13 Apr 2004 - 17 Apr 2004

The Failure of Teton Dam – A New Theory Based on "State Based Soil Mechanics"

V. S. Pillai
Geotechnical Engineer, Vancouver, BC, Canada

V. Muhunthan
Washington State University, Pullman, Washington

N. Sasiharan
Washington State University, Pullman, Washington

Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

THE FAILURE OF TETON DAM – A NEW THEORY BASED ON "STATE BASED SOIL MECHANICS"

V.S.Pillai
Geotechnical Engineer
7753 Ontario Street
Vancouver, BC Canada V5X 3C6

B.Muhunthan
Associate Professor
Washington State University
Pullman, WA, USA 99164

N. Sasiharan
Graduate Student
Washington State University
Pullman, WA, USA 99164

ABSTRACT

Teton Dam failed during its first filling on 5 June 1976. The 405-ft high dam was designed and built using modern standards; therefore its failure received considerable scrutiny from engineering experts. Failure mechanisms suggested, included hydraulic fracture, internal erosion, wet-seam theory, and defects in the abutment rock. None of the investigations, however, were able to explain satisfactorily why the dam breached when the reservoir reached EL.5301.7 ft and only in the vicinity of Sta. 14+00 on the right abutment. The investigation here is focused on this crucial aspect of the failure using the modern framework of fundamental "state based soil mechanics". According to this framework highly compacted soils of low plasticity in an environment of low liquidity index and low confining stress would crack in the presence of high shear stresses. The impervious core (Zone-1) of Teton was constructed of uniform clayey silt of low plasticity and highly compacted and therefore was prone to such a possibility. This paper describes the details of the theory, the investigation, and the conclusions arrived at regarding the potential initiation of Teton failure. Finite element analysis carried out using state based parameters indicate the presence of deep open transverse vertical crack(s) in the core (Zone-1) to a maximum depth of about 32 ft from the crest only in the right abutment and in the vicinity of Sta. 14+00. We conclude that once the water level in the reservoir rose above El 5300.0 ft in the early hours of 5 June 1976 water flowed through the open vertical crack(s), which slowly eroded the crack into a large tunnel leading to the major breach of the dam hours later.

INTRODUCTION

The 405-ft high Teton dam was located in the high plateau of southeastern Idaho (Fig. 1). It failed during its first filling on 5 June 1976. The "sunny-day" failure of the dam resulted in 14 fatalities and a very large economic loss. Its failure was one of the most publicized events at that time involving a large earthfill dam built using current standards. Therefore, this failure received considerable attention from engineering experts around the world. However, the failure assessment and prognosis by experts including those by the Independent Panel (IP, 1976) and the Interior Review Group (IRG, 1980) failed to arrive at a consensus. Failure mechanisms suggested, included hydraulic fractures, internal erosion, the wet-seam theory, and defects in the abutment rock. There, however, remained an unanswered question as to why the dam breached when the reservoir reached El.5301.7ft and initiated only in the vicinity of Sta.14+00 on the right abutment.

The impervious core/water barrier (Zone-1) of Teton was constructed of uniform clayey silt of low plasticity and low liquidity index. Highly compacted soils of low plasticity tend to crack in an environment of low liquidity index, low confining stresses and high shear stresses. None of the previous investigations focused on the possibility of the presence of cracks in the upper portions of the dam. Such possibility is investigated here using the modern concepts of the fundamental framework of "state based soil mechanics" (Pillai and Muhunthan 2001, 2002). The investigation consisted of laboratory tests on Zone-1 material to determine the physical and mechanical parameters and finite element analysis conducted using ABAQUS to simulate the field stress conditions. The results are used to identify the main cause of the Teton failure.

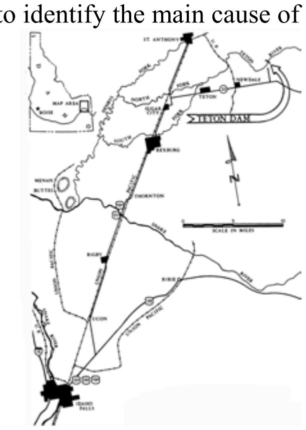


Fig. 1. Location map of Teton Dam (IP, 1976)

BACKGROUND AND FAILURE OF TETON DAM

The design cross section of the Teton dam at the river valley and the right abutment are as shown in Fig. 2 and Fig. 3 respectively. The construction of the dam began in June of 1972 and was completed in November of 1975. The dam was conservatively designed to have a wide impervious core with a head to width ratio of about 1.5 (Figs.2 and 3). The bedrock consisted of open-jointed rhyolite and basalt but was well treated with blanket and curtain grouting. The abutment rock was trenched to provide a large core-rock contact and a long flow path to have a low seepage gradient (Fig.3).

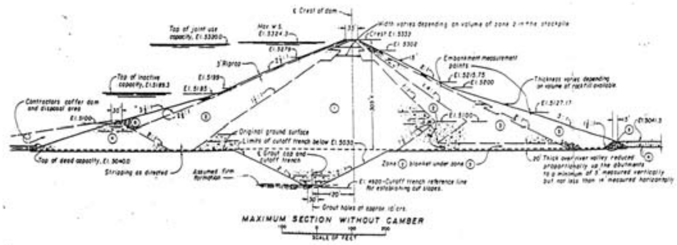


Fig. 2. Design cross section of the dam at river valley section

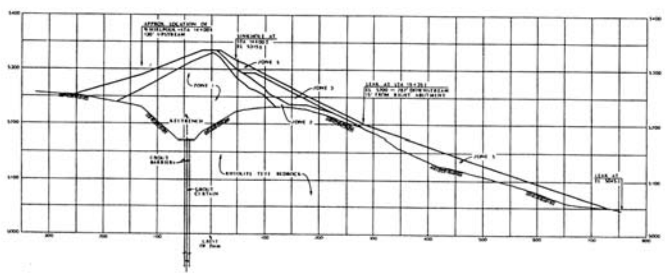


Fig. 3. Cross section of the dam at the right abutment

The impervious core (Zone-1) of the dam consisted of clayey silts of aeolion origin with low plasticity (PI ~ 4) and USCS classification of CL- ML. As per the design and specifications Zone-1 material was placed at average water contents of 1.0% dry of optimum and compacted to a maximum dry density of 98-102 % of the Standard Proctor test (Fig. 4). Similarly the support zone (Zone-2) (chimney filter/drain) was compacted to a high relative density of the order of 65-70 % (IRG 1980).

The first filling of the reservoir began October 3, 1975. The rate of filling of the reservoir was about a foot per day in the early stages; however, it was increased to about 3 ft per day for the most part of May and June 1976. When the dam breached on June 5, 1976 the reservoir had reached only El.5301.7 ft, which was about 22 ft less than the design full pool elevation.

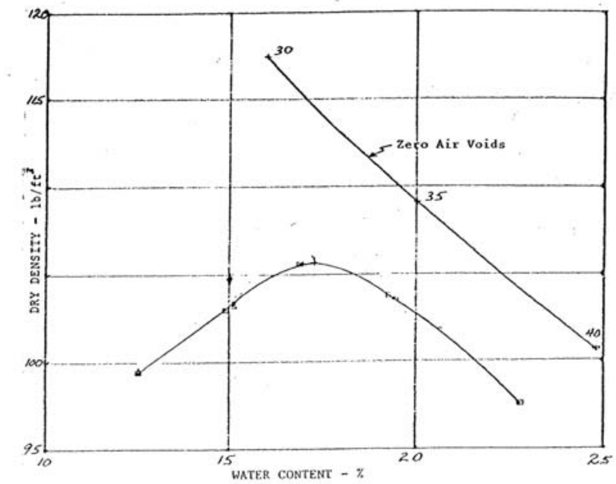


Fig. 4. Typical curve of standard proctor compaction for the Zone-1 material

SURFACE MANIFESTATIONS OF THE DAM FAILURE

On or before June 3, 1976 (Reservoir level was at or below El. 5297 ft), no unusual signs of distress or springs or other increased seepage were noticed downstream of the dam. On June 4, minor evidence of clear seepages appeared downstream, a good distance of 1300-1500 ft from the toe, which was consistent with the raising of the ground water regime due to rising reservoir water level. Late in the evening of June 4 (Reservoir El.5300ft), some dampness was noticed in the right abutment slope at El. 5200 ft. The following morning on June 5 shortly after 7:00 AM (Reservoir El.5301.3 ft) some muddy water was first observed to be flowing from the junction of the embankment and the abutment at El. 5200ft. At 10:30AM, a large leak of about 15 cfs appeared with a "burst" on the downstream at EL 5200.0ft. The leak appeared to emerge from a tunnel of about 6 ft in diameter from inside the embankment and roughly perpendicular to the dam axis at Sta. 15+25. At about 11:00AM, a vortex appeared in the reservoir near Sta. 14+00 above the upstream slope of the embankment. At 11:30 AM, a sinkhole on the downstream slope (El. 5315.0 ft) developed near the crest and above the leaky tunnel. At 11:55AM, the crest of the dam began to collapse between the vortex and the sinkhole, leading to a full breach at 11:59AM (IP 1976).

CONCEPTS OF THE NEW THEORY

Past failure investigations of Teton have paid little attention to its fundamental soil mechanics aspects. We believe that the failure of the Teton dam belongs to a special class of rapid failures brought about by some locations of the highly compacted soils reaching a state of fracture. This fracture state is best addressed by the principles of state based soil mechanics (Pillai and Muhunthan 2001, 2002). The origins of the state based soil mechanics approach lie in the modern concepts of critical state soil mechanics (CSSM). Detail accounts of the CSSM principles, the features, and finite element applications have been presented in a number of publications (Schofield and Wroth, 1968, Schofield, 1980, Muhunthan and Schofield, 2000). Soil behavior under shear stresses including rapid failures is described in CSSM using a normalized state space or equivalently using a normalized space of liquidity index and mean stress. Both approaches are considered here.

RUPTURE AND FRACTURE BASED ON " STATE OF SOILS"

Aggregates of grains that form natural and man-made soil deposits exhibit three distinct classes of behavior (Fig. 5); at large depths, high pressures cause ductile yielding of the aggregates and the layer of sediments to fold; above these depths and at lower pressures aggregates rupture and a layer of sediment faults with the presence of gouge material along the slip planes; near the surface where the pressure is even lower, a layer of sediment fractures or cracks.

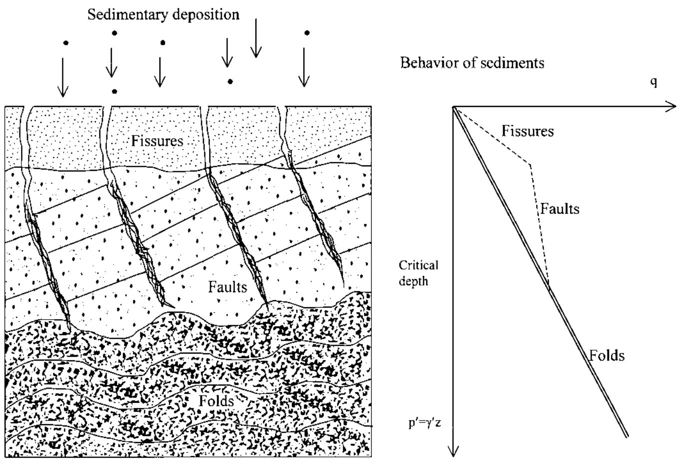
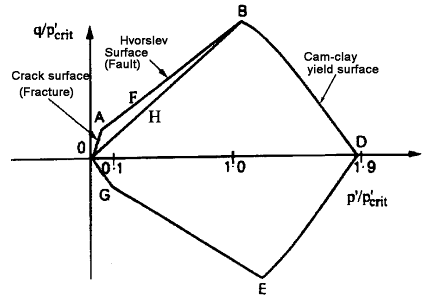


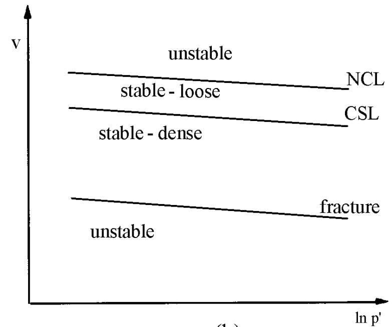
Fig. 5. Folds, faults and fissures in sedimentary deposits (Muhunthan and Schofield, 2000) (schematic)

Critical state soil mechanics captures these simple depositional and structural phenomena of folds, faults, and fractures in soil and sedimentary as well as man-made deposits in a scientific manner. It explicitly recognizes that soil is an aggregate of interlocking frictional particles and the regimes of soil behavior depend in a major way on its density and effective pressure.

In the critical state framework, the state of soils is defined in a 3-D, mean effective normal stress (p), shear stress (q) and void ratio or specific volume (v) space. Limits to stable states of yielding are defined by the state boundary surface in the 3-D, p-q-e space. The 2-D representations of the normalized state boundary surface in the q/p_{crit} - p/p_{crit} and e - ln p spaces are as shown in Fig.6.



(a)



(b)

Fig.-6. Limits of stable states of soils in (a) normalized q/p_{crit} - p/p_{cri}stress space stress (b) v – ln p space (Pillai and Muhunthan,2002) (schematic)

Critical state soil mechanics divides the soil behavior at limiting states into three distinct classes of failure; the limiting lines OA and OG (Fig. 6a) indicate states of soils undergoing fractures or cracks; AB and GE indicate that Hvorslev's Coulomb faults on rupture planes; BD and ED indicate Cam-clay yield and fold of a sediment layer.

Soil states on the crack surface result in the development of unstable fissures and cracks openings. Heavily overconsolidated clays and overcompacted sands at low confining stresses could reach this limiting state. Collapse similar to fracture on the dilative side can also exist on the contractive domain but outside the normal consolidation line (Fig.6b). Such states outside the stable yielding exist in wind deposited loose sands, air pluviated or moist-tamped sands and result abrupt collapse upon shearing of these materials (Pillai and Muhunthan, 2001, 2002). For sands and clayey silts of low plasticity, stable yield behavior occur only within a narrow band on both the looser and denser side of the critical state line (Fig.6b).

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 triaxial specimen the no tension criterion with $\sigma_3 = 0$ results in $p = \sigma_1/3$ or $q/p = 3$ and leads to vertical split cracks which is the case of line OA. For horizontally spalling cracks, $\sigma_1 = 0$ results in $p = 2/3\ \sigma_3$, $q = -\sigma_3$, or $q/p = 1.5$ which is the case of line OG. For clays or silty clays, Schofield (1980) had suggested that the change from rupture to tensile crack occurs at a pressure $p = 0.1\ p_c$, where p_c is the effective confining stress at critical state. This is equivalent an overconsolidation ratio of approximately 20

(Fig.6a)

When the effective stress path crosses the crack surface OA, the soil element 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) and consequently its permeability can increase significantly and instantly. A significant internal/external shear stress at low confining stresses can cause the crossover of the crack-surface OA and a large increase in specific volume. When such condition occurs, the opening within the soil body may be an extensive crack or a local pipe or channel. If such opening (crack/channel) day lights into the water body it could lead to a free flow of water into the downstream slope.

RUPTURE/FRACTURE BASED ON LIQUIDITY INDEX AND CONFINING STRESS

Critical state soil mechanics (Schofield and Wroth 1968) has shown that it is possible to generalize the density or specific volume axis by converting to a liquidity basis. It was further shown that the critical pressure is about 5 kPaat the liquid limit and 500 kPa at the plastic limit. In his Rankine lecture, Schofield (1980) mapped the remolded soil behavior on a liquidity against pressure diagram as shown in Figure 7 utilizing the hundred fold increase in pressure from the liquid limit critical state to the plastic limit critical state which is two log cycles, so the rupture band has half the width of PI and will intersect the line p = 5 kPaat LI = 0.5. This intersection is a consequence of putting the lower limit of Coulomb rupture at p/p_{crit}= 0.1 (Schofield 1980). In the LI-p space, clear boundaries exists that separate the regions of fracture, rupture, and ductile behavior. This is an independent and convenient approach to separate the states of fracture/rupture/ductile yield behavior of the soil using its physical properties.

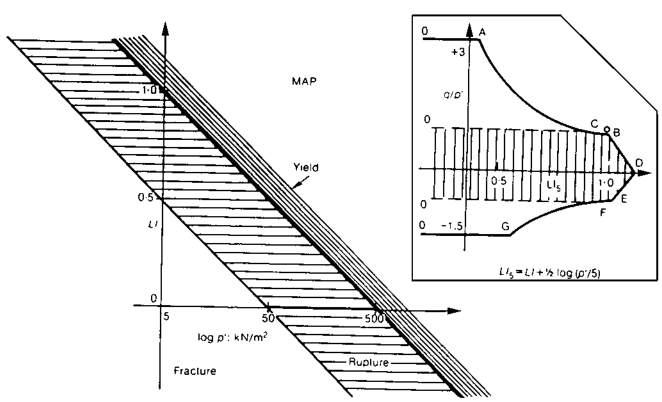


Fig. 7. LI- log p- Liquidity and limits of soil behavior (after Schofield 1980)

Considering a body of soil initially at LI = 0.5 and subjected to an elastic compression the map suggests at shallow depths where p < 5kPa there may be cracks, but for depths where 5 kPa < p < 50 kPa the soil will remain water-tight while deforming. In contrast a body of soil initially at LI= 0 will undergo fracture at depths for which p < 50 kPa or about 3 m of the overburden depth. In other words, the overburden depth should be larger than 3 m to ensure that deformation caused rupture planes (water tight) rather than open cracks. If LI= -0.25, the depth could be about 100 kPa or 6 m of depth. In this view the vertical face of the breach in Teton Dam can be seen as an open fracture in very strong soil, standing to a near vertical height of 6m or more.

In order to identify the band of behavior in which various states of soil lie in the LI-p space, Schofield (1980) defined their equivalent liquidities by projecting these states in the direction parallel to the critical state line towards the ordinate through p = 5 kN/m^2. The equivalent liquidity LI_5 can be shown to be LI_5 = LI+1/2 log (p/5) (Schofield 1980). Therefore, the equivalent liquidity equals liquidity as found in the ground plus a correction for stress. A value of LI_5 of less than 0.5 generally would indicate the fracture zone. Values of 0.5 to 1.0 represent the rupture zone. Values larger than 1.0 represent Cam-clay ductile zone.

The inset of Figure 7 shows the section of the behavior map at constant p: stress ratios q/p will increase as equivalent liquidity falls. In the high equivalent liquidity range, stress ratio increases linearly as liquidity of cam clay falls. The Hvorslev surface gives the rupture limits which allow higher stress ratios as lower values of p/p_{crit} are approached, but at the no tension limits, q/p = 3 in compression, and –1.5 in extension. There is a general increase of limiting stress ratio as equivalent liquidity falls, but this is not a continuous change because there is a change of limiting behavior from contours yield, to discrete rupture , to fracture of stiff fissured soil at equivalent liquidity below 0.5 (Schofield 1980).

The above concepts provide two independent approaches to analyze the cracking of soils particularly in a dam. The first approach makes use of mechanical properties determined from triaxial tests and oedometer tests to separate the three regions of soil behavior, the fractures, the faults, and the ductile yield. The second approach relies on physical properties, plasticity index, and liquidity index to identify such regions. The analysis herein employed both approaches to complement each other.

MATERIAL PARAMETERS

A large database of field and laboratory tests carried out during the post-failure investigations by the IRG and the IP exists in their reports. The laboratory testing herein was focused on the verification of some of the index and mechanical properties. About 1000 lbs of the zone-1 material was obtained from the remnants of the failed Teton Dam. The material was tested for physical and mechanical properties in the laboratory. Tests for physical properties included grain size, plasticity (Atterberg) limits, and proctor compaction curves. Mechanical tests included CU triaxial tests on remolded soils, UU triaxial compression tests, and consolidometer compression curves on compacted samples at w_{opt}-1, w_{opt}, and w_{opt}+1 to obtain constrained modulus at various confining stress levels.

The soil material that formed the impervious core of the dam (Zone 1) was derived from aeloian deposits and consisted of a uniform clayey silt (CL-ML) of low plasticity (PI~4), 80 percent passing through #200 sieve and about 15% of clay fraction (<2 micron). The average liquid limit (LL) was 23% and plastic limit (PL) was 19%. These values are plotted in Fig.-8 along with the idealized family of critical state lines for different soils. It can be seen that the Zone-1 material conforms well to similar materials with different plasticity characteristics.

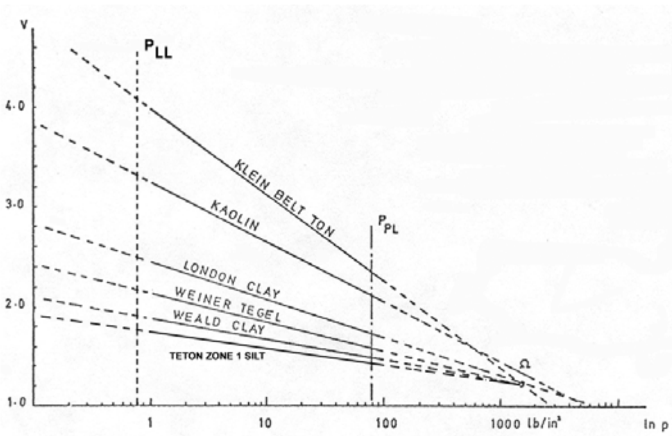


Fig. 8. Family of critical state lines in terms of LL and PL (modified after Schofield and Wroth-1968)

FEM ANALYSIS OF SOIL STATES AND "CRACK SURFACE" IN q-p STRESS SPACE

Finite element analyses were carried out for the longitudinal section of the dam. Longitudinal section was chosen because it captures all the variation along the bottom profile (berms, slopes, etc.). Plane strain condition is assumed to prevail along the section.

The analyses used an elasto-plastic model with modified cam-clay yield curve (Roscoe and Burland 1968). The CSL line with a slope M divides the yield curve into two regions, dry and wet sides. Porous elastic option is used to describe elastic behavior inside the yield curve. It is assumed valid for small strains (<5%) and is a nonlinear isotropic model in which the pressure varies as an exponential function of volumetric strain. The model parameters used in the analysis are shown in Table 1.

The model had five layers to simulate the construction of the dam. In the first step, the top four layers were removed and the remaining layer was analyzed. This was to allow the geostatic stress field to reach equilibrium with initial conditions, applied load, and boundary conditions. Subsequently, each layer was activated strain-free to simulate the construction steps. The strain free activation scheme was adopted to avoid creation of strain by the deformation of the previous layer. From the analysis, the shear stress (q) and the mean stress (p) were obtained along the longitudinal section and contours of q/p ratio were drawn.

Table 1. Material Parameters

Critical state parameter	Value
κ	0.012
λ	0.062
Γ	1.80
M	1.1
ν	0.3
G (psf)	40000
p'_c (psf)	3500

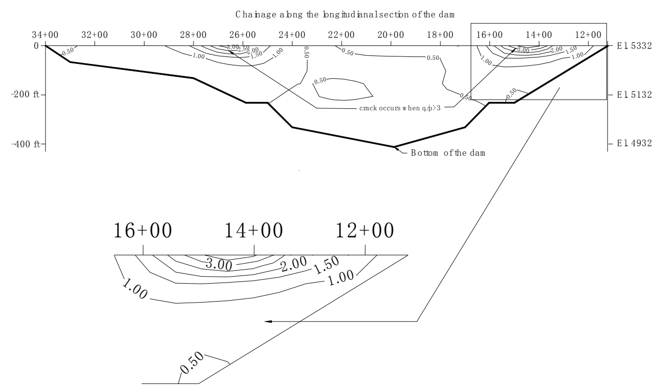


Fig. 9. Mapping of the contours of q/p stress-ratios and zones of potential crack(s) in the Cross-Valley Section

ANALYSIS OF SOIL STATES AND "FRACTURE/ RUPTURE" IN LI_5- p SPACE

As described before, the transition of soil behavior from the crack surface region to stable Hvorslev fault region occurs at an equivalent liquidity index of 0.5 corresponding to a confining stress of 0.8 psi or 5 kPa; or zero liquidity index at confining stress of 8psi (50kPa). Similarly, the Hvorslev-Coulomb rupture regime changes to ductile Cam-clay regime at 80 psi (500kPa) (Fig. 7). This was further confirmed by a series of consolidometer tests with Zone 1 samples compacted at varying initial liquidity indices. For various confining stresses, the corresponding equivalent liquidity indices LI_5 were determined and their position in the LI_5-p space were identified. This was transferred to the cross-valley section for the respective confining stresses. A mapping of the contours of equivalent liquidity index for the valley crosses section of the Teton dam was made as shown in Fig. 10.

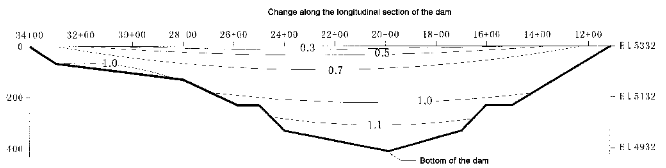


Fig. 10. Mapping of the contours of equivalent liquidity index (LI_5) and zones of potential crack(s) in the Cross-Valley Section

DISCUSSION

The state based soil mechanics theory presented here suggests that zones with stress ratio q/p larger than 3 would indicate the presence of a vertical split or crack (Fig. 6). Such zones can be identified for the Teton Zone 1 from the q/p contours shown on Fig.-9. These results clearly show that at the end of construction the dam core had developed such vertical cracks at two locations, Sta.14 + 50 in the right abutment and Sta.26 + 50 in the left abutment. The cracks at Sta. 14+50 were 32 feet deep from top of the crest while they were only 10 feet deep at Sta.26+50 (see Fig. 9). The state based theory further suggests that a contours of q/p ratio less than 3 would indicate the stable nature of the compacted soil which is the case for soil elements at depth and particularly below 32 feet (Fig. 9). Therefore, we conclude that the failure of the Teton dam was initiated as a result of water flowing through an open vertical crack on the right abutment near Sta. 14+50 during the first filling, which slowly eroded the crack into a large tunnel leading to the major breach.

The zone-1 core was capped by a 3-foot layer of sand and gravel roadbed which was subjected to continual vibration and compaction by the vehicular traffic inhibiting cracks in the layer. Further, the material parameters of the granular bed, their packing, and the characteristics were different from zone-1 material to exhibit cracking. As a result is was likely that the cracks below in the core zone apparently might not have day-lighted onto the roadbed to be visible during the first filling. However, numerous transverse cracks day lighted the roadbed in the left abutment soon after the dam breach, mostly near Sta. 26+50, where the q/p ratio was near or larger than 3 for shallow depths.

The map of the contours of liquidity independently confirms the above conclusions on the initiation of the Teton failure. Fig. 10 indicates shallow depths to about 30 ft between Sta. 14+00 and Sta.+ 16+00.

Because of the low plasticity (PI ~ 4), the liquidity index was very sensitive to placement water content and its influence on the performance of the soil core, under rapidly changing confining and shear stress conditions, particularly at the abutments. At the steep abutments, depth of the soil column decreases; consequently the soil elements were subjected to decreased confining stress. In effect, the soil columns in the abutments were in the Hvorslev regime while those in the valley section of the dam were in or near the ductile (Cam clay) regime. Again the changes in the deformability were further disrupted by the benches, which apparently caused significant differential deformations and increased shear stresses at some locations..

In earth structures such as Teton dam, fill materials are generally placed at or near the optimum water content to achieve a high density. The construction specification generally used the "optimum water content" as the reference point. At this state the material is partially saturated (80-85%), near plastic limit (PL) (low liquidity index), has higher stiffness, constrained modulus, and strength. For this placement condition, the state of soil (3>q/p> 1.2) remains in the Hvorslev regime of the stress-space (Fig. 6). However, if the placement water content is increased, the liquidity index will be increased. Consequently the material will become less stiff and more ductile. With increased confining stress or water content, the equivalent liquidity index would increase and consequently the state of soil can quickly migrate into the Cam-clay yield regime (1.2>q/p>0). The soil would then deform with positive pore water pressure response. Because of the low plasticity index of the Teton core (Zone-1), small changes in water content played a significant role in altering its liquidity index and the mechanical properties including the potential for cracks/rupture and ductility.

The concepts presented may also help explain some of the misgivings of previous investigations. We believe that the hydraulic fracture ((Seed et al, 1976, Sherard, 1987) and its relevance to the failure of the dam is fundamentally flawed (See also Muhunthan and Schofield 2000). Except for the shallow depths of 30 to 35 feet in some location, the q/p stress ratio is significantly lower than 3 (fracture level), which indicates fracturing of the soil would be difficult with increasing depth (Fig.-9). For hydraulic fracture to occur, the soil element must be subjected to seepage water, which can cause (a) physical wetting of the soil first and then (b) a corresponding hydraulic pressure in the soil. The physical wetting and saturation of the soil increases the liquidity index of the in-situ soil and consequently the soil element becomes more ductile and the material tighter and less permeable (Fig.7) (also the q/p ratio drops off quickly, Fig.6a). That is the stress-path moves significantly to the right to a more ductile and stable yield (Cam-clay) regime. Some researchers (Leonard and Davidson, 1984) characterized this phenomenon as "collapse on wetting", which is a misnomer considering that the stress path simply migrated from the stable Hvorslev regime to the stable ductile Cam-clay regime. On the second point, (b), the hydraulic pressure due to the water seepage would have a limited opposite effect of reducing the effective stress of the soil element. Any such reduction in effective stress due to the seepage pressure will be more than offset by changes in the mechanical properties (ductility) of the soil. The net effect is that the movement of the stress-path of the soil element is to the right and towards the Cam-clay regime (Fig. 6). Therefore the notion of "hydraulic fracture" by water pressures equal or less than the reservoir head, which could initiate a failure of the dam has no scientific basis.

We also conclude that the "wet seam" theory postulated during post-failure investigations (Leonards, 1987, Hilf, 1987) is fundamentally flawed. The majority of the core material on

Zone-1 was placed at a negative liquidity index (0.25 – 0.50) or in the Hvorslev regime in the stress-space (Figs. 6, 7). When seasonal rains and snow condition interrupted the material placement during construction, some layers might have been placed at wetter than the average or near liquidity index of unity. When subjected to large stresses, such pockets of material would fall into the Cam clay ductile regime and deform like potter's clay, "wet-seams" or wet-pockets producing positive pore water pressure. This was the case for a few random pockets/layers of fill that were affected by the rain/snow when full stripping and replacement of such layers were not possible during the construction. Although such layers were of low strength and stiffness, they provide more impermeable mass relative to the surrounding material and would have had no adverse effect on the performance of the dam.

The original design specifications of Teton dam stipulated placement water content of optimum minus 1% to optimum for the core which had only a small plastic index (PI<4). Based on our analysis, we believe that this was the fundament error in the design concept in leading to the demise of the dam. The placement water content represented an initial liquidity index of zero or negative, which allowed considerable depth of the core to be prone to fracture (Fig. 10). Without compromising the compacted density, for this material an additional one to two percent water content would have provided adequate equivalent liquidity index of at least 0.5 or more for most of the placed fill. This would have kept the entire fill intact in the Hvorslev regime where the material would have been stiffer, stronger and water tighter except for the top 5 to 10 feet (freeboard regime). Therefore, it is evident that the lack of knowledge at that time of the combined effect of liquidity and confining stress in controlling the mechanical behavior of Zone 1 contributed in a major way to the Teton dam failure. For the design of earth-structures, the theory based on the "state based soil mechanics" provides a better understanding of the physical and mechanical behavior of a broad spectrum of soils including that of Teton dam, which are subjected to different loading conditions.

CONCLUSIONS

A new theory is postulated for the failure of Teton dam based on the concepts of fundamental soil mechanics. Based on our investigation and discussion, it can be concluded that:

1. A transverse crack(s) or large opening(s) had developed in the core (Zone-1) to a maximum depth of 32 feet below the crest (top of the core) at the right abutment near Sta. 14+00. The analysis further indicates that much shallower cracks existed in the core in both abutments under the steep rock slopes. When the reservoir level rose to the level of the deepest crack, water flowed freely barreling downstream into the chimney drain (Zone 2).

2. The internal cracks might not have day lighted through the 3-ft thick granular roadbed which was subjected to constant vehicular traffic and compaction. Also, the parameters that affected the core were different from those of the overlying roadbed granular fill.

3. The uniform clayey silt (CL-ML) that was used for the core of Teton dam fitted well into the CSSM model that was developed for other soils with different plasticity. Although the clayey silt had relatively high values for the liquid limit (LL~23) and plastic limit (PL~19), the plastic index was relatively small (PI~4 or less). Consequently the liquidity index was very sensitive to the initial placement water content and its subsequent changes in mechanical properties due to varying confining stress. This phenomenon was a significant contributor to the cracking of the dam. Therefore, for clay-silt cores, it is more prudent to have the construction specification refer the "placement water content" with respect to the plastic limit (PL), than of the optimum water content.

4. A combination of material parameters such as the low plasticity of the core, the sensitivity of the liquidity index of the material to water content, its variation under the subsequent confining stress condition, and their influence on the constrained modulus played a key role in the cracking of the core. It appears that these aspects of fundamental soil mechanics and the phenomenon of cracking were not recognized in the original design of the dam.

5. The theoretical models based on "state based soil mechanics" used in this study provides a better scientific understanding of the mechanical behavior (stress-deformation) relating to the initial state of soil in the stress-space and the physical properties such as liquidity index and water content.

Acknowledgements: This study was sponsored by the Geomechanics and Geotechnical systems of the National Science Foundation under the grant CMS-0234103. The authors also wish to thank Chris Ketchum of USBR for supplying samples and allowing us to collect additional material from the remnants of the failed dam. We also thank Kathy Cox of the Department of Civil Engineering at Washington State University for her assistance with the preparation of the paper. The opinion expressed in this paper are those of the authors and do not necessarily represent those of any other individual or organization.

REFERENCES

Atkinson, J.H., 1993. An introduction to the mechanics of soils and foundations, McGraw Hill, London.

Independent Panel (IP), 1976. Report to U.S. Department of Interior and State of Idaho on Failure of Teton Dam, U.S. Government Printing Office, Washington, D.C.

Interior Review Group (IRG), 1980, Failure of Teton Dam-Final Report, USBR, Engineering and Research Centre, Denver.

Leonards, G.A. and Davidson, L.W., 1984. Reconsideration of failure initiating mechanisms for Teton Dam. Proc. Int. Conf. Case Histories in Geotechnical Engineering, St. Louis, Mo, May 7-11, Vol. III, pp. 1103-1113.

Leonards, G.A. (Ed) , 1987. Special Issue on Dam Failures, Engineering Geology, 24, Nos 1-4.

Hilf, J.W, 1987, The wet seam and the Teton Dam Failure, Engineering Geology, 24, pp 265-278.

Muhunthan, B and Schofield, 2000. Liquefaction and Dam Failures, GeoDenver-2000, Denver, Colorado.

Penman, A.D.M, 1987. Teton Investigation- A review of Existing Findings, Engineering Geology, 24, pp 221-237.

Pillai, V.S., and Muhunthan, B, 2001. A Review of the initial static shear (K) and confining stress (K) on failure mechanisms and earthquake liquefaction of soils, Proc. 4th Int. Conf. On Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics. San Diego, CA, March 26-31, paper # 1.51.

Pillai, V. S., and Muhunthan, B. (2002). Discussion of An investigation of the effect of soil state on the capacity of driven piles in sands, by Klotz, E.U. and Coop, M.R., Geotechnique Vol. 52(8), 620-621.

Roscoe, K. H. and Burland, J. B. (1968). On the generalized stress-strain behavior of wet clays. *Engineering Plasticity*, University Press, Cambridge, 535-609.

Roscoe, K.H., Schofield, A.N., and Thurairajah, A , 1963. Yielding of clays in states wetter than critical. Geotechnique, 13 (2), 211-240.

Schofield, A.N. 1966. “Original teaching on Cam-clay”, Lecture notes, Cambridge University Engineering Department.

Schofield, A.N. 1980. Cambridge geotechnical centrifuge operations, 20th Rankine Lecture, Geotechnique, 30(3), pp.227-268.

Schofield A.N. and Wroth, P, 1968. Critical State Soil Mechanics, McGraw-Hill.

Seed, H.B., Leps, T.M., Duncan, J.M., and Bieber, R.E., 1976. “Hydraulic fracturing and its possible role in the Teton dam failure” Appendix D of Report to U.S. Department of the Interior and State of Idaho on Failure of Teton Dam by Independent Panel to Review Cause of Teton Dam Failure, pp.D1-D39.

Sherard, J.L. 1987. Lessons from the Teton Dam Failure, Engineering Geology, 24, pp 239-25.