the critical state friction angle and the fully remoulded residual shear strength and plotting the results in q-p' plane would be sufficient for OC and NC sensitive soils respectively.

When a more extended design approach is required the instability line should be determined for void ratios ranging from the intrinsic compression line to the sedimentation compression line (Burland 1990). For the structural properties in natural conditions, undisturbed soil sampling involving carved blocks from the surface, the Laval (La Rochelle *et al.* 1981) or Sherbrooke block sampler (Lefebvre & Poulin 1979) are required. The resulting soil dependent instability lines can be used to asses the effect of influencing factors, such as temperature, strain rate and destructuration.

It is the hypothesis of the authors that destructuration reduces the slope of the undrained instability line (larger void ratio) and that both temperature effects and strain rate effects will result in compressive yielding in time as long as the stress ratio (q/p') stays within the applicable instability line.

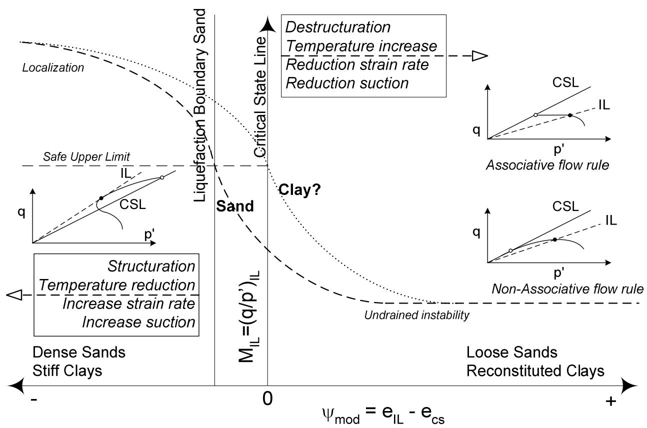

Figure 11. Schematic presentation of instability line concept and influencing soil behaviour aspects on isotache surface and resulting slope of instability line M_{IL}.

5. CONCLUSIONS AND RECOMMENDATIONS

The presented instability line framework, which is in line with work by Lade (1992) and Chu *et al.* (2003), is partly based on Critical State Soil Mechanics and gives insight in soil mechanical effects like viscosity, strain rate and structure.

Depending on the type of project on hand and allowable risk involved, a conservative or more actual strength approach can be applied.

As a conservative estimate it is recommended to apply the critical state friction angle as design boundary for OC soils and the instability line corresponding to the residual friction angle for soils sensitive for destructuration and for slopes where failures have occurred.

For specific projects, where the impact of destructuration on the geotechnical parameters can be quantified, the undrained instability line concept is applicable. The elegance of the concept is that it gives insight in the effects of various geotechnical aspects and allows for prevention of initiation of instability.

The key implications of this concept are that soil disturbance needs to be quantified and that the in situ void ratio is of significant importance. Both in-situ testing, block sampling and advanced laboratory void ratio dependent testing ranging from intrinsic properties to undisturbed natural structured soil conditions are therefore important development areas. Both strength and stress anisotropy of soils influence the soils response and sensitivity for destructuration. Both laboratory and numerical research developments need to address these issues.

It is recommended to verify the influence of temperature, strain rate and destructuration on the instability line framework for a wide range of saturated and unsaturated soils. The authors are presently investigating the material behaviour of organic soils in general using both field (Boylan & Long 2006) and laboratory testing. Furthermore, sampling induced disturbance of organic soils resulting from various samplers (Mathijssen *et al.* 2008) and the applicability of the CSSM framework in general and the presented instability line framework are subject of investigation.

6. ACKNOWLEDGMENTS

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