Examination of cracking potential in the low-plasticity core of an earth dam

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ABSTRACT
In the construction of earth dam cores, it is desirable to use soil with sufficient plasticity and stiffness, and low permeability, preferably from sources near the dam site. However, in some instances, the available fine-grained soils are silty and do not have the desired plasticity. Use of such material in the dam core can increase the potential for cracking. The Doosti earth dam in northeast Iran has been constructed with low-plasticity material and may be subject to such risk. In the current paper, the potential for cracking in the core of this dam has been examined using the principles of Critical State Soil Mechanics. This approach was used previously to investigate the failure of the Teton Dam, which had a low-plasticity core. By calculating stress ratios in a longitudinal section of the dam core, regions with potential for cracking are identified using a cracking criterion. The cracking potential is also examined using the soil Liquidity Index. It is concluded that cracking potential exists in some zones up to certain depths below crest elevation of the Doosti Dam.

RÉSUMÉ
Dans la construction des noyaux de barrage en terre, il est souhaitable d'employer le sol avec la plasticité et la rigidité suffisante, et la basse perméabilité, de préférence des sources près du barrage. Cependant, parfois, les sols fins existants sont principalement vaseux et n'ont pas la plasticité requise pour le matériel de noyau, et ceci peut augmenter le potentiel pour fendre dans le noyau de barrage. Le barrage en terre de Doosti en Iran du nord-est a été construit avec le matériel de bas-plasticité et peut être sujet à un tel risque. Dans le papier courant, le potentiel pour fendre dans le noyau de ce barrage a été examiné a basé à l'examen du développement des contraintes de traction dans le noyau, utilisant les principes de la mécanique de sol d'état critique. Cette approche a été employée précédemment pour étudier l'échec du noyau de barrage de Teton de bas-plasticité. Par des rapports calculateurs d'effort dans une section longitudinale du noyau et d'examiner de barrage le critère de fissuration, des régions avec le potentiel pour déceler fendre sont identifiées.

1 INTRODUCTION

Cracking in earth dams may be caused by various factors such as: differential settlements of the dam foundation, horizontal displacements due to soil expansion and contraction, differences between the stiffnesses of various parts of the dam or those between the soil and the adjacent solid parts, etc. Generally, when tensile or shear strains exceed the levels that can be tolerated by the dam material, cracking may occur in the dam surface or body. If cracks occur at depth and are not repaired, they may eventually lead to the collapse of parts of the dam. The combination of stresses, the type of soil, and it's moisture content can have an important influence on the dam cracking potential. Generally, highly compacted, low plasticity soils tend to crack in an environment of low liquidity index, low confining stresses and high shear stresses. Soils with lower water content are more subject to cracking, and a relatively small tensile strain may cause them to crack. Depending on soil type and factors such as those indicated above, tensile strains beyond a range of approximately 0.15% to 0.48% may lead to cracking in soils.

The principles of Critical State Soil Mechanics (CSSM) (Schofield and Wroth, 1968) may be used to investigate the potential for cracking based on an examination of stresses that develop in the dam core and checking them against a cracking criterion. The concept of Liquidity Index (LI) may also be used to examine cracking potential. Sasiharan and Muhunthan (2006) used these criteria to investigate the potential for cracking in the low-plasticity core of the failed Teton dam, and showed that results obtained from such analyses are consistent with observations reported during dam failure.

2 CRITICAL STATE SOIL MECHANICS AND SOIL CRACKING

2.1 Boundaries of ultimate conditions for sedimentary masses

Muhunthan and Schofield (2000) indicated that based on their depth, natural and man-made soil deposits may exhibit three distinct behaviours (Figure 1).

In deep layers, intense pressure causes ductile yielding of the mass and results in the sedimentary soil mass to fold. In regions above the aforementioned depths, the pressure is lower, and the soil mass fails in the form of sliding of the layers with respect to each other, leading to the formation of faults along sliding surfaces. Near the ground surface, where pressures are lower than those at the two regions described above, layers split, and cracks appear in the soil surface. Sasiharan and

Muhunthan (2006) indicated that CSSM may be used to identify the regions where such behaviours occur, since it relates soil behaviour to its density and effective stress. The next sections examine these interpretations of the concepts of CSSM.

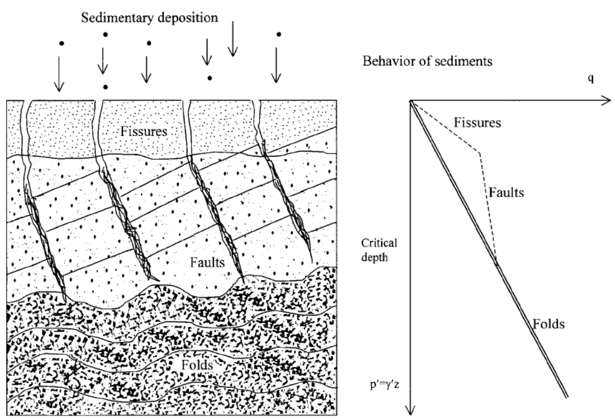

Figure1. Regions with folds, faults and fissures (Muhunthan and Schofield, 2000)

2.2 General soil behaviour in the υ-ln$p$' plane

According to CSSM, compaction and elastic swelling of laboratory specimens follow Equation1 below.

$$v_k = v + kLnp' = const \qquad [1]$$

in which υ_k and υ are the specific volumes at unit and current mean effective normal stresses, p' is the mean effective normal stress and k is the slope of variation. As illustrated in Figure 2(a), a particular value of υ_k specifies a line in the υ v.s. lnp' plane that corresponds to the mass of particles consistent with that of a certain point such as A in this plane. Elastic compaction and swelling parameters define the gradient of this line and υ_k depends on the mass density. No sliding (plastic deformation) between particles is expected to occur during pure elastic deformations in an ideal soil such as that defined in the Cam-Clay model. However, as plastic deformation takes place, a new arrangement of particles is formed (particle positions related to each other change). This behaviour results in the moving to elastic compaction and swelling lines with the same gradient but different values of υ_k; and, the transitions between the lines demonstrate plastic volume changes from a certain condition to a new one such as from point A to B.

2.3 Critical states in the *q*-*p*' space

In the CSSM framework, states of soils are defined in a 3D (p', υ, q) space in which q is the shear (deviatoric) stress. Limits to stable states of yielding are defined by the state boundary surface in this 3D space. A 2D representation of the normalized state boundary surface in the q/p'_{crit}-p/p'_{crit} plane is shown in Figure 2(b). In this figure, p'_{crit} is the mean effective normal stress at critical state for the current specific volume (or void ratio).

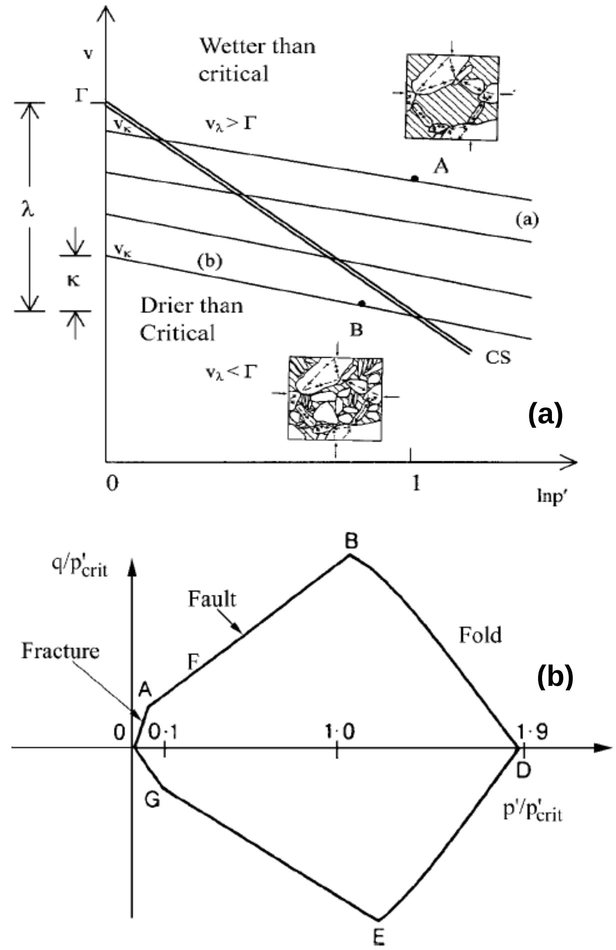

Figure 2. Concepts of CSSM used to differentiate soil states (a) Compression and swelling of soils. (b) Limits of stable states of yielding in the modified stress space (Muhunthan and Schofield, 2000)

Soil behaviour at limiting states may be divided into three distinct classes of failure. The limiting lines OA and OG in Figure 2(b) indicate states in which the soil is undergoing fractures or cracks; AB and GE indicate Hvorslev's Coulomb faults on rupture planes; and the BD and ED lines indicate Cam-clay yielding and folding of the soil. Soil states on the crack surface result in the development of unstable fissures and crack openings.

The 'no tension' or 'limiting tensile strain' criteria are the most widely used among the alternative theories to quantify tensile fracture (Schofield, 1980). For the case of triaxial loading, the no tension criterion with q/p' = 3 leads to vertical cracks, which is the case on line OA. For horizontal cracks, it results in q/p' = 1.5, which corresponds to the stress states on line OG. For clays or silty clays, Schofield (1980) suggested that the change from rupture to tensile cracks occurs at a pressure p' =

0.1 p'_c, where p'_c is the effective mean normal stress at critical state. This is equivalent to an overconsolidation ratio of approximately 20. When the effective stress path crosses the crack surface OA, the soil element in that location begins to disintegrate into a clastic body and unstressed grains become free to slide apart. In that case the average specific volume of the clastic mass can increase, large voids/cracks develop, and consequently the soil mass permeability can increase significantly. When such condition occurs, openings in the soil body may lead to the formation of extensive cracks or local channels. In an earth dam, if such openings daylight into the reservoir and makes its way into the downstream side, it could lead to a free flow of water from the reservoir to the downstream side.

3 THE DOOSTI DAM

The Doosti dam is an earth and rockfill dam with silty clay core, located on the common border of Iran and Turkmenistan at the north 35°75' latitude and east 61°9' longitude. The dam height above the foundation level is 79m at its deepest location; its crest length is 655m; and, the dam width is 428 m at foundation and 15 m at crest elevations. The dam cross-section and zoning are shown in Figure 3.

The dam primary purpose is to supply water for the city of Mashad and for agriculture in the Sarakhs plain. As shown in Figure 3, the impermeable part of the dam includes a silty clay core with a permeability of about $75*10^{-7}$cm/s, placed on the intact foundation bedrock. The average plasticity index (PI) of the material found locally for the dam core was approximately 4, making it necessary to take into account provisions required in the design and construction of low-plasticity core dams.

For example, according to the dam specification, in order to provide sufficient contact between the silty core material and the adjacent foundation, galleries, and valley walls, the soils placed at these locations should have a plasticity index (PI) of about 15. Therefore, soils in these areas were mixed with sufficient amounts of Bentonite to achieve the required PI.

While the primary criteria in dam design involve slope stability and seepage control, science and experience in this field have demonstrated that controlling deformations in the body and foundation of dams is also of paramount importance. Furthermore, a dam must be designed in such a way that crack formation potentials are minimized or eliminated. It is worth mentioning that if, for some reasons, cracks form in an earth dam, their propagation should be controlled.

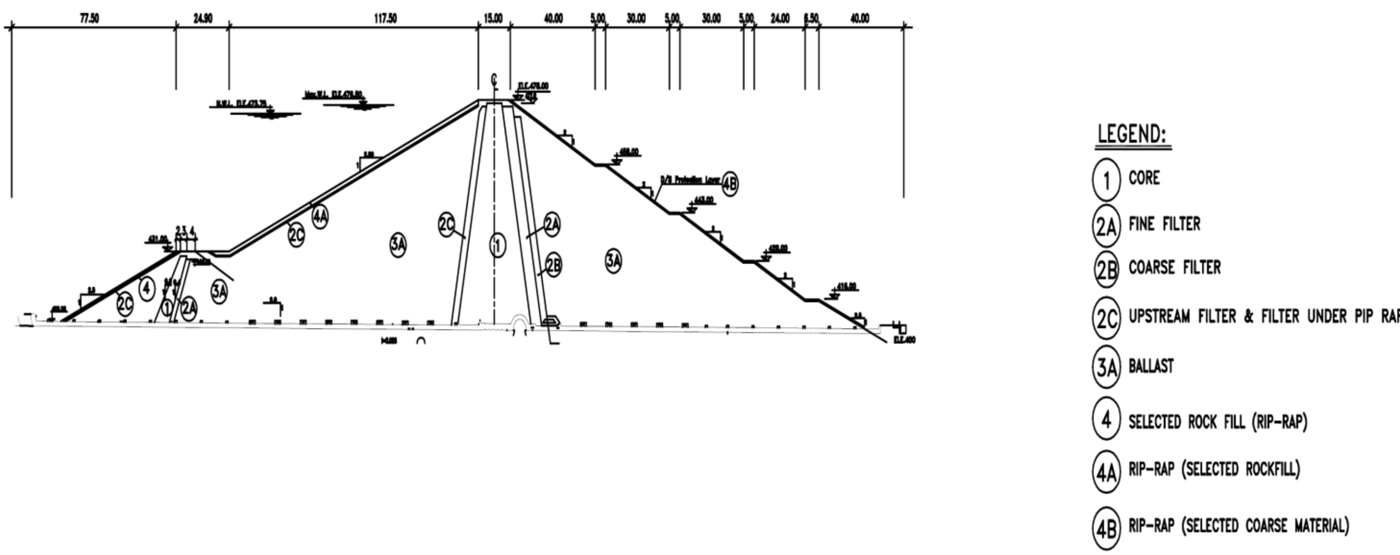

Figure 3. Cross section and material zoning of the Doosti Dam (Toosab, 2005)

4 DOOSTI DAM NUMERICAL MODELING

4.1 Introduction

Earth dams are typically analyzed for two conditions. The first is the condition during and to the end of construction. The construction operation of a dam consists of placing and compaction of various soil and rock types in layers with thicknesses in the order of 0.2 m to 1m. The need for considering stages of construction in the analysis of dam behaviour is currently well accepted. Results of analyses obtained from assuming one stage and several stages of construction for a homogeneous earth dam on solid foundation has revealed that while there is no considerable difference between the calculated stresses obtained from the two analyses, the difference in the resulting deformations is significant.

The second condition typically considered in earth dam analyses is that of first impounding. During this stage, the forces and effects of water on the dam shell, core and foundation are taken into account in the analysis.

In the current work, in order to examine the cracking potential in the core of the Doosti am, the stress states in the core is examined for the condition at the end of construction since due to the lower water content of the material and lack of the lateral load from the water on the core, it is expected that this condition results in a higher potential for cracking in the core.

For this analysis, a model of the longitudinal section of the dam consisting of a finite difference mesh with 331 elements in the x-direction (along dam length) and 79 elements in the Y-direction (along dam elevation) was constructed. These numbers of element were selected based on the crest length (665m) and dam height (79m). Therefore, the element dimensions were 2 m in the x and 1 m in the y directions.

The model has been presented in Figure 4. In order to consider the staged construction of the dam, 13 stages were assumed such that each stage involved the construction of a 6 meter layer of the dam height. Since the dam foundation and abutments consist of good quality rocks with a deformation modulus much higher than that of the core, in order to minimize the calculation efforts required, the foundation material was not included in the numerical model.

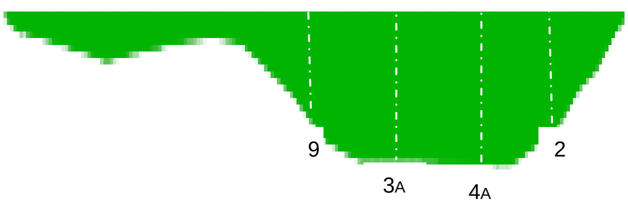

Figure 4.Longitudinal section of Doosti Dam used in the analysis

4.2 The material model used

The Cam-Clay soil model, presented in the framework of CSSM was used for the analysis of Doosti dam. Results of triaxial, consolidation, and direct shear tests on samples of the core material were used to determine the parameters for this model. Values of soil parameters obtained from tests on various samples were examined and average, representative values were selected for modeling. These parameters are shown in Table 1

4.3 Dam settlements obtained from analysis and their verification

As shown in Fig. 4, dam height along the longitudinal section varies substantially. In order to examine the overall behaviour of the dam, settlements obtained in four transverse sections along the dam length namely Sections 2, 3A, 4A, and 9, were determined, and the results were compared with readings of instrumentations installed at various locations of the sections. Figure 5 compares the settlements measured and calculated at the highest section (Section 4A) of the dam and shows that the analysis captures the actual behaviour with a reasonably good accuracy. Comparisons between measured and calculated settlements in other sections are presented elsewhere (Mazaheri, 2010).

Table 1. Cam-Clay parameters used for the analysis of Doosti dam

value	name of critical state Parameter
0.004	k
0.028	λ
1.352	ν
1.541	N
$2*10^6$	Bulk(kPa)
$1/6*10^3$	P'c(kPa)

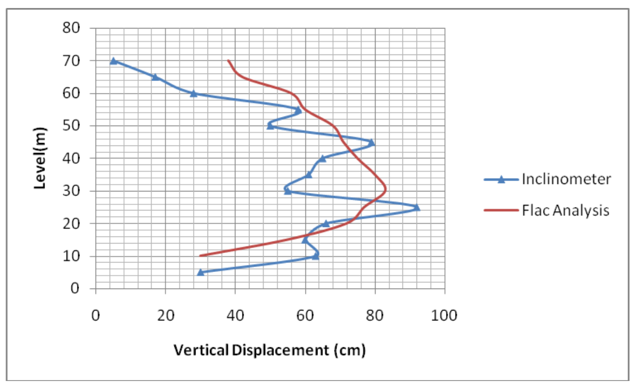

Figure 5. Comparison of measured and calculated settlements in the maximum-height section of the dam

Calculated settlements at the five sections analyzed are presented in Figure 5. As expected, and is seen in this figure, maximum settlement of the dam has occurred in the largest section 4A. The largest settlement occurs at a location about 30 m above the foundation level and is 85 cm. It is noted that due to the irregular shape of the dam longitudinal section, and differences in the elevations of the dam foundation in different sections, shapes of settlement curves and locations of maximum settlements are not quite similar in the various sections.

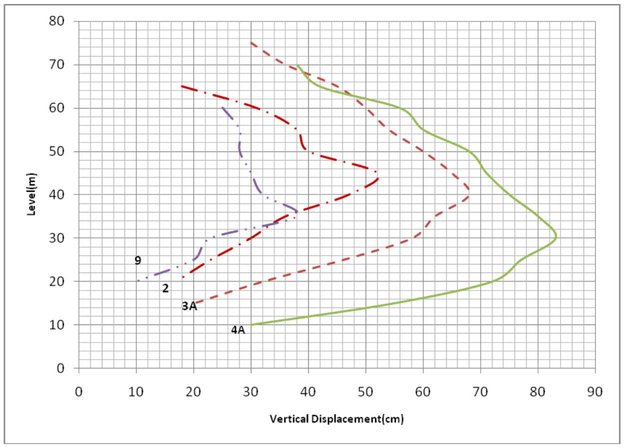

Figure 5. Comparison of settlements at various sections of Doosti dam

4.4 Stress ratios and dam cracking potential

As discussed previously, the CSSM theory indicates that under stress conditions corresponding to triaxial compression, regions with stress ratios (*q*/*p*') greater than 3 are subject to vertical cracks or gaps. Since differences in the stresses that develop in the dam core in the two horizontal directions are not significant, the criterion indicated above obtained based on the stress state in triaxial compression may be used with some approximation in the determination of regions where the potential for cracking exists. Such regions may be determined for the Doosti dam using contours of *q*/*p*' in the dam longitudinal section as shown in Figure 6. The results indicate that at the end of construction, the potential for cracking in the core of the dam exists in two zones within the longitudinal section. These zones are located at stations 296 m and 640 m from the left abutment. Based on the distribution of stress ratios shown in this figure, it may be noticed that the depth of the first zone is about 7 m and the second zone is about 10 m below the dam crest (Figure 7).

It is further noted that while the critical state theory indicates that regions with *q*/*p*'<3 are theoretically considered to remain stable, however, water may flow through cracks that develop in regions with *q*/*p*'>3, and may therefore erode them, resulting in deeper cracks which can extend to depths greater than those obtained from analysis. Sasiharan and Muhunthan (2006) analyzed a longitudinal section of the Teton dam and concluded that failure of this dam initiated at locations where such zones with potential for cracking were identified using the aforementioned procedure based on the CSSM.

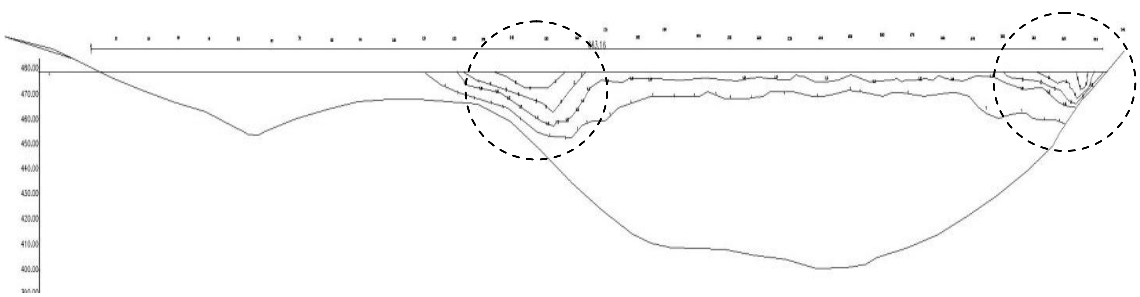

Figure 6.Stresses ratio q/p' diagrams in longitudinal section of Doosti dam

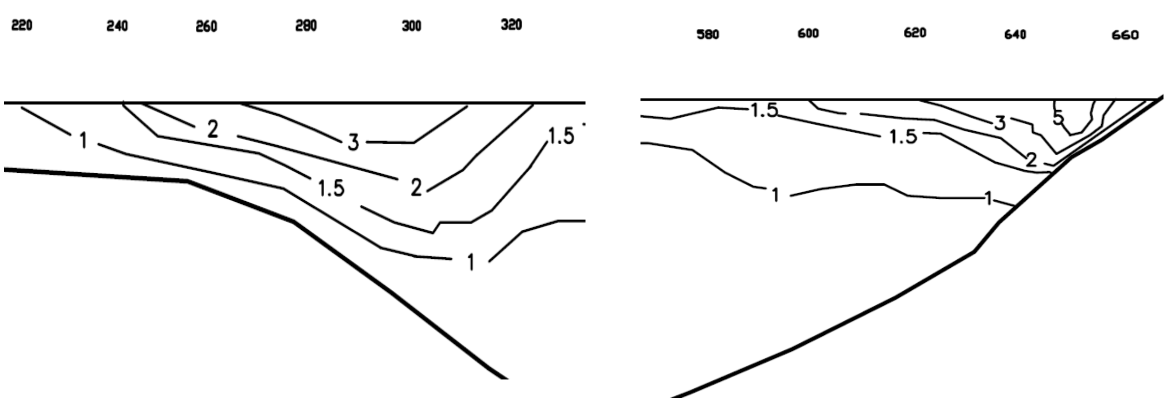

Figure 7. Magnified contours of stress ratios q/p’ on the right and left sides of the longitudinal section of Doosti dam

5 THE LIQUIDITY INDEX CRITERION

5.1 Short summary of the criterion

The Liquidity Index (LI) of a fine grained soil is a measure of the proximity of its water content to the water content at liquid limit, and is defined as follows:

$$LI = \frac{\omega - PL}{LL - PL} \qquad [2]$$

in which ω, *PL* and *LL* are the water contents at current state, plastic limit, and liquid limit, respectively. In using the CSSM theory, it is possible to convert the specific volume or void ratio axis to a measure of the soil liquidity index. In this case, the mean effective normal stress (pressure, p') at critical state corresponding to the LL is about 5 kPa and at the PL is about 500 kPa (Schofield and Wroth, 1968).

In his Rankine lecture, Schofield (1980) mapped the remoulded soil behaviour on a liquidity index v.s. pressure diagram using the hundred fold (two log cycles) increase in pressure at critical state from the LL to the PL (Figure 8(a)). In such diagram, the rupture band has half the width of plastic index (PI) and will intersect the p' = 5 kPa line at LI = 0.5. He showed that this intersection is a result of putting the lower limit of the Mohr Coulomb rupture at p'/p'_{crit}= 0.1. Figure 8(a) shows that there are clear boundaries in the LI-p' space that separate the regions of fracture, rupture, and ductile yielding; and, therefore, it is possible to identify these states of yielding based on the soil index properties. For examples, for a body of soil with LI = 0.5 subjected to an elastic compression, the map suggests that at shallow depths where p' < 5kPa cracks may develop. However, at greater depths where 5 kPa <p' < 50 kPa the soil will experience rupture and remain watertight while deforming. On the other hand, soils with LI = 0 at depths where p' < 50 kPa (about 3m of overburden depth) will undergo fracture. This means that for these soils, the overburden depth should be greater than about 3m to ensure that deformation causes watertight rupture planes rather than open cracks.

Schofield (1980) defined an "equivalent liquidity" corresponding to various soil states by projecting these states in the direction parallel to the critical state line towards the ordinate at p' = 5 kPa. He showed that the equivalent liquidity (LI_5) is obtained from the relationship LI_5 =LI+0.5log(p'/5). Therefore, the equivalent liquidity equals the liquidity at the ground surface plus a correction to the liquidity for the mean effective normal stress. In this case, an LI_5 < 0.5 generally indicates yielding through fracture, 0.5 < LI_5 < 1.0 represents the rupture zone, and LI_5 > 1.0 represent ductile yielding.

Figure 8(b) shows a section of the behaviour map at constant p'. It may be noticed that as the equivalent liquidity increases, the stress ratio q/p' decreases. The Hvorslev surface defined in the CSSM defines the rupture limits. The no tension limits, which correspond to q/p' = 3 in compression, and q/p' = -1.5 in extension specify the fracture zone in which values of LI_5 are low. While the limiting stress ratio generally increases with the decrease in the equivalent liquidity, this change is not continuous since the limiting behaviour changes from ductile yielding to discrete rupture, and then to the fracture of stiff fissured soil, which occurs at an equivalent liquidity below 0.5 (Schofield, 1980).

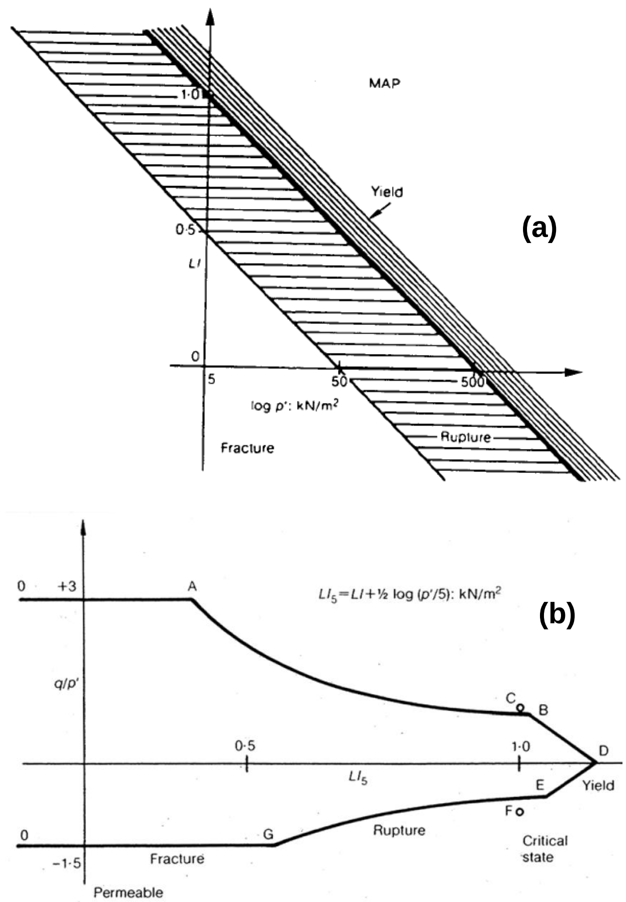

Figure 8. (a) Using the LI v.s. log P' diagram to identify various states of yielding (b) A p'=const. section of diagram (a) (Schofield,1980)

5.2 Application of the criterion for the Doosti Dam

Construction of the Doosti dam embankment started in May 2003 and lasted until August 2004. During this period, moisture content, compaction and some index tests were carried out during placement of each layer.

Averages of results of these tests are presented in Table 2. It may be noticed that the first series of test results were obtained in the early stage of dam construction while the third series were obtained late in the period of construction.

Values of LI obtained from Equation [2] were determined using the results obtained from each test series, and the results are shown in the last column of Table 2. The results indicate negative liquidity indices for all three test series. Based on Figure 8(a), the first series (tested in May 2003) with LI=-0.2 requires a pressure of p'>80 kPa (approximately 4 meters of overburden) so that the soil state will not fall into the zone of cracking. The second and third data set with LI= -0.5 and LI=-0.58 require p> 300 kPa (approximately 15m of overburen) and p>400 kPa (approximately 20 m of overburden) respectively, in order to prevent cracking in the dam core based on this criterion. Considering the dates of collection of the samples, it may be concluded that while the required overburden is provided for the samples collected in May and September 2003, the approximately 20 m overburden needed to avoid cracking in the upper layers placed in April 2004 (four months prior to completion of embankment construction) was not possible, and the potential for cracking, based on this criterion, exists in these layers.

Table 2. Properties of soil samples collected during three stages of dam construction

date	water content	optimum moisture content	LI (%)	pI (%)	LI (%)
May, 2003	14	15/5	26	16	**-0.2**
Sep., 2003	13/5	14/5	24	17	**-0.5**
Apr., 2004	14	13/5	23	17	**-0.58**

It is worth noting that due to the low plasticity index of the core material of Doosti Dam, the liquidity index of these materials is very sensitive to changes in its water content. As a result, a small increase in water content during placement of the fill may reduce the potential for cracking significantly.

This sensitivity to changes in water content may also have an important influence on the behaviour of the core material under rapid changes of shear and confining stresses, especially near the dam abutments. Height of the soil column reduces in near the abutments having steep slopes and, therefore, the confining stresses decrease in the soil elements. In fact, the soil columns near the supports may lie in the Hvorslev region of the CSSM model while the soil columns in the middle of the dam valley are may remain in the deformable region.

6 CONCLUSION

Based on numerical modeling of the Doosti dam and plotting of contours of q/p' ratios using the CSSM and the Cam Clay soil model, theory, two zones along the dam length are identified in which the ratio q/p' exceeds 3 and, therefore, the potential for cracking exists according to this criterion. These regions are located at 296 m and 640 m distance from the left abutment. Depths of these regions from the dam crest are 7m and 10m respectively.

Analysis of the longitudinal section of the dam also showed that the maximum settlement occurs in section 4A (which is the largest section) and is at approximately 40m elevation from the foundation. This maximum settlement is equal to 85 cm which is approximately 1% of the dam height and is expectable.

In order to achieve better strength and stiffness properties in soils, compaction specifications often recommend using moisture contents close to, or slightly lower than optimum water content. However, using the criterion based on the liquidity index of soils, it may be concluded that where the dam core has a low plastic index, such as in the Doosti dam, such water content may result in a liquidity index close to or less than zero, resulting in the existence of a potential for cracking in the dam core even up to a considerable depth. In such cases, it may be possible to slightly increase the water content at compaction without compromising strength and stiffness significantly in order to provide adequate equivalent liquidity index. This can take the soil in most depths to the Hvorslev (watertight rupture) region, where it is stronger and less permeable, except for the soils at the upper 2 to 3m which is typically considered as dam freeboard.

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